



Eleventh U.S. National Conference on Earthquake Engineering
Integrating Science, Engineering & Policy
June 25-29, 2018
Los Angeles, California

NEW DEVELOPMENTS FOR RAPID SEISMIC RISK ASSESSMENT OF WOOD LIGHT-FRAME BUILDINGS

D.J. DeBock¹, D.T. Cook², C.B. Haselton³, K.F. Wade⁴

ABSTRACT

Performance assessments of wood light-frame buildings are complicated by the fact that the “non-structural” components of the building have substantial participation in the buildings seismic response. This makes the application of the FEMA P-58 seismic risk assessment method more difficult without the user creating a detailed structural analysis model of the wood light-frame building (which is impractical). This paper discusses the recent technical developments to enable the more streamlined and practical use of the FEMA P-58 methodology for wood light-frame buildings. This newly-enabled use of FEMA P-58 for this building type will allow the prediction of seismic losses (which enables use for due-diligence applications), prediction of repair times, and the ability to complete a building seismic rating for this building type.

¹ Assistant Professor, Dept. of Civil Engineering, CSU Chico, Chico, CA 95929 (email: ddebock@csuchico.edu)

² PhD Candidate, University of Colorado at Boulder, Boulder, CO 80309 and Research Engineer, Haselton Baker Risk Group (email: dustin@hbrisk.com)

³ CEO, Haselton Baker Risk Group, Chico, CA, 95928 and Professor, Dept. of Civil Engineering, CSU Chico, Chico, CA 95929 (email: curt@hbrisk.com)

⁴ Research Engineer, Haselton Baker Risk Group, Chico, CA, 95928 (email: katie@hbrisk.com)

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Performance assessments of wood light-frame buildings are complicated by the fact that the “non-structural” components of the building have substantial participation in the buildings seismic response. This makes the application of the FEMA P-58 seismic risk assessment method more difficult without the user creating a detailed structural analysis model of the wood light-frame building (which is impractical). This paper discusses the recent technical developments to enable the more streamlined and practical use of the FEMA P-58 methodology for wood light-frame buildings. This newly-enabled use of FEMA P-58 for this building type will allow the prediction of seismic losses (which enables use for due-diligence applications), prediction of repair times, and the ability to complete a building seismic rating for this building type.

Introduction

Wood light-frame (WLF) buildings make up a large portion our building stock, therefore a rigorous and accurate process for assessing the seismic risk posed by WLF buildings is important. However, it is well know that real WLF buildings perform better than many past analytical assessments have shown. As a result, 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 [1]. Advanced risk assessments of WLF buildings have been challenging in the past, because their seismic response depends heavily on nonstructural walls and wall finishes, e.g. [2] has shown that inclusion of gypsum wallboard and stucco (as in a real building) substantially increases seismic resistance of WLF buildings. 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 structural response will be miss-estimated, and the risk assessment will be too conservative as a result.

¹ Assistant Professor, Dept. of Civil Engineering, CSU Chico, Chico, CA 95929 (email: ddebock@csuchico.edu)

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Recent advancements for modeling WLF structures, particularly the FEMA sponsored ATC-116 Project [2], have provided the necessary resources to model nonstructural walls and finishes in WLF buildings effectively. However, performing nonlinear response history analyses for WLF buildings is still not practical for a common risk assessment, such as a Probable Maximum Loss (PML) assessment. This study enables building-specific, site-specific, seismic risk assessments of WLF buildings by developing a Seismic Response Prediction Method specifically for predicting nonlinear structural responses of WLF buildings, without the user needing to build a nonlinear structural model. It is intended that this response prediction method be used within the FEMA P-58 analysis framework [3].

Method

The structural response prediction engine for WLF buildings is developed from a large database of nonlinear response history analyses of archetype WLF buildings. The following is an outline of its development:

1. Develop a suite of archetype WLF building designs.
2. Develop nonlinear models of the WLF buildings, accounting contributions from structural and nonstructural finishes .
3. Analyze the nonlinear models using nonlinear response history analysis and record Engineering Demand Parameters (EDPs, i.e. Interstory Drift Ratio (IDR) and Peak Floor Acceleration (PFA)).
4. Develop a statistical model that provides expected EDP values, including their correlations and uncertainty, based on building period, strength, and number of stories (strength and stiffness can be computed directly within the response prediction method, based on component quantities within the building).

Archetype WLF Buildings

Seven baseline archetype designs are developed for this study. Each design represents a common form of WLF construction: one and two story single family and one through five story multi-family dwellings. From these baselines, a total of 54 archetypes are created by varying quantities of interior walls, varying the nonstructural finishes (e.g. siding vs. stucco), and varying the roof weight (e.g. tile vs. composite).

