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Effect of epsilon-based record selection on fragility curves of typical irregular steel frames with concrete shear walls in Mashhad city

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Abstract

In this paper, the seismic vulnerability of Mashhad city, as the second largest city in Iran, has been investigated using analytical fragility curves. Disaggregation analysis is first performed in order to identify the target epsilon at different hazard levels. The disaggregation results revealed different epsilon values at the first mode period of two representative structures, in the case of 72-, 475-, and 2,475-year return periods. Nonlinear incremental dynamic analyses are then performed for two representative models of a typical steel frame with a concrete shear wall, using independent suites of acceleration time histories that are selected based on the target epsilons. Structural limit states are defined on each incremental dynamic analysis curve, and the corresponding damage measures are estimated. The results show that if ε is neglected in the considered simulations, then the predicted median structural capacities is decreased by around 10%, 15%, and 18%, respectively, for the three abovementioned hazard levels.

Keywords: Record selection; Hazard disaggregation; 3D analysis; Epsilon; Collapse capacity; Strong ground motion; Spectral shape; Vulnerability

Introduction

Mashhad is the second largest city in Iran, and different seismic zonations have shown that the city rest on a seismically active region. One of the main tools for the risk estimation in urban buildings is the seismic vulnerability evaluation in a proper scale for a specific level of seismic hazard. EMS-98, HAZUS, and Risk-UE provide a good and useful library of fragility curves, but they do not really capture the characteristic of Iranian buildings and construction process. Different efforts have been conducted to develop the fragility models for Tehran. Japan International Cooperation Agency (JICA 2000) has developed hybrid fragility curves for Tehran, using available data from major earthquakes. This paper, as part of large studies, is focused on the development of the fragility curve for the most common buildings in the Mashhad city for spectral acceleration (S_a) and epsilon (ε).

In this way, the analytical fragility curves for the existing buildings in Mashhad are one of the present paper goals. The analytical seismic fragility, as an amount of probability of damage, is calculated in this appear based on seismic hazard analysis, selection of typical structures, structural analysis, damage criteria, and probability distribution function.

Based on 2012 municipality census data, the conventional steel frames with concrete shear walls (as the most common types of new buildings) comprise nearly 48% of the residential construction in the Mashhad city during the past decade. Since the seismic behavior of buildings cannot be specified one by one, in order to reduce the number of structural models, it is required that the representative buildings is selected with the aim of being a good sample for a large group of existing buildings by statistical study on desired area. Therefore, the focus in this study is based on two typical five-storey steel frames with cast-in-place concrete shear walls, and for low-rise and high-rise buildings, more studies are needed in the future.

By selecting the representative structural model, the dynamic analysis can be carried out using the appropriate

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suites of ground motion records. In this way, careful ground motion selection can result in the same reduction in the bias and the variance of structural response as can be gained using advanced intensity measures (IMs). Epsilon (ε), as a spectral shape indicator and a predictor of nonlinear structural responses, is an efficient parameter to reduce the bias in the structural response (Mousavi et al. 2011). The previous studies had shown that epsilon is more effective than finding records with appropriate magnitude and distance values (Baker and Cornell 2006). Therefore, disaggregation of seismic hazard should be carried out to identify the target magnitude, the target distance, and the target epsilon at different hazard levels for ground motion selection (Bazzurro and Cornell 1999).

For damage estimation of structures, employing an appropriate damage assessment method is essential to derive the corresponding fragility curves. Many researchers have used from different criteria, e.g., drift, acceleration, or energy indices (Estekanchi and Arjomandi 2007). However, it should be noted that for the seismic assessment of structures with planar irregularities, a damage measure should be able to reflect three-dimensional (3D) structural response features such as torsion and bidirectional response. The effective damage calculation method is used for irregular building in plan that was defined by Jeong and Elnashai (2005, 2006).

In this paper, first, disaggregation analysis on Mashhad city was performed to identify the target epsilon at different hazard levels. In order to perform nonlinear incremental dynamic analysis by employing OpenSees platform, independent suites of the acceleration time histories were selected based on the target epsilon. Structural limit states (immediate occupancy (IO), life safety (LS), and collapse prevention (CP)) are defined on each incremental dynamic analysis (IDA) curve, and the corresponding damage measure is estimated (FEMA-356). In the last step, the proposed method accounts for the multistorey as well as the asymmetry of the structure using multidirectionality of earthquake motions. The variability of the fragility curves is considered, and the corresponding probability of damage is obtained, in the case of 72-, 475-, and 2,475-year return periods.

Methods

Site seismic hazard

Probabilistic seismic hazard analysis (PSHA) aims to quantify these uncertainties and combine them to produce an explicit description of the distribution of future shaking that may occur at a site. Seismic Hazard Map of Iran (1999) has shown that the Mashhad city is mainly exposed to earthquake, and Figure 1 shows active fault map and level of hazard in the area. There have been three major studies on Mashhad seismic hazard. One was done by IIEES for Holy Shrine site located in downtown and the other one by Hafezi-Moghaddas (2007) for the whole area. Recently, Zolfaghari and Ghafory-Ashtiany (2012) have also carried out an independent PSHA study for the city of Mashhad. PSHA was carried out in a city central region to calculate the expected strong ground motion parameters based on seismicity of the region within the radius of 200 km around the city at the ground level by Zolfaghari and Ghafory-Ashtiany (2012), and S_a is used as IM.

