NOVEL APPROACH TO GENERATE COLLAPSE FRAGILITY CURVES FOR SPECIAL MOMENT RESISTING FRAME SYSTEMS

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Abstract

Various approaches have been proposed previously by many researchers to obtain fragility curves for special moment resisting frame (SMRF) systems at collapse. However, these approaches are either computationally expensive or not very accurate. In addition, these existing methods do not consider the ground motion spectral shape characteristics, and therefore a correction factor needs to be adopted. The present study aims to provide a novel approach for the estimation of collapse capacity of SMRF systems and associated variability due to record-to-record uncertainty. The deformation capacity of the beams and columns is determined using PEER ATC-72, and the pushover curve is determined by assuming a constant drift throughout the height. The conditional spectrum (CS) with conditional mean and conditional standard deviation is generated for the particular site at multiple intensity levels based on a single causal earthquake. The randomness associated with the elastic CS is represented by randomly sampled values from a normal distribution. The displacement-based procedure is adopted to obtain the displacement demand analytically for the randomly sampled values. The proposed approach is applied to 3-, 5-, and 8-story SMRF systems and the fitted fragility functions are obtained using the maximum likelihood method. The results of the proposed approach are then compared with that of nonlinear response history analysis (NLRHA) and previous studies. The proposed approach shows good agreement with the results obtained using NLRHA.

Keywords: conditional spectrum; displacement-based procedure; fragility curves; spectral shape factor

1. Introduction

One of the integral parts of performance-based earthquake engineering is the fragility curve which provides the probability of exceedance of a specified limit state at different intensity measures. The use of collapse fragility curves has also been included in FEMA P695 [1] as design validation. The collapse fragility curve is a plot that provides the information on the probability of collapse at particular ground motion intensity. Various methods exist for the generation of fragility curves at different limit states. Incremental dynamic analysis (IDA) implemented by Vamvatsikos and Cornell [2] is one of the methods that uses a generic set of ground motions at different intensity levels to uncover the behavior of a structure from elastic state to the point when the structure collapses. Generally, a structure is considered to reach the collapse state when it attains global dynamic instability even though various definitions of the collapse exist [1]. When IDA is performed for a multi-degree-of-freedom (MDOF) system, a very high computational time needs to be allocated for obtaining the fragility curves. Moreover, for site-specific collapse assessment, multiple-stripe-analysis (MSA) proposed by Jalayer [3] is used which requires reselection of the ground motions at each intensity level. The need to run the analysis at multiple intensity levels increases the computational effort. Further, the selection of the ground motion at each intensity level in MSA makes the procedure more burdensome. Nonlinear response history analysis (NLRHA) at varying intensity levels is computationally expensive, thus, simplified methods have been sought.

With the intent of reducing the computational effort, many researchers have developed simplified methods to estimate the median collapse capacity and the associated variability, such as Vamvatsikos and Cornell [4], Dolšek and Fajfar [5], Han and Chopra [6], Silva et al. [7], etc. Even though these simplified methods are computationally efficient and impose simplification in different ways, most of these methods, except Silva et al. [7], require modeling as the methods rely heavily on the results of nonlinear static analysis. In Silva et al. [7] which uses displacement-based philosophy, the collapse capacity of the structure, adopted from [8], is estimated by using simplified formula as a function of the strain limit of concrete and steel. Haselton et al. [9] have shown by analyzing large database of beams and columns that the section deformation capacity is governed by many factors such as yield strength of steel, confinement ratio, etc. which will be observed in the global capacity of the structure. Due to this reason, Silva et al. [7] may not accurately predict the deformation capacity of the structure. Besides, most of the simplified methods are developed based on the generalized spectrum and do not consider distinctive peak spectral shape of ground motions in the case of rare events [10, 11]. Thus, the spectral shape correction factor needs to be adopted from FEMA P695 [1] for the computation of median collapse capacity to make it compatible with the site characteristics.

Furthermore, the estimation of the record-to-record variability using empirical equations or predefined default value provides inconsistency as the dispersion is controlled by various parameters such as site characteristics, structural configuration, seismic intensity and engineering demand parameter (EDP) being considered [12]. One of the possible ways to estimate the collapse capacity is to use a conditional spectrum (CS) with conditional mean and conditional standard deviation that give a realistic spectral shape of ground motion at a particular site based on a causal earthquake extracted from hazard deaggregation. However, the reselection of the ground motions at each intensity level makes this process tedious. Fox and Sullivan [12] have estimated the ground motion variability associated with the spectral shape using an analytical method for a particular intensity level with varying degree of nonlinearity of SDOF system. But, for the accurate estimation of collapse capacity, deformation capacity needs to be identified by using nonlinear static analysis which leads to additional computational and modeling effort. Thus, even if various methods exist for the estimation of the collapse fragility curve, to the best of our knowledge, no method predicts the collapse capacity and associated variability considering site-specific hazard characteristics at different intensity levels.

