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EFFECTS OF CONSTANT AND ELF STRENGTH PROFILES ON THE PERFORMANCE OF WOOD FRAME SHEAR WALL STRUCTURES

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ABSTRACT: Prior numerical studies to evaluate the sensitivity of modeled collapse performance of wood-frame wood structural panel shear wall structures have been limited to the 1- to 5-story index building models used in the FEMA P695 study. Investigated parameters included inherent damping, hysteretic behavior, P-delta effects, and strength profiles. Results showed that greater overstrength and a tapered strength profile from small at the roof level to large at the 1st story level had better collapse performance (i.e., lower collapse risk) and helped to support changes to design requirements in AWC SDPWS-2021. Sensitivity studies performed on 3-, 4- and 5-story building models reported herein expand on the prior evaluation of overstrength and strength profile effects. In addition, results from the study of a 6-story building model are also reported. Structural performance was evaluated using the probability of collapse per the FEMA P-695 methodology, and additional performance metrics such as energy dissipation and maximum story drift were investigated. The results of this investigation show the importance of the strength profile in Light Frame Shear Walls where a Constant strength profile along the building's heights displays the worst performance.

KEYWORDS Wood-frame shear wall, seismic analysis, collapse performance, FEMA P-695

1 INTRODUCTION

The collapse performance of wood-frame wood structural panel (WF/WSP) shear wall structures has been numerically investigated in the past [1-6]. Different parameters, such as inherent damping, hysteretic behavior, P-delta effects, and vertical strength profile, have been modified to evaluate the collapse performance using the FEMA P-695 methodology [7]. This study was conducted to expand previous studies on 3-, 4- and 5-story building models [1] and, for the first time by the authors, to evaluate a 6-story WF/WSP shear wall building model. Vertical strength profile is studied more in-depth by looking at different results, such as the sequence of yielding during a pushover analysis, energy dissipation at different intensity levels, earthquakes, and collapse performance. In addition, the influence of Rayleigh damping was quantified in terms of the probability of collapse and energy dissipation. Counterintuitively it was found that a constant strength profile along the building's height worsens the performance instead of being a conservative design.

2 NUMERICAL MODELS

The 1- to 5-story building models were taken from the FEMA P-695 wood-frame models. These systems were designed using ASCE/SEI 7-22 [8] and AWC Special Design Provisions for Wind and Seismic (SDPWS-2021) [9] requirements. The 6-story building model was developed assuming the same basic principles used for the 1- to 5-story building models.

Numerical WF/WSP shear models were created by modeling the load-deformation response at each level as a shear spring. The shear spring at each level was made using the CUREE-SAWS [10] model, which represents the nonlinear hysteretic behavior that accounts for pinching, stiffness, and strength degradation and is governed by ten different parameters [10-11].

Since diaphragms were idealized as flexible, torsion was not considered, and 2D numerical models developed in OpenSees were deemed sufficient to represent the buildings [12]. The shear wall response was modeled

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using the SAWS 10-parameter hysteresis model with the backbone used in this study shown in Figure 1 as "Reference Backbone." The reference backbone, representing low-aspect ratio shear wall behavior, was used at all stories except with different scaling employed that varied peak strength to assess strength profile effects while preserving the backbone load-deformation shape.

In addition to the material nonlinearities incorporated in the numerical models, geometric nonlinearities (P-delta effects) were included explicitly. P-delta effects were modeled using a leaning column with no flexural capacity and significant axial stiffness carrying the gravity load.

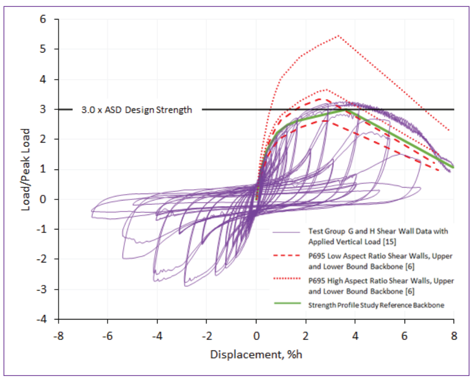

Figure 1 Reference Backbone Curve (green) vs. Test Data (purple) and FEMA P695 Wood Example (red) [1]

The strength profiles to be investigated herein are two for each structure. A "Constant" strength profile where the first story lateral force computed using the equivalent lateral force procedure is assigned as the strength along the building's height. The second strength profile ("ELF") is considered to be the same as the lateral forces computed using the equivalent lateral force procedure.

