Earthquakes and Structures, *Vol. 11, No. 3 (2016) 445-459*
DOI: http://dx.doi.org/10.12989/eas.2016.11.3.445

Ground motion selection and scaling for seismic design of RC frames against collapse

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(Received October 1, 2014, Revised August 16, 2016, Accepted September 6, 2016)

Abstract. Quantitative estimation of seismic response of various structural systems at the collapse limit state is one of the most significant objectives in Performance-Based Earthquake Engineering (PBEE). Assessing the effects of uncertainties, due to variability in ground motion characteristics and random nature of earthquakes, on nonlinear structural response is a pivotal issue regarding collapse safety prediction. Incremental Dynamic Analysis (IDA) and fragility curves are utilized to estimate demand parameters and seismic performance levels of structures. Since producing these curves based on a large number of nonlinear dynamic analyses would be time-consuming, selection of appropriate earthquake ground motion records resulting in reliable responses with sufficient accuracy seems to be quite essential. The aim of this research study is to propose a methodology to assess the seismic behavior of reinforced concrete frames at collapse limit state via accurate estimation of seismic fragility curves for different Engineering Demand Parameters (EDPs) by using a limited number of ground motion records. Research results demonstrate that accurate estimating of structural collapse capacity is feasible through applying the proposed method offering an appropriate suite of limited ground motion records.

Keywords: ground motion selection; scaling method; incremental dynamic analysis; reinforced concrete frames; fragility curves; collapse limit state

1. Introduction

Global collapse is defined as the disability of structural system to sustain gravity loads due to excessive lateral displacement resulting in reduction of story shear resistance and instability of the whole load-resisting system. An appropriate collapse limit state criterion is used to estimate the structural collapse capacity. This parameter can be quantified via defining a limit state for selected response parameter (EDP) and intensity measure in collapse range (Ibarra *et al*. 2005). In general, seismic design of structures at collapse limit state necessitates calculating the structural demand and capacity parameters. Numerous research studies have been carried out over the past decade to investigate the effects of structural and seismic uncertainties on the behavior of structures (Kappos 2001, Kown *et al*. 2006, Padgett *et al*. 2007, Asgarian *et al*. 2016). Results of these studies

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http://www.techno-press.com/journals/eas&subpage=7 ISSN: 2092-7614 (Print), 2092-7622 (Online)

place emphasis on the significant impact of ground motion characteristics due to the random nature in comparison to other sources of uncertainty (such as material properties and design assumptions) on nonlinear response of structural systems. In order to quantify the collapse probability of RC frame structures via producing collapse fragility curves researches have been conducted (Jalayer *et al.* 2003, Haselton *et al.* and Zareian *et al.* 2007) concluding that seismic uncertainties (both aleatory and epistemic) play a significant role in dispersion of structural seismic response. As a result, the necessity for producing fragility curves by using a limited suite of appropriate ground motion records to predict the damage probability becomes apparent.

The aim of this research is to evaluate the collapse probability of reinforced concrete frames through accurate estimation of seismic fragility curves for a specified EDP by using a limited number of ground motion records and investigating the sensitivity of results to the ground motion selection and scaling method. RC frames under investigation are first designed by Direct Displacement-Based Design philosophy (Priestley *et al.* 2007) and collapse fragility curves are produced through applying IDA process to RC sample frames under limited selected and scaled ground motion records. Maximum inter-story drift and spectral acceleration at the fundamental period of vibration ($Sa(T_1)$) are assumed as structural response parameter and intensity measure, respectively. Then, through applying the proposed methodology to sample frames a limited number of proper ground motion records are selected and fragility curves obtained by using the selected suite of ground motions are compared with corresponding fragility curves based on all existing records. COM3 finite element software (Maekawa *et al.* 2003) is used in this study and all dynamic analyses are implemented on two-dimensional RC moment frames.

2. Finite element modelling, ground motions set and analysis

To assess the collapse probability of frame types under investigation two single-bay 4 and 6 stories RC moment frames are studied. Characteristics of structural members and loading are

Table 1 Characteristics of RC frames

Concrete Compressive Strength (kg/cm^2)	Steel yield strengths (kg/cm^2)	Elasticity Modulus of Steel (kg/cm^2)	Live Load on Beams (kg/m)	Importance Factor	Live Load Factor	Dead Load on Beams (kg/m)	Design PGA (g)
250	4000	2000000	1000	1	0.20	4000	0.35

Table 2 Cross-sectional properties of structural members for 4-story RC frame

		1st Floor	2nd Floor	3rd Floor	4th Floor
Cross-sectional properties of Columns	Width (cm)	40	40	40	40
	Height (cm)	40	40	40	40
	Corner Rebars	4ϕ20	4ϕ20	4ϕ20	4ϕ20
	Side Rebars	8ϕ20	8ϕ20	8ϕ20	8ϕ20
Cross-sectional properties of Beams	Width (cm)	40	40	40	40
	Height (cm)	40	40	40	40
	Top Rebars	3ϕ20	3ϕ20	3ϕ20	3ϕ20
	Bottom Rebars	3ϕ20	3ϕ20	3ϕ20	3ϕ20

