

# New Developments in FEMA P-58 Seismic Risk Assessment of Wood Light-Frame Buildings

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## Abstract

This study demonstrates the feasibility of advanced performance assessments of wood light-frame buildings, using the FEMA P-58 method. Accounting for stiffness and strength contributions from nonstructural walls and wall finishes in the building seismic resistance is shown to be an essential aspect of the performance assessments, particularly for shorter buildings (*i.e.*  $\leq 2$  stories). Additionally, the results suggest that the seismic resistance of these buildings can be estimated from component quantities, rather than by dynamic analysis of a robust nonlinear structural model. This newly-enabled use of FEMA P-58 for this building type will allow the prediction of seismic losses (which enables uses for both mortgage due-diligence applications and insurance company risk assessments), prediction of repair times, and the ability to complete a building seismic rating for wood light-frame buildings. This study utilizes advanced structural analysis methods, from the ongoing ATC-116 Project, as part of the performance assessment, but it also outlines near-term future work that will streamline the analysis process by not requiring any structural modeling, thereby reducing the assessment time to the order of hours rather than days.

## Introduction

There is a multifaceted need to better understand the risks associated with wood light-frame (WLF) buildings. These buildings make up a large portion of the built environment and, therefore, are likely to compose a significant portion of the overall seismic risk to communities, to organizations holding

mortgages in seismic areas, and to both insurance and reinsurance companies. WLF structures are also the largest asset for many WLF building owners (*e.g.* homeowners), so the financial risk posed to their owners is substantial and may not be softened by a large and diverse asset portfolio (the majority of homeowners do not carry earthquake insurance). For all of these reasons, a more rigorous and accurate process is needed for assessing the seismic risk posed by WLF buildings.

Since it is well known that real WLF buildings perform better than many past analytical assessments have shown, risk assessments for WLF buildings are commonly based on judgment and past experience, and tend to be generic in nature, *e.g.* HAZUS vulnerability functions (FEMA 2003). Advanced risk assessments of WLF buildings have been challenging in the past, because their seismic response depends heavily on nonstructural walls and wall finishes. Filiatrault et al. (2010) conducted shake-table experiments for a WLF building with and without nonstructural finishes and found that inclusion of gypsum wallboard and stucco (as in a real building) substantially reduces drift demands and improves the seismic performance of the building. Therefore, structural response calculations for WLF buildings that are used to conduct risk assessments must include the effects of nonstructural components and finishes; otherwise, the probability of collapse and drift demands will be over-estimated, and the risk assessment will be too conservative as a result.

Recent advancements for modeling WLF structures, particularly the ongoing FEMA sponsored ATC-116 Project

(Kircher et al., 2016), have provided the necessary resources to model nonstructural walls and finishes in WLF buildings effectively. This current study aims to leverage the work by the ATC-116 Project team, and to then compliment that work, with this current study having the following three specific goals:

- 1) Demonstrate the feasibility of advanced seismic risk assessments, using the FEMA P-58 method, for WLF structures.
- 2) Lay the initial ground work for developing advanced FEMA P-58 vulnerability functions (*i.e.* loss curves) for WLF buildings, which can be tailored to the individual building-specific characteristics of those buildings.
- 3) Lay the ground work for creating a method to statistically predict structural responses, based on these key building characteristics, using a large database of detailed structural models and nonlinear dynamic response-history analyses. This would enable users to conduct the FEMA P-58 risk assessment of WLF buildings without needing to develop a structural model.

## Methodology

Seismic risk assessments for five WLF buildings are performed using nonlinear dynamic structural analyses conducted with OpenSees (PEER 2016). The FEMA P-58 damage and loss assessment is completed using the Seismic Performance Prediction Program (SP3 2016); SP3 is a web-based software platform that employs the FEMA P-58 methodology (FEMA 2012). The FEMA P-58 method uses Monte Carlo simulation techniques to integrate site hazard, structural responses of the building, inventory of building components, and component fragilities to quantify seismic risks.

The buildings range from one to four stories, and each is representative of a common form or residential construction. Two versions of each building are analyzed - one that approximates the seismic resistance based on the lateral strength and stiffness of the whole building, including structural and nonstructural walls and wall finishes, and another that approximates the seismic resistance based only on the shear wall panels that are part of the design lateral force resisting system (LFRS).

The seismic risk assessments are compared to each other. In addition, they are compared to existing vulnerability functions for WLF buildings that are based on judgement, experience, and observations from past earthquakes (specifically with data from the 1994 Northridge earthquake).

## Designs

Five archetype designs are developed for this study. Each design represents a common form of residential WLF construction:

- 1) One-story single-family dwelling (Single-1).
- 2) Two-story single-family dwelling (Single-2).
- 3) One-story multi-family dwelling (Multi-1).
- 4) Two-story multi-family dwelling (Multi-2)
- 5) Four-story multi-family dwelling (Multi-4)

The Single-1 building has a 2,000 sq. ft. footprint with an aspect ratio of approximately 1.5. The Single-2 building has a square footprint of approximately a 1,000 sq. ft. Both single family dwellings utilize only exterior walls for the design LFRS.

The Multi-1 building is 56 ft. x 128 ft. (7,168 sq. ft.), housing eight units with a central corridor in the long direction of the building. The Multi-2 building represents a typical two-story, three-unit row house. It is 36 ft. x 72 ft. (2,592 sq. ft. footprint). The one and two story multi-family dwellings utilize only exterior walls for the design LFRS in the long direction. Exterior walls and some party walls are utilized for the LFRS in the short direction.