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 [4], and the ANSI/AWC Special Design Provisions for Wind and Seismic [5]. The shear walls are proportioned according to ASCE/SEI 7-10 [6]. The baselines are designed short for period spectral acceleration $S_{DS}=1.0g$, 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. Additional archetypes, designed for lower and higher levels of design base shear, are also developed.

Structural Modeling and Analysis

The WLF buildings are modeled in OpenSEES [7]. 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 OSB, gypsum wallboard (gyp), stucco, and siding. As an example, the structural and nonstructural finishes contributing to the strength of the first story of the 3-story baseline archetype are shown in Table 1 (it has a 48' x 96' footprint with floor and roof design seismic weights of 237 kips and 149 kips, respectively).

Table 1. Total length of full height wall finishes (nonstructural finishes italicized), excluding individual segments under three feet, for the first story of the 3-story baseline building.

Finish	Direction 1	Direction 2
7/16" OSB, 8d nails @ 6" O.C.	136 ft.	48 ft.
7/16" OSB, 8d nails @ 3" O.C.	56 ft.	104 ft.
<i>OSB-Nonstruc</i>	<i>24 ft.</i>	<i>24 ft.</i>
<i>Nonstruc Gyp (as a primary finish)</i>	<i>256 ft.</i>	<i>320 ft.</i>
<i>Nonstruc Gyp (as a secondary finish, i.e. with OSB)</i>	<i>216 ft.</i>	<i>176 ft.</i>
<i>Stucco (as a secondary finish, i.e. over OSB)</i>	<i>56 ft.</i>	<i>72 ft.</i>

Backbones for each of the different wall finishes are based on [2]. Floor and roof masses are modeled as 1.05 times the design seismic weight, as recommended by [8]. P-Delta loads are 1.05 times the design dead load, plus an expected live load of 10 psf on the floors. For comparison, a set of models accounting for only the design lateral force resisting system and no nonstructural finishes (referred to as the "bare structure") are also developed.

Each model is analyzed with Incremental Dynamic Analysis (IDA, [9]) using the suite of 44 far-field ground motions from FEMA P-695 [8], and IDR and PFA are recorded at each intensity level. Based on these responses, a structural response engine is developed that does the following:

1. Compute stiffness and strength of the building, based on quantities of structural and nonstructural finishes (consistent with the method in [2]).
2. Output nonlinear structural responses, based on hazard (i.e. spectral acceleration), stiffness, strength, and number of stories. Responses are based on expected values of EDPs, as well as their dispersion and cross-correlations with each other.

Example Results

Seismic risk assessments are performed for the archetype buildings and for generic WLF buildings using the response prediction engine. The FEMA P-58 risk assessment is carried out with the Seismic Performance Prediction Program (SP3 [10]). 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, are populated in the models using normative quantities for similar residential structures. Results are presented in this section to illustrate important trends. Loss estimations are presented as loss ratios, i.e. the building loss normalized by the cost of the building.

Importance of Accounting for Strength and Stiffness from Nonstructural Components

The contributions of nonstructural components to the seismic resistance of WLF buildings are very important for realistic structural response predictions and risk assessments. Figure 1 shows that ignoring the contribution of nonstructural components can lead to significant overestimation of seismic risk. As the buildings get taller, the effects of nonstructural components are less pronounced, because their contribution to the total strength is smaller relative to the design lateral force resisting system; however the effects of nonstructural components are still important for all of the heights considered in this study (up to five stories).

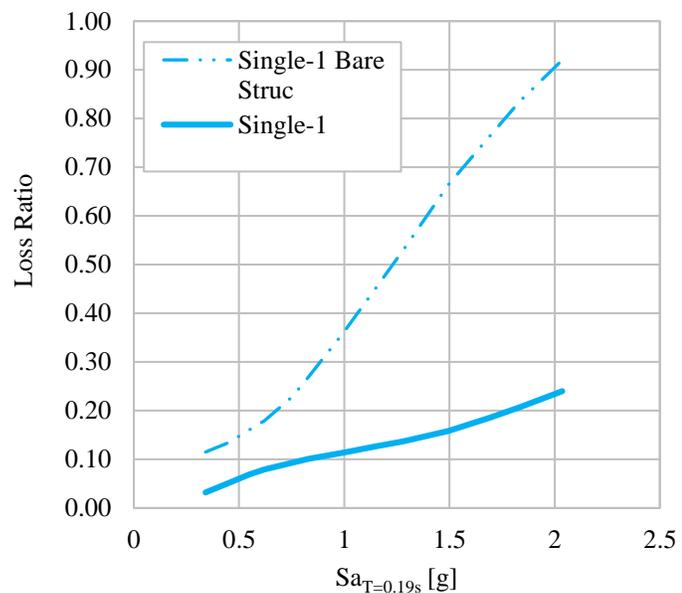


Figure 1. Vulnerability function computed for the one-story single family home (Single-1), with and without consideration of nonstructural finishes in the structural response analysis.