Then, we performed hazard disaggregation analysis for determining target M, R, and ε at probabilities of exceedance of 50%, 10%, and 2% in a 50-year return period, and a sample of the results is shown in Figure 2.

At each of the given hazard levels, the disaggregation results revealed different target epsilon values. For the first mode period and in high hazard level (2,475 years), hazard is dominated by M = 6 to 6.5, R = 0 to 10 km, and $\varepsilon > +2.0$. For relatively high hazard level (475 years), it is also dominated by epsilon values between +1.0 to +2.0, and for low hazard level (72 years), it is limited to −1.0 to 0. Therefore, as shown in Table 1 for each hazard level, independent suites of acceleration histories were selected based on the target epsilon.

Structural selection criteria

Literature surveys (2012 census data) were compiled on the existing typology for residential buildings that has been constructed in the Mashhad city during the last 12 years. It was shown that conventional steel frames with cast-in-place concrete shear walls are most common systems of the residential construction over the past few years. The shear walls in these buildings often can bear walls and the steel frame which is only designed for vertical loads. Lateral loads are transferred by diaphragms to the shear walls, and the steel frame may provide a secondary lateral force resisting system. In this way, 100 existing structures were considered from the database inventory building in a regional study. Since there is low dispersion in the design and construction methods in desired area, it is assumed that the median and the standard deviation values can be suitable criteria for building selection for estimating damage. The median and the standard deviation for structural dimensions of the 100 selected buildings are shown in Table 2. A description of representative model is presented in the 'Overview of mathematical modeling' section.

Based on a statistics study on considering structures, a reasonable sample of buildings has been selected as a representative of the great group of structures. It means that the structural geometric dimensions should be close to the average values. But in this paper, we have selected two models between the upper and lower limits of shear wall dimensions. Shear wall length in Y direction was

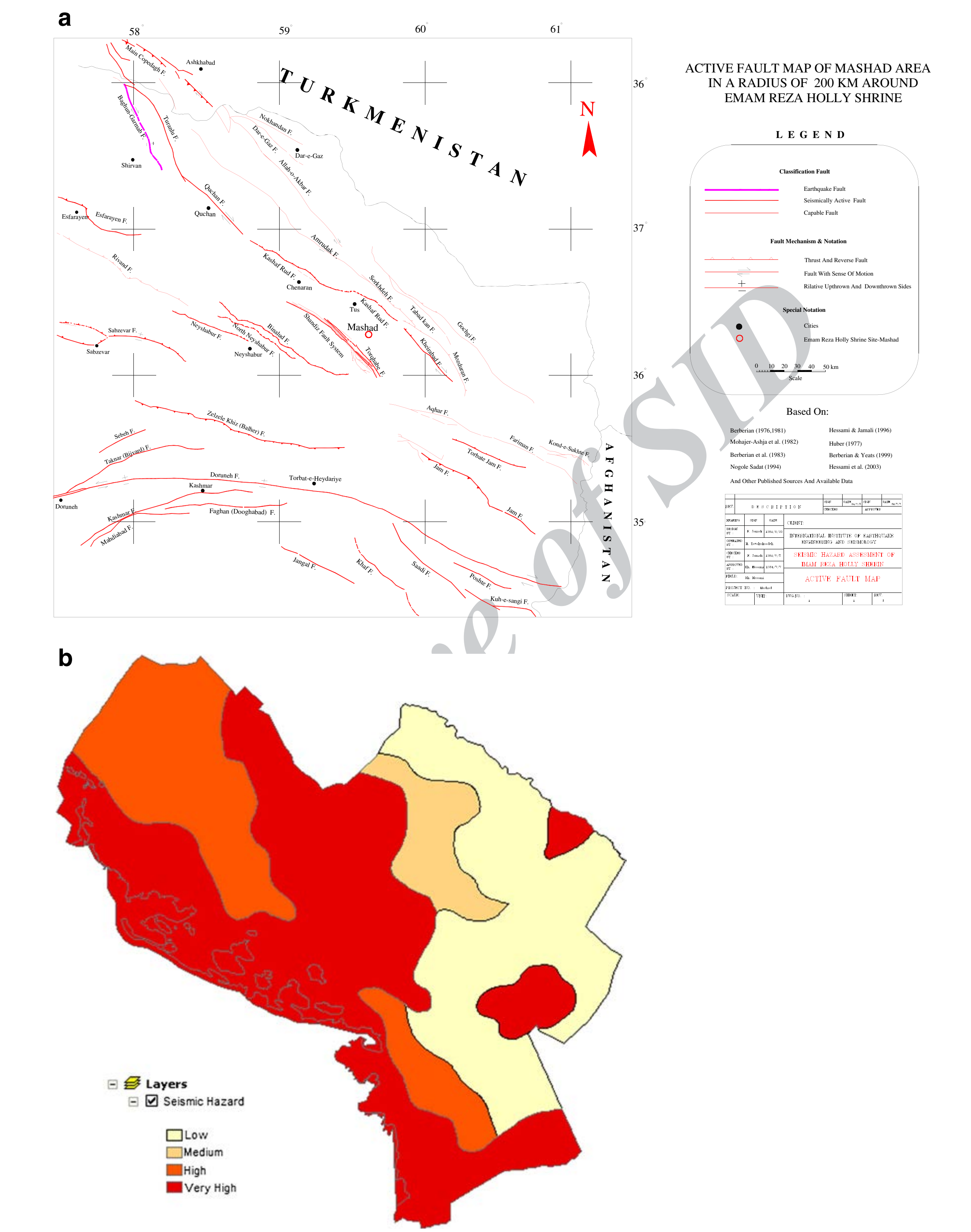

Figure 1 Regional seismology of the Mashhad city: (a) active faults and (b) seismic hazard map. Adapted from Hafezi-Moghaddas (2007).

