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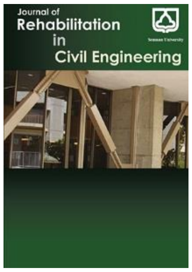

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Considering the Yielding Displacement Uncertainty in Reliability of Mid-Rise R.C. Structures

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ABSTRACT

In structural analysis and design, there are always uncertainties in determining loads and capacities. Structural reliability quantitatively considered uncertainties in analysis and design procedure. One of the well-known criteria to assess structural reliability is the Total Reliability Index (TRI) of structures. Yielding Displacement (YD) is an important component for calculations of TRI. Due to the changes in the analysis method, input type, normalization procedure, and the definition of target displacement, there are uncertainties in YD calculation. In structural reliability studies, both loads and resistance parameters are modeled as random variables. Therefore, the YD can be considered as a random variable. This study utilizes incremental dynamic analysis (IDA) to calculate TRI in mid-rise reinforced concrete moment resistant frames with intermediate ductility. The effect of uncertainty caused by YD is calculated based on pushover dynamic analysis. The reliability indices for the six structures of 3, 5, and 8 stories and three and five-span reinforced concrete moment frames show that the uncertainty caused by the YD reduces the TRI, but does not affect the seismic performance of the structure, significantly.

1. Introduction

After Northridge earthquake in 1994, extensive studies were carried out to find the role of uncertainty in the structures performance evaluation and its contribution to the seismic risk reduction [1]. There are different methods for seismic performance evaluation and safety estimation of structures. Each method considered the specific aspects of structural behavior via deterministic or probabilistic approaches. Probabilistic methods can consider the effects of various sources of uncertainty [2,

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3]. The reliability index is one of the most acceptable approaches among probabilistic methods. Reliability theory is a branch of the total probability theorem. This theory can model the uncertainty in the loading characteristics and structural parameters such as material properties, geometric dimensions, and nonlinear structural analysis; so nowadays it plays a key role in the analysis, design and safety assessment of structures [1, 4, 5 and 6].

Extensive studies have been conducted by various researchers on the types of uncertainties and their impact on the probability of structural failure and reliability index. Cornel et al. (2000) studying the SAC project, considered the seismic uncertainties in a probabilistic framework based on structural performance and reliability theory [2]. Based on performance and reliability theory, Humburger (1996) proposed an approach that considers a variety of uncertainties in the estimation of seismic demand, structure capacity, and reliability index, quantitatively [7]. Yun et al. (2002) presented an algorithm that could consider the epistemic and random uncertainties in the analysis and provided a simple method for estimating the confidence level for satisfying the performance level in a given hazard [8]. Considering the aleatory and epistemic uncertainties, Dolsek (2011) postulated a simplified method for seismic risk assessment based on a closed-form solution for estimating the mean annual frequency of exceeding a limit state [9]. Yazdani et al. (2017) proposed a three-parameter lognormal distribution to describe uncertainty in structural seismic demand and recommended this distribution for reliability assessment in collapse prevention (CP) limit state [10]. Yazdani et al. (2018) showed that considering epistemic uncertainty in record selection and probabilistic distribution dramatically affects the reliability index [11]. Gaxiola-Camacho et al. (2018) proposed an alternative reliability-based methodology for the safety evaluation of structures under seismic loading. They emphasized that to more accurately estimate the probability of failure and reliability index, major sources of nonlinearity and uncertainty must be considered [12]. Noori and Memarpour (2018) investigated the incident angle of ground motion as one of the sources of uncertainty in the seismic response of buildings. Their Results demonstrated that the inter-story drift ratio increased between 30 to 33 percent due to the orientation of excitation [13]. Ge and Kim (2020) presented an approach to determine the most appropriate probabilistic parameters to update the damage propagation prediction model under uncertainty. This approach can improve the accuracy and reliability of life-cycle management of a deteriorating structure [14]. Nguyen et al. (2020) determined the failure probability of a planar steel frame using O-FCD method. They considered the uncertainties of the material and geometry parameters based on Monte Carlo simulation [15]. Rahgozar et al. (2021) studied the seismic reliability of controlled rocking steel cores (CRSCs) for low- and mid-rise archetypes using extensive nonlinear dynamic analyses based on a set of random variables. Their results indicated that the safety requirements of CRSCs are satisfied and the design procedure is reliable. They concluded that the failure probability for mid-rise CRSCs is more than low-rise archetypes [16]. Pouraminian et al. (2021) studied seismic reliability of low-rise moment resisting frame RC buildings using probabilistic analysis, Monte Carlo simulation, and Latin hypercube technique considering different uncertainties. Their results showed that the reliability of the buildings is over 90%. Increasing the building height decreases the reliability by about 3.5% per story [17].

TRI is a general criterion for assessing the safety of structures. This criterion considers the wide range of minor structural damages in small earthquakes to collapse in severe earthquakes at the same time [18]. Okano and Maegawa (2001) offer equations to calculate the TRI based on probability theory [19]. Then Takada and Yamaguchi (2002) suggest a two-step algorithm, based on load and resistance factor design for TRI assessment using nonlinear dynamic analysis [20]. In these studies, the basis of TRI calculations is total probability theorem and incremental dynamic analysis (IDA). Usually, In IDA ductility factor is selected as damage measure (DM). Ductility factor is defined as the ratio of maximum nonlinear displacement to yield displacement (YD). So, any error in estimating YD can enter an error in ductility factor that in turn introduces uncertainty in TRI calculations [20]. Therefore, the uncertainty of YD affects the accuracy of seismic performance evaluations. To obtain the YD, structural capacity curves are often used [21]. The structural capacity curve is used as the linear approximation of the structural response of static pushover analysis. Vaziri Vafa and Tasnimi (2014), revealed that bilinear capacity curves do not take into account the decrease of strength in the high displacement of non-linear static methods. So, it cannot be considered as a proper method under severe earthquakes. However, in nonlinear dynamic analysis, if the structures have not strength degradation, the responses are acceptable [22]. Fragiadakis and Vamvatsikos (2011) showed that the results of the IDA can be plotted in the format of the structural capacity curve (called dynamic pushover curve), which matches with the static pushover curve before collapse [23]. In this case, due to the different responses of the structure under earthquake records, the YD has uncertainty and may affect TRI calculations.