The Multi-4 building represents a typical four-story apartment building. The building dimensions are 56 ft. x 96 ft. It houses six units per floor and has a central corridor in the long direction. Exterior walls and corridor walls are utilized for the design LFRS in the long direction. Exterior walls, party walls, and interior bearing walls are utilized for the design LFRS in the short direction.

Building weights for proportioning their lateral systems are given in Table 1.

Table 1. Summary of building weights.

Building	Design seismic weight each floor [k] <sup>1</sup>	Design seismic weight roof [k] <sup>1</sup>
Single-1	NA	68.7
Single-2	46.7	35.3
Multi-1	NA	218.6
Multi-2	102.4	81.0
Multi-4	276.9	148.6

<sup>1</sup>Seismic weight and dead load are equal for the WLF archetype buildings in this study.

The lateral system of each building is wood oriented strand board (OSB) shear walls. Design strengths of the shear walls are determined by the ANSI/AWC National Design Specification for Wood Construction (AWC, 2012), and the ANSI/AWC Special Design Provisions for Wind and Seismic (AWC, 2008). The shear walls are proportioned according to

ASCE/SEI 7-10 (ASCE 2010) for a design short period spectral acceleration  $SDS=1.0g$ , response modification factor  $R=6.5$ , and importance factor of 1.0; the resulting design base shear coefficient is  $C_S=0.154g$  for all of the test buildings.

Descriptions and quantities of structural and nonstructural walls and finishes are documented in Table 2 through Table 6. Only full height wall segments approximately three feet long or longer are reported in the summary tables, because the contribution of shorter walls to the lateral seismic resistance is not considered. First story wall quantities are reported for each building, because nonlinear seismic responses of the WLF buildings tend to concentrate in the bottom story.

Table 2. Single-1: Total length of full height wall segments with each wall finish (nonstructural finishes italicized).

Finish	Direction 1	Direction 2
7/16" OSB, 8d nails @ 6" O.C.	32 ft.	32 ft.
<i>OSB-Nonstruc</i>	<i>14 ft.</i>	<i>36 ft.</i>
<i>Nonstruc Gyp (on a wall with OSB)</i>	<i>46 ft.</i>	<i>68 ft.</i>
<i>Nonstruc Gyp (not combined with OSB)</i>	<i>110 ft.</i>	<i>100 ft.</i>
<i>Stucco (on a wall with OSB)</i>	<i>46 ft.</i>	<i>68 ft.</i>

Table 3. Single-2: Total length of full height wall segments with each wall finish in the first story (nonstructural finishes italicized).

Finish	Direction 1	Direction 2
7/16" OSB, 8d nails @ 6" O.C.	32 ft.	32 ft.
<i>OSB-Nonstruc</i>	<i>8 ft.</i>	<i>8 ft.</i>
<i>Nonstruc Gyp (on a wall with OSB)</i>	<i>40 ft.</i>	<i>40 ft.</i>
<i>Nonstruc Gyp (not combined with OSB)</i>	<i>64 ft.</i>	<i>64 ft.</i>
<i>Stucco (on a wall with OSB)</i>	<i>40 ft.</i>	<i>40 ft.</i>

Table 4. Multi-1: Total length of full height wall segments with each wall finish in the first story (nonstructural finishes italicized).

Finish	Direction 1	Direction 2
7/16" OSB, 8d nails @ 6" O.C.	88 ft.	0 ft.
7/16" OSB, 8d nails @ 3" O.C.	0 ft.	50 ft.
<i>OSB-Nonstruc</i>	<i>56 ft.</i>	<i>66 ft.</i>
<i>Nonstruc Gyp (on a wall with OSB)</i>	<i>150 ft.</i>	<i>112 ft.</i>
<i>Nonstruc Gyp (not combined with OSB)</i>	<i>592 ft.</i>	<i>592 ft.</i>
<i>Stucco (on a wall with OSB)</i>	<i>78 ft.</i>	<i>116 ft.</i>

Table 5. Multi-2: Total length of each wall finish along full-height wall segments in the first story (nonstructural finishes italicized).

Finish	Direction 1	Direction 2
7/16" OSB, 8d nails @ 6" O.C.	72 ft.	16 ft.
7/16" OSB, 8d nails @ 3" O.C.	0 ft.	32 ft.
<i>OSB-Nonstruc</i>	<i>8 ft.</i>	<i>36 ft.</i>
<i>Nonstruc Gyp (on a wall with OSB)</i>	<i>84 ft.</i>	<i>84 ft.</i>
<i>Nonstruc Gyp (not combined with OSB)</i>	<i>226 ft.</i>	<i>144 ft.</i>
<i>Stucco (on a wall with OSB)</i>	<i>40 ft.</i>	<i>84 ft.</i>

Table 6. Multi-4: Total length of each wall finish along full-height wall segments in the first story (nonstructural finishes italicized).

Finish	Direction 1	Direction 2
7/16" OSB, 8d nails @ 6" O.C.	144 ft.	48 ft.
7/16" OSB, 8d nails @ 3" O.C.	128 ft.	0 ft.
19/32" OSB, 8d nails @ 2" O.C.	0 ft.	104 ft.
<i>OSB-Nonstruc</i>	<i>32 ft.</i>	<i>36 ft.</i>
<i>Nonstruc Gyp (on a wall with OSB)</i>	<i>372 ft.</i>	<i>188 ft.</i>
<i>Nonstruc Gyp (not combined with OSB)</i>	<i>184 ft.</i>	<i>348 ft.</i>
<i>Stucco (on a wall with OSB)</i>	<i>64 ft.</i>	<i>84 ft.</i>

## Structural Modeling and Analysis

The WLF buildings are modeled in OpenSEES (PEER 2016). Structural models include stiffness and strength contributions from all walls and wall segments that are approximately over three feet long. Wall finishes for determining the lateral strength of the buildings are those listed in the section that describes the building designs - OSB, gypsum wallboard, and stucco.