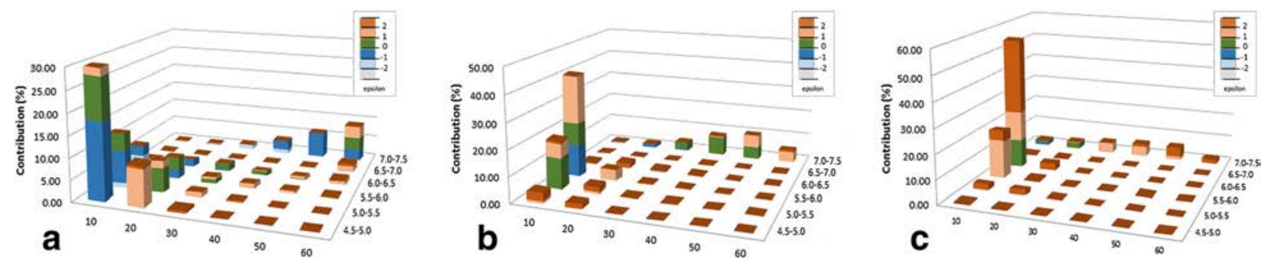

Figure 2 The hazard disaggregation results. (a) 72 years, **(b)** 475 years, **(c)** 2,475 years at T = 0.54 s.

equal to 7.8 (close to 6.4 + 1.18 according to Table 2) for the first model, and it was equal to 5.2 (approximately 6.4 − 1.18) for the second model.

Two existing five-storey steel frame with shear wall buildings have been selected. The structures were designed according to Iran's seismic code (standard no. 2800, 2nd edition, 1999) and were constructed 7 years ago. The specifications of these structures are shown in Table 3 and Figure 3.

Overview of mathematical modeling

The performance-based earthquake engineering requires structural models to be accurate for frequent and rare ground motions which mostly contribute to damage, financial loss, and collapse risk. To analytically predict the inelastic response of such structural systems under seismic loads, the building structure should be accurately described. Using reliable analytical software and definition of strength of materials, yielding behavior of elements, the effects of confinement in boundary elements, concrete crack or crash, and strain hardening and stiffening deterioration phenomena at large deformations are necessary.

The models were analyzed herein by employing OpenSees software. The masses are lumped at floor levels, whereas the horizontal degrees of freedom are defined. The Rayleigh damping with a specified ratio of ξ = 0.05 was assigned at all of the vibration modes, and the effect of nonstructural elements was not considered.

All of the beam end connections within the structure are assumed to be pinned. Therefore, the beams are modeled as elastic elements. These models are built with nonlinear beam column element for columns (batten column with double I section) and shear walls as well as the P-delta effects are taken into account. Fiber elements were used in all of the nonlinear elements, and the spread of plasticity along the elements was considered.

Table 1 Disaggregation results for a central point in Mashhad

	Return period (years)		
	72	475	2,475
M	4.5 to 5.0	5.5 to 6.0	6.0 to 6.5
R (km)	0 to 10	0 to 10	0 to 10
ε	−1.0 to 0.0	+1.0 to 2.0	Larger than +2.0

Table 2 Median and standard deviation of geometric dimensions for 100 samples

	Floor area (m^2)	Shear wall length		$A_{shear\ wall}/A_{floor}$ [a]	
		X (m)	Y (m)	X	Y
Median	146	8.70	6.40	1.63	1.20
SD	33.05	1.80	1.18	0.4	0.28

[a]Area of cross section for shear walls in plan to floor area multiple to 100%.

Table 3 Specifications of selected building

Code	Number of stories	Dimensions		Height, M	Wall length		W (ton)	T_1 (s)
		X (m)	Y (m)		X (m)	Y (m)		
5SW-1	5	16.70	9.80	16.90	9.0	7.6	827	0.54
5SW-2	5	15.40	9.80	17.10	8.8	5.2	794	1.01

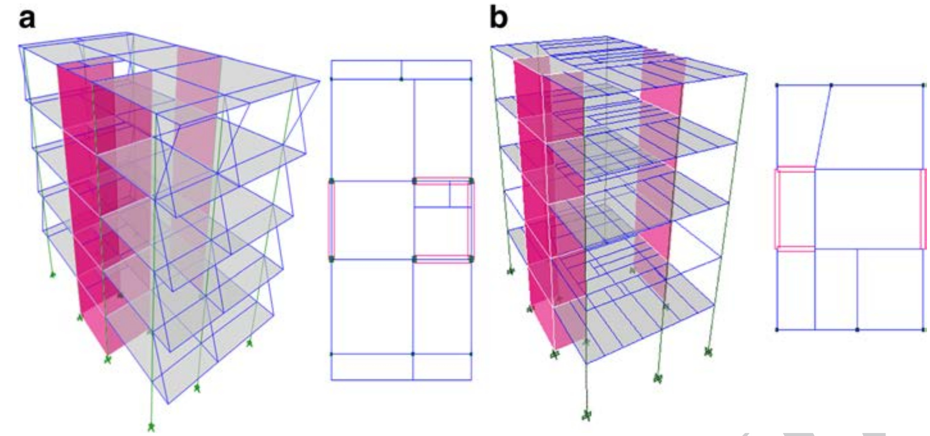

Figure 3 The plan and 3D view configurations of representative structures. (a) 5SW-1 and **(b)** 5SW-2.

