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Influence of structural properties and hazard level on seismic loss estimation for light-frame wood structures

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ABSTRACT

The financial loss for residential light-frame wood structures during moderate to strong earthquakes has a substantial impact on society due to the large stock of this building type in the US. The sensitivity of financial losses as a function of structural properties and seismic hazard level is examined in this paper for a two-story residential woodframe structure representative of a North American floor plan. The strength and stiffness of the structure were correlated with the change in the nail schedule for the shearwalls as well as construction quality. The effect of these variants on the short and long term financial loss was then investigated through loss simulations which utilize assembly-based vulnerability (a method to estimate total loss for a structure due to a natural hazard based on individual component losses). The impact of seismic hazard level on financial loss estimation was also examined for three locations representing different seismic hazard levels. It was concluded that there exists an intensity sensitive region for strength and stiffness which limits the effectiveness of improvements for small or large earthquakes. In addition, it was shown that the effect of construction quality in high seismic zones was disproportionate compared to the effect in low seismic zones.

1. Introduction

Light-frame wood buildings represent the main type of residential construction in North America. They have performed satisfactorily during earthquakes from a life-safety standpoint but can be quite vulnerable to damage and subsequent losses. More than half of the estimated $40 billion loss from the 1994 Northridge earthquake was associated with the damage to woodframe buildings. Although collapse is rare for code-designed woodframe buildings, the damage sustained by these structures and the cost to repair them following an earthquake can result in financial ruin for home owners without adequate earthquake insurance. The performance of woodframe structures has not been explicitly addressed by design specifications to date. However, the seismic research community began to investigate woodframe loss related issues and their societal impact following the 1994 Northridge earthquake. Studies and tools to estimate the loss on a large (regional) scale (e.g. HAZUS program [1] from the Federal Emergency Management Agency) were developed for multiple hazards to help with the decision making process in the public and private sectors such as regional infrastructure planning, disaster relief resource allocation, and risk assessment. The accuracy of these regional loss estimation procedures usually depends on the accuracy of the models (fragilities) used to represent the loss behavior of the individual types of structures within the modeling process. Detailed studies related to specific building types were also conducted for steel [2] and concrete buildings [3]. But only limited research (e.g. [4]) related to losses for woodframe structures has been performed. This is likely due to the complexity of the nonlinear dynamic response, the relatively low cost of a single woodframe building, and the significant contribution to seismic response and resulting losses from the nonstructural components. At the same time, there exists significant opportunity for new research that leads to a better understanding of the financial impact of earthquakes on woodframe structures. Research projects such as post-disaster damage/loss surveys, nonstructural component behavior, and construction quality examination for woodframe structures will be very beneficial to the verification of current loss-related models/procedures.

Although it is apparent from experience that higher seismic hazard and weaker structures will typically result in larger losses over time, the quantitative relationship between loss and the structural and earthquake parameters has not been studied thoroughly for woodframe structures. The assembly based vulnerability (ABV) framework [5] proposed by Porter provides a quantitative procedure to estimate losses for a woodframe structure from earthquakes. This is done by summing up damage and costs from individual damageable components based on nonlinear time history results. Pei and van de Lindt adopted the ABV based method in their development of a vulnerability model and applied it to

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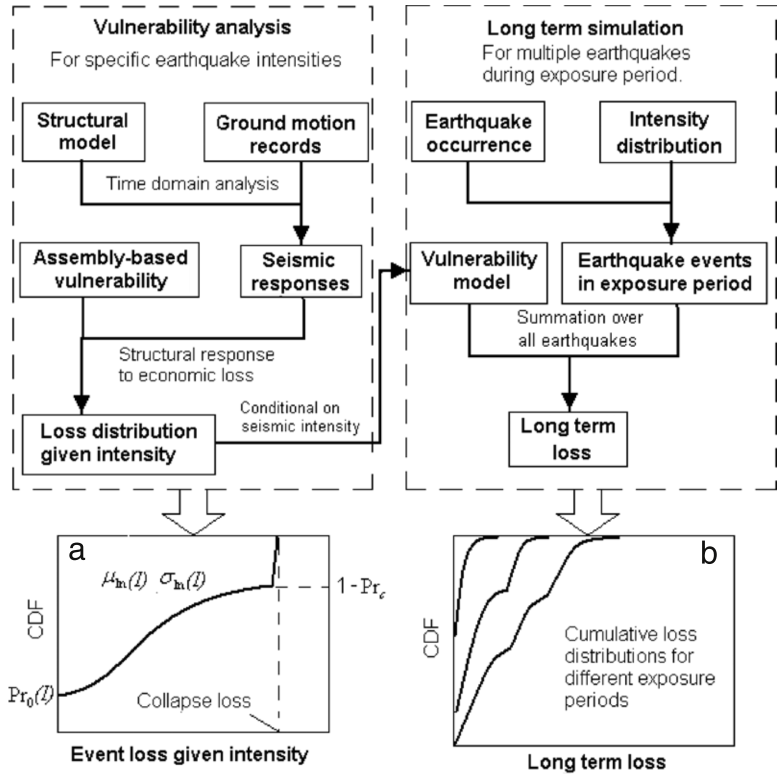


Fig. 1. Loss estimation framework.

develop a long term loss simulation framework [6]. The entire procedure is incorporated into the newly developed software package Seismic Analysis Program for Woodframe Structures (SAPWood) [7]. With these available methods and tools, this study focused on quantitatively investigating the interaction between the losses to woodframe buildings and the major causes of that loss, specifically seismic hazard level and structural properties.

2. Loss estimation procedure

A sensitivity study examining earthquake induced losses to woodframe buildings requires a comprehensive loss estimation procedure to assess losses for a variety of structural configurations and earthquake hazards. In this study, the financial loss was calculated using the Monte-Carlo type simulation procedure developed by Pei and van de Lindt in [6]. A summary of the procedure is presented in Fig. 1.