Backbones for each of the different wall finishes are based on the ongoing FEMA sponsored ATC-116 Project (Kircher et al., 2016). Masses and gravity loads in the structural models are based on the design dead and live loads, but modified to approximate their expected values according to the modeling recommendations in FEMA P695 (FEMA 2009). Floor and roof masses are modeled as 1.05 times the design seismic weight. P-Delta loads are 1.05 times the design dead load, plus an expected live load of 10 psf on the floors (*e.g.* 25% of a 40 psf design live load).

Each building is analyzed two ways: 1) considering all components (structural and nonstructural) that contribute to lateral strength of the buildings; 2) considering lateral strength of the designed LFRS only (referred to hereafter as the “bare structure”). The bare structure model is consistent with structural modeling procedures that are commonly used in seismic risk assessments for other building systems, such as reinforced concrete shear wall systems, for which the contribution of nonstructural components to the overall seismic resistance is relatively small. This study considers structural modeling techniques with and without nonstructural walls and finishes included, so that their impact on the overall risk assessment for WLF buildings can be measured.

A single, two-dimensional model of each building (ten total) is used to approximate its seismic response. A full FEMA P-58 analysis would require the two orthogonal directions of a building to be analyzed separately. For the purpose of this pilot study, however, structural analysis is performed for only one orthogonal direction, and seismic resistance (*i.e.* strength, stiffness, etc.) is assumed to be identical in both directions. This assumption is accurate for the bare structure (*i.e.* just the walls that are part of the design LFRS) but is a slight simplification of the behavior of the full building (including non-structural components). For cases where the quantities of nonstructural walls and finishes differ significantly for the two orthogonal directions, the average of those quantities for the two directions is used to create the structural model. Changes in the peak strength due to this simplification are generally small (less than 10%).

Nonstructural walls and wall finishes significantly impact the lateral response, particularly for the single-story buildings, as shown in Figure 1, Table 7, and Table 8.

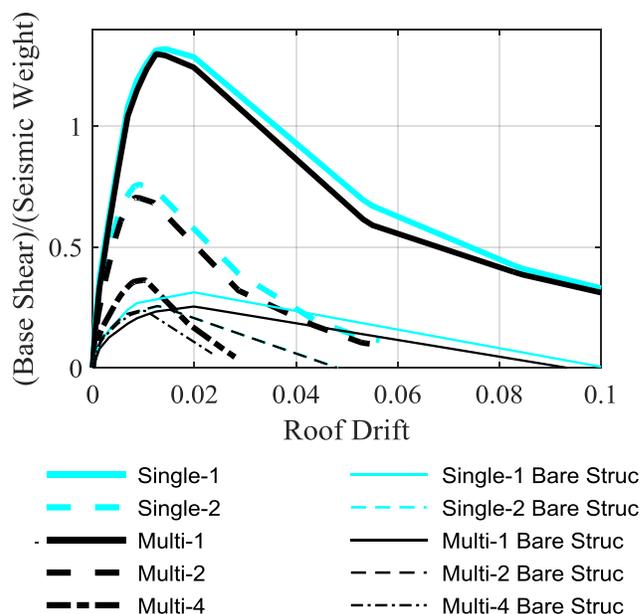


Figure 1. Static pushover plots for the test buildings, normalized by seismic weight. Bare structure models (*i.e.* only the design LFRS is modeled) are plotted with thinner line weights, but with the same colors and line styles as the corresponding full models. The Single-2 bare structure model and Multi-2 bare structure model have nearly identical response, and are therefore difficult to distinguish.

Table 7. Summary of key building properties.

Building	$T_{1,Model}$ [s]	${}^1\Omega_{Model}$	Ratio of $\Omega$
Single-1	0.19	8.6	4.2
Single-1, Bare Struc	0.43	2.0	
Single-2	0.27	4.9	3.0
Single-2, Bare Struc	0.55	1.6	
Multi-1	0.19	8.4	5.1
Multi-1, Bare Struc	0.45	1.6	
Multi-2	0.28	4.6	2.8
Multi-2, Bare Struc	0.48	1.7	
Multi-4	0.50	2.4	1.5
Multi-4, Bare Struc	0.68	1.5	

<sup>1</sup>Overstrength ( $\Omega$ ), based on  $C_S=0.154$  g

Collapse resistance for each building is determined from Incremental Dynamic Analysis (IDA, Vamvatsikos and Cornell, 2002), using the FEMA P-695 far-field ground motion set. The set consists of 22 ground motion time histories, each

with two orthogonal components (44 ground motion histories total). To be consistent with the hazard model that is used in the FEMA P-58 risk assessment, ground motion intensity is measured as the geometric mean spectral acceleration ( $S_a$ ) of the pair of ground motion time history components.

The intensity causing collapse for a given ground motion is the lowest geometric mean  $S_a$  for which one (not both) of the ground motion components cause the building to collapse. This method approximates the collapse resistance of the three-dimensional building with both ground motion components applied simultaneously (as in a real earthquake), assuming that torsional response is minimal and seismic resistance is similar in both orthogonal directions of the building. This approach differs slightly from the collapse capacity prediction approach of FEMA P-695, which uses collapse capacity for a single direction.

The median intensity causing collapse that is used for the risk assessment is also adjusted by a spectral shape factor (per FEMA 2009). The spectral shape factor accounts for differences in spectral shape of the FEMA far-field ground motions vs. the spectral shape of ground motions that are particularly hazardous to the buildings, *i.e.* those with large positive epsilon (Baker and Cornell 2005). Increases in collapse resistance due to spectral shape range from 26% to 34% for the buildings considered. The spectral shape adjustment could be circumvented by selecting custom ground motion sets for each ground motion intensity whose characteristics, *e.g.* epsilon, are targeted to the disaggregated site hazard (Baker and Cornell 2006, Lin et al. 2013), but that approach is not taken in this study.

