Evaluation of an alternative seismic design approach for rigid wall flexible wood roof diaphragm buildings through probabilistic loss estimation and disaggregation

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ABSTRACT

Rigid Wall Flexible roof Diaphragm (RWFD) buildings, commonly referred to as “big-box” buildings are the most prevalent type of construction for low-rise industrial and warehouse facilities in the United States (US). These buildings usually incorporate rigid-in plane concrete tilt-up walls and flexible wood roof diaphragms, which is a commonly seen construction technique in the Western United States. Due to their vulnerability in high seismic areas (e.g. California) observed in past earthquakes, an alternative design methodology was introduced in the FEMA P1026 document to account for the response of the flexible roof diaphragm. The FEMA P1026 design approach has been validated through numerical collapse assessment studies. In this study, the Performance-Based Earthquake Engineering framework, introduced by the Pacific Earthquake Engineering Research (PEER) center, is combined with Monte Carlo Simulation to evaluate, in a probabilistic sense, the earthquake-induced economic losses for these structures. The results are presented in terms of expected losses for two hazard intensities: Maximum Considered Earthquake (MCE) and Design Earthquake (DE), while loss disaggregation plots for collapse and no-collapse losses are also presented. The results demonstrate the ability of the FEMA P1026 design approach to reduce earthquake losses

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compared to current code-conforming RWFD buildings. Additionally, the results can provide damage and loss information for modeling of these types of buildings within a resilient community and other spatially focused analyses.

Keywords: Probabilistic loss assessment; Performance-based engineering; Fragility curves; Nonlinear time history analysis; Flexible wood roof diaphragm; Tilt-up walls; Seismic design provisions.

1. Introduction

Life safety associated with building failures has been extensively accounted for in structural design provisions and standards against extreme ground shaking. However, economic losses from earthquake-induced damage to structural and nonstructural components have not been incorporated into the evaluation of design methodologies and mitigation strategies. Next generation Performance-Based Earthquake Engineering (PBEE) framework introduced in the United States (US) by the Pacific Earthquake Engineering Research center (PEER) considers certain metrics to assess the seismic performance of buildings, which integrate economic losses into the design process [1-3]. The PBEE framework accounts for evaluation, design and construction of structures to satisfy seismic performance criteria expressed in terms of dollars, death, and downtime; metrics that are meaningful to stakeholders and building owners [4-6]. Site-specific hazard analysis (e.g. [7-11]), nonlinear structural modeling, damage assessment through fragility analysis (e.g. [12-16]), and economic loss assessment (e.g. [17-20]) are the main steps of such frameworks. A number of studies has been reported in the literature over the last decade focusing on economic loss assessment of building structures within the context of PBEE including, for example, studies on reinforced concrete [18, 19, 21-24], wood [25, 26] and steel frame [20, 27] buildings. Additional studies have focused on earthquake loss estimation methodologies, parameter sensitivity studies, or both for risk management and mitigation practices (e.g. [28-33]).

Rigid Wall Flexible wood roof Diaphragm (RWFD) buildings (commonly referred to as “big-box” structures) are one of the most prevalent construction types for warehouse, retail, and industrial facilities, in regions of high seismicity in the United States. Photos of typical RWFD building construction in the United States are shown in Figure 1. These types of buildings have shown poor seismic performance during past earthquakes, including the 1964 Alaska, 1971 San Fernando, 1987 Whittier Narrows, 1989 Loma Prieta, and 1994 Northridge earthquakes [34-36], resulting in significant economic losses and operation disruptions. These types of buildings are characterized by rigid in-plane walls and flexible in-plane roof diaphragms. Studies by Koliou et al. [37] showed that current US code provisions do not satisfy collapse prevention objectives for RWFD structures under maximum considered earthquake ground motions. This is mainly because design provisions are based on assumed yielding of the walls rather than yielding of the flexible roof diaphragm. To address this issue, an alternative seismic design approach was introduced and evaluated under extreme ground shaking by Koliou et al. [38] and adopted as a FEMA design guideline [39] for RWFD buildings with wood roof diaphragms, which are typically encountered on the west coast of the US.