The shear walls undergo both shear and flexural deformations. The studies have shown that the shear strength is a function of several parameters such as axial load ratio and horizontal steel ratio. Since the shear strength was enough larger than the flexural strength of the representative structure, the shear failure is definitely an undesirable behavior.

Verification of material properties

In order to calibrate the plastic behavior of batten columns and shear walls, several parameters have been examined in the mathematical model. Therefore, the influential parameters should be validated against the existing experimental test results. Very few tests on entire steel frames with concrete shear wall systems have been performed, especially the ones incorporating details representative of current practice in Iran (Shokrzadeh and Tasnimi 1995). For accurate modeling, the parameters were verified based on existing experimental results by Thomsen and Wallace (1995). The experimental tests are used in order to calibrate the material behavior, which are shown in Figure 4 in which a very good match is observed. There were low differences between the analytical hysteretic loops and the ones obtained experimentally as seen in Figure 4.

It usually use a unique type of reinforcing steel named AIII grade (f_y = 400 MPa), and compressive strength of concrete used for design of the shear walls was 25 MPa in the Mashhad city. The concrete compressive strengths of testing ranged between 19 and 37 MPa, with mean of 26.3 MPa, and similarly for the reinforcement yield stress from 341 to 504 MPa, with mean 445 MPa with yield strain of approximately 0.002 for all specimens.

The parameters of steel02 material for batten columns and for accurate modeling were verified based on existing experimental results obtained by Jafari and Hossaini-Hashemi (2008). The concrete01 for shear walls was used, and the material was modeled as 'unconfined' with peak strength achieved at a strain of −0.002 and minimum post-peak strength achieved at a strain of −0.008. The reinforcement was assumed to have a post-yield modulus equal to 2% of the elastic modulus.

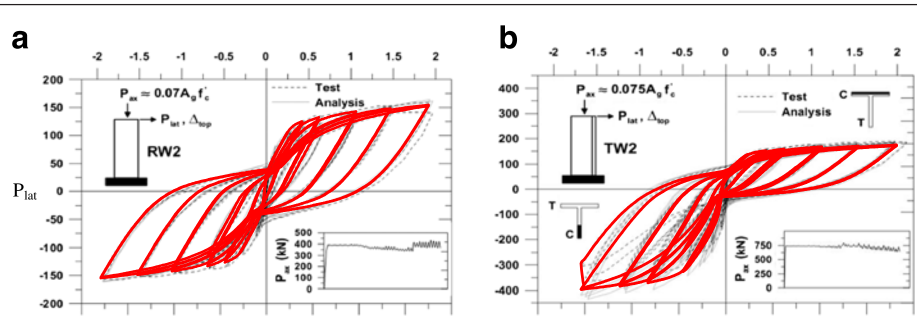

Figure 4 The test and shear wall simulation results: (a) rectangle wall and (b) T-shaped wall.

Ground motion database/selection

Earthquake record database

Earthquake record selection often is considered with the aim of accurately estimating the response of a structure at a specified ground motion intensity, as measured by spectral acceleration at $S_a(T_1)$.

The M6.5 scenario database of SGM records as shown in the 'Appendix' has been used for the time history analysis (Hatefi and Ghafory-Ashtiany 2010). This database belongs to a bin of relatively large magnitudes of 6.0 to 7.6 and moderate distances recorded on II and III soil types. Soil types are classified as type 1, 2, 3, or 4 in accordance with the descriptions defined in 2800 Iran's seismic code and based on geotechnical information. Most of the areas of the Mashhad city are located on II or III soil type. Each record from the database contained two horizontal components for use in the dynamic analysis.

Record selection criteria

When selecting ground motions for dynamic analysis, the efforts should be made to find records with ε-filtration values (as an indicator of spectral shape). It is seen that selecting ground motions based on their epsilon (ε) values is more effective than magnitude (M) and distance (R) (Baker and Cornell 2006). Two studies provide a comparison of the results mentioned here. Zareian (2006) found that a change from $\varepsilon = 0$ to +2.0 caused an approximately 45% to 50% increase in the expected collapse capacity. Haselton and Baker (2006) found that a change consistent with ε causes a 50% shift in the median collapse capacity for some S_a levels. In other works, it has been shown that ε effect scan changes the predicted probability of collapse by 20% to 30% for benchmark project located in the south of downtown Los Angeles (Goulet et al. 2006).

The record selection in the current study is based on the records which have compatible epsilons with the target epsilon of ground motion disaggregation in specified scenarios of different hazard levels in the Mashhad site. The epsilon values of ground motion at T_1 are derived. Epsilon is defined as a measure of the difference between the spectral acceleration of a ground motion record and the mean obtained from an attenuation prediction equation (here, Campbell and Bozorgnia 2008).

The disaggregation results reflected the expectation (−1.0 to 0.0), (+1.0 to +2.0), and (+2.0 to 3.0) for 50%, 10%, and 2% probability of exceedance in 50 years, respectively. Consequently, the database records are classified according to predicted epsilon for analysis of the structure.