In this study, a novel approach for the generation of fragility curves with the consideration of ground motion spectral shape characteristics is presented for special moment resisting frame (SMRF) systems. This study specifically deals with two objectives. First, it provides a method to estimate the capacity curve by making various assumptions. Second, it builds on the work by Fox and Sullivan [12] to predict the collapse capacity and associated record-to-record variability at multiple intensity levels using the elastic variability

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associated with CS. In order to check the accuracy of the proposed method, 3-, 5-, and 8-story SMRF systems are designed following ASCE 7-10 [13] for seismic design and ACI 318-11 [14] for detailing. The two dimensional (2D) model of these structures is constructed in OpenSEES [15], and the properties of equivalent SDOF systems are obtained from pushover analysis of MDOF systems. The equivalent SDOF is then subjected to NLRHA to gauge the effectiveness of the proposed approach.

2. Capacity Curve Estimation

Unlike in Silva et al. [7], in the present method, the displacement capacity at collapse is obtained using the capacity curve. First, pre-capping and post-capping rotation capacity of the beam and columns are obtained from PEER ATC-72 [16] which is based on the work of Hasleton et al. [9]. The moment capacity of beam and columns are obtained when the data of longitudinal reinforcement and imposed axial load (in case of column) is obtained from the design. Knowing the displacement and moment capacity of the beams and columns, the pushover curve of the MDOF system is generated assuming constant drift throughout the height. For this, beam-sway failure mechanism is considered since the proposed methodology is meant for the code designed SMRF systems. Similarly, the mechanism is considered to form only at the ends of the beams and base columns. For yield deformation of MDOF system, the yield rotation θ_y , suggested by Priestley et al. [17] is adopted as,

$$\theta_y = 0.5\varepsilon_y \frac{l_b}{D_b}, \tag{1}$$

where ε_y , l_b and D_b denote the yield strain of steel, bay length and depth of the beam, respectively. The above equation takes into account the deformation contribution due to beam, column, joint and member shear defomation; the contribution of the column, joint and member shear deformation is taken as 40%, 25% and 10% of the beam deformation, respectively. The rotational springs at the ends of the beams and base columns are assumed to reach the yield rotation at the same instant and the base shear capacity V_{base} , is obtained from,

$$V_{base} = \sum_{j=1}^{m} M_{cbase,i} + \sum_{i}^{n} \frac{M_{b,i}^{l} + M_{b,i}^{r}}{l_{b,i}} L, \tag{2}$$

where $M_{cbase,j}$ is the moment of the j^{th} column at the base and m is the number of columns at 1^{st} story. The moment of left and right beam at i^{th} story are denoted by $M_{b,i}^{l}$ and $M_{b,i}^{r}$, respectively. Similarly, n, L and $l_{b,i}$ denote the total number of stories, total length of frame parallel to loading and the beam length between column centerlines at i^{th} story, respectively. Here, the demand introduced by gravity loads is not considered.

Once the yield state of the MDOF system is known, for each step increment in the drift (say 0.1%), the inelastic portion of the moment springs is assumed to undergo the same amount of rotation. Based on the corresponding plastic rotation, the moments are estimated and then used in Eq. (2). Further, hardening and softening of moment springs in the beam may change the deformation contributions of joints and columns. However, for simplicity, once the structure enters into the inelastic region, such deformations are ignored. The collapse state is then identified from the generated pushover curve when the strength drops to 20% of the ultimate capacity. Once the capacity curve is generated, the MDOF system is converted into an equivalent SDOF system using,

$$m_e = \frac{\left(\sum_{i=1}^{n} m_i \Delta_i\right)^2}{\sum_{i=1}^{n} m_i \Delta_i^{2}}, \tag{3}$$

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$$H_e = \frac{\sum_{i=1}^{n}(m_i \Delta_i h_i)}{\sum_{i=1}^{n}(m_i \Delta_i)}, \tag{4}$$

where m_e and H_e denote effective mass and effective height of equivalent SDOF system, m_i is the story mass, Δ_i is the displacement at each floor level, and h_i is the story height. Further, the base shear capacity is transformed in terms of spectral acceleration S_a, using,

$$S_a = \frac{V_{base}}{m_e}. \tag{5}$$

The accuracy of the method is evaluated with the results of nonlinear static analysis of case study buildings which will be described in Section 5. The capacity curve of case study buildings, later described in Section 5, obtained from the nonlinear static analysis and the proposed method is shown in Figs. 1(a)-(c) which shows a good match despite many simplifications.