The present study investigates the effects of YD uncertainty in the TRI of mid-rise RC structures with intermediate moment resisting frames. Then using the concepts of dynamic pushover analysis, target displacement, and bilinear modeling of dynamic pushover curve, a method for investigation the effect of YD uncertainty on the calculation of TRI is proposed. TRI is developed for RC residential buildings, located in metropolitan Tehran (with very high seismicity), designed according to the Iranian building Codes [24-26]. Ground motion records would be selected by CMS method that is an important consideration in IDA and dynamic pushover analysis. The results of this research can lead to a more accurate evaluation of structural seismic performance and safety based on TRI.

2. Material and methods

2.1 Numerical simulation procedure

Structural failure happens because the applied loads are greater than structural capacity. In probabilistic evaluation, structural performance is expressed as a frequency of exceeding specific limit states. So, a probabilistic-based process can be used to determine the probability of limit states occurrence and calculate the reliability index of structures. The probability of exceeding the limit state (failure probability) depends on three key stochastic models: ground motion hazard curve, nonlinear dynamic demand, and structural capacity that calculated as Eq. (1) [27]:

$$P_f = [D > C] \tag{1}$$

where, C and D are seismic demand and capacity, respectively. Generally, to calculate the probability of failure (P_f), it is necessary to introduce two variables, namely damage measure (DM) and intensity measure (IM). The importance of IM selection is scaling of ground motion records, expression of the relationship between seismic attributes and structural behavior, and the availability of the seismic hazard results for it. In this study, spectral acceleration corresponding to the fundamental period with 5% attenuation $S_a(T_1.5\%)$is selected as IM [28]. The severity of the earthquake in the site is calculated using the probabilistic seismic hazard curve. Seismic hazard function ($H_{Sa}(s_a)$) provides the mean annual frequency of exceedance of the particular spectral acceleration (S_a). The hazard curve is a plot of the probability of exceedance of a spectral amplitude versus the spectral amplitude for a given period. This plot is linear when plotted in log-log scale and a straight line fit in the range of hazard levels of interest will have the functional expression in the form of Eq. (2) [8].

$$H_{Sa}(S_a) = k_0(S_a)^{-k} \tag{2}$$

where, k and k_0 show the slope and intercept of the regression line, respectively. To calculate these parameters, spectral accelerations corresponding to the considered hazard levels must be determined. In this study, spectral accelerations corresponding to hazard levels of 10% in 50 years (return period of 475 years) and 2% in 50 years (return period of 2475 years) for the regression through the power law function, in the metropolitan Tehran are used [29].

In IDA, the structure is exposed to the effects of several different records of earthquakes, and their mean responses are considered in the reliability index calculations [30]. IDA curve is the diagram of *IM* versus DM. The nonlinear dynamic analysis method of power law is used for probabilistic assessment in a wide range of intensity levels [31]. This method obtains the median of IDA diagram in logarithmic space using linear regression analysis, based on:

$$X = a\left(\widehat{D}\right)^{-b} \tag{3}$$

where, X and $\widehat{D}$ are medians of IM and DM, a and b are the intercept and slope of the IDA curve in logarithmic space, respectively. In order to estimate the reliability index, it is necessary to calculate the failure probability. To consider the failure probability in all ranges of damage measures at the same time, the total failure probability is calculated as [19]

$$P_f = \int_0^\infty H_\mu(\mu) f_{\mu_{cr}}(\mu) d_\mu \tag{4}$$

where, P_f is the total failure Probability, μ is the ductility factor, and $f_{\mu_{cr}}(\mu)$ is the probability density function of critical ductility factor. $H_\mu(\mu)$ is the hazard probability distribution function of the response ductility factor that is achieved by combining the seismic hazard curve and IDA curve as:

$$H_\mu(\mu) = k_a \mu^{-k_b}\ ; \quad \mu_n \le \mu \le \mu_{n+1} \tag{5}$$

where, $k_a = k_0 a^{-k}$and k_b = -k*b. By dividing the variable of damage measure to different intervals (μ_n to μ_{n+1}), the probability of failure in any interval of ductility factor (P_n) yields as Eq. (6):

$$P_n = \int_{\mu_n}^{\mu_{n+1}} k_a \mu^{-k_b} * f_{\mu_{cr}}(\mu) d\mu = \{\Phi(Z_{n+1}) - \Phi(Z_n)\}. k_a \exp(-k_b\lambda) . \exp\left({k_b^2 \xi^2}/{2} \right. \quad (6)$$

Where, $\lambda = E[\ln(\mu)]$, $\xi = Var[\ln(\mu)]^{1/2}$ and $\Phi(z)$ is the standard normal probability distribution function. Z_n and Z_{n+1} are standard normal variables that are calculated as:

$$Z_n = \frac{\ln \mu_n - (\lambda - k_b \xi^2)}{\xi}$$
$$Z_{n+1} = \frac{\ln \mu_{n+1} - (\lambda - k_b \xi^2)}{\xi} \quad (7)$$