The median collapse capacity for each building is reported in Table 8. The record-to-record logarithmic standard deviation of the collapse capacity is approximately 0.4 for all of the buildings. However, a total dispersion of 0.5 is assumed for the FEMA P-58 analyses to account for additional sources of variance not considered in the structural models, such as variability of construction quality.

Engineering demand parameters (floor accelerations and interstory drifts) for a wide range of ground motion intensities are also determined from the IDA results. The engineering demand parameters (EDPs) are not modified to account for spectral shape effects. As a result, EDPs may be slightly overestimated for high intensity events and slightly underestimated for low intensity events. These potential biases could be eliminated by selecting site and hazard specific ground motions (*e.g.* Lin et al. 2013). As with collapse resistance, additional dispersion is added to EDP demands to account for sources of variability that are not explicitly represented in the structural analyses (FEMA P-58).

Table 8. Summary of collapse resistance.

Building	Median $S_a$ Causing Collapse [g]	Increase from Nonstructural
Single-1	$S_{aT=0.19s} = 4.32$	170%
Single-1, Bare Struc	$S_{aT=0.19s} = 1.60$	
Single-2	$S_{aT=0.27s} = 2.81$	98%
Single-2, Bare Struc	$S_{aT=0.27s} = 1.42$	
Multi-1	$S_{aT=0.19s} = 4.41$	186%
Multi-1, Bare Struc	$S_{aT=0.19s} = 1.54$	
Multi-2	$S_{aT=0.28s} = 2.86$	87%
Multi-2, Bare Struc	$S_{aT=0.28s} = 1.53$	
Multi-4	$S_{aT=0.50s} = 2.11$	18%
Multi-4, Bare Struc	$S_{aT=0.50s} = 1.79$	

### FEMA P-58 Modeling and Analysis

The risk assessments are performed using the Seismic Performance Prediction Program (SP3) which executes the FEMA P-58 methodology through a web-based interface. The WLF building models are assessed at a Seismic Design Category D site in Los Angeles (at 33.996°N and 118.160°W). Figure 3 shows the ground motion hazard for the building site, which comes from the U.S. Geological Survey (USGS 2014).

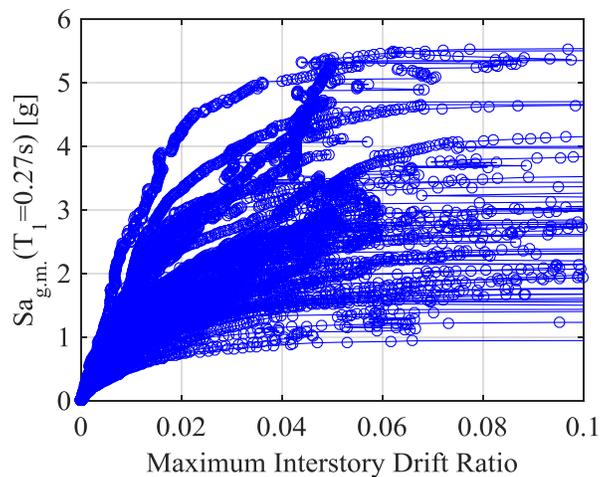


Figure 2. Incremental dynamic analysis results for the Single-2 building.

The FEMA P-58 models of the buildings include component fragility representations of all structural and nonstructural walls that are part of the building designs. Additional components, such as HVAC and plumbing, were populated in

the models using normative quantities for similar residential structures. Losses due to collapse and due to peak drifts and floor accelerations are considered in the assessments. Losses due to residual drift are not considered in the present study, but will be looked at more closely in future studies.

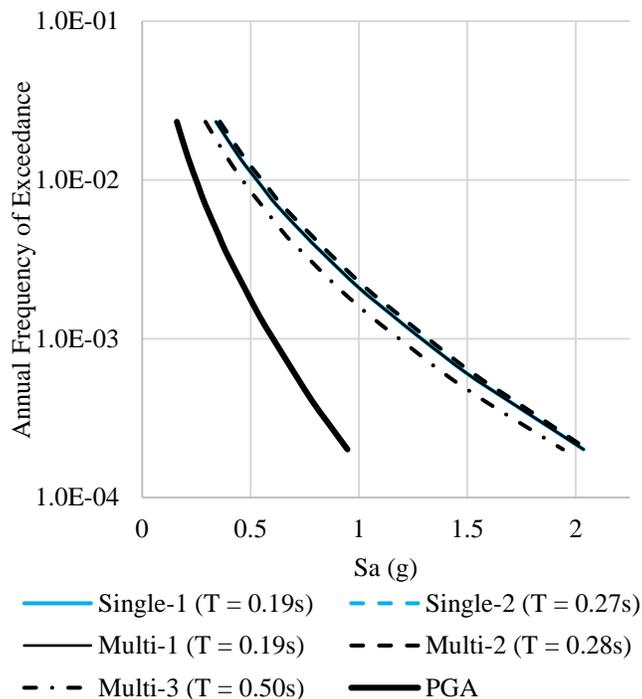


Figure 3. Site Hazard used for all building models.

### Risk Assessment Results

Seismic risk assessments are performed for each of the buildings, and selected results are presented in this section to illustrate important trends. Loss estimations are presented as percent loss, *i.e.* the building loss normalized by the replacement cost (also commonly referred to as a damage ratio, *DR*).