3. Estimation of Statistical Parameters for Fragility Curves

Knowing that the spectral shape depends upon the site characteristics and hazard level, hazard deaggregation data is extracted from USGS at each intensity level. The CS is constructed using the empirical equation provided by Baker [18]. In this study, a single causal earthquake and ground-motion prediction equation (GMPE) are used for CS calculation. This may suppress the variability [19]; however, the exclusion of multiple causal earthquakes, as well as multiple GMPEs, does not alter the procedure described here.

To generate the CS, a conditioning period (T^*) is chosen; normally, the period which has higher influence on the EDP- first mode in collapse assessment. Using the hazard deaggregation data at multiple intensity levels, CS are constructed. Since the CS estimate the ground motion variability on an elastic response spectra accurately, herein, it is assumed, as in Fox and Sullivan [12], that the record-to-record variability in the fragility curves can be estimated using the dispersion of elastic CS. The steps for the computation of the structural response at different parameters are described below:

Step1: Construct the CS at multiple intensity levels using seismic deaggregation data. Obtain the mean as well as standard deviation associated with the elastic CS.

Step 2: Construct the capacity curve of MDOF system as explained in Section 2 and convert it into an equivalent SDOF system. Identify the yield displacement, the initial period, and the displacement capacity of the equivalent SDOF system.

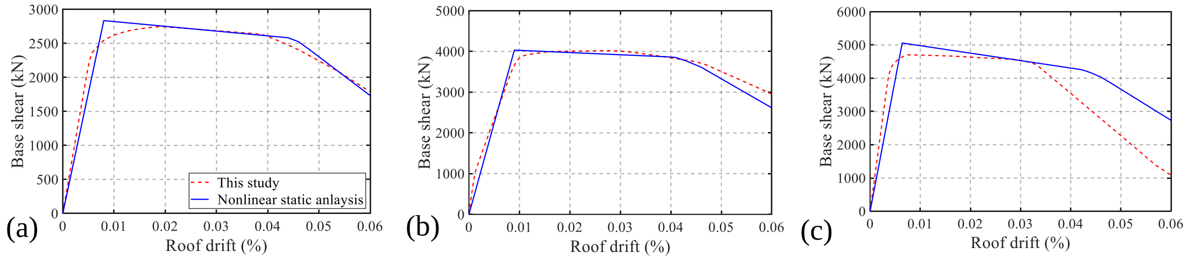

Fig. 1. Comparison of capacity curve obtained using proposed approach and nonlinear static analysis for (a) 3-story; (b) 5-story; and (c) 8-story SMRF systems.

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Step 3: Generate the random values K from standard normal distribution truncated at ±2.0 standard deviations to avoid unrealistic ground motions. It is observed that around 1000 random values will give stable results regardless of the number of times the same structure is analyzed.

Step 4: Define the initial displacement based on the equal displacement rule and accordingly ductility μ of the SDOF system. Similarly, the effective period T_{eff} of the SDOF system is calculated from,

$$T_{eff} = T_i \sqrt{\frac{\mu}{1 + \alpha\mu - \alpha}}, \tag{7}$$

where T_i is the initial period of the equivalent SDOF system and α is the post yield stiffness.

Step 5: Calculate the elastic spectral displacement demand $S_{d,elastic}$ based on T_{eff} using,

$$S_{d,elastic} = \exp\left(\overline{S_d(T_{eff})} + K_i \sigma_{\ln}\right), \tag{8}$$

where $\overline{S_d(T_{eff})}$ is the mean spectral displacement at the effective period, $\sigma_{\ln}$ is the associated standard deviation obtained from Baker [18], and K_i is the i^{th} random value of generated samples.

Step 6: Calculate the equivalent viscous damping ξ_{eq} based on the ductility, spectral modification factor η as a function of equivalent damping, and the revised displacement demand S_d of SDOF system due to random ground motion as,

$$\xi_{eq} = 0.05 + 0.565 \frac{\mu - 1}{\pi\mu}, \tag{9}$$

$$\eta = \sqrt{\frac{0.07}{0.02 + \xi_{eq}}}, \tag{10}$$

$$S_d = \frac{S_{d,elastic}}{\eta}. \tag{11}$$

Step 7: Check the revised displacement with the initially defined displacement. In case the discrepancy is high, assign the revised displacement as an initial guess and go to Step 4. If the discrepancy is within the tolerance, terminate the iteration and move to Step 4 to solve for K_{i+1}.

