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ABSTRACT

The seismic vulnerability functions for portfolio-level loss estimation are typically developed for general classes of buildings which may not be suitable to assess building-specific risks. Performance-based earthquake engineering (PBEE) provides the means to conduct building-specific seismic risk assessments. However, such assessments often rely on computationally-intensive analytical frameworks such as incremental dynamics analysis (IDA) which poses a challenge for many types of risk assessment projects. To expand its accessibility, FEMA P-58 outlines a simplified method to predict the nonlinear responses of buildings in which the scope is limited to lower levels of inter-story drifts (less than 4%). This limitation restricts its application to ductile structures, particularly when predicting the vulnerability of modern special moment frame systems. To overcome this shortcoming, this paper proposes an enhanced methodology by which the nonlinear responses of some common structural systems can be predicted by interpolating from a structural response database, itself developed by IDA. The database adopted in the current study consists of structural responses of 61 distinct modern buildings with variety of heights (number of stories), construction material, and lateral load resisting systems. Two building reference models, light-wood frame and special reinforced concrete moment frame with varying

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heights, are selected to validate the performance of the proposed statistical method. The predicted structural responses for these buildings are benchmarked against the corresponding IDA results. The estimated vulnerability of buildings based on the enhanced simplified method is in good agreement with IDA results. The proposed framework can be used in expedited seismic risk evaluations to estimate the losses of buildings in a large portfolio of diverse structures.

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The seismic vulnerability functions for portfolio-level loss estimation are typically developed for general classes of buildings which may not be suitable to assess building-specific risks. Performance-based earthquake engineering (PBEE) provides the means to conduct building-specific seismic risk assessments. However, such assessments often rely on computationally-intensive analytical frameworks such as incremental dynamics analysis (IDA) which poses a challenge for many types of risk assessment projects. To expand its accessibility, FEMA P-58 outlines a simplified method to predict the nonlinear responses of buildings in which the scope is limited to lower levels of inter-story drifts (less than 4%). This limitation restricts its application to ductile structures, particularly when predicting the vulnerability of modern special moment frame systems. To overcome this shortcoming, this paper proposes an enhanced methodology by which the nonlinear responses of some common structural systems can be predicted by interpolating from a structural response database, itself developed by IDA. The database adopted in the current study consists of structural responses of 61 distinct modern buildings with variety of heights (number of stories), construction material, and lateral load resisting systems. Two building reference models, light-wood frame and special reinforced concrete moment frame with varying heights, are selected to validate the performance of the proposed statistical method. The predicted structural responses for these buildings are benchmarked against the corresponding IDA results. The estimated

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vulnerability of buildings based on the enhanced simplified method is in good agreement with IDA results. The proposed framework can be used in expedited seismic risk evaluations to estimate the losses of buildings in a large portfolio of diverse structures.

Introduction

Among natural hazards that affect the United States, earthquake has remained as one of the most devastating large-scale catastrophes. The 1989 Loma Prieta earthquake followed by the 1994 Northridge earthquake in highly urbanized regions of California warned the stakeholders on the scale of the financial damage to the urban communities. In such a context, catastrophe modeling (or in short, CAT modeling) stands out as one of the main tools in hands of risk analysts, enabling them to quantify the risk which in turn is used to make informed decisions on preparation and mitigation. In recent decades, catastrophe models have evolved and proliferated deeply into private and public sectors to provide more analytical, engineering-based procedures in risk management. Analytical approaches have particularly influenced the vulnerability evaluation framework in CAT models which estimates the conditional damage to structures given a measure of ground motion intensity. The FEMA P-58 framework [1], derived from the procedures developed by the Pacific Earthquake Engineering Research Center (PEER) and the Applied Technology Council (ATC), has standardized the methodology to derive vulnerability functions for different classes of building assets. This methodology substitutes the conventional nonlinear static analysis (a.k.a. pushover analysis) with incremental dynamic analysis (IDA) to quantify the conditional response of the buildings to ground shaking as the main source of aleatory uncertainty. Despite its many advantages, the computationally-intensive IDA does not suit the limited resources available to many types of risk assessment projects.