Finally; the total failure probability is equal to the sum of probability of failure in ranges of ductility factors from zero to infinity, which is derived from Eq. (8)

$$P_f = \sum P_n \quad (8)$$

After calculating the total failure probability of the structure using standard normal cumulative distribution function (Φ), TRI (β) for structure is calculated as:

$$\beta = \Phi^{-1}(1 - P_f) \quad (9)$$

2.2. Uncertainties in YD

Uncertainty can be regarded as a property of the system which describes the defects in human knowledge about a system and its development. There are two sources of uncertainty in engineering: natural uncertainty (statistical) and knowledge uncertainty (epistemic). Ignoring effects of uncertainties leads to structural designs with incorrect knowledge of the possible range of behavior [32]. Using the theory of reliability, these uncertainties can be introduced in mathematical relations and safety considerations to be entered in the process of analysis and design of structures.

In the present study, due to the selection of the ductility factor as the damage measure, YD affects the ductility factor and structural parameters obtained from IDA curves. So, TRI may be changed. Also, YD estimation methods have uncertainties that affect the TRI calculation. Regarding the definition of ductility factor, the annual probability of exceedance and the demand-based ductility factor, in any range, can be calculated as [19]:

$$H_\mu(\mu) = H_\mu\left(\frac{D}{D_y}\right) = H_\mu\left(\frac{D}{m_{D_y}} \frac{m_{D_y}}{D_y}\right) \quad (10)$$

$$H_\mu(\mu) = k_a (\frac{D}{m_{D_y}})^{-k_b} (\frac{D_y}{m_{D_y}})^{k_b} \quad (11)$$

where, D, D_y and m_{D_y} are displacement response, YD and the mean value of YD, respectively. Considering the probability density functions of critical ductility factor μ_{cr} as $f_{\mu cr}\left(D/m_{D_y}\right)$, and probability density functions of YD as $f_{D_y}(D_y)$, and the hazard probability distribution function of the response ductility factor as $H_\mu\left(D/m_{D_y} . m_{D_y}/D_y\right)$, the probability of failure P_f is expressed by:

$$P_f = \iint_0^\infty H_\mu(\frac{D}{m_{D_y}} . \frac{D_y}{m_{D_y}}) * f_{\mu cr}\left(\frac{D}{m_{D_y}}\right) * f_{D_y}(D_y) d\frac{D}{m_{D_y}} dD_y \quad (12)$$

so, to calculate the probability of failure, the dual integral must be solved. By assuming log-normal distribution for probability density functions of ductility factor and YD, the failure probability function in each interval is simplified as Eq. (13) [20].

$$P_f = \{\Phi(Z_{n+1}) - \Phi(Z_n)\}k_a * \exp(-k_b\lambda_1) * \exp\left(\frac{k_b^2\xi_1^2}{2}\right) * \exp\left\{\frac{k_b(k_b-1)\xi_2^2}{2}\right\} \tag{13}$$

where, $\lambda_1 = E[\ln(\mu)]$, $\xi_1 = Var[\ln(\mu)]^{1/2}$, $\xi_2 = Var[\ln(D_y)]^{1/2}$, Z_n and Z_{n+1} is the standard normal variables that is obtained from Eq. (14).

$$Z_n = \frac{ln(\mu_n) - (\lambda_1 - k_b\xi_1^2)}{\xi_1}$$
$$Z_{n+1} = \frac{ln(\mu_{n+1}) - (\lambda_1 - k_b\xi_1^2)}{\xi_1} \tag{14}$$

The term $\exp\left\{\frac{k_b(k_b-1)\xi_2^2}{2}\right\}$in Eq. (13), reflecting the YD uncertainty. By calculating the failure probability in any range of ductility factor and using Eqs. (8) and (9), the probability of total failure and TRI for structures is calculated considering the YD uncertainty.

2.3. Design and modeling of structures

In this paper, six moment resisting reinforced concrete structures of 3, 5, and 8 stories with three and five spans of intermediate ductility, are evaluated. The study is focused on mid-rise structures. Therefore, the number of floors is limited to 8 floors [33-35]. These structures are designed in accordance with the fourth edition of the Iranian seismic code [24] and Iranian National Building Regulations [25, 26].

These residential structures are regular in plan and elevation. The middle and sides spans are 5 and 4 meters, respectively. Each frame is assumed to be part of the lateral load resisting system of a building with a rigid diaphragm. Roofs are one-way slabs (joist) and the height of each story is 3.0 m. Structures are located in metropolitan Tehran (with very high seismicity) with soil type II. Soil–structure interaction was not considered and the bases of the columns at the ground floor are assumed to be rigid. The material properties are assumed to be identical for all structures as: (a) reinforcing steel yield strength, F_y= 400 MPa; (b) concrete compressive strength, f_c= 25 MPa. Plan and elevation of structures are shown in Fig.1 and their fundamental periods are given in Table 1. Beam and column cross sections of all structures are shown in Fig.1 and Tables 2 and 3.