Step 8: Once the displacement demand S_d is obtained for all the random values, take the CS with a new intensity level and repeat the process from Step 3.

Step 9: When the displacement demands are obtained for all the random values at varying intensity levels, the relevant statistical parameters of fragility function are obtained using the maximum likelihood method. It should be noted that the bilinear idealization holds true when the structure does not exceed the displacement capacity. But, there may be cases where the displacement demand obtained using the above mentioned technique may result in a higher value than displacement capacity, which may not yield similar property such as T_{eff}, due to the degradation in strength in actual structure. However, for the generation of fragility curves, it is necessary to accurately estimate the fraction of collapse rather than actual displacement beyond displacement capacity of the structure. Thus, the above mentioned technique should be viewed as a procedure to estimate the fragility function rather than a technique to predict displacement demand for rare events.

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4. Application of the Procedure

4.1 Case study buildings

The proposed approach is applied to 3-, 5-, and 8-story RC SMRF systems that comply with ASCE 7-10 [13]. The site of the buildings is located in San Jose, CA with the coordinates 37.29°N,121.76°W and the soil is characterized as stiff soil (class D). The mapped MCE$_R$ spectral response acceleration parameters are $S_S = 1.548g$ and $S_1 = 0.6g$ at short periods and 1 s period, and the corresponding 5% damped design spectral acceleration values are $S_{DS} = 1.032g$ and $S_{D1} = 0.6g$ (where g is gravitational acceleration). For all the case study buildings, the plan dimension is 24 × 24 m. The bay width in both directions is 8 m, and the floor height is 3 m. The plan of the case study buildings is shown in Fig. 2 (a), and the elevation of 8-story SMRF system is shown in Fig. 2 (b). The slab thickness of 125 mm is taken for all case study buildings. The buildings are designed as a perimeter frame for office and accordingly, live load is obtained from ASCE 7-10 [13]. The compressive strength of the concrete is 41 MPa, and the yield strength of longitudinal and transverse reinforcement is taken as 420 MPa. The section dimension and reinforcement detailing of beams and columns are given in Table 1 and Table 2, respectively. The 3-, 5-, and 8-story SMRF systems are designated as 3-SMRF, 5-SMRF and 8-SMRF, respectively. The total seismic weight of the 3-, 5-, and 8-story buildings are 22661 kN, 35826 kN, and 56348 kN, respectively.

4.2. Finite element modeling and equivalent SDOF system

Two dimensional (2D) finite element modeling is carried out in OpenSEES [15]. The beam and column springs are modeled using the Modified Ibarra Krwawinkler model that employs degrading model. The energy degradation parameter is assumed to be infinitely high in order to avoid cyclic degradation. The hinge property is estimated using the empirical equation developed by Haselton [9]. The hinges for the beam are provided at each end while that for the column is provided only at the base of first story. The upper portion of the column element is modeled as elastic. Moreover, the elastic portion of the beam and columns are modeled using an elastic beam-column element. The cracked stiffness of the beam and columns are taken as M_n / φ_y as per the recommendation of Priestley et al. [17] where M_n is the nominal moment and φ_y is the nominal yield curvature. The gravity loads which are not explicitly included in the 2D model are taken into account by using the leaning column. The assigned gravity loads for this leaning column is $1.0D + 0.25L$ where D and L are dead load and live load, respectively. Nonlinear static analysis is employed in case study buildings to obtain the capacity curve and then the MDOF system is transformed into an equivalent SDOF system as explained in Section 2. The modeling of elastic damping is done using 5% mass proportional damping specified at the first mode elastic period. The numerical integration of the equations of motion is accomplished using the Newmark constant average acceleration method ($\beta = 0.25, \gamma = 0.5$).

4.3 Ground motion selection

Conditional spectra are generated at various intensity levels using Baker [18]. To estimate the CS, a single causal earthquake for varying intensity levels are obtained for T^* from hazard deaggregation information provided in USGS. In case, information at a particular T^* is not available in the USGS, linear interpolation is used to define the causal earthquake. Then GMPE by Campbell and Bozorgnia [20] is used to obtain the deterministic uniform hazard spectra for the corresponding causal earthquake. The epsilon value is modified as explained in Lin et al. [19] in order to match the ordinate of uniform hazard spectrum at T^*. The intensity levels considered in this study have probability of exceedance of 0.5%, 1%, 1.5%, 2%, 5%, 10% and 30% in 50 years. In this study, T^* is fixed at 0.85 s, 1.2 s and 1.5 s for 3-, 5-, and 8-story SMRF systems. Once the CS at different intensity levels are constructed, selection and scaling of ground motions are done to select sets of 40 accelerograms as shown in Figs. 3(a)-(c). The Jayaram et al. [21] algorithm is used for the selection of the ground motions that match the median and deviation of the CS. Moreover, 22 pairs of ground motions from FEMA P695 [1] are also selected to quantify the difference in the median collapse capacity when ground

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motions matched with code based spectra are applied. These ground motions are scaled as suggested in FEMA P695 [1].