Figure 4 shows expected losses vs. hazard for the five buildings studied. At low hazard levels, *i.e.* 50%-10% probability of exceedance in 50 years, the expected losses are on the order of 10% of the replacement cost for all of the test buildings. The one-story and four-story buildings have slightly lower expected losses (relative to their replacement costs) than the two-story buildings. The one-story buildings have lower losses because they experience very little damage at low intensities due to their high stiffness and strength. The four-story buildings suffer very little damage in their upper stories, so the relative magnitude of repair costs to replacement cost is lower.

For rarer ground motion intensities, *i.e.* having less than 10% probability of exceedance in 50 years, the expected losses for the buildings diverge, revealing a trend in height. The taller buildings perform worse when subjected to rare strong ground motions. Trends in expected loss vs. height for the test buildings are shown in Figure 5. The decrease in performance coinciding with an increase in height is attributed to decreasing overstrength and collapse resistance with increasing height (see Table 7 and Table 8). If only the bare structure version of the buildings were analyzed, there would be no significant difference in overstrength between the building variants (refer to Table 7) and these trends would not be observed.

The impact of including stiffness and strength contributions from nonstructural walls and components is illustrated for two specific buildings: Single-1 (Figure 6) and Multi-4 (Figure 7). Omitting stiffness and strength contributions from nonstructural walls and finishes results in conservative loss estimates. For the one-story buildings, such as the Single-1 building, risk assessments that do not consider stiffness and strength contributions from nonstructural walls and finishes are grossly inaccurate. For the four-story building, however, the difference is less substantial and may be acceptable for risk assessments where a moderate level of conservatism is tolerable.

To examine the impact of considering strength and stiffness contributions from nonstructural walls and finishes on overall seismic risk assessments for all of the buildings, expected annual losses (*i.e.* average building loss per year) and expected losses for a design basis event (*i.e.* an event with approximately 10% probability of exceedance in 50 years) are reported in Table 9. Expected annual loss is computed by integrating the expected loss curves with the site hazard. The added seismic resistance from nonstructural walls and finishes considerably impacts the risk. Figure 8 shows that the change in loss due to additional seismic resistance from nonstructural walls and finishes is inversely related to the added resistance (in terms of strength) for the test buildings.

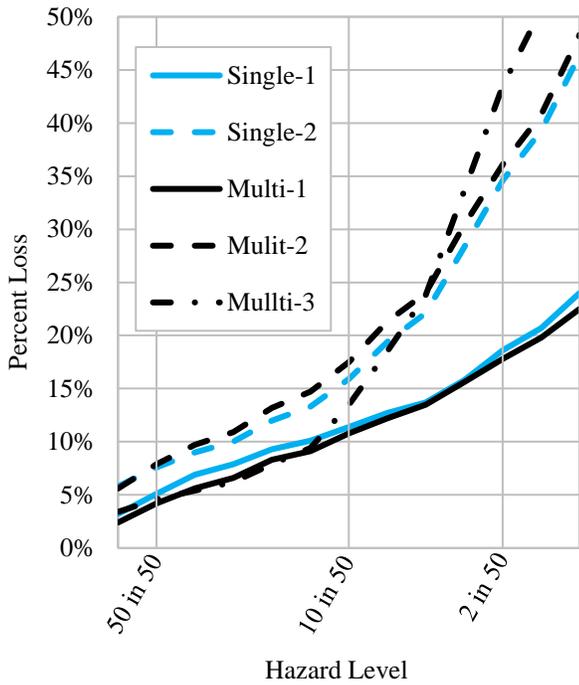


Figure 4. Expected building loss vs. hazard. Hazard level is expressed as percent probability of being exceeded in 50 years.

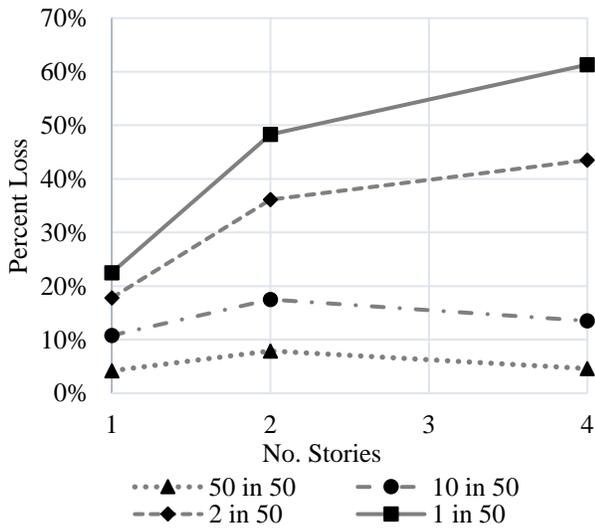


Figure 5. Expected building loss vs. number of stories for the multi-family dwellings at four different levels of seismic hazard.

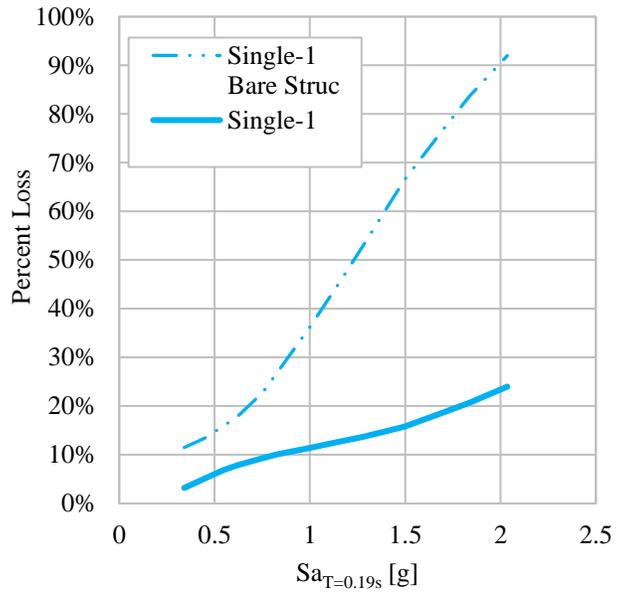


Figure 6. Expected losses for the one-story single family home (Single-1) vs. ground shaking intensity.

