RAPID ESTIMATION OF FRAGILITY CURVESUSING ENDURANCE TIME METHOD

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ABSTRACT

Randomness of earthquakes' inherent causes scattering of structural responses. The most complex analytical methods such as risk, hazard and performance based design try to estimate seismic responses properly. Most of the time, damage probability of structures are studied using fragility curves. Multi-strip analysis (MSA) and incremental dynamic analysis (IDA) are the most dynamic analysis methods which evaluate seismic responses in different intensity measures (IMs). Numerous nonlinear dynamic analyses needed in mentioned methods to evaluate response distributions lead to complexity and time consuming process of them. Endurance time analysis (ETA) evaluates structural responses in different IMs by using artificial intensifying acceleration functions with least dynamic analysis. In this paper a new approach has been suggested to obtain fragility curves rapidly using ETA. Hence, the capability of ETA is evaluated to determine fragility carves by making use of equivalent SDOF instead of MDOF system. Results show that ETA method applied to an equivalent SDOF system predicts MSA fragility curves obtained by analysing a MDOF system with appropriate accuracy, by applying an uncertainty factor of 0.6 to lognormal standard deviation of ETA method.This approach reduces hugecomputational efforts and consumed timewhich are spent on the otherproccesswith a neglecting tolerance.

INTRODUCTION

To study precisely on responses of buildings subjected to earthquake, some complex seismic analysis have been developed that each one gives some results with different difficulties.

Hazard analysis, seismic risk assessment and performance based design are some of advance approaches inthe seismologic science.They study probability of exceedance in responses from limit state. The probability of exceedanceis illustrated by cumulative distribution functions called fragility curves.

Fragility functions use structural response distributions obtained from nonlinear dynamic analysis. There are many analysis methods which evaluate response distributions of structures. Multi-strip analysis (MSA) is the most well-known procedure which has been proposedrecently.MSA consist of a set of nonlinear time history analysis in which various ground motions (GMs) are scaled to desire intensity measure (IM) and repeated by increasing IMs' level (Jalayer(2003)). Therefore, distribution of engineering demand parameters (EDPs) could be evaluated in each IMs' level. The number of selected GMs is an important parameter in MSA. To achieve an accurate response distribution, various GMs should be selecected and structural analysis for different desired IMs.This procedure leads toa time consumig process. Moreover,Endurance time analysis (ETA) method is basically a time history analysis except that in ETA, structures are subjected to an artificial intensifying acceleration function and structural responses are monitored during the analysis. On the other hand, ETA method is a simple dynamic pushover test that efforts to predict structural demands at deferent intensity measures (IMs). This method has been originally proposed as a dynamic analysis method by Estekanchi et al. (2004). In the ETA method, each time in related to a specific intensity of earthquake in a defined performance level. So, performance of a structure at different IMs can be assessed with an ET analyzing. However, three predesigned excitations (acceleration functions) can be used to get more precise results. Then, averages of their results are considered as responses of the structure.Jamshidi and Estekanchi (2012) have studied the possibility of estimationof fragility curves by using ETA. In this paper a rapid method is introduced to obtain fragility curves with an acceptable accuracy.

Some studies have been tried to simplify the structural models. Fajfar (2000) represents a new approach which estimates a MDOF responseby an equivalent SDOF system. Furthermore, many researches have been performed to evaluate the accuracy of this approach in nonlinear analysis (Saadaie&Nassarasadi (2012)).In this paper, fragility curves achieved by ETA method and equivalent SDOF systems are compared with those of MSA method and MDOF systems.

METHODOLOGY

In this paper, in order to attain a simple and rapid procedure to estimation of fragility functions,following steps have been excecuted:

1) Simplifyingthe MDOF model to its equivalent SDOF system by employing the method proposed by Fajfar.
2) Reponse distributions and fragility functions of equivalent SDOF system are evaluated utilizing nonlinear dynamic ETA.
3) The variation of response distribution and fragility curves of MSA analysis on the MDOF system are obtained. Similarly, the same procedure is employed for ETA on the equivalent SDOF system. At the end,the results achieved by the two methods are compared.