To expedite the risk assessment process, FEMA P-58 also proposes a simplified approach which only applies to situations where the story drift is less than 4%, below which the P-delta effect is usually less significant. This limitation restricts the application of FEMA P-58's simplified method especially for ductile buildings. This paper proposes a new methodology to overcome this limitation through using a developed database of IDA-based structural responses and interpolation within them. This database includes records of structural responses from 61 different building models with various construction materials, number of stories, and lateral load resisting systems to re-calibrate the FEMA P-58's parameters and estimate seismic responses for new building types which do not exist in the IDA-based database. Following the development of the methodology, four reference models are selected to validate its performance with varying construction type and heights: two light frame wood and two special reinforced concrete moment frames. The structural responses of the test buildings are compared against the results of the IDA analyses. Good agreement between the outcomes of the enhanced simplified method and the IDA-based method enables expedited risk assessment by employing the enhanced simplified method to estimate the vulnerability of many buildings in a large portfolio. Next section reviews the FEMA P-58 simplified method followed by a description of the proposed methodology.

Review of the FEMA P-58 Simplified Method

FEMA P-58's simplified method estimates the median values of the building's engineering demand parameters. The seismic responses are assumed to be independent in each horizontal axis, and the building is assumed to be regular in plan and elevation and less than 15 stories tall. Furthermore, the story drift ratios cannot be larger than four times the corresponding yield drift ratio, and the story drift should not exceed 4%. These assumptions limit the application of the P-58 simplified methodology to many buildings such as ductile buildings with large drift capacity. Even for buildings which satisfy these assumptions, the demands generated by the simplified method have larger uncertainties compared to the IDA analysis results.

Simplified Analysis Procedure

To calculate the lateral strength of the building which is one of the input variables in the simplified methodology, a static analysis on a linear model is performed. The derivation of the engineering demand parameters in the simplified method uses the floor, story, and height numbering system shown in Figure 1.

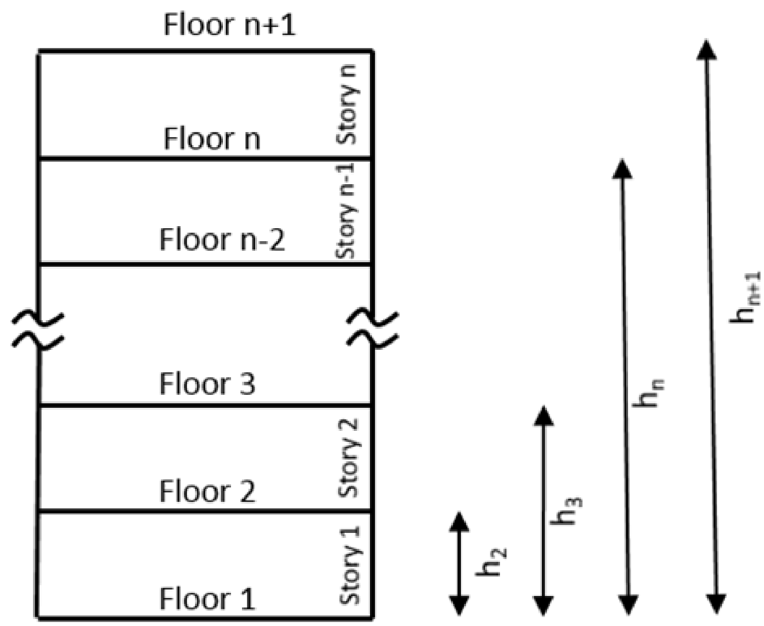

Figure 1. Definition of the floor, story, and height numberings

The base shear in each direction results from a pseudo lateral force due to earthquake shaking can be approximated as follows:

$$V = C_1 C_2 S_a(T_1) W_1 \tag{1}$$

where C_1 is the adjustment factor for inelastic displacements, C_2 is the adjustment factor for cyclic degradation; $S_a(T_1)$ is the 5% damped spectral acceleration at the fundamental period of the building at the selected level of ground motion, and W_1 is the first modal effective weight, defined as:

$$W_1 = \frac{(\sum_{j=2}^{n+1} W_j \emptyset_{j1})^2}{\sum_{j=2}^{n+1} W_j \emptyset_{j1}^2} \tag{2}$$

where w_j is the lumped weight at floor level j and $\emptyset_{j1}$ is the j^{th} floor ordinate of the first mode deflected shape. W_1 should be larger than 80% of the total building weight and can alternatively be calculated as $C_m W$, where C_m is as defined in ASCE/SEI 41-13 and shown in Table 1 [2] and W is the total weight of the building:

C_1 and C_2 are defined as follows:

$$\begin{aligned} C_1 &= 1 + \frac{S-1}{0.04a} \quad for\ T_1 \le 0.2sec \\ &= 1 + \frac{S-1}{a+T_1^2} \quad for\ 0.2 < T_1 \le 1.0sec \\ &= 1 \qquad for\ T_1 > 1.0sec \end{aligned} \tag{3}$$

$$\begin{aligned} C_2 &= 1 + \frac{(S-1)^2}{32} \quad for\ T_1 \le 0.2sec \\ &= 1 + \frac{(S-1)^2}{800T_1^2} \quad for\ 0.2 < T_1 \le 0.7sec \\ &= 1 \qquad for\ T_1 > 0.7sec \end{aligned} \tag{4}$$

where a is taken 130, 130, 90, 60, and 60 for ASCE/SEI 7-10 [3] site classes A, B, C, D and E, respectively, and S is the strength ratio calculated as:

$$S = \frac{S_a(T_1)W}{V_{y1}} \tag{5}$$

where V_{y1} is the estimated yield strength of the building in its first mode and is extracted from the HAZUS-MH MR4 technical manual in this framework. C_1 and C_2 are taken as 1 when S is less than 1.

Table 1. Values of Effective Mass Factor C_m.

No. of Stories	Concrete Moment Frame	Concrete Shear Wall	Concrete Pier-Spandrel	Steel Moment Frame	Steel Concentrically Braced Frame	Steel Eccentrically Braced Frame	Other
1–2	1	1	1	1	1	1	1
3 or more	0.9	0.8	0.8	0.9	0.9	0.9	1

The pseudo lateral force in x^{th} floor can be calculated as follows:

$$F_x = C_{vx} V \tag{6}$$

where C_{vx} is defined as:

$$C_{vx} = \frac{W_x h_x^k}{\sum_{i=2}^{n+1} W_i h_i^k} \tag{7}$$

where W_i and W_x are the weight of i^{th} and x^{th} floors, respectively, and k is equal to 2.0 for the buildings with first mode period larger than 2.5 sec. and equal to 1.0 for the buildings with first mode period less than 0.5 sec. For building periods between 0.5 and 2.5 seconds, the value of k can be interpolated. The calculated lateral forces (F_x) are applied to each floor, and the Direct

Displacement Design (DDD) methodology proposed by Pang and Rosowsky [4] is used to generate the story stiffness and the inter-story elastic drift, Δ_i, for each floor, with the target drift for each level of shaking extracted from ASCE/SEI 41-13. The inelastic inter-story drift for each floor is calculated as follows:

$$\Delta_i^* = H_{\Delta i}(S, T_1, h_i, H) \times \Delta_i \tag{8}$$

where $H_{\Delta i}(S, T_1, h_i, H)$ is the drift correction factor defined as:

$$\ln(H_{\Delta i}) = a_0 + a_1 T_1 + a_2 S + a_3 \frac{h_{i+1}}{H} + a_4 (\frac{h_{i+1}}{H})^2 + a_5 (\frac{h_{i+1}}{H})^3 \quad for\ S \geq 1, i = 1\ to\ N \tag{9}$$

where H is the total height of the building and T_1 is the first modal period of the structure.

The peak acceleration of the first floor is the peak ground acceleration (PGA), and for the other floors, the floor accelerations are calculated using the following equation:

$$a_i^* = PGA \cdot H_{ai}(S, T_1, h_i, H) \quad for\ i = 2\ to\ N+1 \tag{10}$$

where $H_{ai}(S, T_1, h_i, H)$ is the acceleration correction factor defined as:

$$\ln(H_{ai}) = a_0 + a_1 T_1 + a_2 S + a_3 \frac{h_{i+1}}{H} + a_4 (\frac{h_{i+1}}{H})^2 + a_5 (\frac{h_{i+1}}{H})^3 \quad for\ S \geq 1, i = 2\ to\ N \tag{11}$$

The peak velocity of the first floor is the peak ground velocity (PGV). Peak ground velocity is obtained by dividing the spectral velocity at a period of one second by 1.65 [5, 6]. The spectral velocity at 1 second and the PGV can be calculated as follows:

$$S_V(1.0\text{sec}) = \frac{S_a(1.0\text{sec})}{2\pi} g \tag{12}$$

$$PGV = \frac{S_v(1.0\text{sec})}{1.65} \tag{13}$$

The peak velocities for the other floors are calculated using the following equation:

$$v_i^* = H_{vi}(S, T_1, h_i, H) \times v_{si} \quad for\ i = 2\ to\ N+1 \tag{14}$$

where $H_{vi}(S, T_1, h_i, H)$ is the velocity correction factor defined as:

$$\ln(H_{vi}) = a_0 + a_1 T_1 + a_2 S + a_3 \frac{h_{i+1}}{H} + a_4 (\frac{h_{i+1}}{H})^2 + a_5 (\frac{h_{i+1}}{H})^3 \quad for\ S \geq 1, i = 2\ to\ N \tag{15}$$