(a)

(b)

(c)

(d)

Figure 1: (a) and (b) Photographs of typical RWFD building construction process in the US (photo credit: J. Lawson), (c) and (d) photographs of typical RWFD commercial buildings (photo credit: J.W. van de Lindt)

The scope of this study is to quantify the expected earthquake-induced economic losses in RWFD buildings through probabilistic loss estimation and disaggregation. The expected losses due to collapse and repair of the buildings were considered along with inventory losses accounting for the response of nonstructural components (i.e. steel storage racks), given their economic significance for the functionality of RWFD structures. Furthermore, the estimated economic impact of the proposed seismic design [38, 39] was evaluated and compared with conventional US design provisions to demonstrate the efficiency of the proposed design approach. RWFD buildings that host steel storage racks and are commonly used for commercial and retail products (e.g. Home Depot, Costco, and Walmart) are the primary focus of this study.

2. Overview of alternative seismic design approach

An alternative seismic design approach for RWFD buildings that incorporate rigid in-plane tilt-up walls and flexible wood roof diaphragms was adopted in the FEMA P1026 document [38, 39] as an alternative design methodology to explicitly account for the roof diaphragm flexibility in the design procedure. The proposed approach introduces a specific response modification coefficient (R-factor) in the roof diaphragm design (R_{dia}) along with a separate R factor that is currently used in U.S. seismic provisions for the design of the vertical elements of the seismic force-resisting system (SFRS). Based on analytical studies by Koliou et al. [40] and previous post-earthquake observations, typical failure modes denote concentrated damage at the roof diaphragm edge, therefore the alternative seismic design approach facilitates the distribution of the roof diaphragm yielding towards the roof center/mid-span. In a force base design sense, this concept was introduced with a response modification factor R_{dia} of 4.5 for the design of the flexible roof diaphragm along with an amplification factor of 1.5 for the roof shear forces on a distance of 10% of the diaphragm span from both side edges (see Figure 2). Furthermore, a semi-empirical fundamental period (T_1) that takes into account diaphragm response was introduced in this approach for RWFD buildings incorporating wood roof diaphragms and perimeter concrete shear walls as follows [38]:

$$T_1 = \frac{0.0019}{\sqrt{C_w}} h + 0.0020L \quad (1)$$

where, L is the roof diaphragm span in ft. (1 m= 3.28 ft.), h is the height of the in-plane shear walls in ft. (1 m= 3.28 ft.), and C_w is calculated according to equation (2) [41] .

$$C_w = \frac{100}{A_B} \sum_{i=1}^x \left[\frac{A_i}{1 + 0.83 \left(\frac{h_i}{D_i} \right)^2} \right] \quad (2)$$

where, A_B is the area of the base of the buildings in ft^2 ($1 \text{ m}^2 = 10.76 \text{ ft}^2$), A_i is the web area of the in-plane shear wall i in ft^2 ($1 \text{ m}^2 = 10.76 \text{ ft}^2$), D_i is the length of the in-plane shear wall i in ft. ($1 \text{ m} = 3.28 \text{ ft.}$), h_i is the height of the in-plane shear wall i in ft. ($1 \text{ m} = 3.28 \text{ ft.}$), and x is the number of in-plane shear walls.

The proposed alternative design methodology (FEMA P1026) was evaluated to satisfy collapse prevention objectives and it was found that it considerably improved the performance of RWFD buildings compared to current US codes and standards (ASCE 7-10, 2012 IBC) [38].

Figure 2: Roof diaphragm design shear forces for alternative seismic design approach – FEMA P1026

3. Overview of loss assessment methodology

To assess the vulnerability of RWFD buildings, the PBEE framework established by the PEER center [1] was used in this study. The PEER-PBEE framework consists of four analysis steps including seismic hazard characterization, structural analysis, damage analysis, and loss assessment, which are mathematically related by the total probability theorem, resulting into the mean annual rate of the decision variable ($G[DV]$):

$$G[DV] = \iiint G[DV | DM] \cdot f[DM | EDP] \cdot f[EDP | IM] \cdot f[IM] dDM \cdot dEDP \cdot dIM \quad (3)$$

where, $G[X/Y]$ represents the complementary cumulative distribution function of random variable X conditioned on a random variable Y , $f[X/Y]$ represents the probability density function

of random variable X conditioned on a random variable Y , IM denotes the intensity measure, EDP denotes the engineering demand parameter, DM denotes the damage measure, and DV denotes the decision variable.