Nonlinear dynamic analyses

The nonlinear IDA method involves carrying out a sequence of dynamic analyses in which the intensity of S_a (T_1) is considered incrementally increased until a specific limit state (Vamvatsikos and Cornell 2002). Based on the existing methodologies, failure may be defined by some different methods. First, each of the distinct records was incremented and run until an S_a level was reached at which the maximum interstorey drift ratio (MIDR) grew rapidly implying dynamic instability. Second, the rate of decrease of stiffness with increasing record intensity that exceeds beyond a prescribed MIDR is considered doubtful at 10% (Cornell et al. 2005).

In this work, the time histories for the horizontal displacement were, in the case of the shear walls, reported as the OpenSees output. Both horizontal component of a ground motion are used for IDA analysis, and the results were derived for the ground motion records selected based on the target epsilon. MIDR as the engineering demand parameter is correlated to damage within the structure. Figure 5 shows MIDR from the IDAs for both models using 35 ground motions. In the figure, the IM is the ground motion 5%-damped spectral

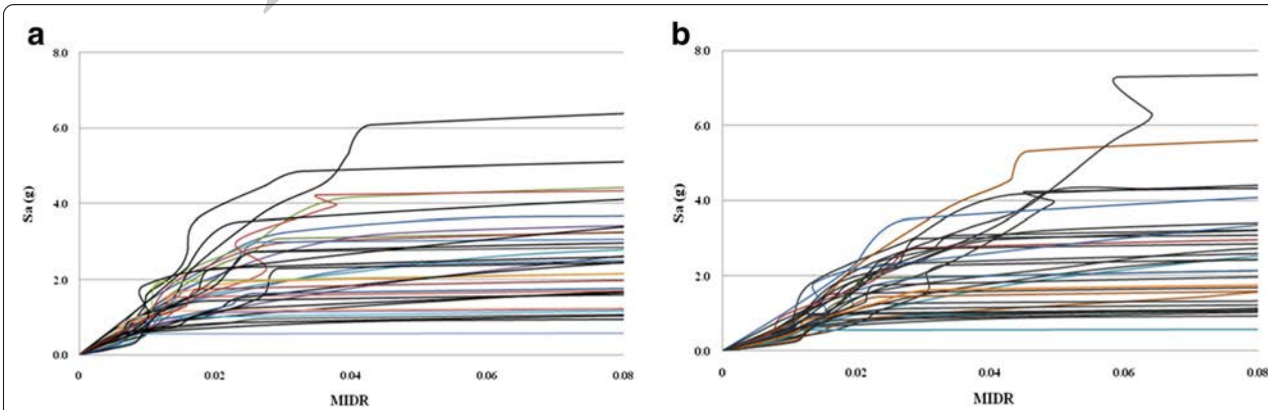

Figure 5 IDA curves for two representative models. (a) 5SW-1 and **(b)** 5SW-2.

acceleration at the first mode period ($S_a(T_1)$) normalized by 'g.' The colored lines are individual IDA curves, and the flat line at the end of the curve represents the collapse region for the particular ground motion (Ibarra and Krawinkler 2004). The different hazard levels can be marked in each figure with horizontal lines, and the intersection of the hazard level line with individual IDA curves leads to results by counting.

Damage index

Except for a few brittle systems and acceleration-sensitive elements, building damage is primarily a function of building relative displacements, rather than force. Hence, successful prediction of earthquake damage to buildings requires reasonably accurate estimation of building drift response in the inelastic range. In this paper, interstorey drift damage indices are selected as one of the most extensively used damage measures.

Structural drift limit states (e.g., IO, LS, or CP) based on FEMA-356 are defined on each IDA curve, and the corresponding capacities are calculated. Furthermore, for the seismic assessment of structures, a damage index should be able to reflect three-dimensional structural behaviors such as torsion and bidirectional response. The procedure advocated in this paper for the damage assessment of structure with planar irregularities is achieved by Jeong and Elnashai (2006). The methodology had been established for one-storey irregular building, while here, we have applied for five-storey irregular structure. Based on the assumption that a critical storey governs the overall damage state of the building, the 3D damage measure of a critical storey is employed as the response variable.

At first, individual shear wall damages are obtained from the transient drift ratio of 3D model for each shear wall. Then, to combine the damage to all frames of a given direction, a weighting mechanism was developed based on effective gravity loads of each frame, such that the importance of each frame to total building damage is a function which that frame resides as well as bearing areas. The local damages were combined using the following equation at each storey in x and y directions separately (Jeong and Elnashai 2006). Finally, the maximum amount of damage index in the critical storey and critical direction is taken (the damage ratio is taken as the maximum over time during seismic loading). The