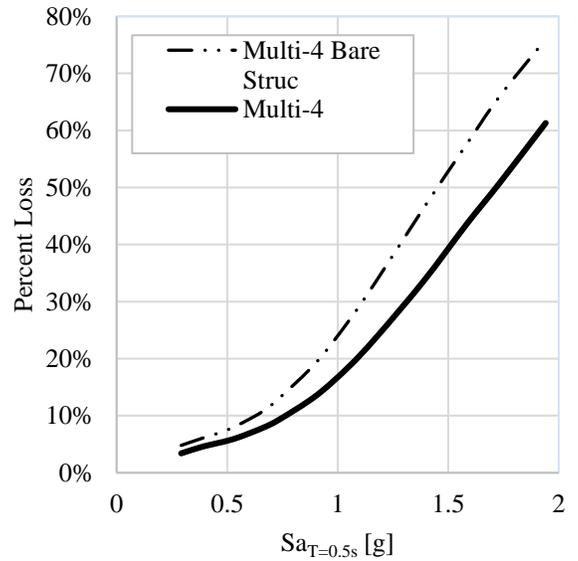


Figure 7. Expected losses for the four-story apartment building (Multi-4) vs. ground shaking intensity.

Table 9. Expected losses for each building.

Building Model	Expected Loss (as a fraction of replacement cost)	
	Annualized	Design Basis Event
Single-1	0.17%	11%
Single-1, Bare Struc	0.51%	36%
Single-2	0.26%	16%
Single-2, Bare Struc	0.60%	46%
Multi-1	0.15%	11%
Multi-1, Bare Struc	0.50%	37%
Multi-2	0.27%	18%
Multi-2, Bare Struc	0.53%	41%
Multi-4	0.19%	14%
Multi-4, Bare Struc	0.26%	19%

component categories for two of the structures. The component category “Exterior Walls” includes damage to the exterior wall cladding (stucco) and damage to the structural portions of the walls (studs and OSB paneling). The “Other Nonstructural” category includes components such as the HVAC, piping, and electrical systems. Since stucco has a tendency to crack and damage at low levels of drift, repair to the stucco on the exterior walls drives much of the estimated financial loss of these buildings, particularly for low to medium intensity events.

It can also be observed that collapse potential impacts the seismic risks of the Single-1 building (Figure 9) much less than the Multi-4 building (Figure 10). That is because the collapse resistance of the Single-1 building is significantly increased by nonstructural walls and finishes (see Table 8). If nonstructural walls and wall finishes are not considered in the structural analysis, then collapse dominates the Single-1 risk assessment.

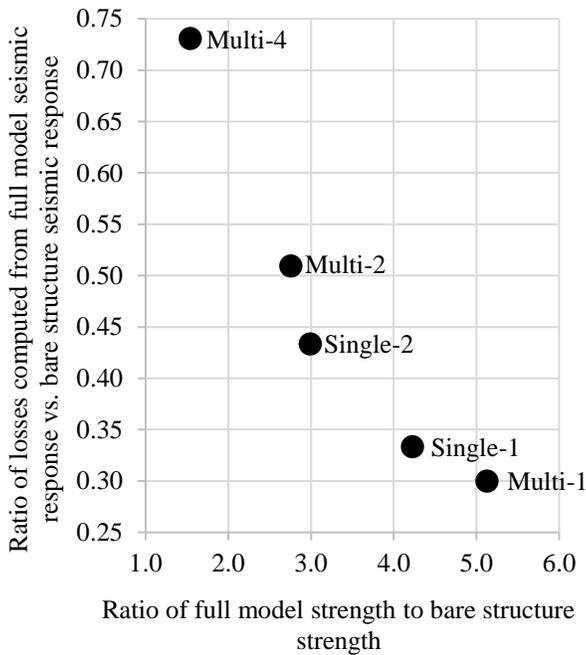


Figure 8. Inverse relationship between relative changes in expected annual loss and relative changes in strength from considering nonstructural walls and finishes.

A unique feature of building specific risk assessment methods like FEMA P-58 is the ability to observe the individual building components’ contributions to the overall seismic risk and estimated losses. Figure 9 and Figure 10 show breakdowns of the estimated financial loss by important

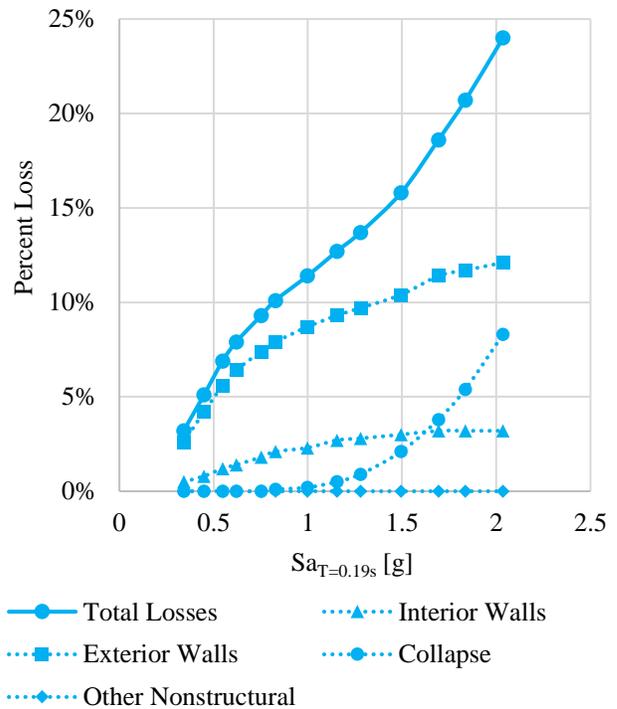


Figure 9. Single-1 disaggregated seismic risk by component type.