STUDIED MODEL

A 2Dsteel moment resisting frame has been analysed according to Iranian seismic code (ISC)and designed based on National Building code (INBC) section 10. It is assumed that the frame elements resist 2.5$^{ton/m}$ and 1$^{ton/m}$, dead load and liveload, respectively. General dimensional information of the frame and its profile sections are shown in Figure 1.

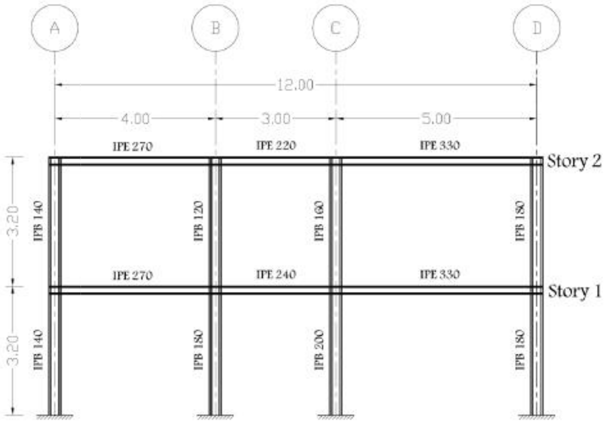

Figure 1.2D moment resisting steel frame geometry and profile section of frames elements

MDOF system can be simplified to an equivalent SDOF presented by Fajfar. Based on this approach, the MDOF sysytem isequallized to a lumped mass SDOF system in which its nonlinear behaviour is determined from pushover analysis and its equivalent mass is extracted from modal analysis. The schematic procedure of simplying MDOF to equivalent SDOF is illustrated in Figure 2.

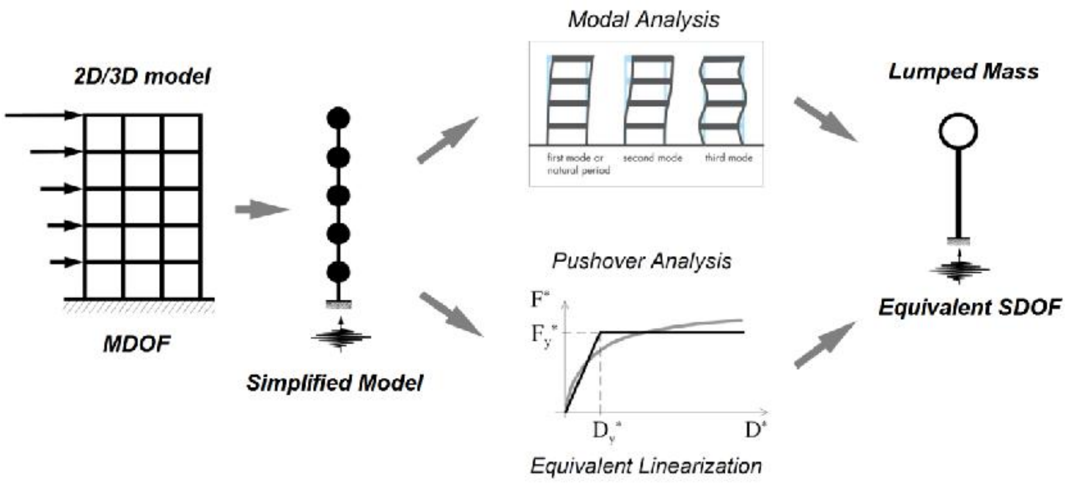

Figure 2. Fajfar procedure to evaluate the equivalent SDOF from a real MDOF system Fajfar (2000)

Dominant period T=0.753^{sec} and first mode shape vector ϕ = [0.4467 1] are evaluated by modal analysis in 2D-frame with story mass vector M= [3.5 3.25]. Therefore, Equivalent mass M*=5.888^{KN} and transformation factor Γ=1.2191 are concluded from Fajfar procedure. Pushover and fitted bilinear curves determined by nonlinear static analysis are illustrated in Figure 3.