The reference floor velocity, v_{si}, is calculated using the following equation:

$$v_{si} = PGV + 0.3 \frac{T_1 V_{y1} \Gamma_1 \delta_i}{2\pi W_1 \delta_r} g \tag{16}$$

where Γ_1 is the first mode participation factor; δ_i and δ_r are the elastic displacement of Floor *i* and the roof, respectively.

Table 2 lists the values for coefficients a_0 through a_5 for all described demand parameters. These values are extracted from FEMA P-58 separately for structures less than 9 stories tall and structures with 10 to 15 stories in height.

The residual drift ratio, Δ_r, are obtained as follows:

$$\begin{aligned} \Delta_r &= 0 && for\ \ \Delta \le \Delta_y \\ \Delta_r &= 0.3(\Delta - \Delta_y) && for\ \ \Delta_y < \Delta\ \le 4\Delta_y \\ \Delta_r &= \Delta - 3\Delta_y && for\ \ \Delta\ \ge 4\Delta_y \end{aligned} \tag{17}$$

where Δ is the median inter-story drift and Δ_y is the yield drift ratio, obtained from the HAZUS-MH MR4 [7] technical manual.

A pseudo IDA analysis utilizing the above formulation and FEMA P-695 22 far-field ground motion and scaling procedure is performed [8]. The pseudo lateral force for each scaled ground motion record at each intensity following by the peak inter-story drifts, accelerations, velocities, and residual drifts of each floor are calculated using the above formulation. The $S_a(T_1)$, PGA, and PGV are computed for different ground motions and shaking intensities to complete the IDA analysis.

Table 2. Correction factors for story drift ratio, acceleration, and velocity in the original simplified method [1].

Demand		Frame Type	a_0	a_1	a_2	a_3	a_4	a_5
1-8 Stories	Story Drift Ratio	Braced	0.900	-0.120	0.012	-2.650	2.090	0
		Moment	0.750	-0.044	-0.010	-2.580	2.300	0
		Wall	0.920	-0.036	-0.058	-2.560	1.390	0
	Floor Velocity	Braced	0.150	-0.100	0	-0.408	0.470	0
		Moment	0.025	-0.068	0.032	-0.530	0.540	0
		Wall	-0.033	-0.085	0.055	-0.520	0.470	0
	Floor Acceleration	Braced	0.660	-0.270	-0.089	0.075	0	0
		Moment	0.660	-0.250	-0.080	-0.039	0	0
		Wall	0.660	-0.150	-0.084	-0.260	0.570	0
9-15 Stories	Story Drift Ratio	Braced	1.910	-0.120	-0.077	-3.780	6.430	-3.420
		Moment	0.670	-0.044	-0.098	-1.370	1.710	-0.570
		Wall	0.860	-0.036	-0.076	-4.580	6.880	-3.240
	Floor Velocity	Braced	0.086	-0.100	0.041	0.450	-2.890	2.570
		Moment	-0.020	-0.068	0.034	0.320	-1.750	1.530
		Wall	-0.110	-0.085	0.110	0.870	-4.070	3.270
	Floor Acceleration	Braced	0.440	-0.270	-0.052	3.240	-9.710	6.830
		Moment	0.340	-0.250	-0.062	2.860	-7.430	5.100
		Wall	-0.130	-0.150	-0.100	7.790	-17.520	11.040

Simplified Method Application

To benchmark the performance of the original P-58 simplified method, 2-story and 4-story light frame wood buildings and 2-story and 12-story concrete moment resisting frame buildings are analyzed. The demands are predicted by the simplified approach, that is, by performing a pushover analysis followed by a pseudo-IDA procedure, as well as by performing full IDA analyses on the detailed models. Figure 2 compares the vulnerability functions derived from the application of the simplified method against those derived from demands predicted by the full IDA analyses using detailed structural models. The comparisons show significant differences between the vulnerability functions obtained by the two procedures particularly in terms of predicting the loss ratios for higher spectral acceleration values. The vulnerability functions derived by P-58's simplified method only predict the loss ratios up to spectral acceleration levels of around 0.4g for the selected buildings. Examining the differences at the demand values identifies main culprit to be the inter-story drift limitation (less than 4%) in the FEMA P-58 simplified method. This limitation is highly restrictive to the applicability of the method to 'ductile' buildings with relatively large displacement capacity.