The procedure mainly consists of two types of simulation, namely the event loss simulation and long-term loss simulation. The event loss simulation focuses on earthquake-induced event loss, which is defined in this study as a random variable that represents the loss for a given building due to an event with a predefined seismic intensity level. Firstly, the structural model of the building was established for time history analysis. A suite of earthquake ground motions were scaled to a particular intensity level and applied to the structural model to obtain the structural responses. Secondly, the loss samples of the building were obtained through the assembly-based vulnerability procedure which evaluates the cost of each damageable component in the building based on structural responses. Note that these samples represent event loss associated with just one intensity level (I). This event loss was then modeled as a four-parameter controlled distribution shown conceptually in Fig. 1(a). The zero-loss probability, Pr_0, indicates the probability of having no loss in an earthquake with given intensity I; the collapse-loss probability, Pr_c, represents the probability of complete collapse of the building under the given intensity I, which (of course) results in maximum loss; the remaining mean and remaining dispersion, $\mu_{\ln}$ and $\sigma_{\ln}$, are the parameters of a log-normal distribution fit based on the loss samples between the zero and maximum losses. The sudden increase in the CDF is caused by the existence of the collapse loss value as an upper limit on the event loss. In this study, this sudden increase was idealized as a straight portion of CDF curve starting at the loss value corresponding to 95% of the collapse loss, thus making it a uniform distribution for this final portion. Then, this process was repeated for a range of intensity levels of interest which resulted in event sample groups associated with different seismic intensities. Finally, the relationship between these parameters and seismic intensity can be established either in explicit functional form or in an empirical way. This relationship, termed herein as a vulnerability model, serves as a comprehensive probabilistic representation for the loss resistance of the structure against various intensity levels. In this paper, the impact of structural properties on the performance of woodframe buildings is examined through this vulnerability model, with the performance defined as earthquake induced losses.

The other simulation type, long term loss simulation, should be conducted when cumulative loss over a certain exposure period is of interest. The objective of this simulation is to obtain the distribution model for long term loss which is illustrated in Fig. 1(b). In this procedure, the vulnerability model is combined with uncertainty models for earthquake occurrence and intensity. For a specific location, the total number of earthquake occurrences can be generated from a Poisson distribution (calibrated using historical data). Then the intensity for each earthquake event can be generated from the distribution model established based on the seismic hazard curve data for the building site of interest from, for example, the US Geological Survey (USGS) database. Once the intensity samples are generated, the vulnerability model can be used to generate loss distributions for each individual earthquake event. The cumulative loss is then found by adding the individual losses together.

Table 1
Wall model backbone parameters.

Pattern	Quality	K_0 (N/mm)	F_0 (N)	r_1	X_u (mm)	r_2	X_{u1} (mm)	λ_b	F_r (N)
2/12	Ideal	2851	33 569	0.01	32	−0.05	38	−1.00	374
	Missing field nail	2441	33 302	0.01	35	−0.05	42	−1.00	356
	Missing top line	2364	31 581	0.01	30	−0.05	37	−1.00	334
	Missing bottom line	2277	30 700	0.01	30	−0.05	37	−1.00	320
	Missing 20% overall	2101	29 077	0.01	26	−0.05	32	−1.00	276
	Missing edge line	1692	15 879	0.01	31	−0.05	38	−0.60	160
4/12	Ideal	2087	17 752	0.01	27	−0.05	32	−0.70	196
	Missing field nail	1786	16 987	0.01	26	−0.05	31	−0.80	187
	Missing top line	1769	15 879	0.01	29	−0.05	35	−0.70	178
	Missing bottom line	1786	15 074	0.01	28	−0.05	34	−0.70	169
	Missing 20% overall	1664	14 216	0.01	27	−0.05	32	−0.50	151
	Missing edge line	1191	9 430	0.01	30	−0.05	36	−0.70	107
6/12	Ideal	1721	11 632	0.01	23	−0.05	28	−1.00	129
	Missing field nail	1646	11 053	0.01	23	−0.05	28	−0.70	125
	Missing top line	1541	10 213	0.01	25	−0.05	30	−0.70	116
	Missing bottom line	1471	10 577	0.01	26	−0.05	31	−0.70	116
	Missing 20% overall	1366	9 216	0.01	25	−0.05	30	−0.60	102
	Missing edge line	1051	6 685	0.01	26	−0.05	32	−0.60	76

Obviously, the cumulative losses to a particular structure are controlled largely by the seismic hazard profile (occurrence and intensity) at the building site and the exposure time. The impact of these variables was investigated in this paper by comparing the long term loss statistics.

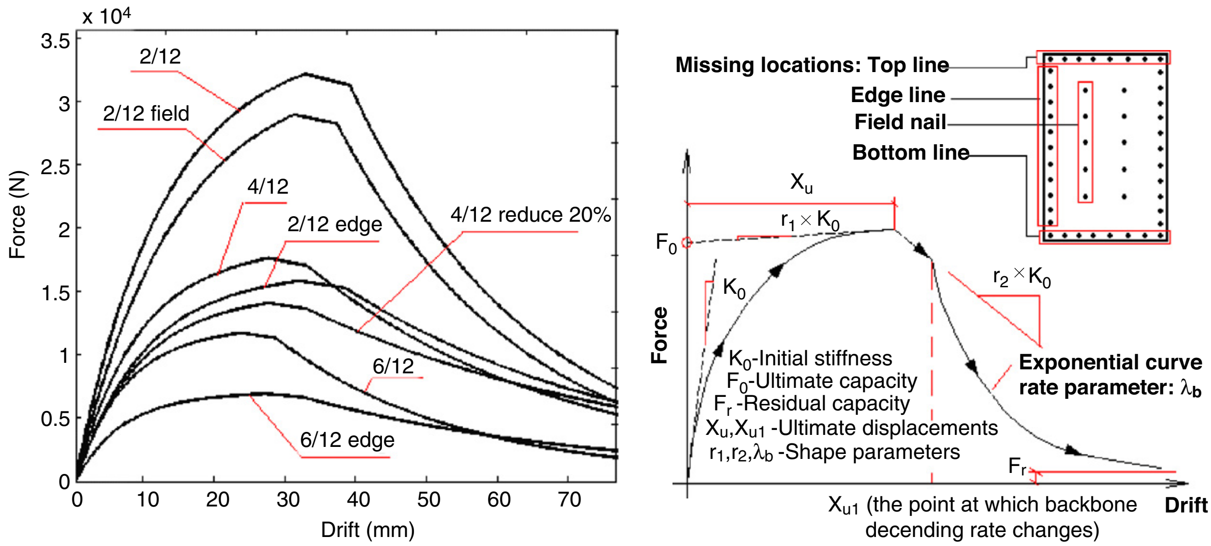
3. Structural properties and wall modeling