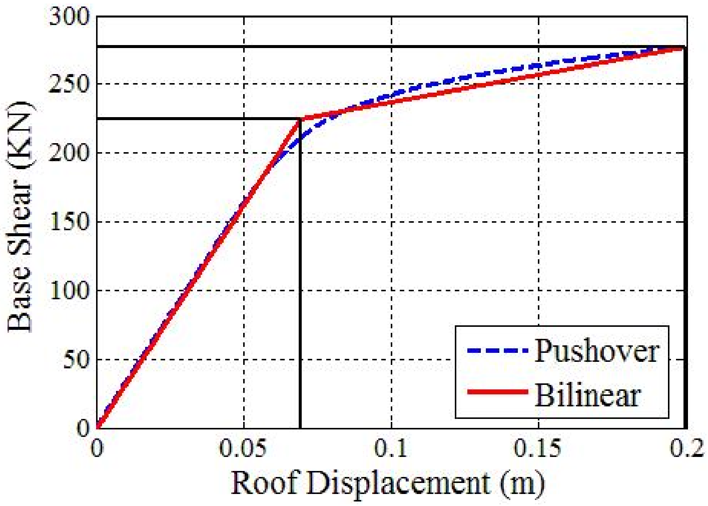

Figure 3.Pushover and bilinear fitted curve resulted from nonlinear static analysis

ANALYSIS

The accuracy of MSA method is depended on selected GMs. In this way, a numerous earthquake events including 44 GMs which are categorized in soil type II ($375^{m/s} \leq V_{s,30} \leq 750^{m/s}$)based on ISC and known as far-fault (Rjb>15^{km}) have been selected from PEER ground motion database[1]. Selected GMs with their specific record sequence number (RSN) defined in PEER ground motions database are tabulated in Table 1.

Nonlinear behaviour of the MDOF system is considered in dynamic analysis. In the procedure, each GM is scaled to IM levels and is applied to the frame. Therefore, there are 44 separate nonlinear dynamic analysis in each IM level. In this paper, MSA procedure is included 15 IM levels that leads to 660 separate analysis.

Three ET acceleration functions used in this research (ETA20e set) have been designed in such a way that their response spectrum remains proportional to that of average of seven real ground motions recorded on a stiff soil condition. These seven accelerograms are recorded on soil type II of ISC. Average response spectrum of the seven ground motions is used as the target response spectrum in creating ET acceleration functions (Riahi, 2009). The frame is subjected to ETA20e set and its results are extracted. Results of ET

1. http://peer.berkeley.edu/products/strong_ground_motion_db.html

Table 1. Selected far fault ground motions located in soil type II.

RSN#	Event	Year	Mag	Rjb (km)	Vs30 (m/s)	RSN#	Event	Year	Mag	Rjb (km)	Vs30 (m/s)
41	Lytle Creek	1970	5.33	103.4	450.3	511	N. Palm Springs	1986	6.06	38.2	684.9
45	Lytle Creek	1970	5.33	17.9	684.9	512	N. Palm Springs	1986	6.06	51.9	684.9
47	Lytle Creek	1970	5.33	90.5	425.3	586	New Zealand-02	1987	6.6	68.7	424.8
55	San Fernando	1971	6.61	111.4	438.3	587	New Zealand-02	1987	6.6	16.1	424.8
58	San Fernando	1971	6.61	92.2	477.2	596	Whittier Narrows-01	1987	5.99	25.9	545.7
88	San Fernando	1971	6.61	24.7	376.1	731	Loma Prieta	1989	6.93	41.7	391.9
89	San Fernando	1971	6.61	61.8	669.5	740	Loma Prieta	1989	6.93	19.9	488.8
124	Friuli, Italy-01	1976	6.5	102	659.6	745	Loma Prieta	1989	6.93	71.3	376.1
135	Santa Barbara	1978	5.92	23.8	438.3	750	Loma Prieta	1989	6.93	79.2	597.1
288	Irpinia, Italy-01	1980	6.9	22.5	500	1159	Kocaeli, Turkey	1999	7.51	141.4	659.6
291	Irpinia, Italy-01	1980	6.9	27.5	530	1162	Kocaeli, Turkey	1999	7.51	31.7	424.8
293	Irpinia, Italy-01	1980	6.9	59.6	659.6	1168	Kocaeli, Turkey	1999	7.51	293.4	659.6
294	Irpinia, Italy-01	1980	6.9	51.7	460	1169	Kocaeli, Turkey	1999	7.51	53	659.6
299	Irpinia, Italy-02	1980	6.2	41.7	500	1170	Kocaeli, Turkey	1999	7.51	51.2	424.8
302	Irpinia, Italy-02	1980	6.2	22.7	530	1172	Kocaeli, Turkey	1999	7.51	164.2	659.6
304	Irpinia, Italy-02	1980	6.2	64.4	460	1184	Chi-Chi, Taiwan	1999	7.62	19.9	549.6
323	Coalinga-01	1983	6.36	55	408.9	1211	Chi-Chi, Taiwan	1999	7.62	38.7	574.7
352	Coalinga-01	1983	6.36	38.1	438.3	1214	Chi-Chi, Taiwan	1999	7.62	56.7	411.5
424	Coalinga-08	1983	5.23	17.8	617.4	1600	Duzce, Turkey	1999	7.14	131.2	523
439	Borah Peak, ID-01	1983	6.88	84.8	424.8	1619	Duzce, Turkey	1999	7.14	34.3	659.6
472	Morgan Hill	1984	6.19	31.9	622.9	1620	Duzce, Turkey	1999	7.14	45.2	471