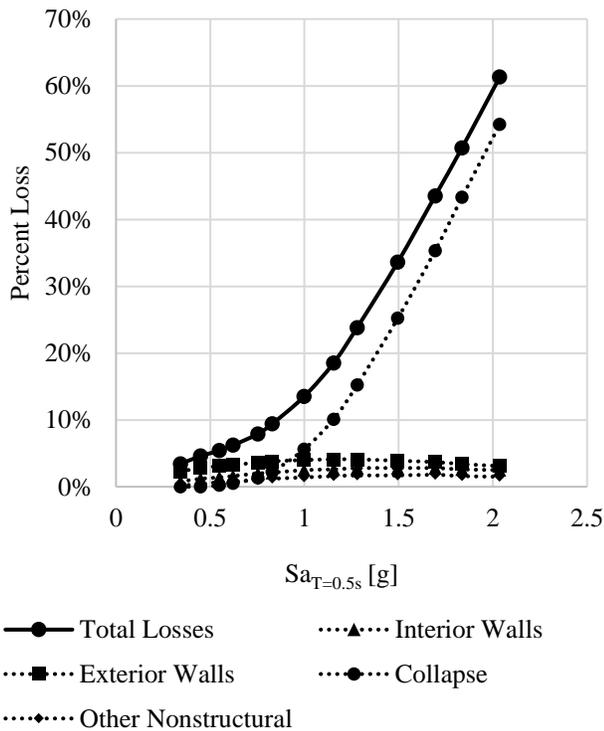


Figure 10. Multi-4 disaggregated seismic risk by component type.

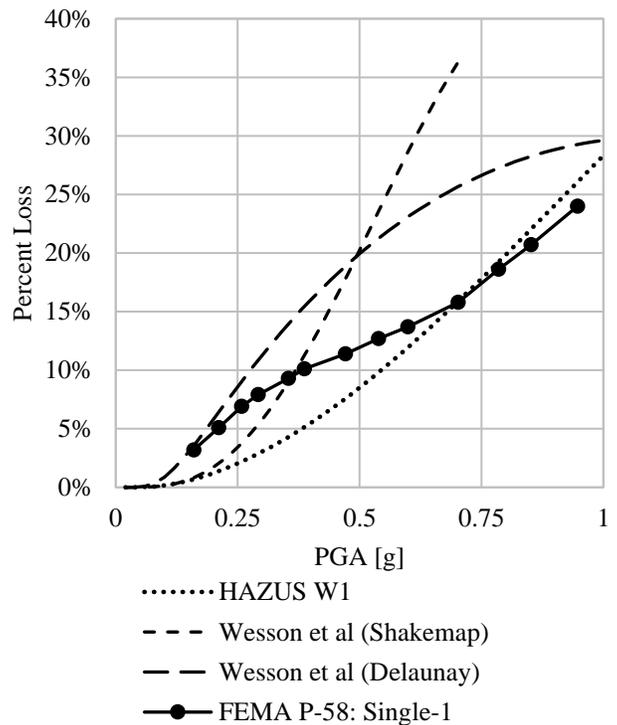


Figure 11. Vulnerability of single-family one-story buildings, computed from different sources.

Figure 11 shows the results for the Single-1 building benchmarked against previous risk assessments of single-story WLF structures using other methodologies. It can be seen that the P-58 results give comparative yet distinctly unique predictions of loss when compared with both HAZUS predictions of a similar structure and predictions calibrated to data from Northridge Earthquake observations. The HAZUS W1 curve represents the estimated loss for single-story wood structures that are less than 5,000 square feet (FEMA 2003). The two Wesson curves are derived from conditional probability density functions (CPDF) that predict loss based on PGA. The two curves represent two ways of determining the site PGA, shake map vs. the Delaunay triangulation interpolation scheme. Both CPDF curves are reported to be calibrated to the insurance loss data from the 1994 Northridge Earthquake (Wesson et al. 2004) but the Delaunay ignores certain site conditions that are known to occur through the region of the study. Therefore the shake map CPDF is considered the more trusted source. It is clear from Figure 12 that the spread between the traditional predictive curves (HAZUS and Wesson) is still very wide (*e.g.* losses between the two curves range from 8% to 20% for a PGA of 0.5g) and there is much uncertainty in the traditional processes for seismic risk evaluations (*i.e.* methodologies preceding FEMA P-58).

### Future Directions

This study validates the feasibility of FEMA P-58 risk assessments for wood light-frame buildings and shows that including contributions to strength and stiffness from nonstructural walls and finishes is an essential step for completing a meaningful FEMA P-58 risk assessment. However, “feasible” does not necessarily mean “practical” or “simple.” For example, developing a nonlinear structural model that includes stiffness and strength contributions from nonstructural walls and finishes and performing nonlinear dynamic response-history analyses is impractical for nearly all potential users. Our next research steps focus on making rigorous FEMA P-58 seismic risk assessments of wood light-frame buildings feasible for all potential users, where an assessment could be completed in a matter of hours.