Shearwalls are the main lateral force resisting component in light frame wood structures, and are the focus of the design procedure for strength based seismic design. The nailing pattern and sheathing panel thickness (and type) are determined based on the amount of lateral force needed for each individual wall line. Different wall configurations, when subjected to seismic excitation, can lead to very different values of seismically-induced loss in a woodframe building. A quantitative understanding of the sensitivity of loss behavior to changes in structural properties is crucial in order to develop a better understanding of how these design variants should enter into the loss-based decision making process, e.g., the choice of a more robust design that exceeds current code requirements but provides long term loss mitigation benefits. In order to investigate this sensitivity, quasi-static numerical models for different shearwalls were established in the SAPWood-Nail Pattern software package [7] and forced through a reversed-cyclic displacement protocol imposed at the top of the wall. The resisting shear force of the entire wall during the loading process can be calculated based on the principal of virtual work and the hysteretic response of the wall is obtained. Each individual wall hysteresis is fitted to a wall level nonlinear hysteretic spring model (see [8] for details on the model), which is then put into a full system-level model (i.e. a house) in order to perform nonlinear time history analyses to obtain responses for loss estimation. Through this procedure, the difference in shearwall design (properties) is represented in loss simulation as shearwall hysteresis elements having different parameters.

3.1. Nail pattern/schedule and construction quality

Three types of very commonly used nailing patterns (in the US) were investigated, termed as 2/12 (nails along sheathing panel edge spaced at 50 mm (2 inch) with spacing for nails not along the panel edge (field spacing) equal to 305 mm (12 inch)), 4/12 (edge spacing = 102 mm, field spacing = 305 mm), and 6/12 (edge spacing = 152 mm, field spacing = 305 mm) nailing pattern. The standard wall model was selected as a 2.44 × 4.88 m (4 × 8 ft) shear wall with 0.4 m stud spacing. The sheathing to framing nails used in the model were 8d common nails (length = 63.5 mm, diameter = 3.33 mm) which is also widely used in residential light frame wood construction. Fastener parameters were obtained through cyclic tests of fasteners with 11.1 mm thick (7/16 inch) Oriented Strand Board (OSB) sheathing conducted at Colorado State University [8]. Since wood shearwall behavior is largely controlled by the backbone curve of the hysteresis, the backbone curve parameters obtained from the analysis are presented in Table 1. Other factors that might affect the overall structural and shearwall performance, such as construction quality are also shown in that table. An earlier study [9] indicated that construction quality issues in light frame construction usually result from poor on-site construction practice, such as sheathing nails missing the wall stud and thus reducing the ultimate capacity and stiffness of the wall and essentially changing its behavior during an earthquake. In this study, the shearwall models corresponding to variants of poor construction practices were modeled with SAPWood NP, which included missing 20% of the nails, missing one field nail line, and missing one panel edge nail line (vertical nail line, top nail line, and bottom nail line). The locations of these missing fasteners and the parameters used to represent the backbone curve model are presented in the schematic of Fig. 2(b). The wall models for these cases were built and analyzed for each configuration. Then each parameter in Fig. 2(b) was determined through the fitting of the backbone curve model to the simulated response of the shearwall models. Also shown in Fig. 2(a) are the backbone curves corresponding to the model with construction quality deficiencies, and an ideal wall model (no missing fasteners) backbone for reference.

It can be concluded from Table 1 that both the variation in nailing pattern and construction quality result in a "degradation" (or reduction) of the hysteretic parameters, especially those associated with the strength and stiffness of the nonlinear hysteretic spring. Using the parameters for an ideally built 2/12 wall as a benchmark value (full capacity) against which to compare other models with construction deficiencies, the degradation of stiffness and strength parameters under all situations in Table 1 can be illustrated with percent deductions to the key hysteretic parameters in Fig. 3. One can see from the figure that construction quality issues may cause an average stiffness and strength reduction of around 10%–20% (from a missing field nail line to missing 20% of the nails overall). This was believed to fall in a more normal range of construction quality for residential structures, although obviously still not ideal. Missing the edge nail line can cause significant reduction in strength (around 45%), which was assumed in this study to be



(a) Backbone curves for different conditions.

(b) Parameters for backbone curve.

Fig. 2. Degrading of backbone curve for different wall configurations.

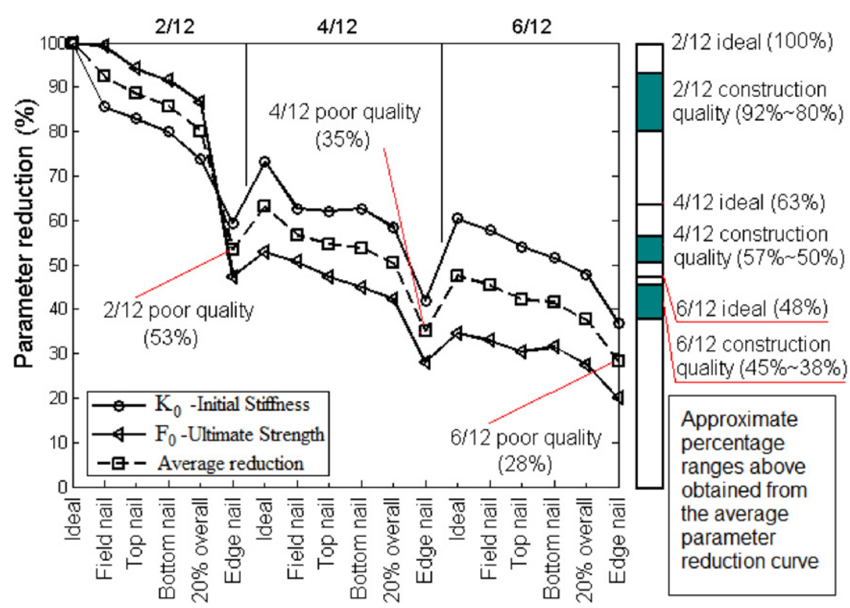


Fig. 3. Quantifying nailing pattern and quality with parameter reduction.

representative of poor construction quality. This percent degradation of the major hysteretic parameters was used later to represent different configurations and construction qualities for all the shearwalls in the example building.