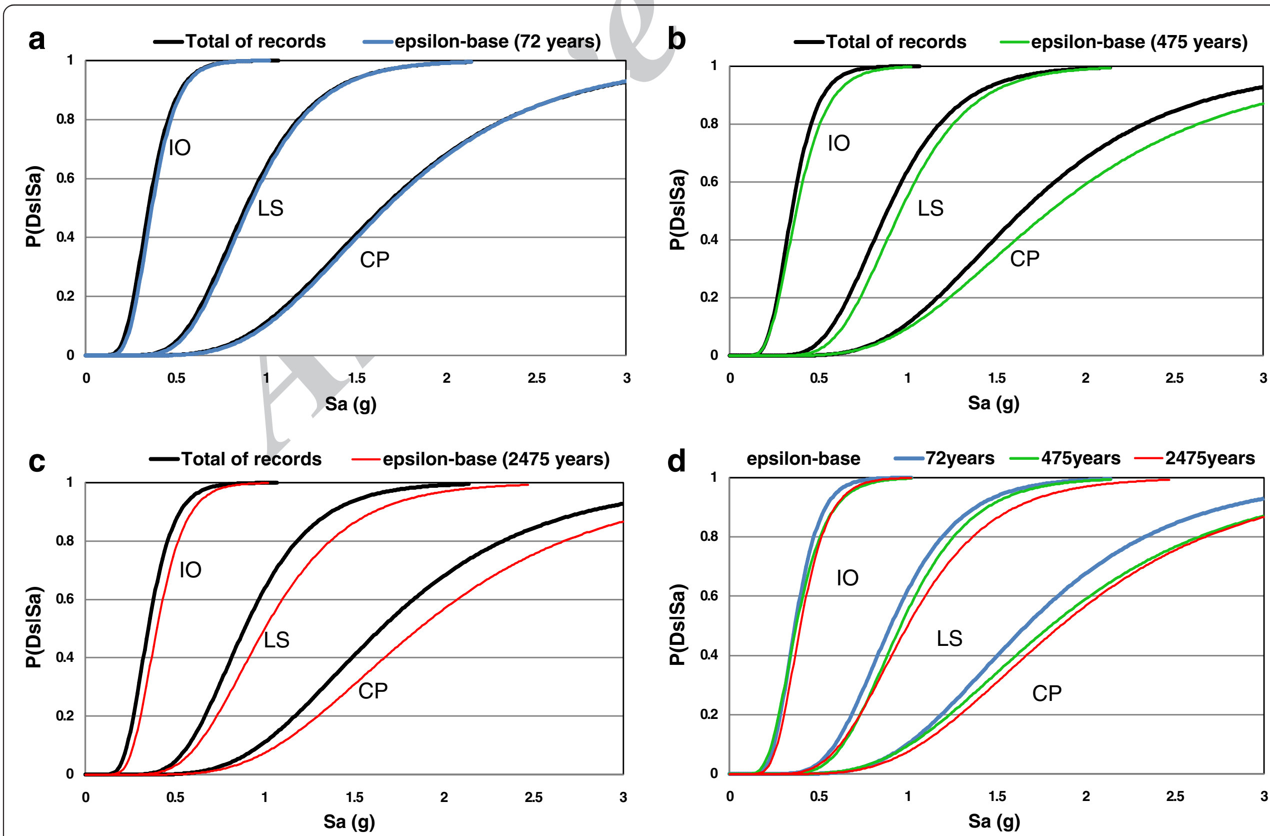

Figure 6 The fragility curves of 5SW-1 model, with and without regard to epsilon. Return periods of **(a)** 72 years, **(b)** 475 years, and **(c)** 2,475 years. **(d)** Final fragility curves using epsilon.

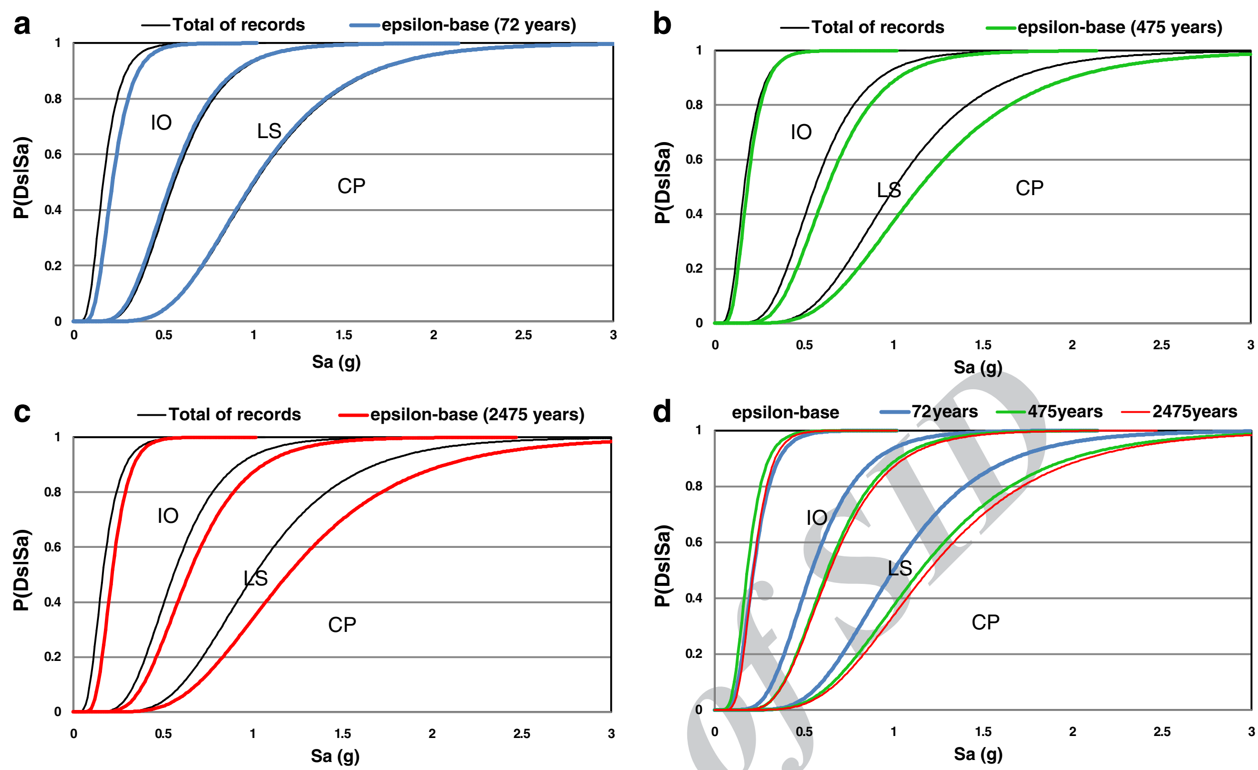

Figure 7 The fragility curves of 5SW-2, with and without regard to epsilon. Return periods of **(a)** 72 years, **(b)** 475 years, and **(c)** 2,475 years. **(d)** Final fragility curves using epsilon.