The shear walls were designed using allowable stress design (ASD) in accordance with SDPWS-2021 [9] and the seismic demands in accordance with ASCE 7-22. Based on test data, each shear wall's overstrength per story (Fu/VASD) was considered equal to 3. F_u is the maximum lateral strength shear wall capacity modeled using the ten parameter material within OpenSees. V_{ASD} is the allowable stress design story shear. In the case of the Constant profile, the maximum lateral strength capacity F_u was constant along the building's height. On the other hand, the F_u in the ELF profile follows the seismic demands at each story in accordance with ASCE 7-22 (Figure 2).

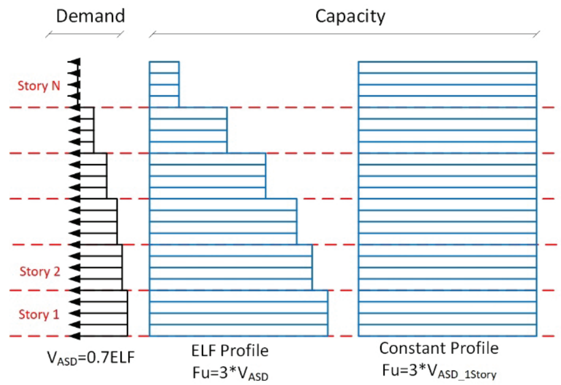

Figure 2 ELF vs. Constant Profile

Finally, Rayleigh damping was incorporated using three different percentages 1%, 2%, and 5% of critical damping. Damping was considered proportional to the building's mass and initial stiffness.

3 METHODOLOGY

The FEMA P695 methodology was used for this parametric study. The numerical models were subjected to static pushover and incremental dynamic analyses to evaluate their collapse performance. Static pushover analyses were performed to quantify overstrength and period-based ductility (μT) parameters. Before assessing the collapse performance of both strength profiles, the dynamic performance at MCE level was quantified using displacement time history and energy dissipation per story.

Incremental dynamic analyses were performed using 44 Far Field ground motions where spectral accelerations were increased until collapse was determined, defined as when the structure could no longer resist lateral loads (dynamic instability) such as from side-sway collapse or exceedance of a specified story drift of 8%. From these results, the probability of collapse for the maximum considered earthquake (MCE) was computed and compared among models.

4 NONLINEAR STATIC PUSHOVER ANALYSES

Nonlinear static pushover analysis was performed using the load pattern proposed in FEMA P695, based on the building's fundamental mode shape. All the structures using both strength profiles, Constant and ELF, were subjected to a lateral load until failure. The pushover curves, in conjunction with analyzing the sequence of yielding along the building's height, provide important insight into the structure's behavior under lateral loads.

Figure 3 displays the pushover curves, including P-Delta obtained for the 4-story model using both strength profiles, Constant and ELF. These curves show how both profiles have the same lateral strength with an overstrength (**Ω**) equal to 1.86. However, the

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displacement capacity is significantly more significant for the ELF profile reaching a roof drift of 0.029 compared to 0.012. While the Constant profile has a ductility of 5.75, the ELF profile has a ductility of 8.66. This difference is important, and the reason behind the better performance is that yielding is concentrated in the first story in the Constant profile. On the other hand, yielding for the ELF profile when subjected to the same lateral load used for the Constant strength case is better distributed along the building's height.