Based on the findings presented in this study, we expect that robust FEMA P-58 risk assessments of WLF buildings can be performed without needing to create a structural model. Instead, the structural responses (*e.g.* drifts and floor accelerations), as well as the collapse resistance, could be predicted using statistical predictive methods derived from our database of wood light-frame nonlinear time-history analyses (where the future method would essentially “do the structural analysis for you”). This streamlined process will involve first

estimating the key seismic response characteristics of the building (e.g. strength, stiffness, pushover ductility capacity, etc.) by using the inventory of building components (structural and nonstructural) and the database of known strengths, stiffnesses, and nonlinear characteristics of each component; this inventory could be based either on the actual inventory of the building or typical “normative” quantities based on building occupancy and age. Structural responses will then be statistically predicted based on these key building characteristics, based on a large database of detailed structural models and nonlinear dynamic response-history analyses.

This substantially streamlined analysis method is expected to be completed in the fall/winter of 2016 (with a status update provided in the oral presentation of this work) and will enable users to conduct the FEMA P-58 risk assessment of wood light-frame buildings without developing a structural model. We anticipate that this will help fill the need for more robust seismic risk assessments of wood light-frame buildings for mortgage/transactional due-diligence analyses, insurance risk assessments, analyses to support more resilient design of new wood light-frame buildings, and possibly retrofit design of soft-story buildings (with some possible further extensions needed for retrofit applications).

## Conclusions

This study includes robust FEMA P-58 risk assessments of five wood light-frame (WLF) buildings. Though the focus of this study is FEMA P-58 risk assessment, and not structural modeling, an important aspect of this study is the inclusion of stiffness and strength contributions from nonstructural walls and wall finishes in the structural models of the buildings, and this was done using component backbone modeling curves similar to those from the ongoing FEMA sponsored ATC-116 Project (Kircher et al., 2016). The result of the FEMA P-58 risk analyses in this paper demonstrate that:

- 1) FEMA P-58 loss predictions for WLF buildings are comparable to other models that are based on past experience, judgement, and observed losses in historical earthquakes. However, an advantage of the FEMA P-58 method is that it provides building-specific risk predictions rather than a less specific and more uncertain prediction for a broad class of buildings (i.e. it allows building-specific nuances to be fully incorporated into the risk assessment). An example of this is shown in Figure 12 where experience-based methods predict a range of loss from 8% to 20% for a peak ground acceleration of 0.5g, whereas the FEMA P-58 approach predicts a specific mean loss of 12% (where this prediction still has uncertainty, but whose uncertainty is less than other experience of judgment-based methods).

- 2) The FEMA P-58 loss assessment of wood light-frame buildings is highly dependent on accounting for the effects of the nonstructural walls and wall finishes on a building’s response, particularly for shorter buildings.
- 3) The FEMA P-58 loss assessment shows that multi-story WLF buildings pose greater seismic risks than single-story WLF buildings. It is notable that this important trend is only observable when nonstructural walls and finishes are included in the structural modeling (i.e. using a shear wall only model would not expose this trend).
- 4) The FEMA P-58 methodology specifically accounts for collapse probability in the assessment and collapse plays a more significant role in the seismic losses of multi-story WLF buildings as compared with single-story WLF buildings. Like conclusion #3, this is only observable when nonstructural walls and finishes are included in the structural modeling.

In addition to demonstrating the feasibility of robust FEMA P-58 risk assessments of wood light-frame buildings, the future work section outlines the vision for making these robust FEMA P-58 seismic risk assessments possible without the user needing to create a structural model. This would make completing the analysis feasible in a matter of hours instead of days, thereby making the method accessible to a broad range of potential users.

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## References

- ASCE (2010). *Minimum Design Loads for Buildings and Other Structures*, ASCE Standard ASCE/SEI 7-10, American Society of Civil Engineers, Reston, VA.
- AWC (2008). *Special Design Provisions for Wind and Seismic*, ANSI/AF&PA SDPWS-2008, American Wood Council, Leesburg, VA.
- AWC (2012). *National Design Specification (NDS) for Wood Construction*, ANSI/AWC NDS-2012, American Wood Council, Leesburg, VA.

Baker, J.W., and Cornell, C.A. (2005). A vector-valued ground motion intensity measure consisting of spectral acceleration and epsilon, *Earthquake Engineering and Structural Dynamics*, 34(10) 1193–1217.

Baker, J. W., and Cornell, A.C. (2006). Spectral shape, epsilon and record selection. *Earthquake Engineering & Structural Dynamics*, 35(9), 1077-1095.

FEMA (2003). HAZUS-MH MR3 Tech. Manual, Wash., DC.

FEMA (2012). Seismic performance assessment of buildings (FEMA P-58). Prepared by ATC for FEMA.

Kircher et al. (2016). “Solutions to the Short-Period Building Performance Paradox: A Focus on Light-Frame Wood Buildings,” Proceedings of the Structural Engineers Association Conference, October 12-15, 2016, Maui, Hawaii.

Lin, T., Haselton, C. B., & Baker, J. W. (2013). Conditional spectrum-based ground motion selection. Part I: Hazard consistency for risk-based assessments. *Earthquake engineering & structural dynamics*, 42(12), 1847-1865.

PEER (2016). OpenSEES (Open System for Earthquake Engineering Simulation). *opensees.berkeley.edu*.

Seismic Performance Prediction Program (SP3) (2016). © Haselton Baker Risk Group, *www.hbrisk.com*.

USGS (2014). U.S. Geological Survey (USGS), *Hazard Curve Application*. US Department of the Interior. <http://geohazards.usgs.gov/hazardtool/application.php>

Wesson, R. L., Perkins, D. M., Leyendecker, E. V., Roth, R. J., & Petersen, M. D. (2004). Losses to single-family housing from ground motions in the 1994 Northridge, California, earthquake. *Earthquake Spectra*, 20(3), 1021-1045.