Seismic Testing and Analysis Program on High Aspect Ratio Wood Shear Walls

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INTRODUCTION

Simpson Strong-Tie introduced their Strong-Wall™ series of prefabricated nailed wood structural panel shearwalls in June of 1998. In addition to several innovative design features, they also introduced a new method of rating the wall’s allowable resistance and pushed the boundaries of what was considered acceptable in maximum aspect ratios for light framed shearwalls resisting high seismic demands. As required by the Acceptance Criteria for Prefabricated Wood Shear Panels (ICBO ES AC130), Strong-Wall performance ratings are derived from reversed cyclic/static testing conducted in accordance with the Sequential Phased Displacement (SPD) protocol. While cyclic tests provide some insight into the force-displacement characteristics of a wall, they cannot predict dynamic response. For this, dynamic testing or nonlinear dynamic analysis is needed, but to the authors’ knowledge neither had been performed for high aspect ratio nailed wood structural panel shear walls until the current work. The research described herein is part of an overall initiative titled the “Earthquake 99 Project”. The Earthquake 99 Project has been designed and managed by TBG Seismic Consultants in collaboration with the Department of Civil Engineering at The University of British Columbia.

LOAD RATING

The new method of rating a wall’s allowable resistance referred to above essentially employs drift limits explicitly in the development of the Allowable Load (limit states design for wood is not commonly used in the US). The procedure is very straightforward provided there is a code specified drift limit and code defined relationships between: 1. The calculated design level deflection and the expected maximum inelastic deflection of the structure during the real earthquake event (a drift amplification factor); and 2. The Strength level (limit states design) and the Allowable Stress Design (ASD) level earthquake demand. The governing equations from the 1997 Uniform Building Code are as follows:

\[
\begin{align*}
\Delta_M &= 0.7R \Delta_S \\
\Delta_{M,\text{Max}} &= 0.025H \\
E_{\text{ASD}} &= E_S / 1.4
\end{align*}
\]

Where:

- \(\Delta_M\) = An estimate of the expected “real” inelastic deflection expected during the earthquake
- \(R\) = The seismic force reduction factor
- \(\Delta_S\) = The calculated Strength design level deflection due to the prescribed design force, \(E_S\), (the building should still be “elastic” at this deflection level).
- \(H\) = The story height
- \(E_{\text{ASD}}\) = The Allowable Stress Design level earthquake demand.
- \(E_S\) = The Strength Design level earthquake demand.

Together with the backbone curve from cyclic testing, these equations are used to establish a maximum resistance, at either the Strength or ASD level, that satisfies the drift limits. The maximum permitted inelastic deflection will change depending on structural system type and other factors, but for most light-framed structures it will be 2.5% of the story height. (Note: in the previous version of the UBC drifts limits were checked against an elastic response level criteria, but

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in the 1997 UBC calculated elastic response level drifts are amplified to expected inelastic “real” levels and then checked against inelastic response level criteria. The purpose of this is to help the designer understand just how large the “real” drifts might be. Although it may not appear to be, the end result is very similar to the previous methodology.)

Setting $\Delta_M$ to 0.025$H$, equation (1) is rearranged and solved for $\Delta_S$. Then from the backbone curve of the cyclic testing, a force associated with that deflection is established. If this force was used as a Strength level factored resistance, then, it would satisfy the amplified drift limit requirements. Because of the relationship shown in equation (3), this force is further reduced by a factor of 1.4 to establish and ASD level design resistance compatible with the UBC inelastic drift limits and the relationship between Strength and ASD.

Several things need to be considered when using this procedure. First, it is assumed that the question of appropriate energy dissipation, ductility and overstrength is implicitly handled through the designated R value for the system being evaluated in this manner. This places a great importance on the part of the Code reviewer who would be establishing Code approval for new products. However, there is an additional requirement of AC130 to help address the situation of too little deformation capacity. If the peak resistance, or Strength Limit State (SLS), observed during the cyclic testing occurs at a deflection less than the maximum allowed inelastic deflection (0.025$H$ in this case), then $\Delta_M$ in equation (1) is set to this lower value when solving for $\Delta_S$ and establishing an appropriate resistance. For this reason the method is sensitive to the specific cyclic protocol used to test the wall. The SPD protocol required by AC130 has been shown by Ficcadenti, Steiner, Pardoen and Kazanjy (1998) to be the most conservative of the commonly used displacement regimes for cyclic testing. Because of this and the quality assurance requirements under AC130 for prefabricated shearwalls, the first cycle backbone curve is used to establish the SLS.