The computer program for the inelastic damage analysis of reinforced concrete buildings IDARC2D [36] is used for IDA and dynamic pushover analyses analysis. This program can model both lumped and distributed plasticity [37]. In this study, the concept of distributed plasticity and Park's three-parameter hysteresis model are used to express the nonlinear behavior of beam and column elements. The three-parameter Park hysteretic model considers stiffness degradation, strength deterioration, non-symmetric response, slip-lock, and a tri-linear monotonic envelope [37]. Depending on the history of deformations, the model shows the hysteresis behavior of an element as it changes from linear stage to deterioration. Values for hysteresis parameters (Stiffness degradation parameter (HC), Strength deterioration parameter (HBD, HBC), and slip-lock parameter (HS) are used for intermediate reinforced concrete moment frames [38].

Table 1. Fundamental periods of structures.

Structure	Period (second)
3 stories-3 spans	0.65
3 stories-5 spans	0.63
5 stories-3 spans	0.71
5 stories-5 spans	0.72
8 stories-3 spans	0.86
8 stories-5 spans	0.87

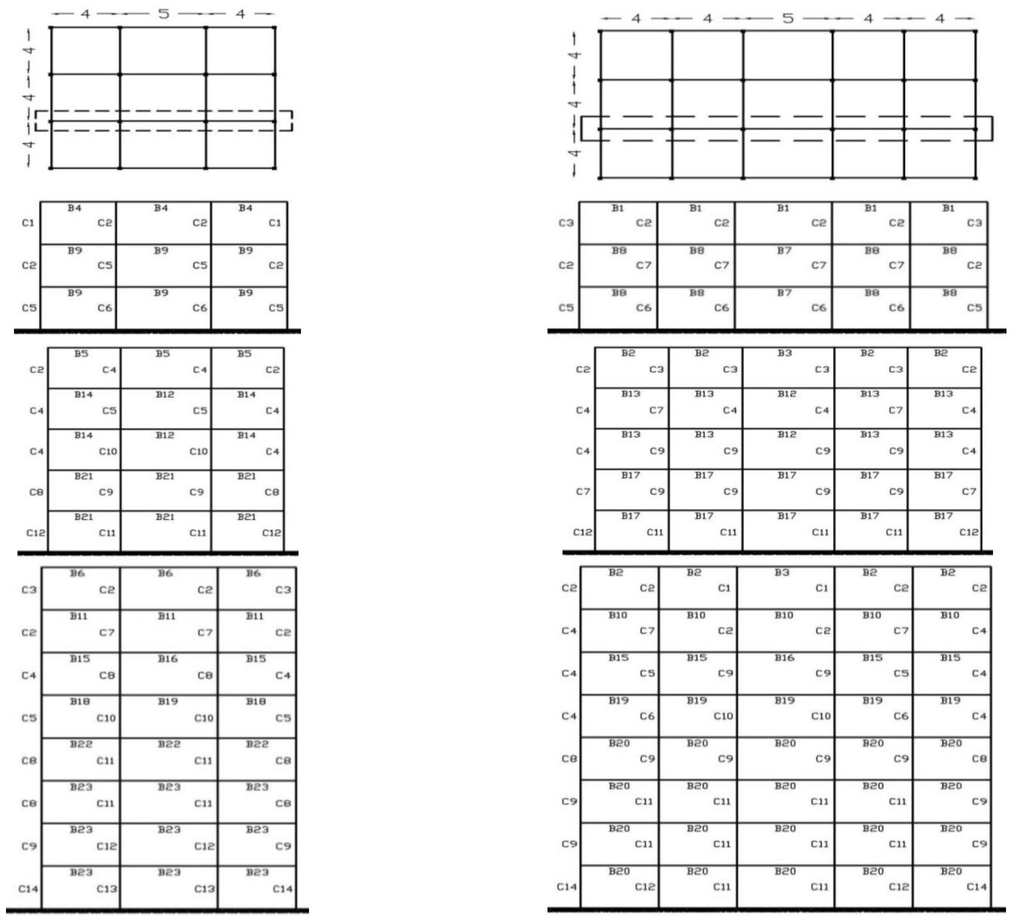

Fig. 1. Plan and elevation of structures.

Table 2. Beam cross sections of structure.

Type	Width (cm)	Depth	Down-Reinforcement	Up-Reinforcement
B1	30	30	1Φ12,1Φ14	4Φ16,1Φ10
B2	30	35	1Φ12,1Φ14	3Φ16
B3	30	35	1Φ12,1Φ14	2Φ18,1Φ16
B4	30	40	2Φ14	3Φ16
B5	35	30	2Φ14	4Φ16,1Φ14
B6	35	35	2Φ14	1Φ22,1Φ20
B7	35	40	1Φ22	3Φ20,1Φ14
B8	35	40	2Φ18	4Φ18
B9	35	45	1Φ22,1Φ14	2Φ20,1Φ22
B10	35	50	3Φ14	1Φ22,1Φ20
B11	35	55	1Φ18,1Φ14	3Φ18
B13	40	45	3Φ14	3Φ20,1Φ18
B14	40	45	2Φ18,1Φ14	3Φ20,1Φ14
B15	40	45	2Φ20,1Φ12	3Φ20,1Φ18
B16	40	50	2Φ18	3Φ16,1Φ18
B17	40	50	2Φ18	4Φ18
B18	40	50	4Φ16	3Φ20,1Φ18
B19	40	60	4Φ14	4Φ16,1Φ10
B20	40	60	4Φ14	3Φ20,1Φ12
B21	45	65	4Φ16,1Φ10	3Φ20,1Φ14
B22	50	50	4Φ16,1Φ14	3Φ22,1Φ16
B23	50	60	3Φ18	3Φ20,1Φ16
B24	50	65	3Φ16,1Φ20	3Φ20,1Φ18

Table 3. Column cross sections of structures.