The four analysis steps of the PBEE framework are briefly discussed below:

- Step 1: Hazard Characterization

Hazard characterization involves a probabilistic seismic hazard analysis (PSHA) in order to determine the mean annual rate of exceedance of the ground motion IMs for a specified building site, building geometry, uncertainties in earthquake size, and distances to the faults. In this study, the spectral acceleration at the fundamental period of the building ($S_a[T_1]$) was the IM used.

- Step 2: Structural Analysis

In this step, nonlinear time history analyses are conducted for selected ground motions (Step 1) to compute the conditional probabilities for the $EDPs$ at increasing levels for ground motion intensities ($f[EDP/IM]$). Incremental Dynamic Analyses (IDAs) are conducted as part of this step. The $EDPs$ considered in this study were the roof diaphragm drift ratio (DDR) for the structural response of the RWFD buildings and the peak ground acceleration (PGA) for the steel storage racks. The DDR is defined as the displacement at the center of the flexible roof diaphragm divided by half of the roof diaphragm length [42].

- Step 3: Damage Analysis

Damage analysis includes the development of fragility functions to probabilistically describe the damage to the building and building components, defined by DMs , as a function of $EDPs$ ($f[DM/EDP]$). In this study, the structural and nonstructural components were identified and grouped based on their type (i.e. roof, storage racks), and relevant $EDPs$.

- Step 4: Loss Assessment

In the last step of the PBEE framework, the structural and nonstructural components DMs are related to the decision variables (DVs), such as the replacement cost, downtime and loss of life, through probabilistic loss models ($G [DV/DM]$). Decision variables associated with economic losses (L_T) related to repair and collapse costs were considered herein.

3.1. Expected loss calculations

The expected economic losses (L_T) are the DV for this study, and are computed accounting for losses due to collapse and repair of the building. Repair losses account for structural damage at the wood roof diaphragm based on predefined damage states as well as inventory losses. The inventory losses are significant for the functionality of RWFD structures and are related to the structural performance of the steel storage racks. Mutually exclusive and collectively exhaustive events of building collapse (C) and no-collapse (NC) at a given seismic intensity were assumed.

4. Monte Carlo simulation

The annual economic losses (L_T), using equation (3), can be computed using different methods, including closed-form solutions [43], direct integration methods [44] as well as stochastic simulations (i.e. Assembly Based Vulnerability –ABV– analysis) [45]. In this study, a multilayer Monte Carlo Simulation (MCS) method [46] was used to conduct the loss assessment study of RWFD buildings designed with different methodologies. A flowchart of the multilayer MCS method considered in this study for one-year interval is presented in Figure 3, which accounts for the uncertainties from all phases of the PEER-PBEE framework. Each analysis phase was conducted in two steps: (i) sample generation step of random parameters with known probability distribution functions, which are essential to describe the uncertainties in analysis techniques and cost estimates, and (ii) analysis step based on deterministic models, used to transfer the uncertainties from input to output parameters at each analysis phase. To accurately

162 evaluate the total economic losses, 100,000 Monte Carlo samples were considered for all
 163 analyses conducted in this study.

164

165 **Figure 3: Multilayer MCS approach for probabilistic seismic loss estimation of RWFD buildings**

166 Two different seismic hazard levels (*IM*) were considered for the loss assessment: (i)
 167 Maximum Considered Earthquake – MCE and (ii) Design Earthquake – DE, which is defined as
 168 2/3 of MCE per ASCE 7. Furthermore, per ASCE 41, MCE hazard level corresponds to a 2% in
 169 50 years probability of exceedance, while the DE level is associated with approximately 10% in
 170 50 years probability of exceedance. The design spectra for these hazard levels per ASCE 41-13
 171 [47] are presented in Figure 4 for high seismic zone ($S_{D1}=0.6$ and $S_{D5}=1.0$), site class D and 2%
 172 viscous damping ratio. Step 1 of the PEER-PBEE methodology was not applied in this study
 173 since only two representative seismic hazard levels (DE and MCE) were considered to evaluate
 174 the total building losses. Furthermore, response analyses using nonlinear models were conducted
 175 (described later in this paper) to compute the conditional probability of exceeding a certain
 176 damage state given an IM ($f[DM|IM]$), combining steps 2 and 3 of the PEER-PBEE framework.
 177 Therefore, Equation (3) is modified as follows for this study.

$$G[DV | IM] = \int G[DV | DM] \cdot f[DM | IM] dDM \quad (4)$$

For each IM , either collapse or no-collapse was randomly simulated based on the conditional probability of collapse, computed through Incremental Dynamic Analysis (IDA) [48] for the RWFD building models. In the case of collapse, the total economic loss was randomly generated according to the probability distribution of the building replacement cost plus costs associated with the building's demolition and debris removal from the site. If there is no-collapse, for each damage component, a damage state (DM) given an IM was randomly generated using the fragility curves for each DM (see Figure 5). This step is followed by the generation of the repair cost according to its probability distribution function, while, the total loss for each component (L_j) was computed by the sum of all the simulated component repair costs. Repeating this process for all components included in the analyses, the total economic loss (L_T) was then computed.