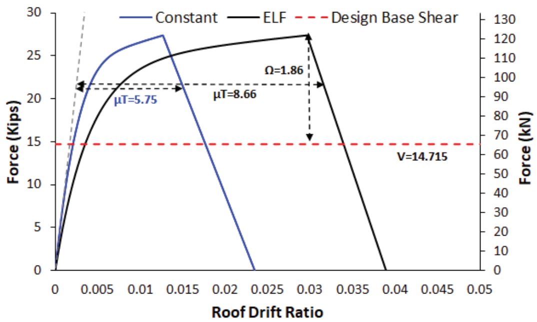

Figure 3 4-Story Pushover Curves

Figure 4 illustrates the performance of the 6-story models when subjected to the increasing lateral load. It can be seen once again that the maximum strength is the same for both strength profiles. However, the displacement capacity of the ELF profile is significantly larger (roof drift=0.011) than the Constant profile (roof drift=0.029). The ductility for the Constant profile is 5.06, while the ELF has a ductility equal to 8.40.

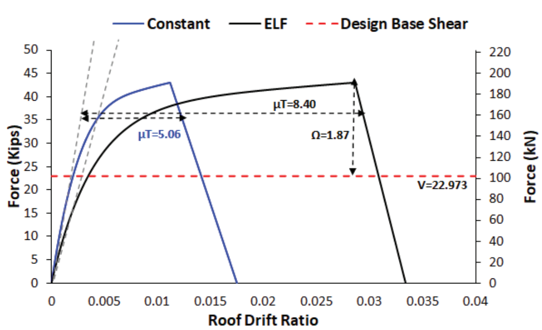

Figure 4 6-Story Pushover Curves

Even though pushover curves for the 4- and 6- story models are presented, the same trends were seen for the 3- and 5-story models.

5 GROUND MOTIONS

The ground motions used in this study were taken from the Far-Field set provided by the FEMA P-695 methodology. The Far-Field record set is twenty-two horizontal component pairs of records (44 in total) from sites located at a distance equal to or greater than 10 km from the fault rupture.

The methodology used to scale the ground motions was also taken from the FEMA P695 methodology and involved two steps: normalizing the individual ground motions with respect to the peak ground velocity and scaling the median acceleration spectrum to the MCE acceleration spectrum at the structure's natural period of vibration. The ground motions were also normalized by the 5-percent damping spectral acceleration at the building's period (S_{MT}). In this way, a scale factor of 2/3 and 1.0 represents the Design Basis Earthquake and Maximum Considered Earthquake (MCE), respectively.

6 DYNAMIC TIME HISTORY RESPONSE AT MCE

Before performing an Incremental Dynamic Analysis where the ground motions are amplitude scaled until the structure collapses, the seismic performance of the models is evaluated at the MCE level earthquake (Scale factor equal to 1). The performance quantification compares the roof drift ratio for both strength profiles. To further understand the complete behavior of the building model with a different strength profile when subjected to a ground motion, the hysteretic energy dissipated per shear wall at each story is computed. The hysteretic energy is calculated integrating the area under the Force (kN) vs. deformation (mm) curve. The results presented herein are for the 2% damping case.

Figure 5 illustrates the roof drift ratio for the 4-story model caused by one of the ground motions scaled at the MCE level. While results for only one ground motion are shown, the trend was the same for other ground motions that caused large displacements. It can be seen that the Constant profile has a residual roof drift ratio of around 0.0159 (roof drift = 7.65in, 194.36mm), while the ELF strength profile shows a residual roof drift equal to 0.048 (roof drift = 2.31in, 58.75mm). The larger residual deformation in the Constant strength profile case is from concentrated yielding in the first story.

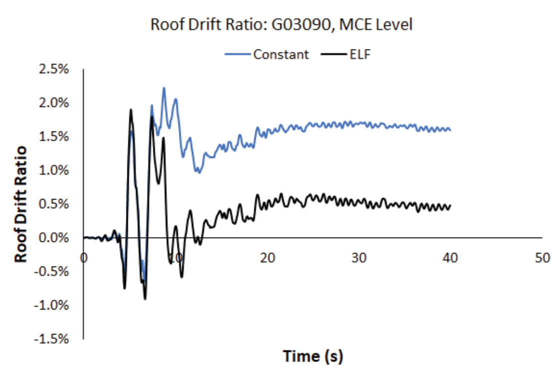

Figure 5 4-Story Roof Displacement Time History Analysis (G03090 MCE, 2% Damping)

Figure 6 and Figure 7 display the hysteretic energy dissipated by the Constant and ELF strength profile when the structure is subjected to the same ground motion (G03090) that caused the roof drift response shown in Figure 5. The dissipation is computed in time as the integral between the story force and story drift [13]. From

Figure 6 for the Constant profile, it can be seen how the first story shear wall is the one that dissipates energy the most (87.2%), meaning that this wall yields the most. This is a straightforward consequence of having first story drift concentrations. This figure also indicates that the last two stories remained elastic because they barely dissipated energy.