Type	Dimension	Reinforcement	Type	Dimension	Reinforcement
C1	30X30	4Φ18	C8	40X40	8Φ16
C2	30X30	8Φ12	C9	40X40	8Φ18
C3	30X30	8Φ14	C10	40X40	8Φ20
C4	35X35	8Φ14	C11	45X45	8Φ18
C5	35X35	8Φ16	C12	45X45	8Φ20
C6	35X35	8Φ18	C13	45X45	14Φ16
C7	35X35	12Φ12	C14	50X50	8Φ22

A reinforced concrete building frame designed and tested at the National Earthquake Engineering Research Center (NCEER) is modeled for verification [37]. Maximum experimental and analytical responses of this structure under Taft record are given in table 4. Also Shear response time history of an exterior column of the first floor is shown in fig 2. It is observed that the results of structural modeling and analysis with IDARC are close to laboratory results, so it estimates an appropriate and acceptable structural response to earthquake loading.

Table 4. Maximum experimental and analytical responses of NCEER structure under Taft record.

Story	Story Shear (Kips)	Drift Ratio (%)	Displacement (in)
Laboratory Results [37]			
Third	3.4	0.23	0.3
Second	4.2	0.24	0.22
First	5.3	0.28	0.14
Analytical results by IDARC			
Third	3.46	0.21	0.293
Second	3.76	0.24	0.216
First	5.03	0.26	0.116

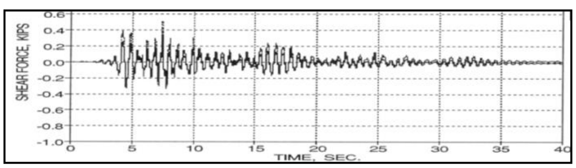

Fig. 2. a. Shear response time history for exterior column of the first floor (Laboratory [37]).

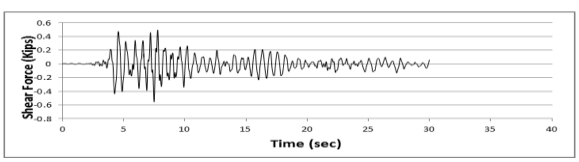

Fig. 2. b. Shear response time history for exterior column of the first floor (IDARC).

2.4. Ground motion records selection

The selection of ground motion records is an important consideration in IDA. Based on the previous studies, 10 to 20 accelerations are required for middle height structures analysis [39]. The selected acceleration should be strong enough to bring the structure to the collapse state. In recent years, extensive studies have been carried out in the context of record selection [28, 40, 41, 42]. Among them, the conditional mean spectrum (CMS) method which was proposed by Baker and Cornell, is used because it incorporates the aleatory uncertainties in earthquake events with all possible magnitudes and distances, as well as the epistemic uncertainties. [41]. In this method, in addition to the regional characteristics including magnitude and distance, the shape of the response spectrum, which is an important factor in the nonlinear response of structures, is considered as a dominant parameter for records selection. The shape of the response spectrum is important because the nonlinear response of multi-degrees of freedom structures subjected to smaller periods than the principle mode in other modes (other than the first mode), and also with the onset of the nonlinear behavior of the structure and reduction of the structural stiffness the original period increases from the initial value [41, 42].

So, to consider the effect of spectrum shape, the process of record selection is done based on the target spectrum. Most regulations introduce the uniform hazard spectrum (UHS) as target spectrum for acceleration selecting, which often leads to a conservative estimate of seismic demand in structures [40]. The CMS is the target spectrum that estimates the distribution of the response spectrum conditioned on the occurrence of a target spectral acceleration value at the period of interest [42].

Based on the above explanations and the results of the probabilistic seismic hazard analysis disaggregation in metropolitan Tehran [29, 43], mean spectrum, conditional mean spectrum, and spectrum of selected records are shown in Fig. 3. It is noteworthy that structures are located on soil type II, in far fault zones, so the earthquakes are selected on soils with average shear wave velocity in the range of 375 to 750 m/s in distances more than 10km [24]. The magnitude of earthquakes and peak ground-motion acceleration are from 4.5 to 7.5 (M_W) and 0.05g to 1g, respectively (Table 5).

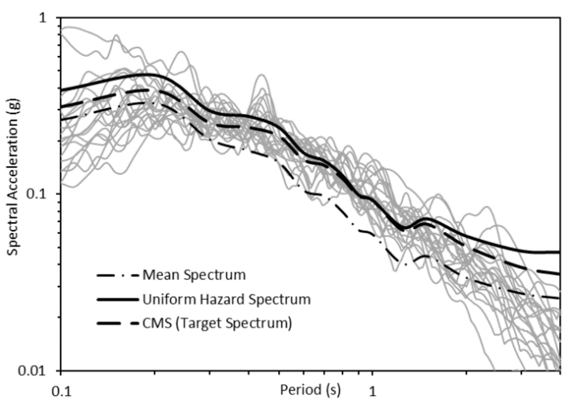

Fig. 2. Regional design spectrum vs records selected by CMS method.

3. Results and discussion

3.1. Incremental dynamic analysis (IDA)

IDA curve is a drawing of the non-linear dynamic behavior of structures under a ground motion record [30]. Since this curve depends on the selected record, the IDA study of one record, alone cannot estimate the actual behavior of structures for other earthquakes that may happen in future events. Hence, 10 to 20 records are required for middle height structures analysis [39]. In IDA curves, the earthquakes are scaled with different scalar coefficients for non-linear time history analysis of structures. Previous studies show that at least 12 different scale factors are required to calculate an IDA curve for a record [28]. The IDA curve was calculated for each of the twenty selected records (table 5). Based on the results, mean (50%) and mean plus or minus standard deviation (e.g. 84% and 16%) can be calculated for different structures [30]. The curves of the mean and mean plus and minus standard deviation of structures response are shown in Fig.4. It is noteworthy that each of these curves was calculated with the help of at least 400 nonlinear time history analyses (20 records and for each record at least 20 different scale factors- 0.05 increment-of spectral acceleration).