Figure 4: Response spectra for the seismic hazard considered in the study per ASCE 41-13

Figure 5: Conditional probability of damage state given the Intensity Measure (IM)

5. Building archetypes

A set of eight building archetypes were used to perform the loss assessment study of RWFD buildings that incorporate wood roof diaphragms varying in aspect ratio and plan dimensions. The building archetypes were selected to represent seismic design category $D_{\max}$ ($S_{D1} = 0.6$ and $S_{DS} = 1.0$) and risk category II [41]. Single-story concrete tilt-up buildings 9.14 m (30ft) tall, 7.62 m (25ft) wide, and 0.24 m (9.25inches) thick incorporating a 0.91 m (3ft) parapet were considered. The horizontal element of SFRS (roof diaphragm) were sheathed with 12mm (7/16") thick oriented strand board (OSB), nailed with 10d common nails (3.76mm x 50.8mm – 0.148" x 2") to wood nailers and fastened to open-web steel joists. Two different designs for the considered archetypes were developed based on: (i) current US seismic codes and provisions (ASCE 7-10) and (ii) the alternative seismic design approach described earlier (FEMA P1026). Following the current US design provisions, the building archetypes were designed with the vertical elements of the SFRS as intermediate precast shear walls for a response modification factor of $R = 4.0$. The archetypes were designed according to 2012 IBC [49], ASCE 7-10 [41], 2008 SDPWS [50] and ACI 318-11 [51]. The re-designed building archetypes per FEMA P1026 also followed the US design codes but additionally incorporated the alternative approach of designing for an $R = 4.0$ the vertical SFRS elements (tilt-up walls) and for an R_{dia} factor of 4.5

the roof diaphragm. The roof diaphragm design shear forces were further amplified by a factor of 1.5 on a distance of 10% of the diaphragm span from both side edges. Moreover, the FEMA P1026-designed archetypes were designed for a fundamental period (see equation (1)), which accounts for the deformations of the flexible roof diaphragm and the rigid-in-plane wall panels. The design of all archetypes are presented in detail in Koliou [52], while a summary of their characteristics is provided in Table 1.

Table 1: Summary of building archetypes characteristics considered in the study

Archetype ID	Footprint (m)	Building size	Design methodology	Reference ID in [39, 52]
1-ED	121.9 x 121.9	Large	ASCE 7-10	HWL_11_N_OSB_RW4_01
2-ED	121.9 x 61.0	Large	ASCE 7-10	HWL_21_N_OSB_RW4_07
3-ED	61.0 x 30.5	Small	ASCE 7-10	HWS_21_N_OSB_RW4_01
4-ED	30.5 x 30.5	Small	ASCE 7-10	HWS_11_N_OSB_RW4_01
1-ND	121.9 x 121.9	Large	FEMA P1026	HWL_11_N_OSB_RD4.5-1.5_01
2-ND	121.9 x 61.0	Large	FEMA P1026	HWL_21_N_OSB_RD4.5-1.5_01
3-ND	61.0 x 30.5	Small	FEMA P1026	HWS_21_N_OSB_RD4.5-1.5_01
4-ND	30.5 x 30.5	Small	FEMA P1026	HWS_11_N_OSB_RD4.5-1.5_01

ED=Design Existing; ND=Design New

6. Cost distribution functions

The cost distributions for collapse and repair were computed using values from the HAZUS MH manual [53] guideline as well as the RS Means Cost Estimating Manuals [54]. The losses due to the building collapse were calculated accounting for the total building replacement cost plus additional costs related to demolition and debris removal from the site, equal to 10% of the total building replacement cost [18]. The replacement cost values were computed per square meter and are tabulated in Table 2.