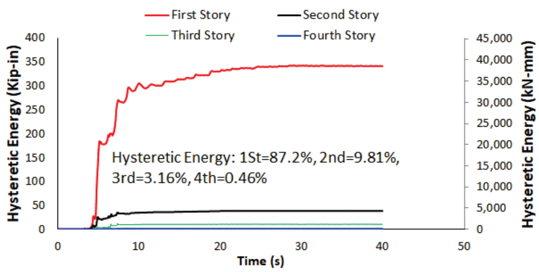

Figure 6 4-Story Energy Dissipation Per Story (Constant Profile, G03090 MCE, 2% Damping)

On the other hand, Figure 7 illustrates the hysteretic energy dissipated by each shear wall per story when the 4-story model has an ELF strength profile. The percentage of the energy dissipated by the first story is 69%, meaning yielding was distributed better along the building's height. Even though yielding distributes more uniformly in comparison to the Constant strength profile, more investigation is required to evaluate the effect of strength profiles that are intermediate between ELF and Constant as well as strength profiles based on targeted strengthening of lower stories to distribute better yielding in the first 2 or 3 stories.

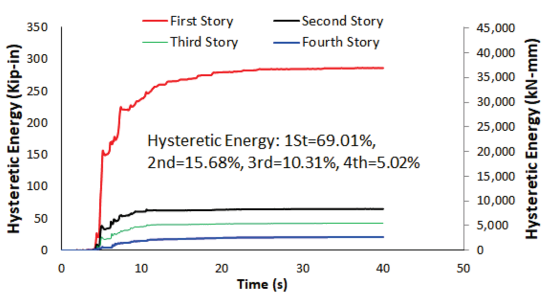

Figure 7 4-Story Energy Dissipation Per Story (ELF Profile, G03090 MCE, 2% Damping)

Figure 8 presents one of the results obtained from the 44 ground motions scaled at the MCE level to which the 6-story model was subjected. This figure shows that the Constant profile strength presents a residual roof ratio of around 0.79% (roof drift = 5.69in, 144.62mm). On the other hand, the ELF strength profile shows a residual roof drift ratio equal to 0.064% (roof drift = 0.46in, 11.68mm). The residual drift ratio is typically dominated by first story deformations, which could be confirmed by looking at the hysteretic energy dissipated by the shear walls.

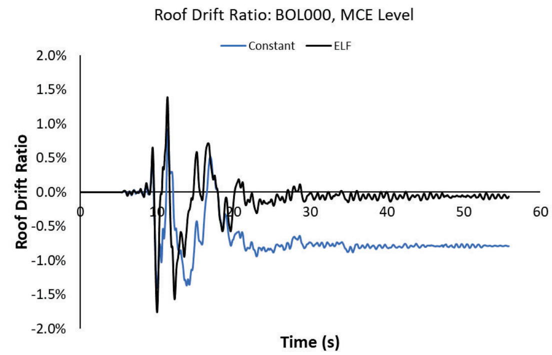

Figure 8 6-Story Roof Displacement Time History Analysis (MCE, Zi=0.02 Ko)

Figure 9 and Figure 10 show the hysteretic energy dissipated by the shear walls along the building's height when subjected to the same ground motion (BOL000) that caused the roof drift response shown in Figure 8. It can be seen how 89% of the dissipated energy, when the Constant profile is used, concentrates in the first two stories. On the other hand, around 71% of the dissipated energy, when the ELF profile is used, focuses on the first two stories. The reduction of the energy dissipated in the first story is around 18% when the ELF profile is used. As a result of having a more uniform distribution of yielding along the building's height, residual deformations decrease when the ELF strength profile is used, and the seismic performance is improved.