4. Example residential building

In order to examine the sensitivity of residential structures, a two story single family home with two bedrooms and a one car garage was selected in this paper as the example structure. This type of residential building represents one of the dominant residential construction types in North America and was used for this illustrative seismic loss study. The building has a total area of about 140 m^2 (1500 sq. ft), and the architectural floor plan is shown in Fig. 4.

The numerical model for the structure was developed using the software program SAPWood [7] with shearwalls represented by nonlinear hysteretic elements as described earlier. This level of numerical model complexity is consistent with typical nonlinear analysis within the seismic wood engineering research community. The seismic mass was assumed to be evenly distributed at the first floor level (19 500 kg [43 kips]) and the roof level (15 000 kg [33 kips]). Four types of damageable components were considered in the loss estimation, which included structural shearwalls, drywall partition walls, doors and windows, and general contents. The repair cost of each component was based on the structural response from nonlinear time history analysis and the component damage fragilities [6]. The maximum amount of loss a component can have was set equal to the replacement cost which was assigned based on existing literature [4]. The unit replacement costs of these components are listed in Table 2. Note that the general contents are not listed in the table but assumed to be worth a total of $50 000 in this study (with half of the value linked to the acceleration at ground level and the other half associated with the 1st floor acceleration) if the numerical model indicates collapse of the structure. By adding up these assigned costs for all the damageable components in the building, the collapse loss of the example

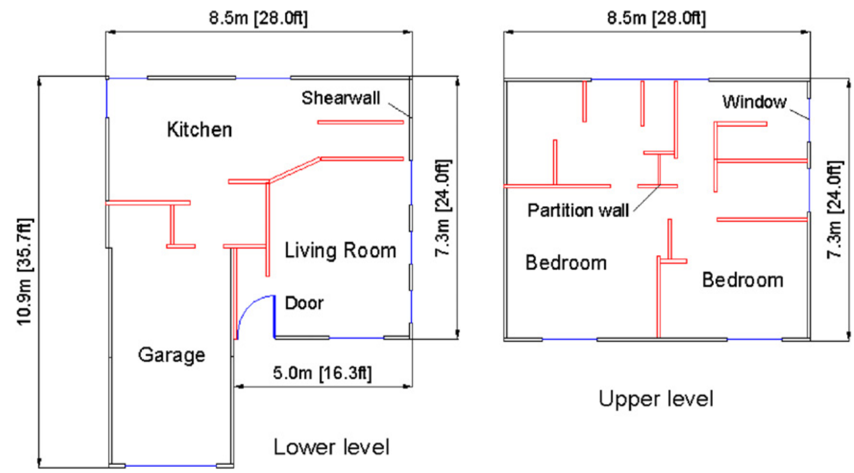


Fig. 4. Example building floor plan.

Table 2
Replacement cost data from CUREE report.

Component	Shearwall	Drywall	Door	Window
Units	5.9 m^2 (64 ft^2)	5.9 m^2 (64 ft^2)	Each 0.9 × 1.2 m (3 × 4 ft)	
Replacement cost (US$)[a]	742	445	190	149

[a] Cost data obtained from CUREE report: Improved loss estimation [4].

structure was calculated to be approximately $75 000. Note that the inclusion of components contributing to the loss in this study was not comprehensive, i.e. the collapse loss value is likely to be smaller than financial loss in a real collapse since only four major components were included. But after the simulated losses were normalized by the collapse loss value, the results and conclusions based on the normalized values may be viewed as being representative for typical residential woodframe building.

5. Loss sensitivity

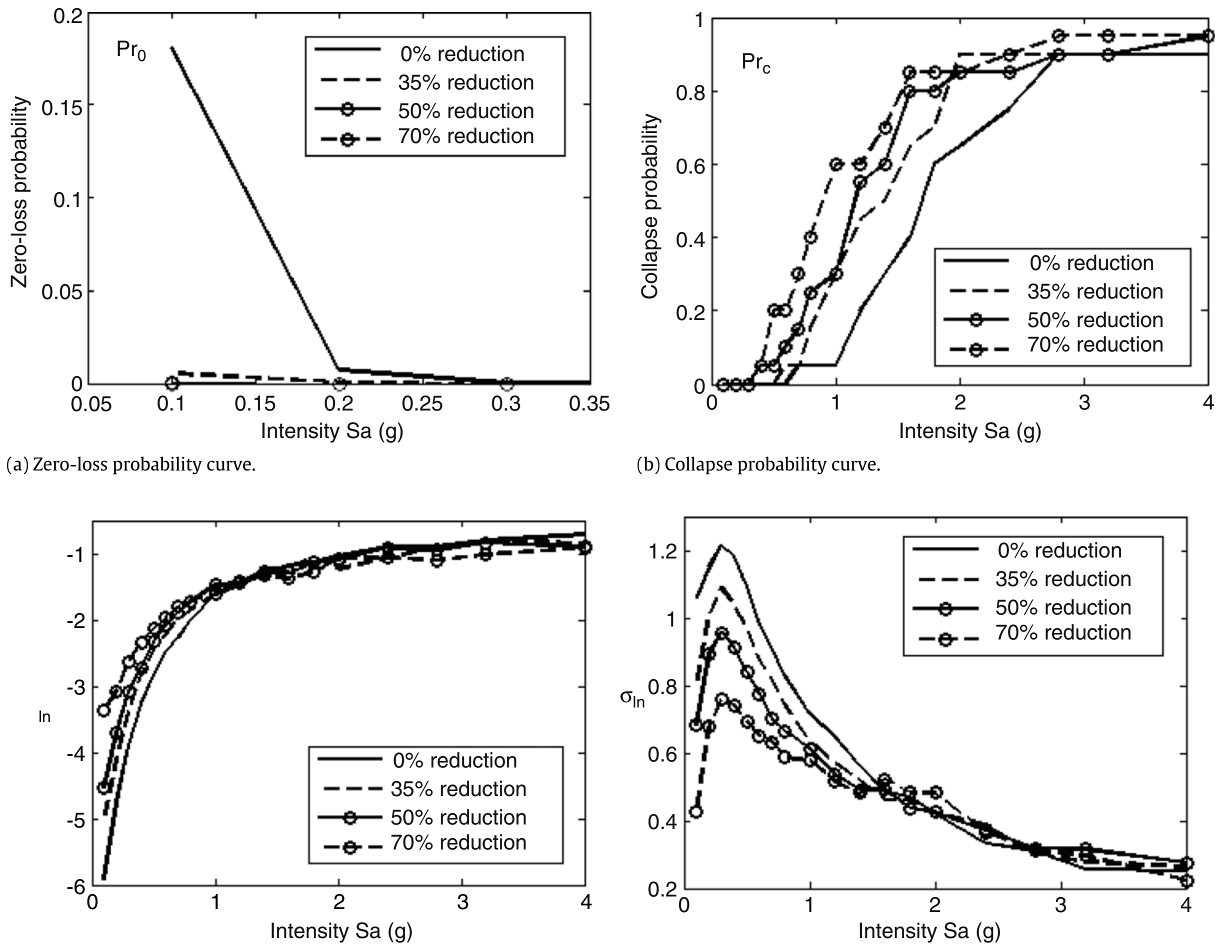
The sensitivity of the seismic-induced loss to seismic hazard and structural parameters was divided into two major parts in this study. The first part focused on the event loss, which was directly related to the structural properties and construction quality. The second part of the analysis focused on long term loss, which varies also with the seismic hazard level and is thus a function of building location and exposure time.