Table 5. Earthquakes characteristics used in IDA (recorded on soil type 2).

Earthquake	Direction	Station	PGA	Distance	Magnitude	Velocity
Tabas	LN	Dayhook	0.327	13.94	7.35	659.6
Manjil	LN	Abbar	0.514	12.56	7.37	724
San Fernando	90	Pasadena Cit Athenaeum	0.11	25.47	6.61	415.1
San Fernando	291	Lake Hughes #9	0.134	22.57	6.61	670.8
Kern County	111	Taft Lincoln School	0.178	38.89	7.36	385.4
Morgan Hill	270	San Justo Dam (L Abut)	0.081	31.88	6.19	622.9
Morgan Hill	67	Gilroy - GavilanColl	0.114	14.84	6.19	729.7
Hector Mine	90	Twentynine Palms	0.066	42.06	7.13	684.9
Sierra Madre	180	LA - City Terrace	0.091	25.69	5.61	365.2
Loma Prieta	250	Anderson Dam	0.244	20.26	6.93	488.8
Loma Prieta	90	Fremont - Mission San Jose	0.106	39.51	6.93	367.6
Loma Prieta	0	Gilroy Array #6	0.126	18.33	6.93	663.3
Loma Prieta	90	Gilroy Array #6	0.17	18.33	6.93	663.3
Loma Prieta	0	Monterey City Hall	0.073	44.35	6.93	684.9
Northridge	9	Arcadia - Campus Dr	0.089	41.41	6.69	367.5
Northridge	279	Arcadia - Campus Dr	0.11	41.41	6.69	367.5
Northridge	260	Alhambra - Fremont School	0.08	36.77	6.69	550
Northridge	270	N Hollywood - ColdwaterCan	0.271	12.51	6.69	446
Northridge	180	La Crescenta - New York	0.159	18.50	6.69	446
Northridge	70	LA - Chalon Rd	0.225	20.45	6.69	740.1

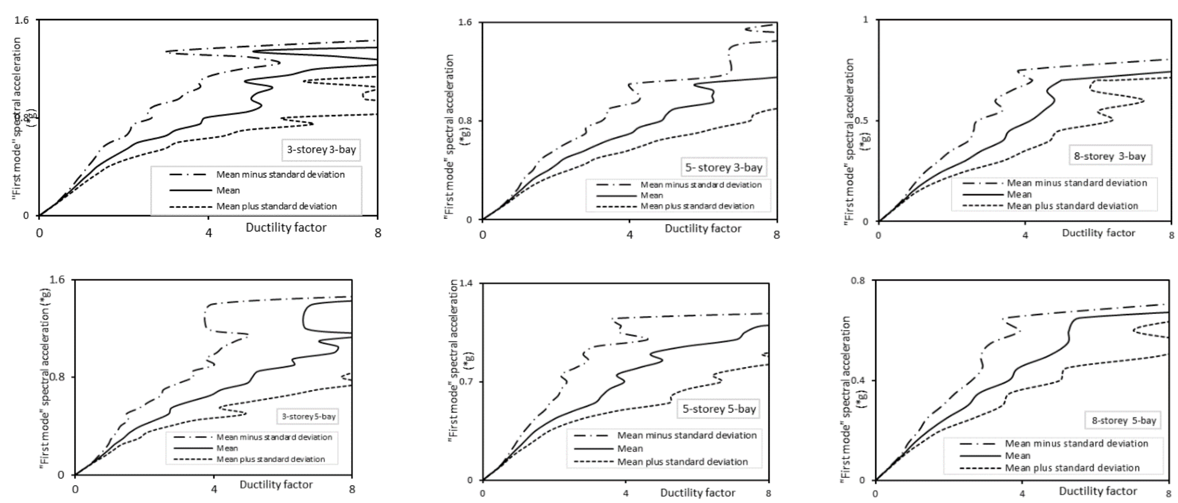

Fig. 4. IDA curves of structures.

3.2. YD calculation

The median and standard deviation of YD are required for considering the YD uncertainties in TRI relationships. Since bilinear static push over curves (capacity curves) were not taken into account the strength degradation at high displacements, they did not provide a suitable response of the structure. But, when the structures do not enter the strength degradation, the responses are acceptable [22]. The results of IDA can be plotted using the same coordinates of the static pushover curve called the dynamic pushover curve. In dynamic capacity curves, roof displacement was plotted as a function of base shear [23]. It was shown that the dynamic pushover capacity curve obtained based on the maximum displacement diagram and respective base shear was almost consistent with the static pushover curve [23]. These curves had a clear initial elastic branch and terminate at a horizontal straight line that shows collapse seismic intensity. At the CP level, the structure has nonlinear behavior and the scatter of structural responses is greater than the structure response before yielding. Studies show that 20 seismic records can provide a good approximation of the CP [4, 8]. Hence, In order to estimate the yield point of the structure, the results of 20 dynamic pushover analyzes are considered [20 and 23]. Obviously, using more records can increase the accuracy of the analysis. Dynamic pushover curves of 5-stories five-span structure were shown in Fig.5.