Effects of Building Height

The effects of height on structural responses and risk assessments for WLF buildings is explored in Figure 2 and Figure 3. As the height increases, the strength (relative to seismic weight) decreases, because the strength contribution of nonstructural components, relative to the total weight of the building, decreases. As a result, taller WLF buildings are more vulnerable to seismic damage than shorter buildings, as seen by comparing vulnerability functions for 3-story WLF buildings to 1-story WLF buildings in Figure 3. A similar observation, that taller WLF buildings are more sensitive to collapse, is made by [2].

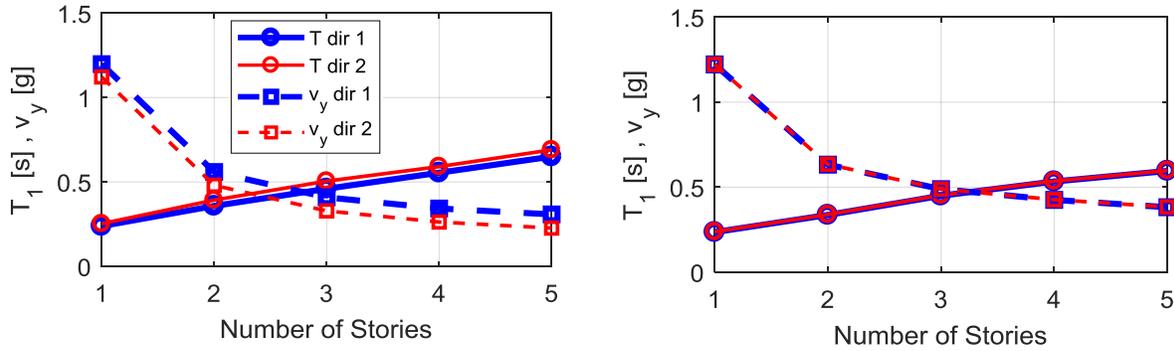


Figure 2. First mode period and ultimate strength vs. height for generic 2:1 aspect ratio multi-family dwellings designed for sites in New York City (left, wind controlled design) and San Francisco (right, seismic controlled design).

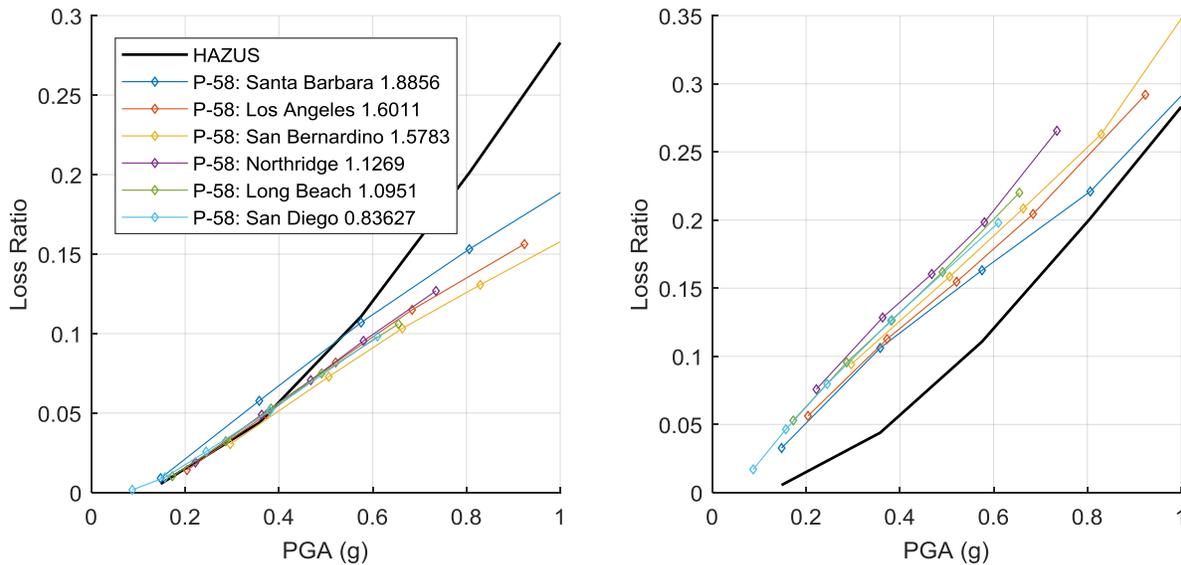


Figure 3. Vulnerability functions for new 1-story WLF buildings (left) and new 3-story WLF buildings (right) designed for different West coast site locations. The trailing number is the short period design spectral acceleration, S_{DS} . The Vulnerability function computed by HAZUS [1] is shown for comparison.