The structural repair costs were generated based on a lognormal distribution, with a mean value given by a percentage of the total building replacement cost for each damage state (highest loss value provided by the HAZUS-MH manual) and coefficient of variation (COV) values equal to 0.1 for all damage states [46, 55]. The structural repair cost for the RWFD structures per

HAZUS-MH manual were accounted as 0.6%, 3.2%, 16.2% and 32.4% of the total building replacement cost for *Slight*, *Moderate*, *Extensive*, and *Complete* damage states, respectively.

Similar to the structural repair costs, the losses arising from inventory damage were estimated using the HAZUS-MH approach and utilizing empirical functions that express the contents/inventory cost for acceleration sensitive nonstructural components (i.e. storage racks) equal to 5%, 25% and 50% of the total building replacement cost for *Moderate*, *Extensive*, and *Complete* damage states, respectively. The contents loss was sampled from a lognormal distribution with mean value equal to the highest loss estimate computed from the HAZUS-MH manual and COV values of 0.1 for all damage states.

Note that the median values and COVs of the cost distributions were logically selected guided from previous studies [55]. Furthermore, a sensitivity study was conducted on COV values varying between 0.1 and 0.3, and it was observed that the loss estimations were not significantly influenced.

Table 2: Replacement cost estimates for the building archetypes studied

Footprint (m)	Building size	Archetype IDs	Total floor area (m ²)	Total replacement cost	Cost per m ²
121.9 x 121.9	Large	1-ED & 1-ND	14,864	\$9,052,800	\$609.04
121.9 x 61.0	Large	2-ED & 2-ND	7,432	\$4,526,400	\$609.04
61.0 x 30.5	Small	3-ED & 3-ND	1,858	\$1,408,600	\$758.13
30.5 x 30.5	Small	4-ED & 4-ND	929	\$704,300	\$758.13

7. Nonlinear simulation models and structural analysis

The ability to accurately capture the structural response of RWFD buildings is crucial in estimating the associated losses. A three step sub-structuring based numerical framework, developed by Koliou et al. [40, 56], was used in this study for nonlinear response analyses of the building archetypes. The numerical framework, as schematically shown in Figure 6, includes the following three steps: (i) development of roof connector database and identification of the Wayne-Stewart [57] hysteretic parameters for each connector type, (ii) development of an

analytical roof diaphragm model in MATLAB [58] accounting for the shear deformation of the roof deck as well as the nonlinear response of the roof connectors (using Wayne-Stewart hysteretic springs) and (iii) development of a simplified building model in RUAUMOKO [59] incorporating roof diaphragm inelastic springs with properties identified in Step 2. A detailed description of the numerical framework is provided in Koliou et al. [40, 56] and Koliou [52].

Figure 6: Three step sub-structure modeling framework used for nonlinear time history analyses

Nonlinear dynamic response analyses were conducted for the building archetypes under the FEMA P695 Far-Field Ground motion ensemble [60] using the three step numerical framework. The FEMA P695 ground motion ensemble includes 22 ground motions, two components each, from all over the world to be representative of the seismicity of the Western United States. To estimate the probability of exceeding a certain limit state for the RWFD buildings, each ground motion was scaled to increasing earthquake intensities (IDAs) [48], starting with a scale factor of 0.1 and increment of 0.1 until complete collapse is achieved. Based on the IDA results, fragility curves for each damage state (including collapse) were generated as a function of the seismic intensity. The median intensity value was defined as the median 2% damped spectral acceleration at the fundamental period of the building archetype for which 50% of the earthquake motions exceed a certain limit state. The roof diaphragm drift ratio (*DDR*) [37, 42, 61] was considered in this study as the representative seismic EDP for the RWFD buildings and the limit states (for each damage state) were defined in terms of the *DDR*. Note that the lateral displacements at the top of the in-plane walls are negligible compared to the displacement of the

roof diaphragm at the mid-span [37]; therefore for practical purposes, DDR is equal to the lateral building drift at the center of the roof.

Furthermore, in order to account for the inventory losses, the response of the steel storage racks which are often found in these types of buildings was considered in this study based on empirical fragility curves generated by Eidinger [62] (i.e. nonlinear response analyses were not conducted for storage racks in this study). A summary of the damage states, in terms of physical damage, and their respective limit states for each component considered in this study (roof diaphragm and storage racks) are presented in Table 3. The physical damage to the roof diaphragm was selected based on literature documents [53] associating physical wood roof damage to damage states. The limit state values for DDR for each damage state were selected based on numerical analyses results on a large set of RWFD building archetypes [37]. The physical description and limit state values for steel storage racks were selected based on the study by Eidinger [62].