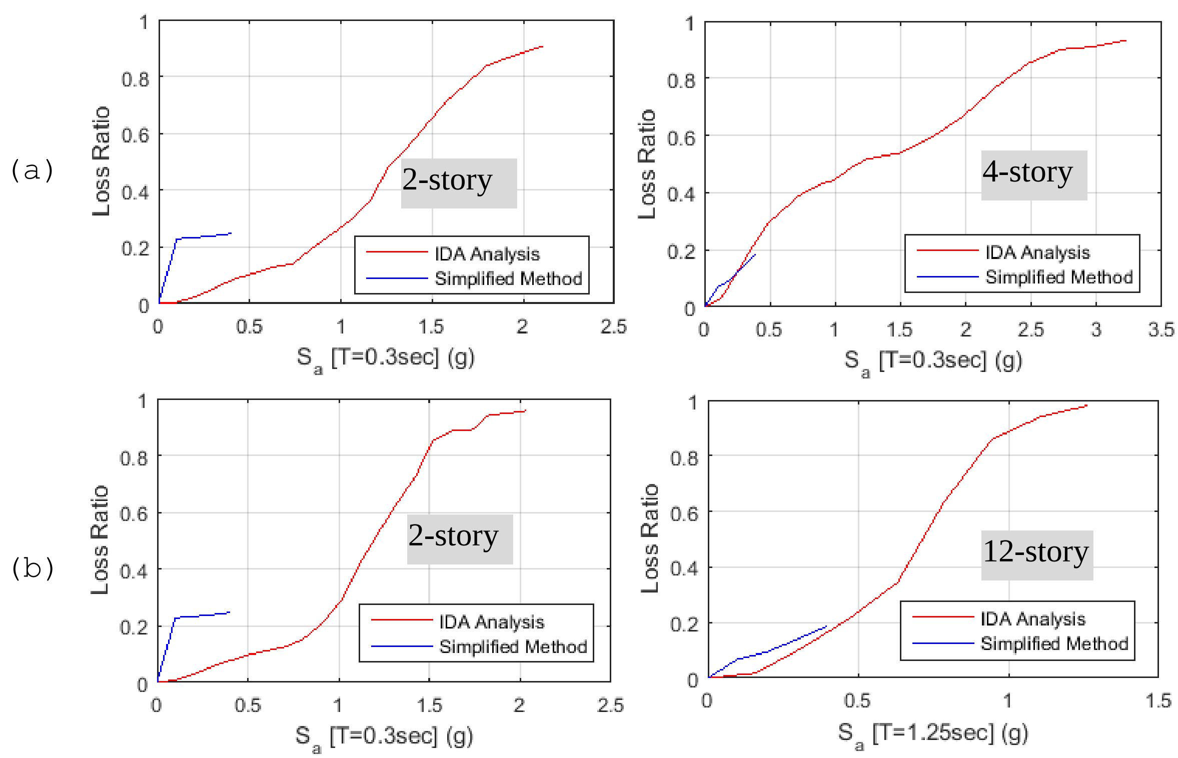

Figure 2. Benchmarking the vulnerability functions derived by the application of the original simplified method against full IDA analysis: (a) light frame wood and (b) reinforced concrete moment resisting frame.

Enhanced Simplified Method

In order to overcome the observed shortcoming for larger seismic demand values, a methodology is proposed by which the nonlinear responses of a given structural system can be predicted by fitting a database of structural responses, which is compiled by full IDA analyses, to a statistical model (i.e. Eqns. 8 to 15).

A database consisting of the structural responses of 61 different reference models with different heights (number of stories), material, and lateral load resisting systems is used to develop statistical regression models for estimating the engineering demand parameters (EDPs). The buildings include 12 concrete moment frame buildings, 27 light frame wood shear wall buildings, 11 modern steel moment frame buildings, and 11 steel braced frame buildings. The same functional forms as in the original P-58 are used to calculate the elastic inter-story drift (Δ_i), peak ground acceleration (PGA), reference floor velocity (v_{si}), and residual inter-story drift (Δ_r). Parameters a_0 through a_5, however, are re-calculated in the improved methodology by fitting a regression model to the existing responses in the available database. The least square algorithm is used to fit the nonlinear peak drift, acceleration, and velocity equations to approximately 100,000 data points, and the modified correction factors are calculated as tabulated in Table 3.