analysis are incremental because of inherent increasing in ET acceleration function intensity. Moreover, EDPs of structures can be resulted in different IMs. In this way, in order to extract the EDPs related to their IMs, the process which has been represented by Riahi (2009) is used.

As it is describe previously, MSA procedure conducts 660 separate nonlinear dynamic analysis while ETA uses only 3 intensifying acceleration functions. This comparison shows that ETA method inherently has more rapid procedure than MSA one.

ANALYSIS RESULTS

In this section, results of the two methods mentioned before are studied and compared. Selected MDOF 2D-frame and its equivalent SDOF have been analysed by MSA and ETA methods. Peak ground acceleration (PGA) and roof displacement are considered as IM and EDP, respectively. Furthermore, Median of response distribution (with 50% probability of occurrence), median minus a deviation (16%) and median plus a deviation (84%) are evaluated in both MSA and ETA. Roof displacements have beenplottedfor increment of PGA in Figure 4. It is observed in Figure 4 that results of MDOF and equivalent SDOF are close to each other. Thissimilarity is also true for results between MSA and ETA.

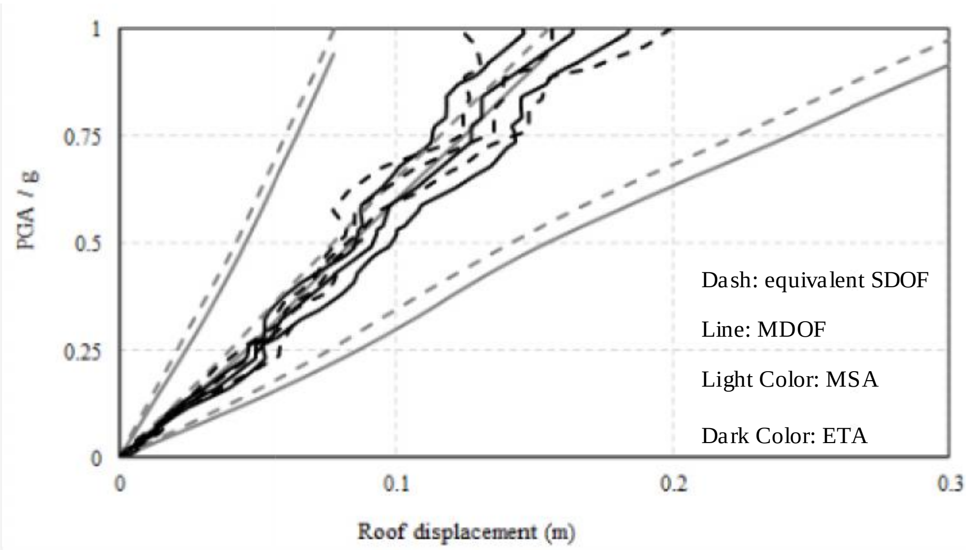

Figure 4. Response distribution of 2D MRF and its equivalent SDOF determined by MSA and ETA

The probability of exceedance of limit state responses are called fragility functions. It indicates probability of a system which experiences desired damage at excess of a specified level. The probability that a component (D) reaches or exceeds damage state threshold (d_i), given a particular IM value can be determined by equation 1:

$$P(D > d_i \mid IM) = 1 - P(D < d_i \mid IM) = 1 - \Phi\left(\frac{\ln(IM_i) - \ln(IM_{Mean})}{{}_{IM}}\right) \quad (1)$$