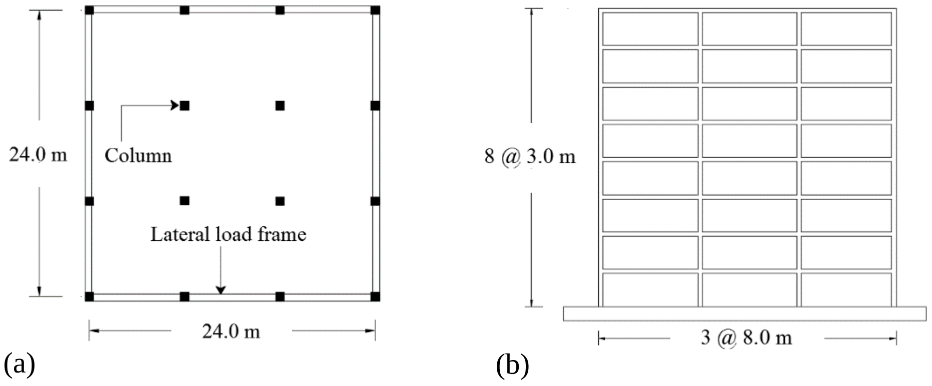

Fig. 2. (a) Plan of case study buildings; (b) Elevation of 8-story SMRF system.

Table 1. Beam section size and reinforcement details

Building	Story	Size (mm^2)	Longitudinal reinforcement		Shear reinforcement*
			Top	Bottom	
3-SMRF	1 – 2	800 × 800	4 No. 28	4 No. 28	4 legs at 100 mm
	3	800 × 800	4 No. 25	4 No. 20	4 legs at 100 mm
5-SMRF	1 – 3	900 × 900	6 No. 28	6 No. 22	4 legs at 125 mm
	4	900 × 750	4 No. 28	4 No. 22	4 legs at 125 mm
	5	900 × 750	4 No. 26	4 No. 20	4 legs at 125 mm
8-SMRF	1 – 4	1000 × 1000	4 No. 30	4 No. 28	4 legs at 125 mm
	5 – 6	1000 × 850	4 No. 28	4 No. 24	4 legs at 150 mm
	7 – 8	1000 × 700	4 No. 25	4 No. 22	4 legs at 150 mm

*No. 8 bar used as shear reinforcement in 3- and 5-story SMRF systems; No. 10 bar used for 8-story SMRF system.

Table 2. Column section size and reinforcement details

Building	Location	Size (mm^2)*	Longitudinal reinforcement	Shear reinforcement**
3-SMRF	Int	800 × 800	12 No. 28	3 legs at 100 mm
	Ext	800 × 800	12 No. 28	3 legs at 100 mm
5-SMRF	Int	900 × 900	10 No. 32	4 legs at 100 mm
	Ext	900 × 900	10 No. 32	4 legs at 100 mm
8-SMRF	Int	1000 × 1000	20 No. 28	4 legs at 100 mm
	Ext	1000 × 1000	20 No. 28	4 legs at 100 mm

*Same section size of column for all stories.
** No. 8 bar used as shear reinforcement in 3-story SMRF system; No. 10 bar used for 5- and 8-story SMRF systems.

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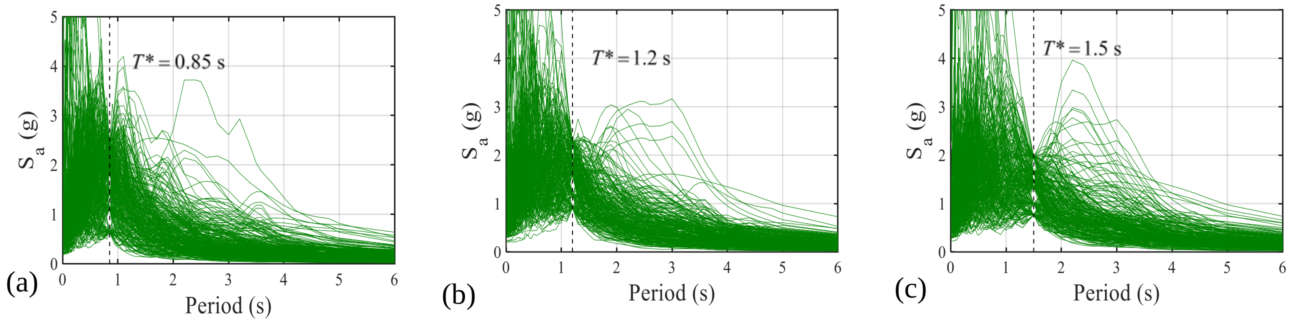

Fig. 3. Ground motion response spectra for ground motions selected at (a) $T^* = 0.85$ s; (b) $T^* = 1.2$ s; and (c) $T^* = 1.5$ s to match the CS mean (μ) and standard deviation (σ).