Table 3 Cross-sectional properties of structural members for 6-story RC frame

		1st Floor	2nd Floor	3rd Floor	4th Floor	5th Floor	6th Floor
Cross-sectional properties of Columns	B(cm)	45	45	45	40	40	40
	H(cm)	45	45	45	40	40	40
	Corner Rebars	4ϕ20	4ϕ20	4ϕ20	4ϕ16	4ϕ16	4ϕ16
	Side Rebars	4ϕ20	4ϕ20	4ϕ20	4ϕ16	4ϕ16	4ϕ16
Cross-sectional properties of Beams	B(cm)	45	45	45	40	40	40
	H(cm)	45	45	45	40	40	40
	Top Rebars	4ϕ20	4ϕ20	4ϕ20	3ϕ20	3ϕ20	3ϕ20
	Bottom Rebars	4ϕ20	4ϕ20	4ϕ20	3ϕ20	3ϕ20	2ϕ20

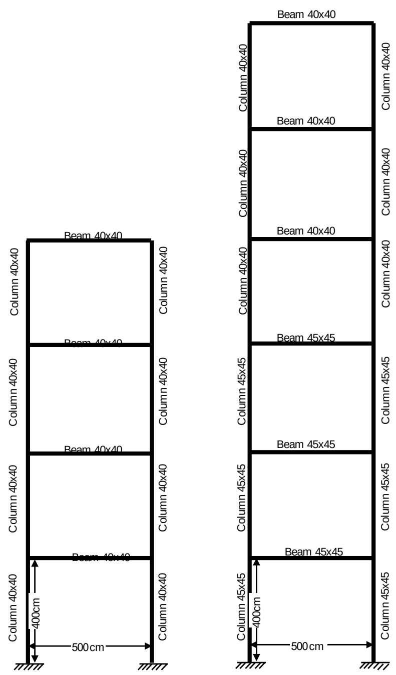


Fig. 1 Structural sections of RC frames

mentioned in Table 1. Cross-sectional Properties of sample RC frames are shown in Tables 2-3, as well. Sections of structural members are appropriately estimated by means of direct displacement-based design procedure (Priestley Method) and Eurocode 8 design displacement spectrum modified based on effective damping (ξ_{eff}). Fig. 1 represents the structural sections of RC frames.

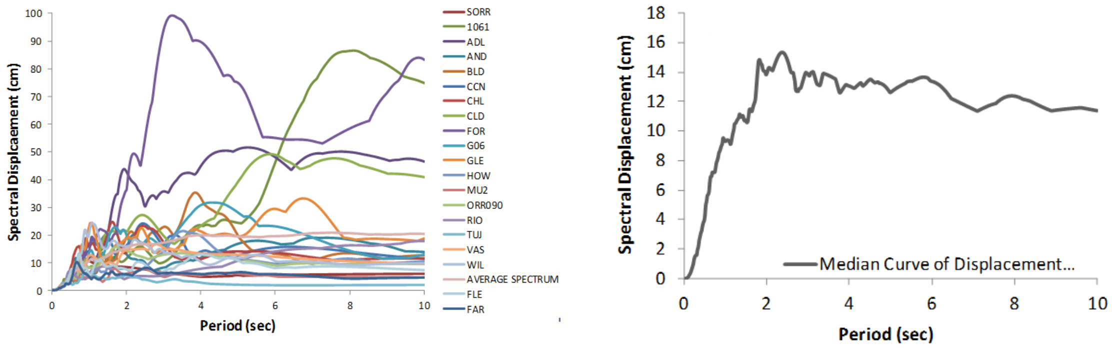
Modelling and analysing the sample RC frames under selected ground motions are implemented by COM3 (Concrete Model in 3D) finite element software to reach global collapse. This program; developed at the University of Tokyo, is capable of modelling the nonlinear behaviour of concrete members under various types of seismic loading. Modelling the RC elements is performed according to governing rules on reinforcing rebars and concrete behaviour before and after cracking. Cracked concrete model includes Shima's tension stiffening model (Shima *et al.* 1987), Maekawa's compression model (Maekawa and Okamura 1983). Structural members are modelled by 3-node fiber element with six degrees of freedom (3 rotational and 3 transitional). Definition of structural characteristics such as cross-sectional area, elasticity modulus of steel and its yield strength, compressive and tensile strength of concrete, rebar percentage of the section and initial Poisson's ratio is required for modelling the fiber element. In this paper the story collapse mechanism due to formation of plastic hinges in beams and columns (flexural and axial yielding) and also instability due to geometrical nonlinearity are considered, and it is assumed that all members have adequate shear capacity at this limit state.

2.1 Strong ground motion data

Time-history dynamic analyses were conducted by means of a selected suite of horizontal earthquake ground motions with moment magnitudes larger than 6.5 and site-to-source distances less than 30 km. The recorded motions; proposed by Shome *et al.* (1999) for soil type C in NEHRP classification (soil type II in 2800 Seismic Code of Iran), are from Pacific Earthquake Engineering Research Center (PEER) strong motion database. Characteristics of the ground motions set; displacement spectra (for damping ratio of 5%) and the 50th percentile (median) of all records have been illustrated in Table 4 and Figs. 2 (a)-(b) respectively.