Table 3. Correction factors for story drift ratio, acceleration, and velocity in the enhanced simplified method.

Demand		Frame Type	a_0	a_1	a_2	a_3	a_4	a_5
1-8 Stories	Story Drift Ratio	Steel Braced	1.142	-0.073	-0.030	-2.744	3.123	0.726
		Concrete Moment	1.188	-0.064	-0.013	-3.555	3.218	0.973
		Steel Moment	1.223	-0.075	-0.031	-2.444	3.168	0.873
		Wood Shear Wall	1.345	-0.046	-0.092	-3.735	2.202	1.179
	Floor Velocity	Steel Braced	0.023	-0.070	0.039	-0.724	0.674	0.823
		Concrete Moment	0.039	-0.082	0.044	-0.796	0.774	0.959
		Steel Moment	0.028	-0.079	0.041	-0.754	0.694	0.891
		Wood Shear Wall	-0.052	-0.111	0.074	-0.681	0.606	0.975
	Floor Acceleration	Steel Braced	0.739	-0.256	-0.111	-0.061	0.301	0.837
		Concrete Moment	0.992	-0.310	-0.121	-0.057	0.453	1.042
		Steel Moment	0.892	-0.310	-0.135	-0.073	0.364	1.011
		Wood Shear Wall	0.988	-0.230	-0.107	-0.329	0.799	1.005
9-15 Stories	Story Drift Ratio	Steel Braced	1.238	-0.083	-0.032	-2.698	3.206	0.964
		Concrete Moment	0.961	-0.054	-0.124	-2.055	2.105	-0.775
		Steel Moment	1.345	-0.090	-0.034	-2.933	3.484	1.048
		Wood Shear Wall	N/A	N/A	N/A	N/A	N/A	N/A
	Floor Velocity	Steel Braced	0.029	-0.089	0.042	-0.842	0.710	0.994
		Concrete Moment	-0.026	-0.107	0.048	0.413	-2.774	2.099
		Steel Moment	0.031	-0.095	0.045	-0.905	0.763	1.069
		Wood Shear Wall	N/A	N/A	N/A	N/A	N/A	N/A
	Floor Acceleration	Steel Braced	0.873	-0.331	-0.132	-0.078	0.356	1.080
		Concrete Moment	0.460	-0.347	-0.081	4.379	11.346	6.656
		Steel Moment	0.981	-0.371	-0.148	-0.088	0.400	1.213
		Wood Shear Wall	N/A	N/A	N/A	N/A	N/A	N/A

The enhanced simplified methodology with the modified correction factors is subsequently applied to the same buildings for benchmarking. The vulnerabilities for the modified methodology are evaluated and compared against those obtained from the IDA analyses for the mean, 10% and 90% percentile loss ratios (Figure 3). The vulnerability functions reveal good agreement between the results of the enhanced simplified method and those from the IDA analysis using detailed numerical models. Given that developing the vulnerability functions by the enhanced simplified method requires considerably less computational resources compared to full IDA analyses, Figure 3 suggests a favorable balance can be reached between accuracy and applicability.