5. Results and Discussion

The collapse fragility functions are developed for 3-, 5-, and 8-story systems using the approach described in Sections 2 and 3. The 5% damped spectral acceleration conditioned at T^* is used as an intensity measure, as it is considered to have a strong correlation with the probability of collapse [7]. The different displacement demands for the 5-story SMRF system at multiple intensity levels obtained using the proposed approach is shown in Fig. 4(a). The dispersion in the displacement demand at a particular level is attributed to the associated standard deviation of CS. Once the displacement demand is obtained, the probability of collapse at each intensity level can be determined, which is then combined with the maximum likelihood method to obtain the statistical parameters of the fragility function as shown in Fig. 4(b). The predicted median collapse capacity is obtained as $1.61g$, $1.60g$ and $1.18g$ for 3-, 5-, and 8-story SMRF systems, respectively. Similarly, the associated record-to-record variability is estimated as 0.52, 0.52 and 0.45, respectively.

The verification of the proposed method is made by comparing the results with the results obtained using NLRHA. Further, previous studies by Vamvatsikos and Cornell [4], Dolšek and Fajfar [5] and Silva et al. [7], termed as SPO2IDA, IN2, and DBELA are also used for the comparison. For NLRHA, IN2 and SPO2IDA, the displacement capacity that corresponds to the point where the system loses 20% of the ultimate strength is obtained using nonlinear static analysis. The generated capacity curve is then converted into an equivalent SDOF system. In NLRHA, suites of ground motions at different intensity levels are applied to an equivalent SDOF system to generate probabilistic function. For IN2, the equal displacement rule is adopted to identify the median collapse capacity while for SPO2IDA, the parameters of the backbone curve are extracted to obtain the statistical parameters. Similarly, IN2, SPO2IDA and DBELA do not explicitly capture the ground motion spectral shape characteristics of a particular site therefore, their median collapse capacity is corrected by the spectral shape factor.

In order to include the spectral shape factor, the median collapse capacity obtained from FEMA P695 [1] ground motions is utilized. Spectral shape factor is obtained by taking the ratio of the median collapse capacity when CMS are applied to the median collapse capacity when FEMA P695 [1] ground motions compatible with design spectra are applied. The spectral shape factor obtained for the SMRF systems is shown in Table 3. This spectral shape factor is used to magnify the median collapse capacity which is predicted by previous studies.

The fragility curves generated using the proposed approach and NLRHA for case study buildings are shown in Figs. 5(a)-(c). The relevant statistical parameters obtained using different methods without consideration of modeling uncertainties are shown in Table 3. The observed difference between the median collapse capacity from the proposed approach with that from NLRHA, IN2 and SPO2IDA is mainly due to the use of different capacity curve with different displacement capacity as tabulated in Table 3. The higher discrepancy in predicted displacement capacity has shown a higher difference in median collapse capacity. Moreover, the predicted record-to-record variability for all cases is higher than the one obtained from NLRHA.

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Table 3. Displacement capacity as well as statistical parameters of collapse fragility computed by different method

Buildings	Methods	Displacement capacity (mm)	Collapse fragility	
			Median $\left(S_a(T_1)[g]\right)$	Deviation $\left(\sigma_{\ln}\right)$
3-SMRF	NLRHA	353	1.64	0.48
	Proposed	353	1.65	0.52
	IN2	353	2.24 (1.14)	-
	SPO2IDA	353	1.84 (1.14)	0.42
	DBELA	225	1.28 (1.14)	-
5-SMRF	NLRHA	599	1.61	0.44
	Proposed	571	1.60	0.52
	IN2	599	2.02 (1.19)	-
	SPO2IDA	599	1.77 (1.19)	0.38
	DBELA	417	1.25 (1.19)	-
8-SMRF	NLRHA	619	1.09	0.39
	Proposed	750	1.28	0.45
	IN2	619	1.7 (1.25)	-
	SPO2IDA	619	1.41 (1.25)	0.40
	DBELA	499	1.06 (1.25)	-

Note: IN2 - incremental N2 method [5]; SPO2IDA - static pushover to incremental dynamic analysis [4]; and DBELA - displacement-based earthquake loss assessment [7].
Values shown in parentheses are the spectral shape factor used for correction of median collapse capacity estimated using different methods.