Table 4 Candidate ground motion records

Record ID	Event	Year	M	R (km)	Station	PGA(g) Component		Selected Component
						X	Y	
FOR	Mendocino	1992	7.1	23.6	Fortuna	0.114	0.116	Y
RIO	Mendocino	1992	7.1	18.5	Rio Overpass	0.459	0.385	X
1061	Duzce	1999	7.1	15.6	Lamont 1061	0.134	0.107	X
FAR	Northridge	1994	6.7	23.9	N Faring Rd	0.273	0.242	X
FLE	Northridge	1994	6.7	29.5	Fletcher Dr	0.162	0.24	Y
G06	Loma Prieta	1989	6.9	19.9	Gilroy Array #6	0.170	0.126	X
AND	Loma Prieta	1989	6.9	21.4	Anderson Dam	0.240	0.244	Y
ADL	Loma Prieta	1989	6.9	21.4	Anderson Dam	0.077	0.064	X
CLD	Loma Prieta	1989	6.9	22.3	Coyote Dam	0.179	0.160	X
ORR090	Northridge	1994	6.7	22.6	Castaic Ridge	0.514	0.568	Y
BLD	Northridge	1994	6.7	31.3	LA-Baldwin	0.168	0.239	Y
MU2	Northridge	1994	6.7	20.8	Beverly Hills	0.444	0.617	Y
TUJ	Northridge	1994	6.7	24	Big Tujunga	0.245	0.163	X
CCN	Northridge	1994	6.7	25.7	Canyon Country	0.222	0.256	Y
CHL	Northridge	1994	6.7	23.7	LA Chalon Rd	0.185	0.225	Y
GLE	Northridge	1994	6.7	17.7	Sunland	0.157	0.127	X
HOW	Northridge	1994	6.7	20	Burbank Howard	0.163	0.120	X
WIL	Northridge	1994	6.7	25.7	Hollywood	0.246	0.136	X
VAS	Northridge	1994	6.7	24.2	Vasquez Rocks	0.139	0.151	Y
SORR	San Fernando	1971	6.6	24.9	Castaic Ridge	0.268	0.324	Y



(a) Displacement spectra of ground motion records (b) 50^{th} percentile displacement spectrum

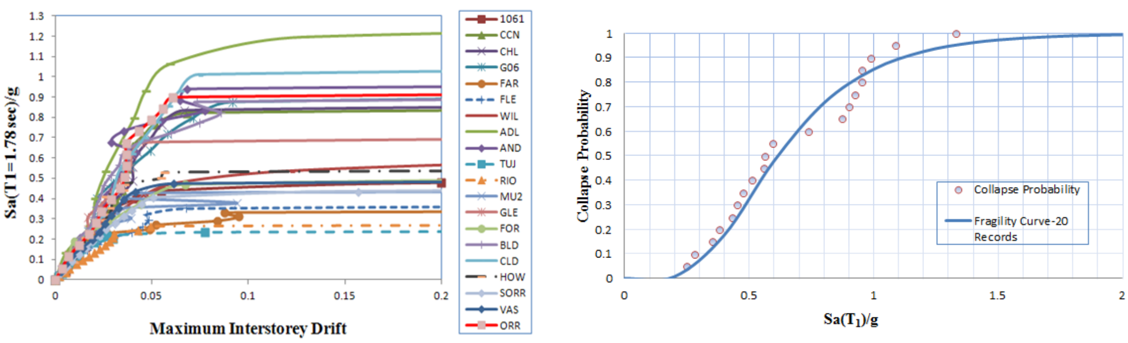
Fig. 2 Displacement spectra and median displacement spectrum of ground motion records

3. Collapse probability and fragility curves

To investigate the structural collapse probability, RC frames are analyzed under selected ground motions set and the dispersion of responses obtained by dynamic analyses is quantified. Nonlinear dynamic analyses are performed by using the ground motion component with maximum Peak Ground Acceleration (PGA) for each of the twenty records in the suite. To implement the IDA procedure, PGA values as intensity measure parameters are increased, according to FEMA 350 recommendations (2000) with the aim of reaching the collapse limit state. Fragility curves are applied to quantitatively estimate the collapse capacity of structures considering the dispersion of dynamic analysis results. Collapse fragility curves are produced by using the computed values of median and standard deviation and the lognormal Cumulative Distribution Function (CDF) which describes the collapse probability (Eq. (1))

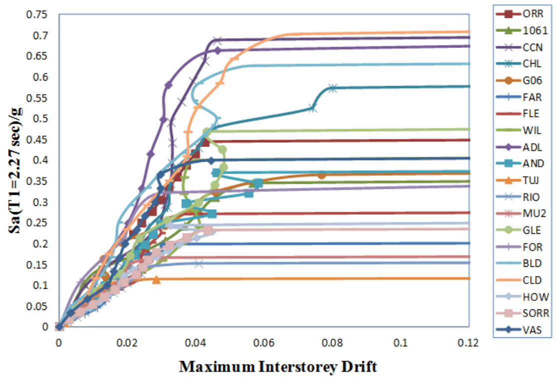
$$P\left[Collapse|IM\right]=\varphi\left[\frac{\ln(IM)-\ln(\mu)}{\sigma}\right] \tag{1}$$

where IM is the Intensity Measure, μ, σ and are mean and standard deviation of the structural collapse capacity respectively and φ is the standard normal distribution function. Considering the

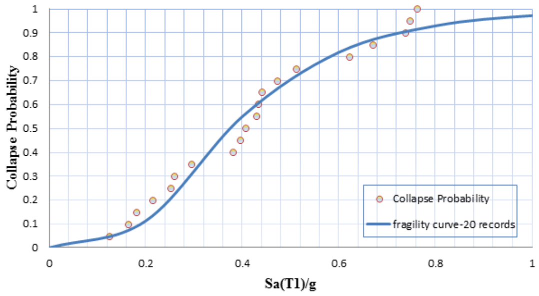


(a) IDA curves of 4-story RC frame (b) collapse fragility curve of 4-story RC frame

Fig. 3 IDA results and collapse fragility curves of 4-story RC frame



(a) IDA curves of 6-story RC frame



(b) collapse fragility curve of 6-story RC frame

Fig. 4 IDA results and collapse fragility curves of 6-story RC frame

effect of the 5% linear elastic spectral acceleration at the fundamental period of structure (Sa (T_1)) on reducing the dispersion of dynamic analyses curves, this parameter is considered as an appropriate *IM* for expressing the IDA and fragility curves. IDA results and fragility curves of sample RC frames are illustrated in Figs. 3-4, respectively.