Comparisons to HAZUS

On average, the FEMA P-58 WLF results for 1-story WLF buildings agree well with HAZUS [1] up to PGA of approximately half of gravity, as shown by Figure 3. This is also the range for which data are available for calibrating the HAZUS vulnerability functions. In the range where data are limited, i.e. for multi-story WLF buildings and for buildings subjected high intensity shaking, the FEMA P-58 results begin to diverge from HAZUS. However, losses at large levels of shaking and for multi-story WLF buildings are computed by the same principles as the 1-story buildings that match HAZUS in the range for which HAZUS has the best data; thereby giving a sense of confidence in the FEMA P-58 results.

To observe the differences between FEMA P-58 and HAZUS at a hazard level that is common for gauging seismic risk, Figure 4 shows a ratio of FEMA P-58 loss to HAZUS loss for the ground shaking hazard with 10% probability of exceedance in 50 years, for one and three story WLF buildings at different sites. The large range of differences observed between FEMA P-58 and HAZUS (but with the FEMA P-58 method generally agreeing with HAZUS in the range for which HAZUS has the most calibration data) illustrates the site-specific, building-specific, nature of the FEMA P-58 method.

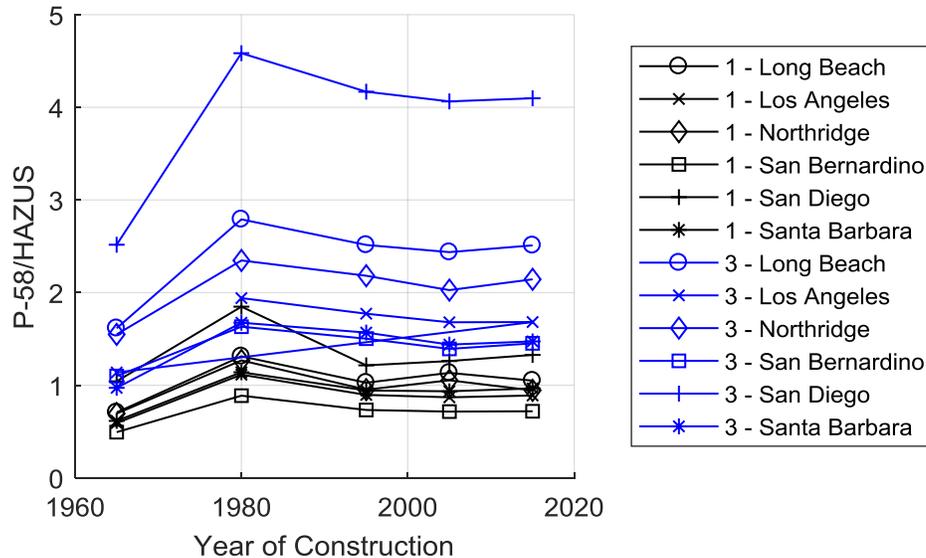


Figure 4. Ratio of the expected loss from the FEMA P-58 method to HAZUS at a hazard with 10% probability of exceedance in 50 years, for one and three story WLF buildings.

Conclusion

This study demonstrates the feasibility of FEMA P-58 risk assessments for wood light-frame (WLF) buildings and shows that including contributions to strength and stiffness from nonstructural walls and finishes is an essential step for predicting the structural responses that are necessary for completing a robust FEMA P-58 risk assessment. We observe that single-story WLF buildings out-perform multi-story WLF buildings, because the contributions of nonstructural walls and finishes to strength and stiffness, relative to the total weight of the building, is more substantial for single-story WLF buildings than multi-story WLF buildings.

However, 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 many uses. Therefore, a structural response prediction method, tailored specifically to WLF buildings, is developed and implemented in the FEMA P-58 framework. This substantially streamlined analysis method is shown to produce reasonable results that (i) agree with HAZUS in the range for which HAZUS has the best calibration data, and (ii) produce trends with height that are consistent with what is expected based on the most recent advances in WLF research (e.g. [2]). This new method will enable users to conduct FEMA P-58

risk assessments of WLF buildings without developing a structural model. We anticipate that this will help fill the need for more robust seismic risk assessments of WLF buildings for mortgage/transactional due-diligence analyses, insurance risk assessments, analyses to support more resilient design of new wood light-frame buildings, and retrofit design of soft-story buildings, which are also handled in the method.

Acknowledgments

Guidance and expertise from members of the ATC-116 Technical Committee, particularly the backbone curves developed by the ATC-116 project team, is gratefully acknowledged. This work is supported in part by the National Science Foundation (NSF) Small Business Innovation Research (SBIR) grants 1519719 and 1632429.

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