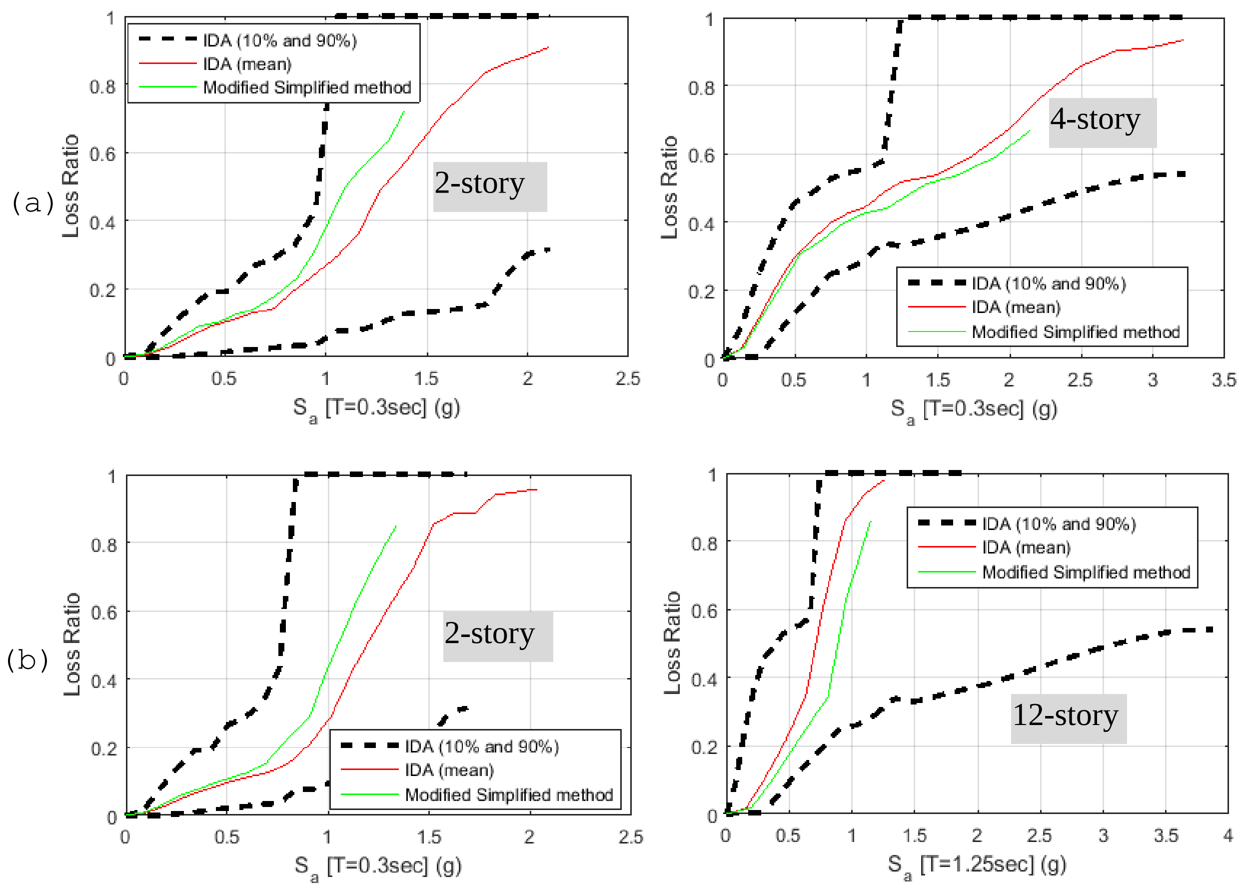

Figure 3. Benchmarking the vulnerability functions derived by the application of the modified simplified method against full IDA analysis: (a) light frame wood and (b) reinforced concrete moment resisting frame.

Conclusion

This research proposes an improvement over FEMA P-58's simplified method for seismic loss assessments. The following conclusion can be drawn:

1. The simplified method proposed by FEMA P-58 restricts its applicability by many assumptions, including the independency in each horizontal axis, regularity in plan and

elevation, number of stories, and the maximum story drift ratios.

2. The limitation on the inter-story drift (less than 4%) has the most significant effect on the demand calculations and the resulting vulnerability functions because this limitation restricts the applicability of the method to ductile structures such as modern special moment resisting frame systems.
3. To evaluate the performance of the FEMA P-58 simplified methodology, 2-story and 4-story light frame wood buildings and 2-story and 12-story concrete moment resisting frame buildings, are used to develop loss functions using both P-58's simplified method and IDA analyses with detailed numerical models. The results reveal that there are noticeable differences between the loss functions developed by the two tracks.
4. To improve the FEMA P-58 simplified method, new regression models are fit to the nonlinear responses of 61 different buildings obtained using IDA analyses on detailed structural models. The modified regression parameters are used to develop the vulnerability functions for the same two buildings which show improved agreement against the vulnerability functions by IDA analyses.
5. The enhanced simplified framework can be used to estimate the losses of buildings in a large portfolio of structures with favorable computation times. The applicability of the proposed modifications to the simplified method is limited to the building classes which were used in this development. Moreover, and since the IDA-based database is set up by analyzing the building reference models against far-field ground motions, the resulting vulnerability functions may not be used for risk assessment studies which are subject to near-field seismic excitations.

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