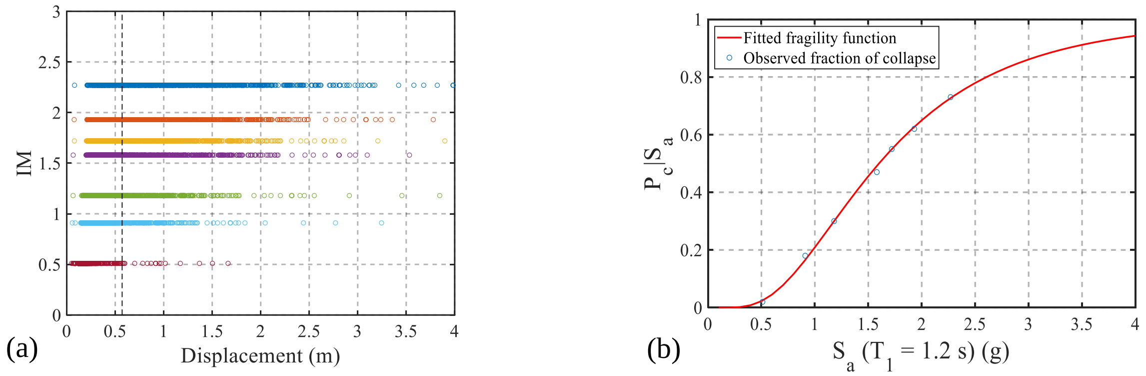

Fig. 4. (a) Response of 5-story SMRF system at different intensity levels; and (b) Fragility fitted function of 5-story SMRF system using the maximum likelihood method.

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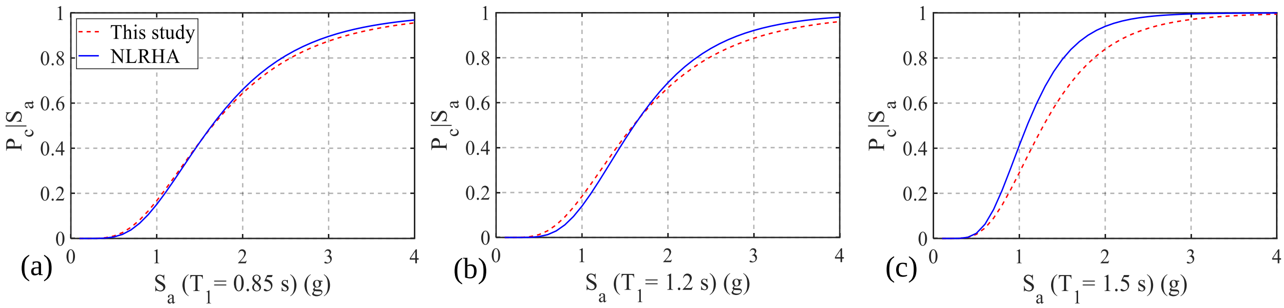

Fig. 5. Comparison of fragility curves for (a) 3-story, (b) 5-story, and (c) 8-story SMRF systems.

6. Conclusions

In this paper, a novel approach for generating collapse fragility curves of SMRF systems is presented. For this, a simplified approach for the generation of pushover curves is proposed and later validated with the results of nonlinear static analysis. Further, CS with conditional mean and conditional standard deviation is generated and the displacement-based approach is adopted to obtain the dispersion of EDP at a particular intensity level. For this, 3-, 5-, and 8-story buildings are used. The accuracy of the method is validated with the results of NLRHA, as well as that of previous studies. The following conclusions can be drawn:

(a) The displacement capacity estimated from the proposed approach and nonlinear static analysis differ by 0%, 13.6% and 21% for 3-, 5-, and 8-story buildings.

(b) The estimates of median collapse, when compared with the results of NLRHA, show a reasonable accuracy of the proposed approach. The errors in the estimation of the median collapse capacity of 3-, 5-, and 8-story SMRF systems are 0.6%, 0.6% and 17%, respectively. The estimations in the median collapse capacity show a higher discrepancy with the difference in the displacement capacity predicted by nonlinear static analysis and the simplified approach. In addition, the proposed method overestimates the associated record-to-record variability in all three cases.

(c) The IN2 method and the SPO2IDA method overestimate the collapse capacity for all case study buildings. However, the median spectral acceleration predicted by SPO2IDA shows higher accuracy in comparison to the IN2 method. Furthermore, the DBELA method underestimates the collapse capacity for all case study buildings. This is attributed to the smaller displacement capacity computed by DBELA when the structure loses its 20% of ultimate strength.