parameters are described in detail in the related article by Jeong and Elnashai. Mathematically, the damage index for x direction is given as

$$D_{gnx} = \sum_{i=1} D_i \frac{W_{i,\min}}{W_{\text{total}}} + \sum_{i \neq j} \left\{ \frac{W_{\text{CF},ij}}{2 \cdot W_{\text{total}}} \left[\max\left(D_i, D_j\right)^2 - D_i \cdot D_j \right] \right\} \quad (1)$$

where D_{gnx} is the x-global damage index in storey n, D_i is the local damage index of shear wall, W_{total} is the total effective weight in storey n, $W_{i,\min}$ is the tributary weight of shear wall i in storey n, and $W_{\text{CF},ij}$ is the common failure consequence weight between local shear walls i and j.

Fragility curve results

As mentioned previously, the most important element at risk is vulnerability of structures in the city, and the vulnerability of a building can be described using fragility curves. Structural damage fragility curves are described (in this research by values of drift ratio) that define the thresholds of different damage states at a specified

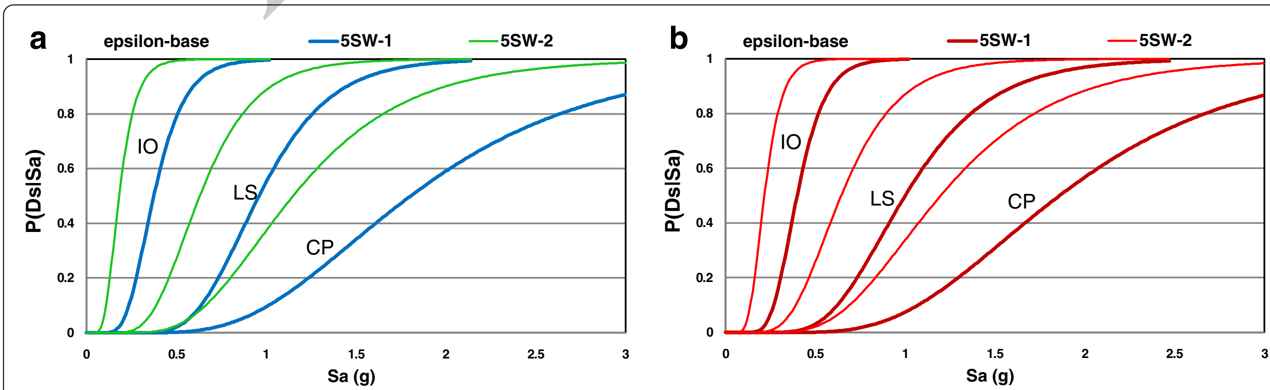

Figure 8 Fragility curves of steel frame with concrete shear walls, with regard to epsilon. Return periods of **(a)** 475 years and **(b)** 2,475 years.

Table 4 Spectral acceleration in the first mode of structure at different levels from PSHA

Model	First mode period (s)	Probability of occurrence		
		50% in 50-year return period (72 years)	10% in 50-year return period (475 years)	2% in 50-year return period (2,475 years)
5SW-1	0.54	0.10 g	0.45 g	0.75 g
5SW-2	1.01	0.07 g	0.16 g	0.28 g

hazard level. Using the strong ground motion indices, S_a, and the damage ratio, fragility curves were constructed and the cumulative probability of occurrence of damage was assumed to be lognormal (Miranda and Aslani 2003). The definition is expressed by the following mathematical formulation:

$$F = P(d > D_i|\mathrm{IM}) \tag{2}$$

where F is the fragility function, P is the probability function, d denotes damage level of structure, D_i presents ith damage states, IM denotes ground motion intensity parameter (S_a), and (i =1 to n) shows different damage states.

Comparison of the results from lognormal fit in Figures 6 and 7 shows higher values of the median damage capacity when record selection based on epsilon is employed. The differences in fragility curves increase with the increase of the epsilon in the record selection as well as hazard level in the site. As shown in Figures 6 and 7, a change of epsilon-based record selection from 72- to 2,475-year return periods reduces the probability of exceedance expected performance levels by less than 20%. Similarly, Figures 6 and 7 show that if ε had been neglected in our simulations, the median predicted structural capacities would be decreased by less than 10%, 15%, and 18% for 72-, 475-, and 2,475-year return periods, respectively. These results demonstrate the importance of ground motion acceleration history selection criteria in accurately predicting building limit states.

Figure 8 shows comparison of fragility curves for two representative structures. According to these curves, we surmise that expected damages for most of the steel frames with concrete shear walls approximately are on these limits.

Table 4 shows the S_a at the first period of the two structural models for three hazard levels in the city of Mashhad (Zolfaghari and Ghafory-Ashtiany 2012). Figure 9 shows the estimated probability of exceedance of expected damage states for the typical buildings.