5.1. Event loss behavior

The structural parameter variation will have a direct impact on the vulnerability analysis and result in different loss behaviors at different intensity levels. A suite of 20 earthquake ground motion records [10] was used in the vulnerability analysis to represent the variation in earthquake ground motion. Based on the relationship between wall element parameters and structural configuration/quality illustrated earlier in Fig. 3, the change in nailing pattern and different levels of construction quality were examined here by incrementally reducing the stiffness and strength parameters of all wall elements in the numerical model with a reduction factor (discount) from 0% to 70% of the full value. A 0% reduction factor corresponds to an ideally built wall with the 2/12 nailing pattern without any construction quality problems. It should be mentioned that this approach assumes that the nailing pattern and construction quality were consistent for all the walls in the structure, which is not necessarily the case, i.e. construction quality varies from wall to wall. However, this style or method of analysis is intended to capture the sensitivity of seismic-induced loss to construction quality in an average sense.

Following the aforementioned vulnerability simulation procedure, the vulnerability parameters for the example house with different reduction factors were calculated and plotted as vulnerability curves in Fig. 5. Note that the seismic intensity was characterized by the spectral acceleration at 0.2 s with 5% elastic damping ratio. For clarity, only four different reduction levels were selected to show the trends. The 0%, 35%, and 50% reduction cases correspond to ideally built walls with the 2/12, 4/12, and 6/12 nailing patterns respectively. The 70% reduction case corresponds to a wall having a 6/12 nailing pattern with poor construction quality, which was the worst case included in this study. One can see from Fig. 5 that the probability of zero loss dropped quickly to zero for all reduction cases as soon as the seismic intensity reached a certain level, indicating minor damage/loss is inevitable for earthquakes with intensities exceeding a certain threshold value. The probability of collapse approaches unit as intensity increases for all cases, with the stronger buildings (with less reduction) showing better performance, i.e. their collapse probability is lower than weaker buildings for most intensity levels. Recall that parameter $\mu_{\ln}$ and $\sigma_{\ln}$ were obtained by fitting the simulated samples excluding the zero loss and collapse cases. Thus these samples were always greater than 0 but smaller than the collapse loss. As a result, the parameter $\mu_{\ln}$ increases with intensity but never exceeds a certain value, while the parameter $\sigma_{\ln}$ decreases as the intensity becomes large (resulting in concentration of samples close to the collapse loss) and approaches 0 (resulting in concentration of samples close to zero loss). The comparison of parameter $\mu_{\ln}$ and $\sigma_{\ln}$ between different reduction cases does not result in a significant conclusion because these parameters only reflect the distribution for part of the simulated samples. With the vulnerability model parameters, the normalized vulnerability curves (corresponding to $S_a = 1.0g$) were developed and are shown in Fig. 6 with loss samples divided by the collapse loss. Note that because of the distribution model used in this study, there is a sudden increase as the CDF curves approaches the 1.0 normalized loss value, resulting in a piecewise CDF function as shown in Fig. 6 (with the simulated samples for the 70% reduction case also shown).

It can be seen from Fig. 5 that structural configuration and construction quality issues will have an impact on the structure's loss vulnerability, as one would expect. The probability of zero loss for all cases drops to essentially 0 after 0.3g intensity. The probability of collapse increases as the seismic intensity increases for all reduction levels. But this increase begins at higher intensity levels for higher quality (less reduced) structures. With the distribution parameters available from vulnerability curves, the event



(a) Zero-loss probability curve.
(b) Collapse probability curve.
(c) $\mu_{\ln}$ curve.
(d) $\sigma_{\ln}$ curve.

Fig. 5. Effects of reduction factor on vulnerability parameters.