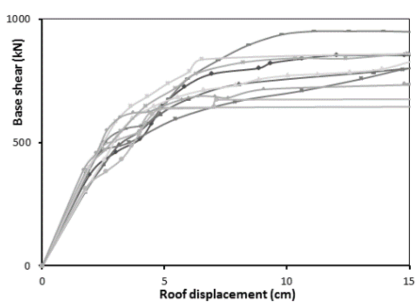

Fig. 5. Dynamic pushover curves for 5 stories five-spans structure.

Therefore, this study hired the concepts of bilinear modeling of dynamic pushover curve (capacity curve), presented in the research of Luca et al, D'Ayala et al, FEMA-350 and FEMA-356 [44, 45, 4 and 21], and target displacement, to calculate the median and standard deviation of YD.

After plotting the dynamic pushover curves for each record, YD for each curve was obtained using bilinear modeling of the capacity curve based on FEMA-356 [22]. Then the median and standard deviation of YD are calculated.

Since the TRI evaluates all of the performance levels from minor damages to global instability; the drift demand values that represent the onset of global instability (CP) in the structure must be estimated and be introduced as the target displacement in the dynamic pushover curve [21]. According to the FEMA-350 guideline, the onset of global instability is the point where the local slope of the IDA curve decreases to 20% of the initial slope of the IDA curve in the elastic region [4]. Fig.6 shows an example of bilinear modeling for dynamic pushover curves (for all structures exposed to Tabas record).

After obtaining equivalent capacity curves for all records, a set of 20 YD data were achieved for each structure. Assuming log-normal distribution for YDs, the median and standard deviation of data for each structure were obtained (Table 6). Now, regarding the YD uncertainty, TRI can be calculated.

Table 6. Median (med) and standard deviation (std) of YD obtained from dynamic pushover analysis

Structure	med (cm)	std
3 St.- 3 Sp.	2.716	0.211
3 St.- 5 Sp.	2.581	0.241
5 St.- 3 Sp.	3.417	0.234
5 St.- 5 Sp.	3.697	0.226
8 St.- 3 Sp.	4.127	0.315
8 St.- 5 Sp.	4.310	0.309

St: stories & Sp: spans

3.3. Total reliability index (TRI)

As mentioned in section (2-1), coefficients k and k_0 were calculated based on the results of the probabilistic seismic hazard analysis (PSHA) of Tehran [29]. PSHA curves for different periods were shown in Fig.7. k and k_0 derived from linear regression on seismic hazard curves on a logarithmic scale, were listed in Table 7.

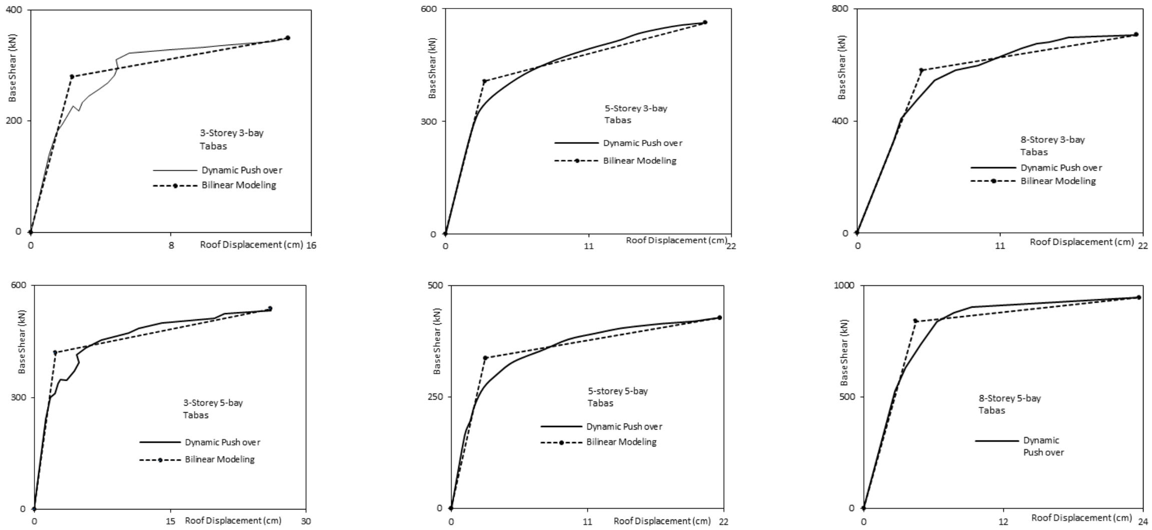

Fig. 6. Equivalent bilinear elasto-plastic model for dynamic pushover curve.

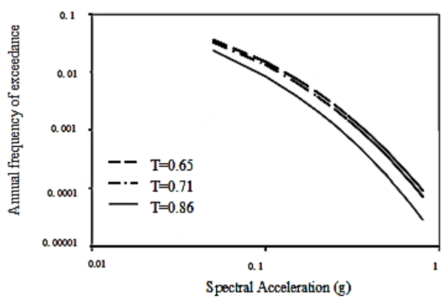

Fig. 7. Seismic hazard curves (Yazdani et al., 2015).

Table 7. Seismic hazard parameters for Tehran.