Conclusions

A procedure for developing fragility curves for spectral acceleration and epsilon to be used for existing five-storey steel frame with cast-in-place concrete shear wall buildings is demonstrated. The methodology employs a proposed technique for calculating the probability of exceedance of three limit states for irregular buildings in plan by Jeong and Elnashai. The distribution of damage indices in each ground motion intensity measure (S_a) are estimated by numerous inelastic incremental dynamic analysis. Unlike alternative methodologies, the proposed procedure allows estimating the fragilities based on weighed combination of interstorey drift demand of the lateral resistance systems, which is an engineering demand parameter closely related to structural damage.

The epsilon-based method was taken into account for earthquake ground motion selection, whereas the disaggregation analysis was implemented on the Mashhad site. The epsilon demands reflected the expectation (−1.0 to 0.0), (+1.0 to +2.0), and (+2.0 to 3.0) for 50%, 10%, and 2% probability of exceedance in 50 years, respectively.

The results show that a change in epsilon-based record selection decreases the probability of exceedance expected performance levels by less than 10%, 15%, and 18% for 72-, 475-, and 2,475-year return periods, respectively. It seems that the regarded typical building (steel frame with concrete shear walls) in high hazard levels is safe for collapse in the site.

Therefore, the proposed method to derive fragility curves is recommended for the probabilistic seismic assessment of typical buildings with significant torsional

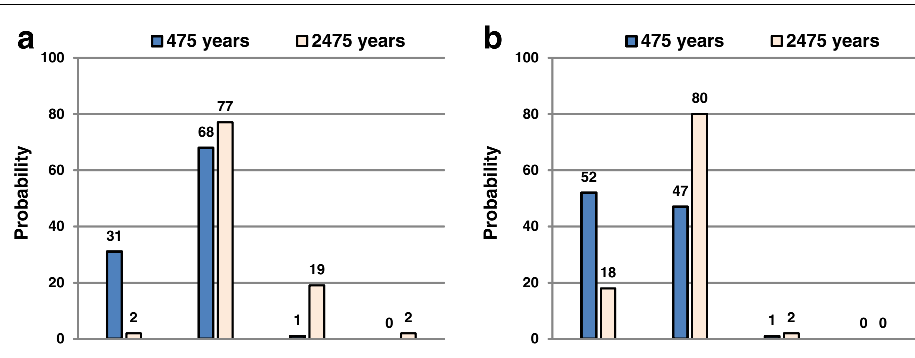

Figure 9 Probability of exceedance of damage levels. (a) 5SW-1 and **(b)** 5SW-2.

and bi-directional responses. Deriving curves leads to more reliable damage assessment in terms of interstorey drifts for typical steel frame with concrete shear walls in Mashhad.

Appendix

The M6.5 scenario database of SGM records is shown in Table 5.

Table 5 M6.5 scenario database of SGM

Number	Event	Year	Station	M	PGA	R	V_{s30}
1	Bandar Abbas	1975	Bandar Abbas	6.1	0.13	36	337
2	Tabas	1978	Boshruyeh	7.4	0.10	55	564
3	Tabas	1978	Tabas	7.4	0.81	54	645
4	Tularud	1978	Talesh	6.0	0.24	15	539
5	Qaen	1979	Khezri	7.1	0.10	75	701
6	Qaen	1979	Gonabad	7.1	0.10	93	529
7	Golbaft	1981	Golbaft	7.0	0.28	13	365
8	Manjil	1990	Qazvin	7.4	0.27	94	456
9	Manjil	1990	Abhar	7.4	0.21	101	291
10	Manjil	1990	Ab-bar	7.4	0.57	41	291
11	Islamabad	1997	Kariq	6.0	0.57	48	589
12	Avaj	2002	Kaboodar-A	6.5	0.16	62	613
13	Avaj	2002	Razan	6.5	0.20	35	314
14	Firoozabad	2004	Hasan-Keyf	6.3	0.84	42	339
15	Firoozabad	2004	Moalem-Kel	6.3	0.29	99	490
16	Enchehborun	2005	Agh-Gala	6.1	0.12	14	341
17	Zarand	2005	Zarand	6.4	0.32	16	226
18	Erzurum	1983	Meteorologi-Ist	6.6	0.17	35	316
19	Adana	1998	Tarum-Ilce	6.2	0.28	48	263
20	Adana	1998	Meteorologi-Ist	6.2	0.13	65	366
21	Kocaeli	1999	Devlet-Has	7.4	0.14	81	348
22	Kocaeli	1999	Meteorologi-Ist	7.4	0.37	101	282
23	Kocaeli	1999	Marmara-Ara	7.4	0.12	43	701
24	Duzce	1999	Bayindirlik	7.1	0.81	36	294
25	Bingol	2003	BayindirlikIsk	6.3	0.50	12	529
26	Northridge	1994	Beverly Hills	6.7	0.49	13	356
27	Northridge	1994	Can country	6.7	0.48	27	309
28	Friuli	1976	Tolmezzo	6.5	0.34	20	425
29	San Fernando	1971	LA - Hollywood	6.6	0.18	40	316
30	Imperial Valley	1979	Delta	6.5	0.34	34	275
31	Landers	1992	Yermo Fire	7.3	0.24	86	354
32	Loma Prieta	1989	Hollister City Hall	6.9	0.25	47	199
33	New Zealand	2010	Heathcote-V	7.0	0.63	43	-
34	Chi-Chi	1999	Chi-CHY101	7.6	0.40	32	259
35	Chi-Chi	1999	Chi-TCU045	7.6	0.47	76	701

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