Where; $P(D > d_i \mid IM)$ is probability of exceedance in ith response from demand threshold given IM, IM_i is the ith intensity measure, IM_{Mean} is intensity measures with probability of exceedance of 50% and ${}_{IM}$ is logarithmic standard deviation of response. Equation 1 is generated by assuming the probability of exceedence is idializezd by lognormal distribution. There are some references presenting different limit damage state definitions which are qualitative mostly. HAZUS-MH MR3 (2003) represents 4 different damage state limitation: slight, moderate, extensive and complete in which it is assumed that minor deformation in connections happen in slight damage up to significant portion of structural elements exceed their ultimate capacities in collapse thresholds.Hence, the parameters of fragility functions resulted from dynamic analysis of MSA and ETA are evaluated based on all four defined thresholds and summarized in Table 2.

Table 2. Parameters of fragility function estimated by MSA and ETA

		Slight		Moderate		Extensive		Complete	
		IM_{Mean}	${}_{IM}$	IM_{Mean}	${}_{IM}$	IM_{Mean}	${}_{IM}$	IM_{Mean}	${}_{IM}$
MDOF	**MSA**	0.1365	0.6703	0.2844	0.7104	0.7760	0.6590	1.7716	0.5620
	ETA	0.1295	0.0838	0.2619	0.1214	0.7672	0.1588	3.0071	0.1000
Equivalent SDOF	**MSA**	0.1573	0.6606	0.3231	0.6728	0.8323	0.6226	1.8423	0.5404
	ETA	0.1293	0.0779	0.2507	0.1412	0.7533	0.1178	3.0071	0.1000

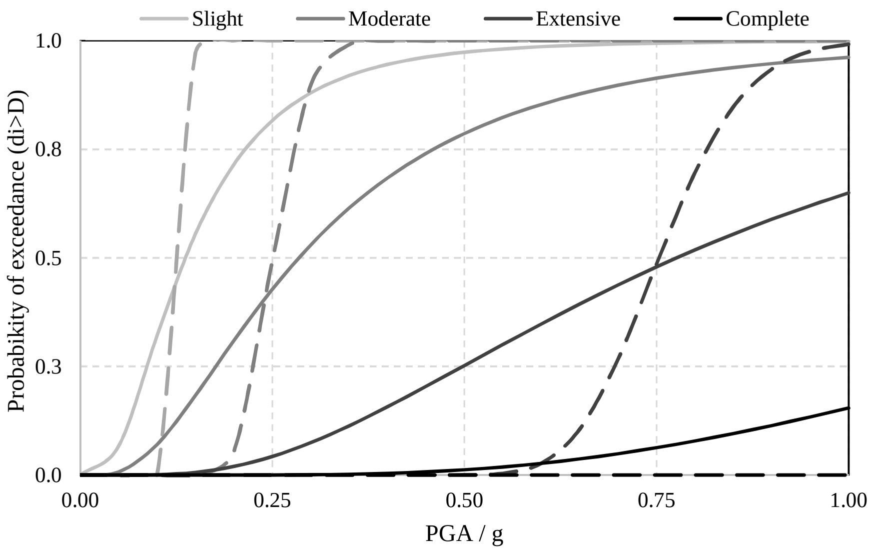

Figure 4 . ETA and MSA obtained fragility curves for SDOF and MDOF respectively

In Figure 4, Fragility curves obtained by ETA from equivalent SDOF are compared with those of MSA on the MDOF frame. As can be seen in this figure, there are too differences between fragility curves of ETA and MSA. However, it is shown in Table 2 that their IM_{mean} are close. It seems that fragility curves could be fitted by modifying standard deviations. Lognormal standard deviations show uncertainty of different aspects of structural responses. The total variability of fragility curves decrbied by lognormal standard deviation is modeled as a combination of uncertainty which is defined by Hazus.

$${}_{Tot} = \sqrt{{}_{IM}^{2} + {}_{M}^{2} + \sum {}_{Ui}^{2}} \quad (2)$$

Where; $_{Tot}$ is the lognormal standard deviation that describes the total variability for structural damage state, $_{IM}$ is lognormat standard deviation due to analysis uncertanity, $_{M}$ is the lognormal standard deviation parameter that describes the uncertainty in the estimate of the median value of the threshold of structural damage state which Hazus consider $_{M} = 0.4$ and $_{Ui}$ is lognormal standard deviation of other possible uncertinaty.