4. Estimating the collapse capacity of structures

Obtaining the *IM* values corresponding to structural collapse capacity reduces the IDA calculations and consequently decrease the number of required ground motions for producing the seismic fragility curves. In order to estimate the $Sa(T_1)$ coefficients equivalent to the structural collapse drift limit, Nonlinear Static Pushover (NSP) curve and Capacity Spectrum (ADRS) of the structure are employed to find the acceleration values near collapse limit (the NSP curves obtained from pushover analysis of RC frames become bilinear according to FEMA356 (2000) to calculate the yield displacement of the structure and ductility parameter).

According to Vamvastikos studies (2002), accurate estimation of seismic demand and capacity of first-mode-dominated MDOF systems is achievable by using the relation between pushover and IDA curves. Thus, proposing a methodology for estimating the structural collapse capacity via converting the pushover curve to IDA approximate curves seems to be feasible.

To achieve this purpose, NSP curve of MDOF frames (in terms of base shear and peak lateral displacement) is first converted to corresponding pushover curve of the equivalent SDOF system via dividing the base shear of MDOF system by first-mode mass participation factor (Γ_1) defined as below (Eq. (2))

$$\Gamma_1 = \frac{\{\varphi\}^T [M]\{1\}}{\{\varphi\}^T [M]\{\varphi\}} \tag{2}$$

where $\{\varphi\}$ is the modal shape and M is the mass matrix of the structure. According to Priestley (2007) $\{\varphi\}$ is obtained based on the deformed shape of the structure. Equivalent displacement of SDOFsimulated system, x^r, is also calculated by Eq. (3).

$$x^r = \frac{x_t}{\Gamma_1} \tag{3}$$

where x_t is the displacement of MDOF system. Secant period (capacity spectrum slope) of the structure is obtained from Eq. (5) by using the structural effective mass (Eq. (4)) and structural effective stiffness for each point of related pushover curve

$$M^r = \{\varphi\}^T \{M\}\{1\} \tag{4}$$

$$T_{\sec} = 2\pi \left[\frac{M^r x^r}{Q_y{}^r} \right]^{1/2} \tag{5}$$

Equivalent spectral acceleration of the SDOF system for each point of pushover curve is defined as the SDOF system load (base shear) on that point divided by the SDOF system equivalent mass

$$Sa^r = \frac{Q}{M^r} \tag{6}$$

Ductility parameter is calculated for each point of the equivalent pushover curve according to the displacement value of that point (via dividing the maximum displacement of each point within collapse range by the yield displacement of the structure). Then the effective damping (ζ_{eff}) is obtained for all curve points based on ductility parameter obtained.

The conventional Capacity Spectrum Method (ATC-40) utilizes the secant period as the effective linear period of the SDOF system to obtain the maximum displacement or performance point (FEMA 440). The maximum displacement is obtained by intersecting the capacity curve of the structure and a demand curve for the effective damping in ADRS format (Fig. 5).

According to FEMA 440 the effective period is generally shorter than the secant period. Secant period (corresponding to the Modified Acceleration-Displacement Response Spectrum) is defined by the point corresponding to the maximum displacement (d_{max}) on capacity curve. A modification factor is used to correlate these periods (Eq. (7)).

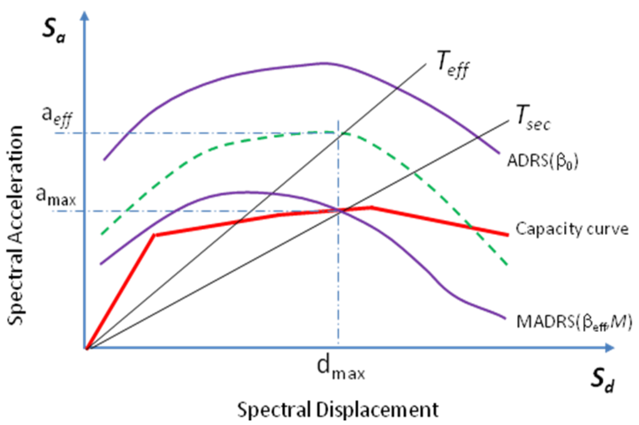


Fig. 5 Modified Acceleration-Displacement Response Spectrum (MADRS)

$$M = \left(\frac{T_{eff}}{T_{sec}}\right)^2 = \left(\frac{T_{eff}}{T_0}\right)^2 \left(\frac{T_{e0}}{T_{sec}}\right)^2 \tag{7}$$

In Eq. (7), T_0 is the structural fundamental period and the effective period (T_{eff}) is obtained from following equations (FEMA 440)