There were two main reasons for developing this new method. First, from field reconnaissance after the Northridge earthquake and testing by Simpson Strong-Tie and others, narrow shear walls were shown to deflect much more than previously thought. Thus any method of rating pre-fabricated narrow walls needed to explicitly account for this. Second, traditional methods of basing a resistance on a yield point are difficult, if not impossible, to use with light-framed shear wall systems. Yamaguchi, Minowa and Miyamura (2000) showed that the shape of the backbone curve and the general area that one might consider to contain the yield point can change because of loading protocol and strain rate. Gad and Duffield (2000) emphasize that the load-deflection curves of framed residential structures are highly nonlinear and in fact have no distinct yield point in the traditional sense. Additionally, while there have been many proposed “definitions” of yield and ultimate strengths for light-framed systems, there has been little research into how these definitions actually relate to the seismic performance of light framed structures. Because of these problems, New Zealand researchers are considering the use of nonlinear time history analysis in conjunction with standardized ground motions and maximum inelastic deformation limits to establish the maximum mass a wall system can be permitted to resist (Deam 1999).

**TESTING PROGRAM**

The main objectives of the Simpson Strong-Tie’s testing program were to:

1. verify the deformation-based load rating method under dynamic loading and the design provisions of the 1997 Uniform Building Code;
2. establish “control tests” using “equivalent” 2:1 aspect ratio wood structural panel shear walls as allowed by Code;
3. ensure that testing was as realistic as possible and representative of actual complete systems; and
4. verify Simpson Strong-Tie’s proprietary nonlinear time history analysis software.

As a part of the “Earthquake 99” project, a new unidirectional shake table was constructed at the University of British Columbia. The table can accommodate structures with maximum plan dimensions of 20 ft. by 25 ft. and a maximum specimen inertial weight of 45,000 lbs. Within these constraints, single story “sub system” tests were developed to investigate the variables of engineered shear wall type, exterior wall finish contribution, interior wall finish contribution, building period, damping (viscous vs. hysteretic) and response to multiple earthquakes. Top of wall height was 8 ft., and plan and elevation views are shown below.
Figure 1: Plan View of Single Story Sub System Test

a) Exterior Load Bearing Walls with Primary Wood Structural Panel Shear walls
b) Interior Load Bearing Wall

**Figure 2:** Details of Wall Elevations

Shaking was parallel to the long dimension of the structure, with one half of the short dimension exterior walls sheathed for stability in that direction. Primary wood structural panel shear walls were placed only in the front and back exterior walls and were located at the beginning and end of each wall. Code shear walls were constructed in accordance with the requirements of the 1997 UBC.

Selection of ground motions was driven by the target design parameters of seismic Zone 4, type A fault, type D soil, 10 km from the fault and 10 percent chance of exceedence in 50 years. The CUREe-Caltech Woodframe Project (2000) has established the Canoga Park record from the 1994 Northridge earthquake scaled up 20% (120 Canoga Park) as one record that meets these requirements. Additionally, another Northridge record from the Sherman Oaks area was chosen because it also met the criteria.

As can be seen below, the response spectrum of each of these records follows very well the trend of the 1997 UBC 5% damped response spectrum.
Figure 3: Comparison of 5% Damped Response Spectra for Ground Motions Considered with 1997 UBC Response Spectrum.

A summary of the peak ground motion parameters is shown below with acceleration time histories following:

Table 1
Peak Ground Motion Parameters

<table>
<thead>
<tr>
<th>RECORD</th>
<th>PGA (g)</th>
<th>PGV (in/s (cm/s))</th>
<th>PGD (in (cm))</th>
</tr>
</thead>
<tbody>
<tr>
<td>120 Canoga Park</td>
<td>0.50</td>
<td>19.7 in (50.0 cm/s)</td>
<td>5.6 in (14.1 cm)</td>
</tr>
<tr>
<td>Sherman Oaks</td>
<td>0.45</td>
<td>21.6 in (54.9 cm/s)</td>
<td>5.2 in (13.1 cm)</td>
</tr>
</tbody>
</table>

Figure 4: 120% CUREe Canoga Park Acceleration Time History
The selection of the inertial mass was driven by the above mentioned design parameters. For short period structures the target design parameters yield an ASD level base shear coefficient of 0.1429.