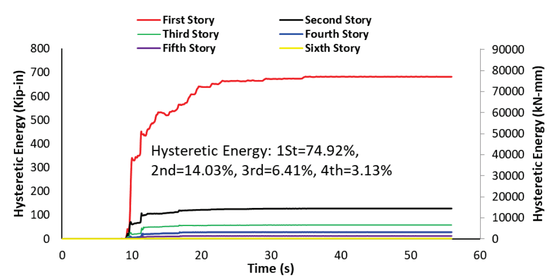

Figure 9 6-Story Energy Dissipation Per Story (Constant Profile, BOL000 MCE, 2% Damping)

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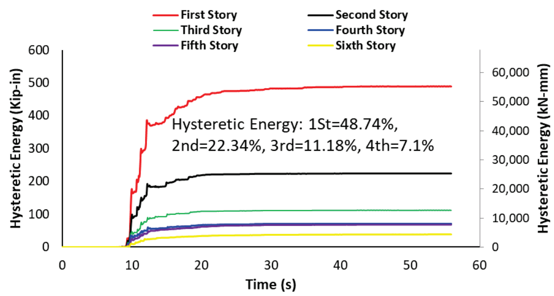

Figure 10 6-Story Energy Dissipation Per Story (ELF Profile, BOL000 MCE, 2% Damping)

7 INHERENT DAMPING ENERGY DISSIPATION

Inherent damping in structures considers energy dissipation from components not explicitly modeled using nonlinear inelastic hysteretic models. It would be challenging to account for all the energy dissipated by all the elements in a structure during an earthquake and the energy dissipated by the lateral resisting system. Therefore, the percentage of the total energy dissipated during an earthquake is typically assigned to inherent damping using either modal damping or Rayleigh damping. Inherent damping evaluated in this study uses Rayleigh damping.

Inherent damping is assigned as an imaginary linear viscous damper within the building using a damping value that is a percentage of the critical damping. The commonly used values assumed vary from 1% to 5%. For instance, FEMA P-695 proposes that the value of inherent damping should be between 2% and 5%.

When Rayleigh damping is used, the damping matrix is formed by considering this matrix proportional to the stiffness and mass matrix. However, the stiffness matrix could change during a seismic analysis when the material and geometric nonlinearities are included in the model. Therefore, there are mainly two options when proportional stiffness damping is considered: initial and tangent stiffness.

In this investigation, 1%, 2%, and 5% proportional to the initial stiffness were considered, leaving the same analyses to be performed using tangent stiffness for future research. However, less energy is expected to dissipate when the tangent stiffness is used because the model's stiffness reduces when yielding occurs. Therefore, this section of the paper shows the amount of energy compared to the total energy taken by the inherent damping. Even though there is nothing proposed for how much energy inherent damping should dissipate as a percent of the total energy dissipated, this type of result could help inform future recommendations.

It was found that the percentage of energy dissipated by inherent damping compared to total energy dissipated for the 44 ground motions scaled to the MCE level depended mainly on the amount of damping assigned and not on the number of stories or the strength profile. The amount of energy dissipated by the inherent damping varies between 10% and 12% for 1% damping, 16% and 20 % for 2 % damping, and 30% and 38% for 5% damping. As previously mentioned, there is no proposed value on how much energy should be dissipated by inherent damping. In the case of wood-frame shear wall structure, this energy represents non-modeled components which, for this study, could potentially represent stucco and gypsum wallboard finish materials, partition walls, and diaphragms that will dissipate energy during an earthquake.

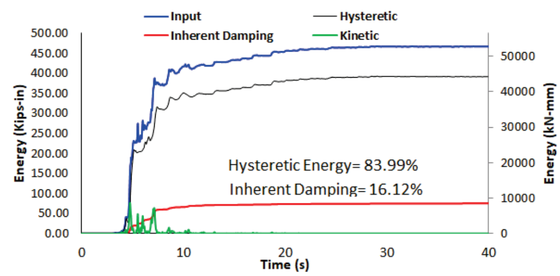

Figure 11 4 Story Energy Balance (Constant profile, 2% Damping)

Figure 11 and Figure 12 show the energy balance using 2% and 5% inherent damping for the 4-story Constant profile. The analyzed ground motion is again the G03090. It can be seen how energy dissipated by the inherent damping increases from 17% to 30.9%. Note that the hysteretic energy shown herein is taken by all the shear walls along the building's height. However, as previously described, the way it distributes depends on the strength capacity profile.