For 1.0<μ<4.0:

$$T_{eff} = \left[G(\mu-1)^2 + H(\mu-1)^3 + 1\right]T_0 \tag{8}$$

For 4.0<μ<6.5:

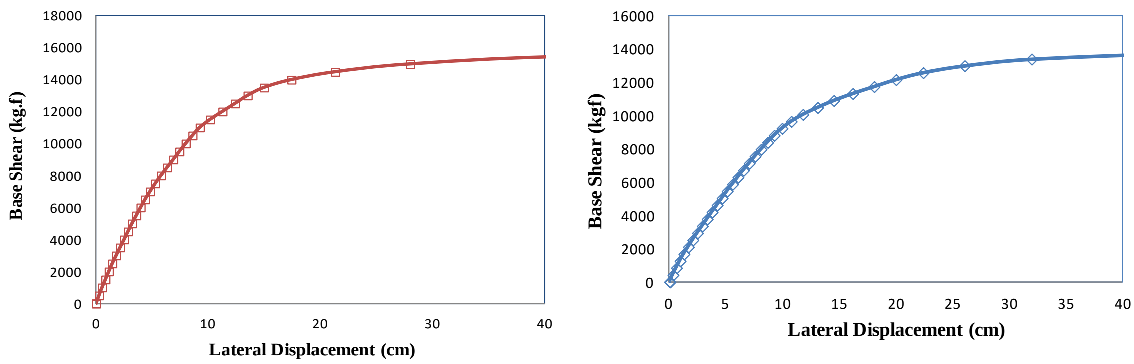
$$T_{eff} = \left[I + J(\mu-1) + 1\right]T_0 \tag{9}$$

As a result, effective period and spectral acceleration for effective damping are obtained for all ground motions in the suite and $Sa(T_1)$ is computed for all points of the pushover curve. The ratio of spectral acceleration of the SDOF simulated system to the spectral acceleration obtained based on (ζ_{eff}) is considered as the scale factor for ground motions. $Sa(T_1)$ value defining an estimated point of IDA curve is obtained by multiplying the computed scale factor to the 5% damped spectral acceleration at the fundamental period. These calculations are implemented until reaching a point at which a certain drop in load-displacement (pushover) curve is occurred. This point refers to the collapse capacity of the structure at which the IDA curve becomes completely horizontal (global instability of the structure is taken place).

Pushover curves obtained from nonlinear analyses of 4 and 6-story RC frames have been illustrated in Fig. 6. Results of computing the Sa (T_1) coefficients for estimating the IDA curves under all ground motions are illustrated in Fig. 7 for 4 and 6-story RC frames, as well. Fig. 8 also shows the estimated and real median (50$^{\mathrm{th}}$ percentile) curves of IDA results for two RC frames.

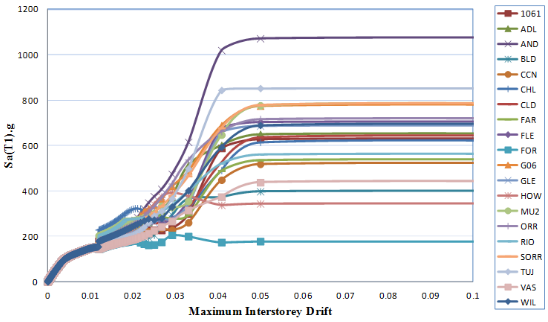
A comparison between the estimated and real fragility curves of 4 and 6-story RC frames has been also made in Fig. 9.

Comparing the estimated and real fragility curves of sample RC frames shows that converting the pushover capacity curve to IDA curves is an appropriate approach to estimate the collapse capacity of structures.

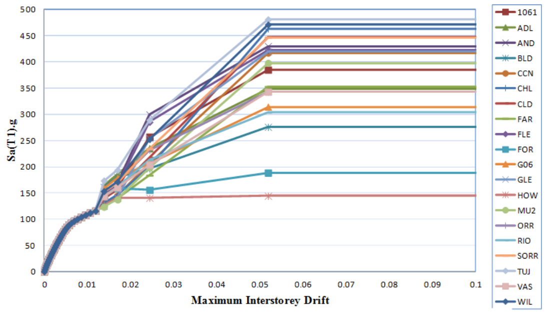


(a) Pushover curve of 4-story RC frame (b) Pushover curve of 6-story RC frame

Fig. 6 Pushover curves obtained from nonlinear analyses of 4 and 6-story RC frames