Of the available Simpson Strong-Walls, four of the 4:1 aspect ratio SW24x8-4 were chosen for the primary lateral force resisting system, with two in the front wall and two in the back wall. No wood structural panel shear walls were placed in the interior load-bearing wall. Each Strong-Wall has an ASD level rated capacity of 1610 lb., for a total design capacity of 6440 lb. This divided by the base shear coefficient yielded a target design inertial weight of 45070 lb. For the “control” tests with traditional shear walls, four 2:1 aspect ratio nailed wood structural panel shear walls with a combined Code-rated ASD level capacity of 7360 lb. (15/32” OSB with 10d common nails at 4/12) were used in the same locations as the Strong-Walls. For the initial one-story sub system tests, considerable concrete inertial mass had to be added to the roof diaphragm to achieve the 45,000 lb. inertial weight representative of the two-story tests.

In order to test realistic structures, gypsum wallboard was applied to the interior of the front and back exterior walls, both sides of the interior load bearing wall and to the bottom of the ceiling/floor joists above (one-story sub system tests were simply the first story of the planned two story tests). A nailed wood structural panel diaphragm was applied to the top of the floor joists and constructed such that its response would be essentially linear throughout the testing. Shear transfer details typical of California engineered construction were used to transfer shear from the roof diaphragm to the exterior shear wall lines, but again these were done conservatively in an effort to constrain nonlinear response primarily to the shear walls. Finally, a three-coat stucco system was applied to the exterior of the front and back walls.

**INITIAL RESULTS AND DISCUSSION**

Initially, the test procedure called for two series of sub-system tests, one for the Code walls and one for the Strong-Walls. For each wall type, the structure would be subjected to both ground motions without any repairs being made in between. However, after the first test with the Code walls, it was apparent that the contribution of the stucco had been severely underestimated. When subjected first to the Sherman Oaks record, the resultant maximum story drift was only 15mm (0.59”). In an effort to achieve more drift, the structure was subsequently subjected to a scaled Canoga Park record. However, instead of the 120% scaling factor mentioned previously, a scaling factor of 150% was used in an effort to utilize the maximum capacity of the table. The resulting drift was almost the same as the previous test. Because of this, it was decided to remove the stucco and then retest with the same two records. The same testing sequence was also carried out for the sub-systems tests with Strong-Walls. No repairs were made to the structures between any of the tests.

At the time of this writing, the eight sub-system tests for the Code walls and the Strong-Walls have just been completed. While a thorough review of the data will take considerable time, some of the initial results are summarized below.

![Figure 5: Sherman Oaks Acceleration Time History](image)
Table 2
Maximum Story Drifts from Shake Table Tests

<table>
<thead>
<tr>
<th>Wall Type</th>
<th>With Stucco</th>
<th>Without Stucco</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sherman Oaks</td>
<td>Sherman Oaks</td>
</tr>
<tr>
<td></td>
<td>150 Canoga Park</td>
<td>150 Canoga Park</td>
</tr>
<tr>
<td>Simpson SW24x8-4 Strong-Wall</td>
<td>6mm</td>
<td>27mm</td>
</tr>
<tr>
<td>Traditional 2:1 Aspect Ratio</td>
<td>15mm</td>
<td>27mm</td>
</tr>
<tr>
<td>Engineered Shear Wall</td>
<td>13mm</td>
<td>109mm</td>
</tr>
</tbody>
</table>

The relative displacement time histories for each system type for the last test are shown below:

Figure 6: Final Test (No Stucco – 150 Canoga Park) Displacement Time Histories

Immediately obvious from Figure 6 is that the permanent drift in the Strong-Wall system is substantially less than the Code wall system. Additionally, the high amount of degradation observed in the Code Walls system shows up as wide
response bands and sustained large deflections. These results lend strong support for the rating method used for the Strong-Walls as discussed earlier, and also show that the Strong-Wall can sustain multiple large earthquake events better than their traditionally engineered shear wall counterparts.