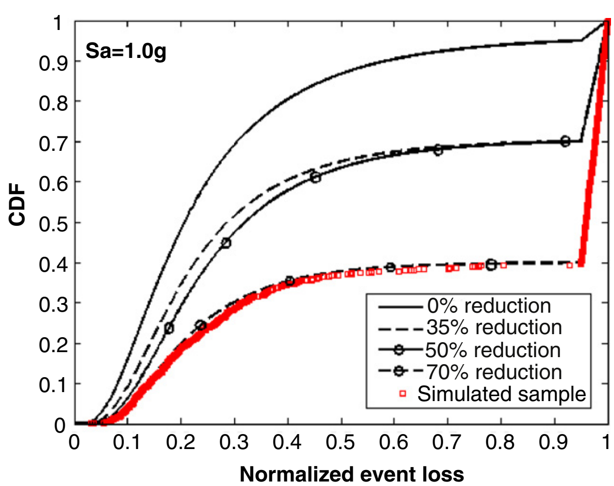
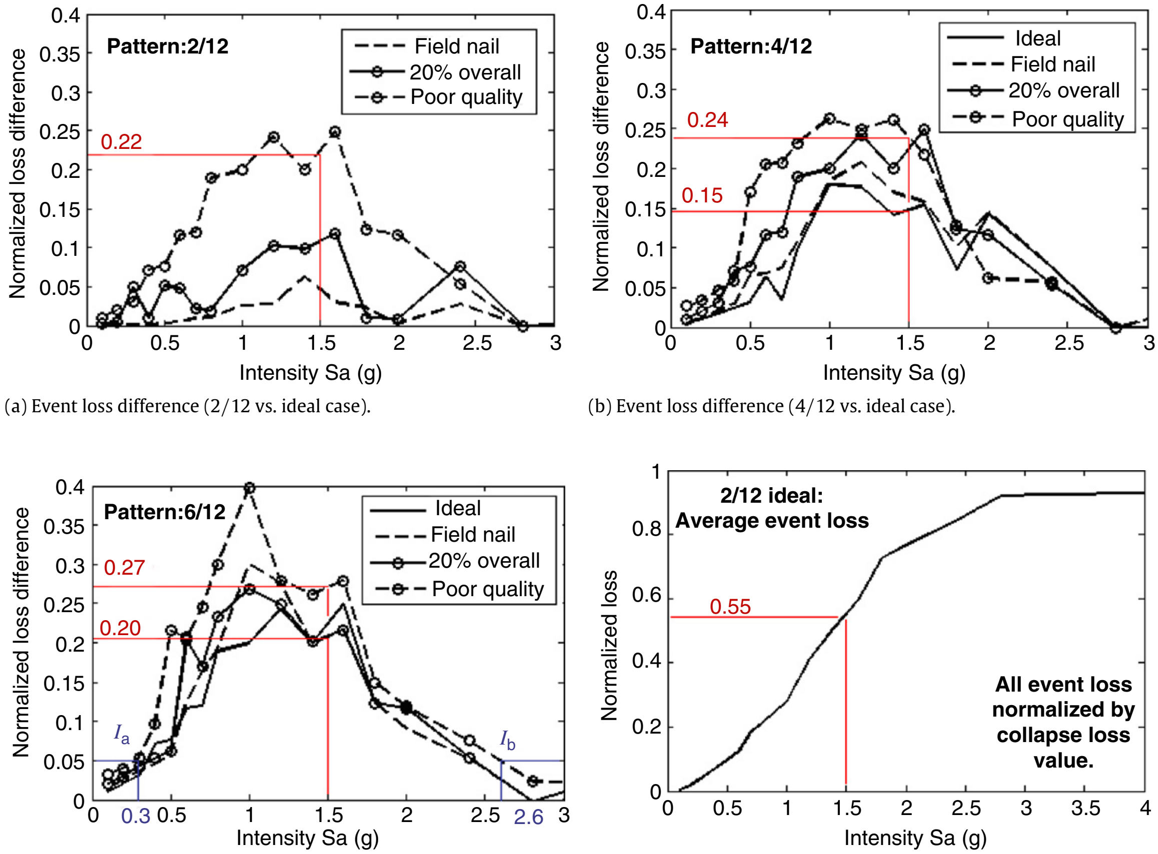


Fig. 6. Normalized event loss distribution for different reduction values.

loss distributions can be obtained for any specific intensity level. Furthermore, the direct comparison of certain loss statistics can be compared among alternative options. For example, in Fig. 7(a), (b), and (c), the difference in the average event loss between several of the reduced cases and the 2/12 ideal case are presented in order to illustrate the difference. The average loss of the full capacity structure (2/12 ideal) was also shown in Fig. 7(d) as a common point of reference.

It can be observed from Fig. 7 that the financial loss of the building with the stronger wall configuration is generally less than that with weaker walls provided the construction quality is the same, which is to be expected for the same seismic mass. However, the house with the 2/12 nailing pattern having poor construction quality under-performs the house ideally built with the 4/12 nailing pattern. This is somewhat notable since this type of dense nail spacing is typically used in building with very high seismic demand. For all nailing patterns, the average event loss generally increases with the decrease of construction quality. The impact of construction quality deficiencies is more notable for stronger configurations. For example, as one can see from Fig. 7, given an earthquake with 1.5g spectral acceleration, the average event loss for the 2/12 nailing pattern structure could range from 55% to 77% (a 22% variation) of the collapse loss depending on the construction quality. The range of average loss was 70% to 79% (a 9% variation) for the 4/12 pattern and 75% to 82% (only a 7% variation) for the 6/12 pattern. This result agrees and reinforces the earlier comment that the benefit observed from quality control is more significant for stronger designs. Among different seismic intensity levels, the difference in average loss was not significant in both the low and high ends of the intensity level. The simulation showed essentially no difference in average loss for intensities over 3g spectral acceleration due to the fact that the model indicated collapse for most simulation cases regardless of quality or nailing pattern. This implies that the financial advantage of having a stronger building or better quality construction would not be significant if the earthquake is very small or very large, but is present for a range of moderate seismic intensity levels. This range was termed herein as the



(a) Event loss difference (2/12 vs. ideal case).

(b) Event loss difference (4/12 vs. ideal case).

(c) Event loss difference (6/12 vs. ideal case).

(d) Ideal case average event loss.

Fig. 7. The impact of configuration/quality issue on average event loss.

Intensity Sensitive Region (ISR) for structural properties (including quality and nail pattern). The ISR was defined in this study as the seismic intensity range between two intensity values I_a and I_b (see Fig. 7(c)), where I_a/I_b is the lowest/highest intensity at which the difference in normalized average event loss between any two configurations with different structural properties reaches a threshold value (0.05 in this case). The range of ISR depends largely on the influential factor (change of nail pattern in this case) and threshold value chosen. The ISR for a single structural configuration usually is not as useful as the comparison between different configurations. The calculation and comparison of ISR (range and position) and loss difference curves for different system alternations can be conducted to identify the most important influential factor to seismic loss in different intensity levels. In the case of this example, the range of (approximately) 0.3g to 2.6g spectral acceleration was the intensity level within which the structural properties considered in this study makes a significant difference. Note that the ISR can also be calculated using other seismic intensity indicators (such as PGA) by using the desired intensity indicator in the vulnerability analysis.