Table 3: Summary of damage and limit states

Assembly Description	Damage State Description	Physical Damage Description	EDP used*	Limit States Description
Flexible roof diaphragm	<i>Slight</i>	Some visible damage in the form of nail slippage and some nail withdrawal toward the center of the roof.	DDR (unitless)	$0.001 < DDR < 0.005$
	<i>Moderate</i>	Some nails connecting blocking elements and nailers partial withdrawal in the roof diaphragm corners.		$0.005 < DDR < 0.010$
	<i>Extensive</i>	Several nails withdrawal from connecting joists, several splitting of sheathing boards, several nails tear through the ends of connecting sheathing boards.		$0.010 < DDR < 0.020$
	<i>Complete</i>	Possible large permanent diaphragm deformation resulting to collapse of out-of-plane walls due to failed out-of-plane roof-to-wall anchorages.		$DDR > 0.020$
Steel storage	<i>Moderate</i>	Loose contents slide to floor	PGA (g)	$0.30 < PGA < 0.70$
	<i>Extensive</i>	Palletized contents slide to floor		$0.70 < PGA < 0.90$

racks	<i>Complete</i>	Structural collapse		PGA>0.90
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*DDR=roof diaphragm drift ratio; PGA=peak ground acceleration (g)

8. Loss estimation and disaggregation

Figure 7 plots, in a semi-logarithmic scale, the annual probabilities of exceedance of the total economic loss for the RWFD building archetypes designed under the two methodologies, i.e. ASCE 7-10 and FEMA P1026, for the two different hazard scenarios considered in this study. The results are presented in the form of normalized total loss. Table 4 summarizes the expected annual losses (EAL) and the associated standard deviation of the losses (SDL) for each building archetype at both seismic intensities. The EAL and SDL values are describing the median and standard deviation values of an empirical cumulative distribution function fitted to the normalized total loss curves presented in Figure 7.

Based on the results presented in Figure 7, it is observed that the FEMA P1026 design methodology has a significant economic impact on large RWFD buildings (Figure 7a and b) for both DE and MCE hazard intensities. Reduced mean annual losses (EAL) of the order of 20-26% and 14-17% are identified for DE and MCE intensities, respectively (see Table 4), when the FEMA P1026 design is adopted compared to ASCE 7-10 design associated losses.

Similarly, for the smaller building archetypes (Figure 7c), considerably smaller economic losses are associated with the FEMA P1026 design for DE hazard intensities with an EAL reduction of 17-19%. However, a smaller amount of loss reduction was found for MCE intensities with an associated EAL reduction of less than 10%. This result suggests that for the smaller RWFD buildings at MCE intensities, the losses associated with structural damage (affected by the new design) are not significant compared to the inventory losses. This is also verified by the loss disaggregation plot of Figure 8. Note that the effect of the FEMA P1026 design on the structural collapse performance of the smaller archetypes is considerably less compared to the larger footprint archetypes [38]. The findings of this study on the loss

disaggregation (i.e. contents and structural losses) align well with observations from previous strong earthquakes (e.g. 1994 Northridge earthquake) in the United States [63].

Figure 9 shows the building loss curves (archetypes with plan view 121.9 x 121.9 m) computed for no-collapse and collapse cases with the total normalized loss curve for designs incorporating both ASCE 7-10 and FEMA P1026 methodologies (MCE seismic intensity). It is observed that for normalized loss of 0.1 ($\approx$ \$1 million), the contribution of no-collapse losses is more than the collapse losses, however, this changes for normalized losses larger than 0.2 ($\approx$ \$2 million) for both design methodologies. This is mainly attributed to the fact that in higher seismic intensities, the collapse damage state is associated with high losses resulting from contents damage (loss), which appear to be higher than the losses resulting from structural damage (see Figure 8). It is also observed from Figure 9 that the collapse losses associated with the FEMA P1026 design are smaller compared to the collapse losses of the archetype designed per ASCE 7-10.

Based on the results presented in this paper, it is shown that the FEMA P1026 design methodology can result in significant savings for RWFD buildings incorporating wood roofs compared to ASCE 7-10 design, therefore, it is an effective design solution both for the building structural response [38] and economic losses.