A series of full-scale two-story tests should be completed by the time of the 2000 WCTE. These tests will use the SW24x8 Strong-Walls in both stories. As before, four of these walls will be placed in the first story, and four more will be placed in the second story stacked over those in the first story.

**ANALYTICAL MODELING**

Currently, there is no user-friendly nonlinear time history analysis program that utilizes state-of-the-art hysteretic modeling elements and is also optimized for shear wall systems. For this reason, Simpson Strong-Tie has developed a proprietary computer program with these capabilities. The program is two-dimensional and considers mass to be concentrated discretely at the story levels. Wall elements can be modeled as linear or nonlinear, and p-delta effects can be explicitly accounted for. Nonlinear models can be simple bilinear representations or more complex representations employing tri-linear backbones, various types of pinching, strength degradation and stiffness degradation. Currently, these elements are based upon the work of Professor A. M. Reinhorn at the State University of New York at Buffalo. However, other elements can be easily added, and those proposed by Dolan (1989) and Foliente, Paevere, and Ma (1998) are being considered for implementation in a future version of this computer program. Multiple hysteretic elements can be defined to work in parallel for each story, and an additional discrete damping element is also available for each level. Viscous damping is set as a percent of critical based on the stiffness of the “primary” hysteretic element. To aid in obtaining a good fit, the results of cyclic tests can be brought in to visually fit the mathematical element. As an additional aid, a cumulative graph of dissipated energy is displayed to compare the cyclic test and the model. Furthermore, the shape of individual loops anywhere in the hysteresis being fit can be easily reviewed.

To help verify the accuracy of the program, a series of shake table tests were conducted in September of 1999 at the University of British Columbia on individual Simpson Single Portal SW16x7-4HD Strong-Walls. These walls are nominally 6:1 aspect ratio and are designed to act as ductile wood frames at garage openings in light-framed construction. The tests were conducted with the assistance of Professor J. D. Dolan of Virginia Tech Polytechnic Institute and State University on a three-hinge frame he designed while obtaining his doctoral degree at the University of British Columbia. The program was then used to model the response of the wall, and the results are compared below.
Figure 7: Test vs. Analysis Displacement Time History for Single Simpson Strong-Tie SW16x7-4HD Using the Northridge Arleta record and 13450 lb. inertial weight

Figure 8: Test vs. Analysis Hysteresis for Single Simpson Strong-Tie SW16x7-4HD Using the Northridge Arleta record and 13450 lb. inertial weight
There are some differences in the predicted vs. observed response, but overall the result is quite good. Peak displacement is well predicted, and the differences in the cycles after the peak displacement are probably the result of differences between the degraded cycles as modeled by the analytical element and the real hysteresis. Early attempts at modeling using only bilinear elements proved to be unsuccessful, as they cannot exhibit the crucial behavior of low deformation hysteresis. While 3% critical damping was used in the analysis above, preliminary analysis of the subsystems is indicating that a value of about 5% critical may be more appropriate for complete structures (when also explicitly modeling the stucco and gypsum boards by including extra hysteretic elements - otherwise much more than 5% would be necessary to account for the energy dissipation of these elements). This supports the findings of Foliente (1995) that when accurate hysteretic models are used, the necessary additional viscous damping should only need to be in the range of 1% to 5% of critical.

The results of these tests also show that even the highest aspect ratio Simpson Strong-Walls can perform very well when rated under the displacement based method previously discussed. Using the base shear coefficient of 0.1429 as defined earlier and the rated ASD capacity of 1460 lb., the inertial mass was 32% higher than it should have been. Furthermore, this test was even more severe because there were no “nonstructural” elements to provide additional damping. Even so, the peak wall displacement was only about 50% of the displacement capacity of the wall.

CONCLUSION

The research described herein is part of a larger effort at the University of British Columbia called the Earthquake 99 Project. In cooperation with the CUREe-Caltech Woodframe project, all data for nonproprietary systems will be released to further the understanding of the performance of light-framed structures subjected to seismic excitation. While the testing is still ongoing, early results indicate that high aspect ratio wood structural panel shearwalls can perform very well when rated in accordance with the displacement-based method outlined in this paper. Finally, good hysteretic models are necessary for accurate time history modeling.

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REFERENCES