5.2. Long term losses

The effect of structural characteristics on losses for an earthquake having a specific intensity is valuable from a scenario viewpoint, i.e. for comparison. In a more practical sense, it is rational to evaluate the impact of a change in structure configuration on cumulative losses over a longer period of exposure in which multiple earthquake events could occur, for example, the anticipated lifetime of the structure or the expected ownership period. In order to perform this evaluation, other sources of uncertainty, such as earthquakes recurrence, must also be included using the long term loss simulation procedure introduced earlier. These additional uncertainties will not affect the vulnerability of a structure against a single event. Thus the long term loss simulations performed in this section were based on the vulnerability results from the previous analysis.

The seismic hazard environment is controlled by the location of the building, thus the sensitivity of losses to seismic hazard level were studied here by examining the long term loss of the example structure at three different locations throughout the US. These locations ranged from seismically active to inactive locations, namely Los Angeles, CA, Portland, OR, and Detroit, MI. The hazard curve for each location was obtained from the US Geological Survey and presented graphically in Fig. 8, showing the annual probability of exceedance for each location versus spectral acceleration.

The period of ownership for a single family home can vary significantly; therefore the exposure periods examined in this study include 5 years for short term ownership, 30 years for typical mortgage payment period, and 75 years for lifetime (plus) ownership. A combination of structural properties and locations were investigated for these three time periods. The long term loss (normalized by the collapse loss) associated with each combination of location and construction quality was computed. It was assumed that the structure will be restored/repaired to the initial status after each individual earthquake during the exposure period. Thus the total loss normalized by the collapse loss value could be greater than unity if multiple severe earthquakes occurred during the simulation period. The statistics of the simulated loss distribution are presented in Table 3, including the probability of zero loss (Pr_0), the median, and the 95th percentile value (labeled Extreme) of

Table 3
Statistics of long term losses.

Location (USA)	Period (year)	Quality								
		Ideal			Normal			Poor		
		Pr_0	Median	Extreme	Pr_0	Median	Extreme	Pr_0	Median	Extreme
2/12 nailing pattern										
	5	0.991	0	0	0.990	0	0	0.989	0	0
Detroit MI	30	0.944	0	0.001	0.944	0	0.002	0.938	0	0.007
	75	0.864	0	0.004	0.859	0	0.006	0.844	0	0.014
	5	0.822	0	0.012	0.802	0	0.017	0.796	0	0.033
Portland OR	30	0.313	0.004	0.097	0.251	0.008	0.144	0.265	0.018	0.208
	75	0.040	0.020	0.246	0.045	0.030	0.378	0.030	0.065	0.476
	5	0.229	0.006	0.107	0.198	0.010	0.131	0.178	0.023	0.200
Los Angeles CA	30	0.002	0.087	0.562	0.002	0.121	1.015	0.002	0.227	1.173
	75	0.005	0.278	1.202	0.005	0.384	1.469	0.005	0.694	1.780
4/12 nailing pattern										
	5	0.989	0	0	0.989	0	0	0.989	0	0
Detroit MI	30	0.938	0	0.004	0.938	0	0.007	0.938	0	0.022
	75	0.844	0	0.009	0.844	0	0.014	0.844	0	0.032
	5	0.799	0	0.023	0.796	0	0.033	0.795	0	0.059
Portland OR	30	0.271	0.011	0.137	0.265	0.018	0.208	0.259	0.041	0.245
	75	0.035	0.042	0.344	0.030	0.065	0.476	0.040	0.130	0.538
	5	0.183	0.014	0.173	0.178	0.023	0.200	0.181	0.051	0.240
Los Angeles CA	30	0.002	0.158	1.037	0.002	0.227	1.173	0.002	0.405	1.280
	75	0.005	0.477	1.583	0.005	0.694	1.780	0.005	1.073	2.403
6/12 nailing pattern										
	5	0.989	0	0	0.989	0	0	0.989	0	0
Detroit MI	30	0.938	0	0.007	0.938	0	0.016	0.938	0	0.026
	75	0.844	0	0.014	0.844	0	0.026	0.844	0	0.039
	5	0.796	0	0.033	0.796	0	0.051	0.799	0	0.068
Portland OR	30	0.265	0.018	0.208	0.255	0.033	0.235	0.261	0.049	0.265
	75	0.030	0.065	0.476	0.040	0.109	0.763	0.045	0.148	0.787
	5	0.178	0.023	0.200	0.178	0.041	0.214	0.177	0.061	0.267
Los Angeles CA	30	0.002	0.227	1.173	0.002	0.333	1.176	0.002	0.472	1.412
	75	0.005	0.694	1.780	0.005	0.925	2.019	0.005	1.327	2.487