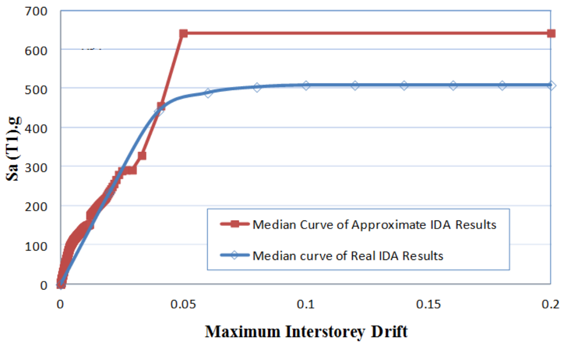


(a) IDA estimated curves for 4-story RC frame

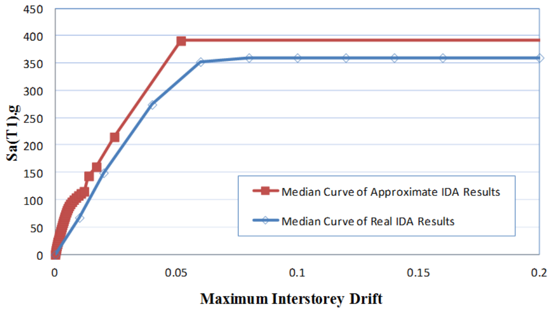


(b) IDA estimated curves for 6-story RC frame

Fig. 7 IDA estimated curves for 4 and 6-story RC frames

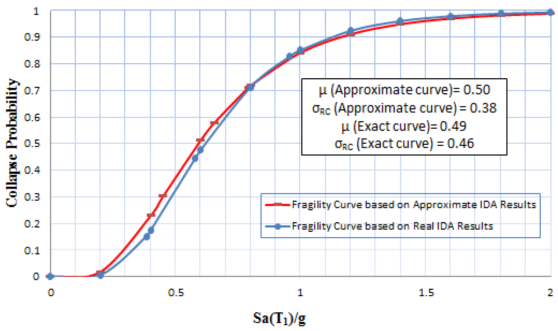


(a) Estimated and real median of IDA curves for 4-story RC frame

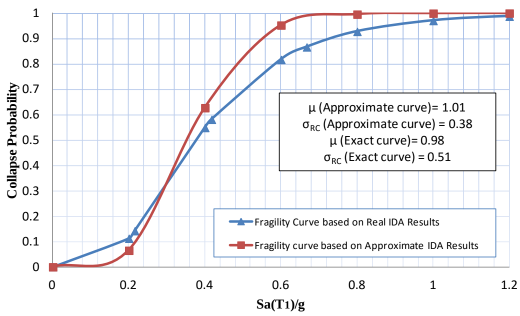


(b) Estimated and real median of IDA curves for 6-story RC frame

Fig. 8 Comparison between the approximate and real median of IDA curves for 4 and 6-story frames



(a) Estimated and real fragility curves of 4-story frame



(b) Estimated and real fragility curves of 6-story frame

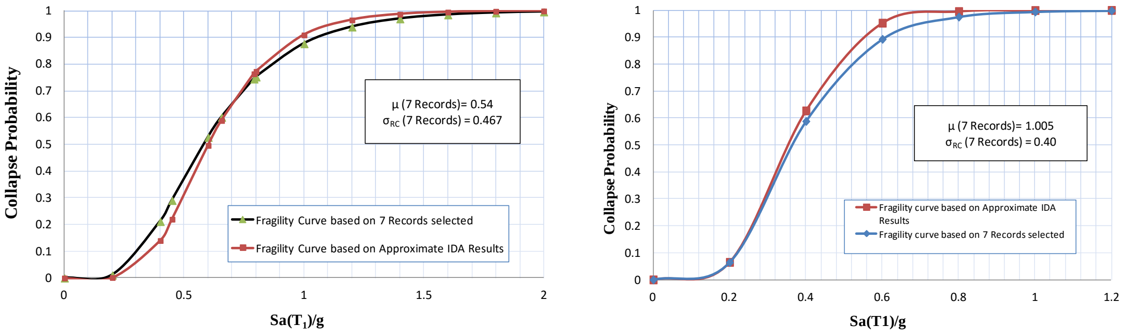
Fig. 9 Comparison between the Estimated and real fragility curves for 4 and 6-story RC frames

5. Selecting a limited number of ground motions to produce the collapse fragility curves

Calculating the collapse capacity limits based on estimated fragility curves is followed by selection of a limited number of ground motions (seven seismic records in general) with the most considerable effect on nonlinear response of the structure. As was stated already, the collapse capacity should be estimated via implementing the nonlinear dynamic analyses on structures under a set of limited ground motion records. To achieve this purpose, seven appropriate ground motions are selected based on the estimated fragility curves of structures via matching the mean (μ) and standard deviation (σ) values of the selected records with their corresponding values in the suite including all ground motions. Then the collapse fragility curve obtained based on seven records selected is compared with the collapse fragility curve resulted from all ground motions in the suite.

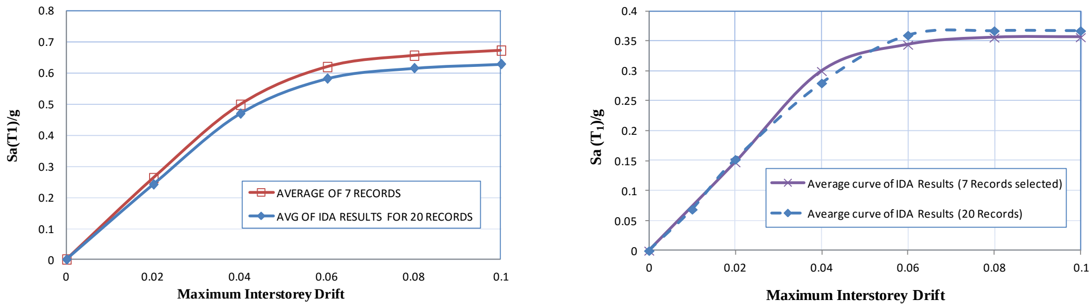
In practice, selection of a limited number of ground motions (seven ground motions as an optimum number) denotes the elimination of ground motions with negligible effect on probabilistic distribution of the structural response.