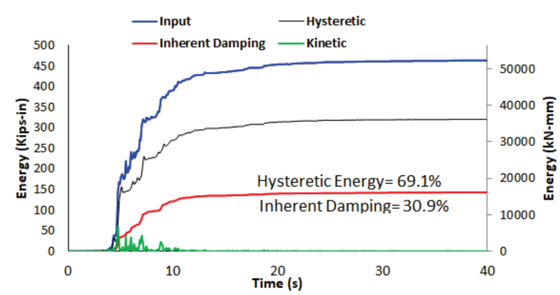

Figure 12 4 Story Energy Balance (Constant profile, 5% Damping)

8 COLLAPSE EVALUATION

The collapse performance of the models is evaluated following the FEMA P-695 procedure. Each building model is subjected to 44 Far-Field ground motions where Incremental Dynamic Analysis [13] is used to assess the model's collapse. Based on previous studies, the collapse is assumed to occur when story drift reaches a value of 8%, or dynamic instability occurs [1]. Once the scale factor that causes the collapse per ground motion is computed, the collapse margin ratio (CMR) is calculated.

The CMR is the ratio between the spectral acceleration that causes the median collapse (22 out of the 44 ground motions) to the spectral acceleration at the MCE level.

The probability of collapse is used as the metric to compare the performance among the building models and strength profiles. The ACMR is a ratio, as described by P-695, that adjusts the CMR with the Spectral Shape Factor (SSF), This accounts for the differences between the spectral shape of rare ground motions in California with the design spectrum or a uniform hazard spectrum. The probability of collapse is computed at the MCE level using as mean the lognormal of ACMR and as standard deviation the uncertainties (β=0.5) [15].

Figure 13 shows the probability of collapse for all the building models using both strength profiles and for different damping ratios. The Constant profile performs in all cases worse than the ELF profile. This could be counterintuitive if one thinks that constant strength at each story along the building's height would be beneficial. The performance worsens with the Constant strength profile because it is prone to drift concentrations in the first story. On the other hand, the ELF profile distributes yielding along the building's height, improving the performance by delaying the first story yielding, contributing to the modeled collapse.

Another important property is inherent damping and how it influences performance. For instance, the 4-story ELF profile modeled probability of collapse varies from 13.1% with 1% damping to 3.6% with 5% damping and is related to the amount of dissipated energy by the inherent damping compared to the total energy dissipated considering both inherent damping and structural system yielding.

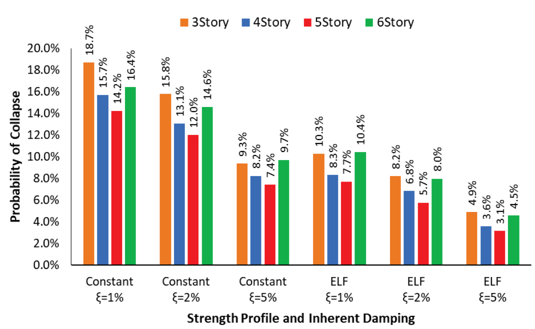

Figure 13 Probability of Collapse Evaluation

9 CONCLUSIONS

This study analyzed different building models ranging in height from 3 to 6 stories. Two different ways these buildings could be designed were considered: a Constant profile where the strength remains constant along the building's height and an ELF profile where a tapered strength from small at the roof to large at the 1st story level is used. In addition, three different values of inherent damping were considered, 1%, 2%, and 5%, proportional to the initial stiffness.