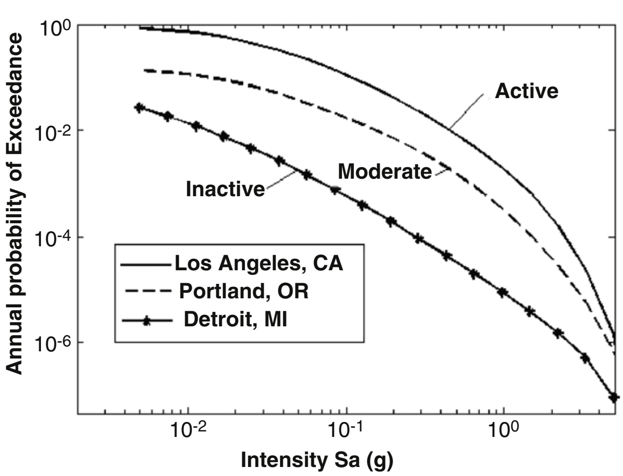
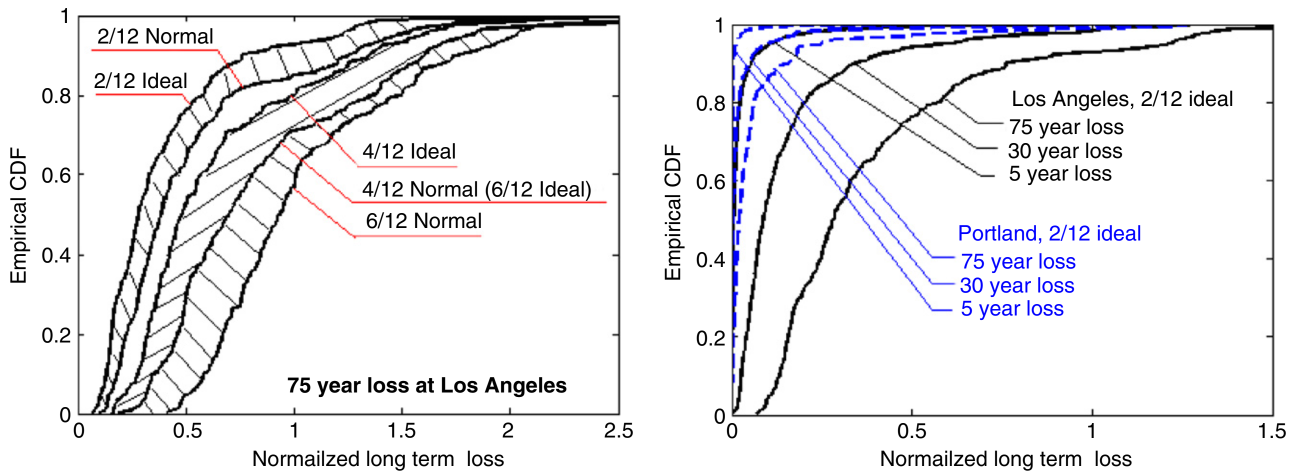


Fig. 8. Hazard curves for different locations.

simulated long term loss samples. The "Normal" quality in the table corresponds to the case with 20% nails either missing or missing the stud, representing the most deficient construction quality among normal quality problems. The empirical CDF curves (CDF values obtained by rank ordering the simulated data) for different configurations, exposure periods, and locations are shown in Fig. 9. Note that curves from Detroit are not presented because the loss is essentially zero for most of the cases.

It can be seen from the Table 3 and the CDF curves shown in Fig. 9 that the location, or more specifically the seismic hazard, significantly influences the loss results for all ranges of exposure period as one would expect. For Detroit, MI, the extreme loss values (the percentile values with a very low probability of exceedance) were only a fraction (less than 5%) of the collapse loss, which means the damage from earthquakes to the structure during the exposure period is very likely to be just cracking of the drywall since the damage is predicted as minimal. Although there is a trend in the simulation statistics, the influence of construction quality and nailing pattern (structural strength) is negligible for the low seismic hazard zones since the lack of earthquake occurrence dominates the problem. When considering a location with higher seismic hazard such as Portland, OR, one would expect some probability of more considerable losses in the 30 and 75 year periods. The loss median value increases with the exposure period but the relationship is not linear. Under the same construction quality, the increase in design strength (e.g. a stronger nailing pattern) can considerably reduce the risk of loss, especially in extreme cases (e.g. normal quality 6/12 configuration resulted in twice as much extreme loss statistics as a 2/12 configuration in 75 years in Portland). Having good quality control can also help reduce the potential seismic loss. In high seismic regions like Los Angeles, home owners of structures with configurations similar to the example structure in this paper are likely to see minor loss in 5 years. The 75 year median loss is estimated at more than a quarter of the collapse loss even for the ideally built stronger (2/12) configuration. In extreme cases, the cumulative loss in 75 year in Los Angeles could exceed the collapse loss value even with strong ideal construction. The impact of construction quality is observed to be significant for all nailing patterns over time (i.e. poor quality doubles the median loss) in high seismic zones. The median long term loss value of a poorly constructed can be more than twice of that of the ideal quality for any nailing pattern. This relative (percentage) increase was more for stronger configuration (2/12) than for weaker ones.



(a) Impact of nailing pattern and quality.

(b) Impact of nailing location and exposure period.

Fig. 9. Empirical CDF curves of long term losses.

6. Summary and conclusions

Based on detailed nonlinear dynamic model responses, vulnerability analysis and long term loss simulation were combined for a typical North American style residential building with variants. These included structural properties, construction quality, and the level of seismic hazard. Both the event and long term loss were assumed to be random variables whose distribution was controlled by the structural and hazard environment inputs. Through the examination of the results, the following conclusions related to the effect of structural properties and seismic hazard on the predicted loss for a typical woodframe building can be made:

There is an Intensity Sensitive Region associated with the modification of structural strength and stiffness. As a direct result, the mitigation of loss due to changes in structural properties or construction quality is limited for very small or very large earthquakes; the expected loss to the example buildings with different configurations used in this study was not affected significantly by structural properties for earthquakes under 0.3g spectral acceleration or the ones greater than 2.6g. Within the Intensity Sensitive Region, the impact of construction quality on event loss is more significant in stronger configurations than in weaker configurations.

As one would expect, seismic hazard level greatly affects the long term loss of the structure. The long term losses for the building in seismically active regions is significantly higher than that in seismically inactive regions. The median loss value for the Detroit area was essentially 0 for all design and construction quality cases, while the loss of the building in Los Angeles can have a median of more than the collapse loss. The reduction in loss from using a stronger nailing pattern was limited, especially in the short term (less than 5 years) and in seismically inactive regions. However, using a significantly stronger structural configuration did considerably reduce the expected losses over the long term in seismically active regions (changing from 6/12 to 2/12 nail pattern brings about 30% decrease in the 95th percentile 75-year loss value and a 60% decrease in median loss value for Los Angeles). The construction quality had a considerable impact on long term loss. For example, for the 6/12 nail pattern design cases, reducing the construction quality from normal to poor resulted in 35% increase in long term median loss values for Portland and 43% increase for Los Angeles, respectively. This effect of construction quality in high seismic zones is disproportionate to the effect in low seismic zones.

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