7. Acknowledgements

The first author gratefully acknowledges a Monbukagakusho (Ministry of Education, Culture, Sports, Science, and Technology, Japan) scholarship for graduate students.

8. References

[1] FEMA (2009): *Quantification of Building Seismic Performance Factors*, FEMA P-695, prepared by Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

[2] Vamvatsikos D, Cornell CA (2002): Incremental dynamic analysis. *Earthquake Engineering and Structural Dynamics*, **31**(3), 491-514.

[3] Jalayer F (2003): *Direct Probabilistic Seismic Analysis: Implementing Non-Linear Dynamic Assessments*. Ph.D. Thesis, Dept. of Civil and Environmental Engineering, Stanford University, Stanford, CA.

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[4] Vamvatsikos D, Cornell CA (2005): Direct estimation of seismic demand and capacity of multidegree-of-freedom systems through incremental dynamic analysis of single degree of freedom approximation. *ASCE Journal of Structural Engineering*, **131**(4), 589-599.

[5] Dolšek M, Fajfar P (2007): Simplified probabilistic seismic performance assessment of plan-asymmetric buildings. *Earthquake Engineering and Structural Dynamics*, **36**(13), 2021-2041.

[6] Han SW, Moon KH, Chopra AK (2010): Application of MPA to estimate probability of collapse of structures. *Earthquake Engineering and Structural Dynamics*, **39**(11), 1259-1278.

[7] Silva V, Crowley H, Pinho R, Varum H (2013): Extending displacement-based earthquake loss assessment (DBELA) for the computation of fragility curves. *Engineering Structures*, **56**, 343-356.

[8] Crowley H, Pinho R, Boomer JJ (2004): A probabilistic displacement-based vulnerability assessment procedure for earthquake loss estimation. *Bulletin of Earthquake Engineering*, **2**(2), 173-219.

[9] Haselton CB, Deierlein GG (2007): *Assessing seismic collapse safety of modern reinforced concrete frame buildings*, Blume Earthquake Engineering Research Center Technical Report No. 156, Stanford University.

[10] Baker JW, Cornell CA (2006): Spectral shape, epsilon and record selection. *Earthquake Engineering and Structural Dynamics*, **35**(9), 1077-1095.

[11] Haselton CB, Baker JW, Liel AB, Deierlein, GG (2009): Accounting for ground-motion spectral shape characteristics in structural collapse assessment through an adjustment for epsilon. *ASCE Journal of Structural Engineering*, **137**(3), 332-344.

[12] Fox MJ, Sullivan TJ (2016): Use of the conditional spectrum to incorporate record-to-record variability in simplified seismic assessment of RC wall buildings. *Earthquake Engineering and Structural Dynamics*, **45**(3), 463-482.

[13] ASCE 7-10 (2010): *Minimum design loads for buildings and other structures.* American Society of Civil Engineers (ASCE), Reston, VA, USA.

[14] ACI (2011): *Building code requirements for structural concrete (ACI 318-11).* American Concrete Institute (ACI); Farmington Hills, MI, USA.

[15] OpenSees (2013): *Open System for Earthquake Engineering Simulation*, Computer Program, University of California, Berkeley, California, USA. (Available from: http://opensees.berkeley.edu [15 October 2013]).

[16] PEER/ATC. (2010): *Modeling and acceptance criteria for seismic design and analysis of tall buildings*, in PEER/ATC 72-1.

[17] Priestley MJN, Calvi GM, Kowalsky MJ (2007): *Displacement Based Seismic Design of Structures*, IUSS Press, Pavia,Italy.

[18] Baker JW (2011): Conditional mean spectrum: Tool for ground-motion selection. *ASCE Journal of Structural Engineering*, **137**(3), 322-331.

[19] Lin T, Haselton CB, Baker JW (2013): Conditional spectrum-based ground motion selection. Part I: hazard consistency for risk-based assessments. *Earthquake Engineering and Structural Dynamics*, **42**(12), 1847-1865.

[20] Campbell KW, Bozorgnia Y (2008): NGA ground motion model for the geometric mean horizontal component of PGA, PGV, PGD and 5% damped linear elastic response spectra for periods ranging from 0.01 to 10 s. *Earthquake Spectra*, **24**(1), 139-171.

[21] Jayaram N, Lin T, Baker JW (2011): A computationally efficient ground-motion selection algorithm for matching a target response spectrum mean and variance. *Earthquake Spectra*, **27**(3), 797-815.

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