Figure 7: Annual probabilities of loss exceedance for: (a) large archetypes - DE, (b) large archetypes - MCE, (c) small archetypes - DE and (d) small archetypes - MCE

Table 4: Summary of loss analysis findings for all building archetypes

Archetype ID	Building size	Hazard intensity	EAL (\$)	SDL (\$)	Design methodology
1-ED	Large	DE	\$3,231,685	\$2,086,091	ASCE 7-10
		MCE	\$4,745,944	\$2,344,468	
2-ED	Large	DE	\$1,582,911	\$1,040,619	ASCE 7-10
		MCE	\$1,848,141	\$1,148,228	
3-ED	Small	DE	\$638,199	\$384,580	ASCE 7-10
		MCE	\$884,681	\$367,764	
4-ED	Small	DE	\$291,407	\$191,355	ASCE 7-10
		MCE	\$475,609	\$184,270	
1-ND	Large	DE	\$2,578,200	\$1,958,843	FEMA P1026
		MCE	\$4,057,409	\$2,190,936	
2-ND	Large	DE	\$1,168,465	\$948,744	FEMA P1026
		MCE	\$1,518,486	\$1,087,713	
3-ND	Small	DE	\$511,657	\$361,369	FEMA P1026
		MCE	\$923,735	\$358,127	
4-ND	Small	DE	\$241,515	\$177,276	FEMA P1026
		MCE	\$442,810	\$171,645

Figure 8: Disaggregation of the annual probabilities of loss exceedance estimated from structural loss and contents loss for archetype 4-ED at MCE seismic intensity

Figure 9: Disaggregation of the annual probabilities of loss exceedance estimated from no-collapse and collapse losses at MCE seismic intensity for archetype: (a) 1-ED and (b) 1-ND

9. Conclusions

In this study, the PEER performance-based earthquake engineering methodology was used to estimate the expected economic losses for Rigid Wall Flexible wood roof Diaphragm buildings for two seismic hazard intensities, i.e. MCE and DE. The main focus of this study was to: (a) compare the expected losses for RWFD buildings designed with current US codes (ASCE 7-10) and with an alternative seismic design methodology (FEMA P1026) and (b) evaluate the

financial advantage of the FEMA P1026 approach. A set of eight building archetypes, incorporating flexible wood roof diaphragms and tilt-up wall panels, were designed with both methodologies for a high seismic location in the United States. The total Expected Annual Losses (EAL) were computed through Monte Carlo Simulation to account for the uncertainties associated with the damage and cost distributions. Two major components contributed to the estimation of the total losses: (i) damage to the flexible wood roof diaphragm (main seismic failure mode) and (ii) damage to steel storage racks, which controls the inventory losses. Inventory losses were included in this study given their economic significance in the functionality of businesses housed in RWFD structures.

The annual probabilities of exceedance of the loss for each building archetype for the two hazard scenarios were computed. It was found that the seismic-induced loss of RWFD buildings was significantly reduced when the FEMA P1026 methodology was adopted in the design process. Large archetypes exhibited an EAL of 20-26% and 14-17% for DE and MCE hazard intensities, respectively, while the smaller RWFD buildings had an EAL of 17-19% and 4-6% for DE and MCE hazard intensities, respectively. It should be noted that the less improved performance of smaller archetypes is also well aligned with the results of their collapse performance found in a previous study by the first and third authors and their co-investigators. Loss disaggregation results were also presented in this study, indicating that at higher intensities (MCE), the collapse losses contribute more than the no-collapse losses for both designs. However, the application of the alternative FEMA P1026 design methodology resulted in reduced collapse losses compared to ASCE 7-10 designs.

It is concluded that the alternative design approach (FEMA P1026) for RWFD buildings incorporating wood roofs, not only improves their collapse response, but is also a cost-efficient solution for businesses housed in these structures and located in high seismic regions.

Acknowledgments

The first and second authors acknowledge the support of cooperative agreement 70NANB15H044 between the National Institute of Standards and Technology (NIST) and Colorado State University. The contents expressed in this paper are the views of the authors and do not necessarily represent the opinions or views of NIST. Thanks are due to Dr. Vipin U. Unnikrishnan for his helpful input regarding the probabilistic analytical studies presented herein. Mr. Dominic J. Kelly and Professor John Lawson are also acknowledged for their contributions with the first and third author in the development of the alternative seismic design methodology for rigid wall flexible diaphragm buildings discussed in this paper.

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