Findings from studies presented herein are summarized as follows:

- The pushover curves showed how the ELF profile improves the displacement capacity and ductility compared to the Constant profile.
- The analyses performed at MCE level earthquakes illustrate the difference in the residual deformations when both profiles are used. Furthermore, evaluating energy dissipated helps explain the extent of concentrated yielding on the first story in the Constant profile.
- A Constant strength profile is more prone to have drift concentrations in the first story resulting in a higher probability of collapse when compared to the ELF strength profile.
- Even though the ELF profile delays drift concentrations in the first story to higher scaling factors compared to the Constant profile, the failure mechanism is still the first story.
- The amount of inherent damping used in analysis directly affects the modeled performance. Therefore, this paper includes data on the amount of energy dissipated based on the damping assigned in the analysis.
- Inherent damping is related to the amount of unknown energy dissipated during an earthquake. However, there is no consensus regarding the amount of energy that should be dissipated by inherent damping. In the case of wood structures, there is likely a wide range given that there is a varied make-up of buildings and multiple components that dissipate energy that is not included in the mathematical model.

10 RECOMMENDATIONS

This study shows the importance of the strength profile on the seismic performance of Light Frame Shear Walls. The trends were the same for all building models from 2- to 6 stories, where the Constant profile performed worse than the ELF profile. In addition, damping was studied using an energy perspective that could help establish a recommended damping ratio value in the future. Even though important conclusions were obtained from this study, the following are recommendations for future research:

- Study profiles vary from the Constant to the ELF profile.
- Investigate profiles that strengthen only the first two stories to move yielding to the upper stories.
- Expand this type of study to other lateral resisting systems.
- Study more in-depth the energy dissipated when tangent stiffness is used instead of initial stiffness.

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11 REFERENCES

[1] Line, P., Charney, F. And J. Linville : Influence of story lateral strength profile on modeled seismic response of wood-frame shear wall buildings. WCTE 2020.

[2] Jayamon, J., Line, P., and Charney, F.: Sensitivity of Wood-Frame Shear Wall Collapse Performance to Variations in Hysteric Model Parameters, Journal of Structural Engineering, 145(1): 04018236, 2019.

[3] Jayamon, J., Charney, F., Flores, F., and Line, P.: Influence of Wall Load-Displacement Shape on Seismic Performance of Wood-frame Shear Wall Structures. WCTE 2016

[4] Jayamon, J., Charney, F., and Line, P.: Evaluation of Damping in Wood-Frame Shear Wall Buildings. WCTE 2016.

[5] Perry, L. Line, P., and Charney, F.: Influence of varying strength, from story to story, on modeled seismic response of wood-frame shear wall structures. WCTE 2018.

[6] Line, P., Charney, F., Linville, J.: Influence of story lateral strength profile on modeled seismic response of wood-frame shear wall buildings. WCTE 2021.

[7] FEMA P695: Quantification of Building Seismic Performance Factors, Federal Emergency Management Agency, Washington, D.C., 2009.

[8] ASCE/SEI 7-22: Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, Reston, VA., 2022.

[9] AWC SDPWS-21: Special Design Provisions for Wind and Seismic, American Wood Council, Leesburg, VA 20175.[9] Folz, B., and Filiatrault, A. A Computer program for Seismic Analysis of Woodframe Structures. CUREE Publication No. W-21, Consortium of Universities for Research in Earthquake Engineering, Richmond, CA, 2001.

[10] Folz, B., & Filiatrault, A. . “Cyclic analysis of wood shear walls.” Journal of Structural Engineering, 127(4), 433-441, 2001.

[11] Folz B., Filiatrault A.: Seismic Analysis of Woodframe Structures I: Model Formulation, Journal of Structural Engineering, 130:1353-1360, 2001.

[12] McKenna, F. and Fenves, G.L.: Open System for Earthquake Engineering Simulation (OpenSEES), Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA., 2017.

[13] Uang, C. M., & Bertero, V. V. (1990). Evaluation of seismic energy in structures. Earthquake engineering & structural dynamics, 19(1), 77-90.

[14] D. Vamvatsikos, C.A. Cornell, Applied incremental dynamic analysis, Earthquake Spectra 20 (2004) 523.

[15] G.G. Deierlein, A.B. Liel, C.B. Haselton, C.A. Kircher, K. Principal, ATC-63 methodology for evaluating seismic collapse safety of archetype buildings, ASCE-SEI Structures Congress, Vancouver, BC 2008, pp. 24–26.

https://doi.org/10.52202/069179-0363