Since ETA is a method based on artificial acceleration functions and also an equivalent SDOF system has been used instead of MDOF, an uncertainity factor emerges. Therefore, a parameter can be defined as $_{Ui}$ in equation 2. Results obtained by MSA applied to MDOF system are considered as a reference.

An uncertainity in the approach which is described in this paper ($_{Ui}$) is calculated to minimize deferences between the fragility curves extracted by the methods. This minimizing procedure shows that by applying an uncertainty factor of $_{Ui} = 0.6$ tolognormal standard deviation of ETA method, MSA fragility curves can be predicted. Fragility curves modified by mentioned process whichare obtained by ETA on equivalent SDOF are compared with those of MSA in the MDOF frame in Figure 6.As can be seen in this figure, modified ETA fragility curves predict the MSA ones with an appropriate accuracy.

To summarize, fragility curves concluded by ETA analysis on equivalent SDOF which is modified by an uncertainty parameter can estimate fragility curves by MSA analysis on MDOF with an acceptable tolerance. It should be mentioned that ETA has a neglegible tolerance considering the great reduction in computational effort and consumed time resulted by ETA.

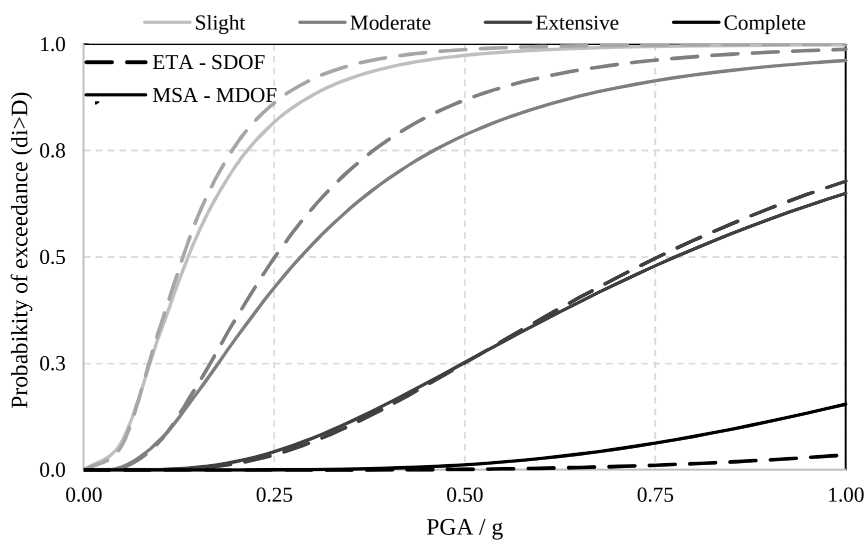

Figure 6 .ETA and MSA obtained fragility curves for SDOF and MDOF respectively

CONCLUSIONS

In this paper a new approach has been studied to achieve fragility carves of a two dimensional steel frame with least time and effort. Fragility carves of the MDOF frame and its equivalent SDOF system have been analysed and compared by MSA and ETA methods, respectively. At first, it has been concluded that their fragility curves are not compatible only their values in 50 % of probability. It concludes that median of responses distributions of the frame for desired EDP is predicted by ETA method while standard deviations is not in the same range. With applying an uncertainty factor equal to 0.6 to lognormal standard deviation of ETA method, because ETA is a method based on artificial acceleration functions and also an equivalent SDOF system has been used instead of MDOF, ETA fragility curves have been modified. Modified curves are so compatible with those of MSA method with an acceptable accuracy. In this research, this approach reduced 660 nonlinear dynamic analysis on a MDOF to only 3 nonlinear dynamic analysis on an equivalent SDOF system. It is very usefull and helpfull for optimizing of time, computational effort and complexity of the proccess to obtain fragility curves and consequently evaluate probability assessment. It should be mentioned that the unceraintity factor (0.6) applied to lognormal standard deviation of ETA method, is obtained only for the frame which is studied in this paper. To obtain this type of uncertainity for other frames and structures, more researchs should be implemented on them.

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