After selecting the most appropriate ground motions considering the dispersion effects, RC frames are analysed via IDA accurate approach under the seven records selected. To ensure that the selected records are reliable enough, mean curve of IDA results and fragility curve obtained based on selected ground motions is compared with mean curve of IDA results and fragility curve based



(a) Estimated fragility curve based on seven and all ground motions for 4-story frame
(b) Estimated fragility curve based on seven and all ground motions for 6-story frame

Fig. 10 Comparing the estimated fragility curve based on seven selected records and estimated fragility curve based on all records in the suite for RC frames



(a) Mean of IDA curves based on seven and all ground motions for 4-story frame
(b) Mean of IDA curves based on seven and all ground motions for 6-story frame

Fig. 11 Comparing the mean of IDA curves based on seven selected records and mean of IDA curves based on all records in the suite for RC frames on all records in the suite. Comparison of the results of seven and all ground motions has been illustrated in Figs. 10-11 for RC frames under investigation.

Considering the results of RC frames, it is concluded that the structural collapse capacity can be accurately estimated via substituting all the ground motions of a suite by seven records selected in addition to reducing the number of required nonlinear time-history dynamic analyses.

6. Generalizing the proposed approach for selection of ground motions

The proposed approach explained above aims to appropriately select and scale the ground motions required for implementing the time-history nonlinear analyses. To investigate the validity of this methodology, it should be examined for structures with more general characteristics. For this purpose, a multi-bay 6-story RC frame structure is studied in this section. The sample structure has been modelled and analysed by Aneshkani (2010) by means of COM3 finite element program under a set of far-field ground motions to investigate the structural and seismic (ground motions properties) uncertainties.

The structural system of the RC frame model is Special Moment Frame (SMF) with bay size and stories height equal to 3.0 meters. The structure under investigation has been subjected to gravity and seismic loads according to Iranian National Building Code (Part 6: Loads, 2004) and Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard No. 2800, 2005) provisions, respectively and has been also analysed according to Iranian Building Code for Concrete Structures (ABA, 2006) and Iranian National Building Code (Part 9: Reinforced Concrete Structures, 2005). Fig. 12 shows the 3D view of the 6-story RC frame structure. Characteristics and design properties of this structure have also been shown in Table 5 as well as the cross-sectional properties of the structure in Table 6.

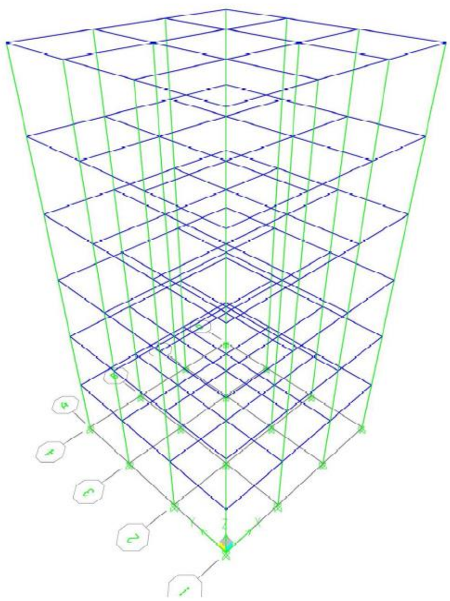


Fig. 12 3D view of 6-story RC frame

Table 5 Characteristics and design parameters of 6-story RC frame structure

Concrete Compressive Strength (kg/cm^2)	Steel yield strength (kg/cm^2)	Elasticity Modulus of Steel (kg/cm^2)	Live Load of floors on beams (kg/m)	Live Load of roof on beams (kg/m)	Live Load Factor	Dead Load of floors on beams (kg/m)	Dead Load of roof on beams (kg/m)	Design PGA (g)	Importance Factor
210	4000	2000000	200	150	0.20	530	500	0.35	1

Table 6 Cross-sectional properties of structural members for 6-story RC frame structure

		1st Floor	2nd Floor	3rd Floor	4th Floor	5th Floor	6th Floor
Columns cross-sectional properties	Width (cm)	35	35	35	35	30	30
	Height (cm)	35	35	35	35	30	30
	Corner Rebars	4ϕ14	4ϕ14	4ϕ14	4ϕ14	4ϕ12	4ϕ12
	Side Rebars	8ϕ14	8ϕ14	4ϕ14	4ϕ14	4ϕ12	4ϕ12
Beams cross-sectional properties	Width (cm)	30	30	30	30	30	30
	Height (cm)	30	30	30	30	30	30
	Top Rebars	3ϕ20	3ϕ20	3ϕ20	3ϕ16	3ϕ14	3ϕ14
	Bottom Rebars	2ϕ20	2ϕ20	2ϕ20	2ϕ16	2ϕ14	2ϕ14

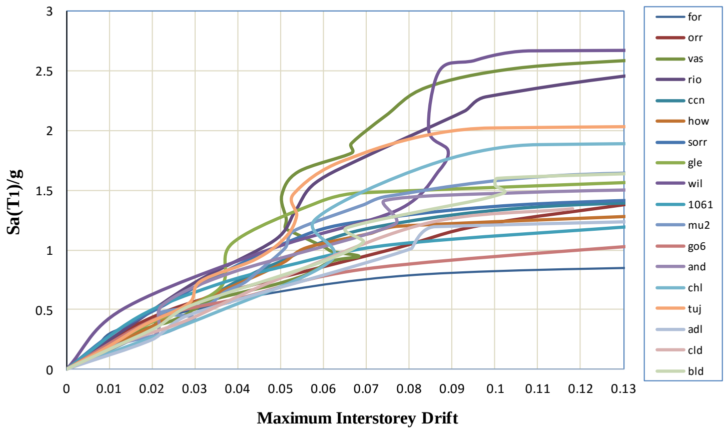


Fig. 13 IDA exact curves of 6-story RC frame

In order to implement the nonlinear dynamic analyses, a 3-meter bay space frame has been selected and modelled by COM3 software. RC frame members have been modelled by fiber element and based on appropriate details for divided cells and longitudinal rebars and stirrups. Considering the RC moment frame fundamental period of T_1=1.2 sec, the spectral acceleration at the first mode period is computed for each seismic record of the suite of ground motions. IDA curves of the RC frame under investigation have been plotted under a set of far-field ground motions (similar to the suite used for analysing the 4 and 6-story RC frames) in terms of $Sa(T_1)$ parameter (Fig. 13).

IDA estimated curves are also obtained based on the proposed method (converting the structural Pushover capacity curve to IDA curves) applied to 6-story RC frame structure. Fig. 14 illustrates the comparison between the estimated and exact median (50th percentile) of IDA curves.

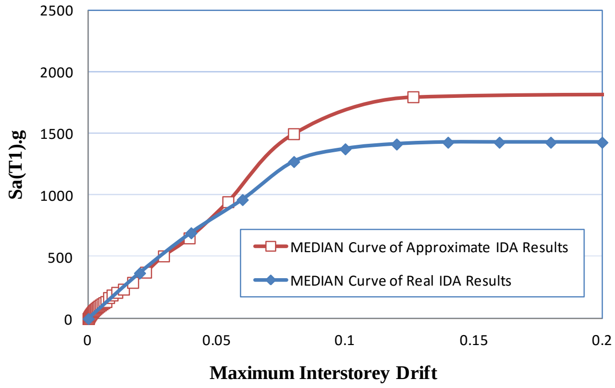
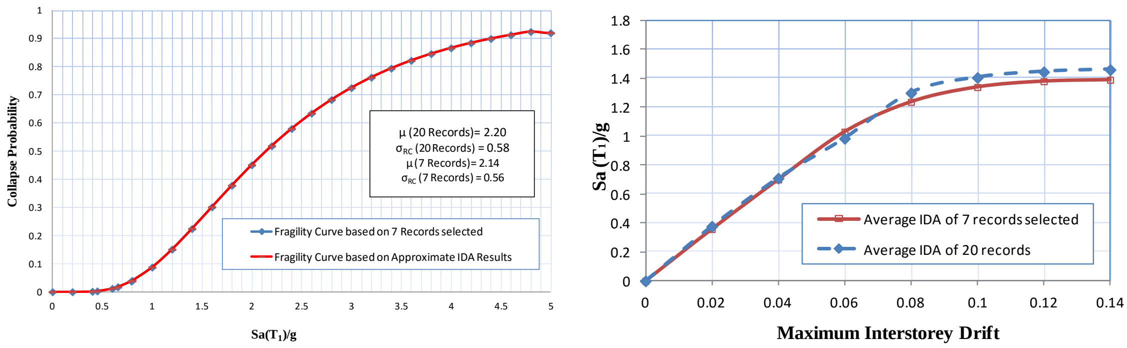


Fig. 14 comparison between the estimated and exact median of IDA curves



(a) Estimated fragility curve obtained by seven ground motions selected and fragility curve resulted from all ground motions in the suite

(b) Mean of IDA estimated results based on seven ground motions selected and mean of IDA results based on all seismic records in the suite

Fig. 15 Comparison between the fragility curve and mean of IDA curves based on seven records selected and fragility curve and mean of IDA curves based on all records in the suite for 6-story RC frame structure

Seven ground motions are selected based on the estimated fragility curves resulted from IDA estimated results. Fig. 15(a) shows the comparison between estimated fragility curve obtained by seven selected records and fragility curve resulted from all ground motions in the suite. Also Fig. 15(b) illustrates the comparison between mean curve of IDA estimated results based on selected ground motions and mean curve of IDA results based on all seismic records in the suite.

7. Conclusions

A theoretical approach to appropriately select and scale ground motions for seismic design of the RC frame structures at the collapse limit state has been proposed herein. To predict the mean (average) seismic response of structures, estimation of IDA and fragility curves has been done via applying the pushover curve obtained from nonlinear static analysis. Reducing the number of required dynamic analyses to obtain the spectral acceleration ($Sa(T_1)$) values and nonlinear structural response (at collapse limit state) is achievable through matching the mean (μ) and standard deviation (σ) values of collapse fragility curve obtained based on the limited ground motions selected with their corresponding values of the collapse fragility curve obtained based on all records in the suite. According to the analytical results obtained from RC frame structures studied, it is concluded that the proposed method is capable of appropriately estimating the IDA and collapse fragility curves of the structures via selection and scaling a limited number (seven is the optimum number) of ground motions. As a result, through applying this computationally advantageous and time-saving method (in comparison with performing the IDA process for a large number of ground motions) seven ground motions selected can be substituted for all seismic records of the suite. The extension of the proposed approach for collapse analysis of structures with higher modes effects, can be the topic of future study in this area.

Acknowledgements

The authors greatly acknowledge Professor Koichi Maekawa at Civil Engineering Department of the University of Tokyo for supporting this study by providing the COM3 software.

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