Structure	K_0	k
3 st.-3 sp.	2688.6	2.519
3 st.-5 sp.	2688.6	2.519
5 st.-3 sp.	1041.1	2.383
5 st-5 sp.	1041.1	2.383
8 st.-3 sp.	345.19	2.297
8 st.-5 sp.	345.19	2.297

In order to apply the power law function on the mean IDA curve (Eq. (3)) and calculation of *a* and b coefficients, IDA results in log-log scale are shown in Fig. 8. Standard deviation values at various intervals of ductility factor shown in Table 8. It is observed that since 8-story structures are unstable in the ductility factor values greater than 6, there is no data in this part. Based on Eqs (6) and (13) the failure probability of structures, with and without regard to YD uncertainty, were calculated at various intervals of ductility factor using coefficients *a*, *b*, *k*, k_0 and standard deviation values ($Sa|\mu_\beta$). The results of total failure probability (Eq (8)) and TRI for all structures are summarized in Table 9.

Table 8. Standard deviation values at various intervals of ductility factor (ξ).

Structure	ductility factor			
	0-2	2-4	4-6	6-8
3 st.-3 sp.	0.223	0.254	0.103	0.132
3 st.-5 sp.	0.214	0.198	0.101	0.102
5 st.-3 sp.	0.249	0.187	0.105	0.162
5 st-5 sp.	0.214	0.222	0.108	0.055
8 st.-3 sp.	0.216	0.191	0.071	-
8 st.-5 sp.	0.307	0.206	0.066	-

The results showed that applying YD uncertainty at reliability index relationship increased structures total failure probability and consequently decreased the TRI. As the height of buildings increased, the effect of YD uncertainty on the TRI increases, so that TRI increased from 0.47% in a 3-story structure to 1.96% in an 8-story structure. The reason was that according to Table 6, with increasing the height of the structures, the amount of YD standard deviation also increased, which ultimately led to an increase in the total failure probability and a decrease in TRI.

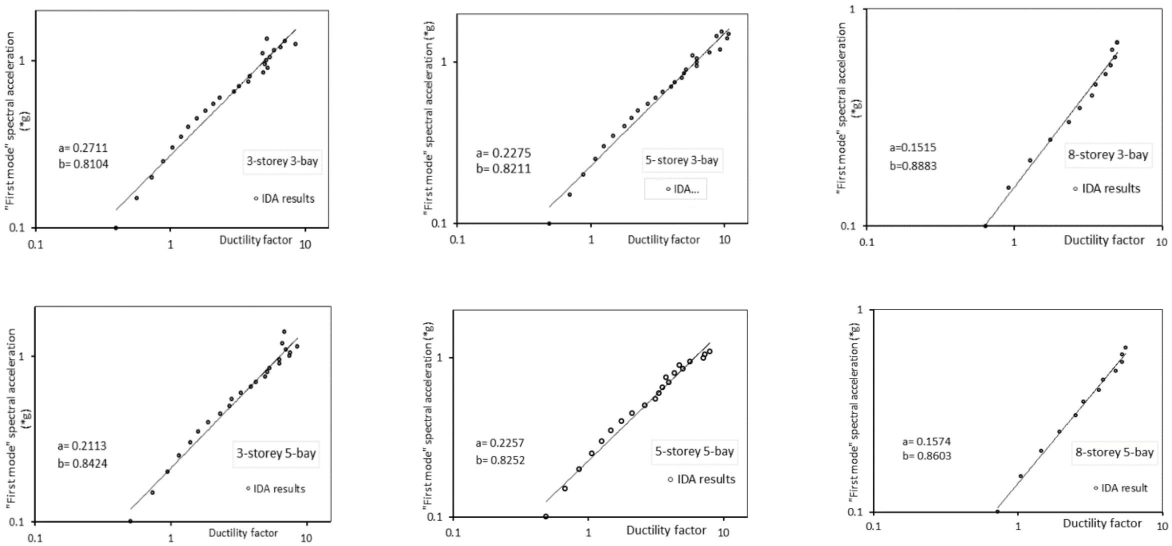

Fig. 8. power law form coefficients (a and b).

Generally, the effect of YD uncertainty on the TRI of the studied structures was small (changes less than 2%) and both cases showed a slight difference in the safety of the structure. Thus, it can be proposed that for evaluation of the seismic performance of mid-rise structures based on TRI, taking the YD uncertainty because of spending a lot of analysis and time can be ignored such as the uncertainty of live load and damping of structures [21].

From Table 9, the results showed that with increasing the height of the structures, the value of the total reliability index increased; so that 8-stories structures had the greatest reliability. The reason was that increasing the height of structures, increased structures periods, and decreased seismic hazard parameters in accordance with Fig. 7 and Table 7. Decreasing effects of hazard parameters prevail over structural parameters (In particular, random uncertainties on the structural demand and slope of IDA curves).

Table 9. TRI of Structures.

Structure	Without YDU		With YDU		Change percentage (%)
	P_f	β	P_f	β	
3 st. 3 sp.	0.00099	3.0932	0.00104	3.0785	0.47%
3 st.- 5 sp.	0.00114	3.0511	0.00123	3.0282	0.75%
5 st.-3 sp.	0.00102	3.0843	0.00107	3.0700	0.46%
5 st.-5 sp.	0.00101	3.0872	0.00106	3.0728	0.47%
8 st.-3 sp.	0.00064	3.2204	0.00072	3.1865	1.05%
8 st.-5 sp.	0.00061	3.2341	0.00076	3.1708	1.96%

st: stories, sp: span.

Hence, due to the effect of seismic hazard parameters (k & k_0), the failure probability decreased and TRI would increase. On the other hand, structures with the same height had almost the same TRI. In these structures, the seismic hazard was the same. Regarding Tables 6 and 8, it can be seen that increasing the values of the standard deviation of YD and standard deviation of ductility factor in different intervals reduces the TRI.

Fig. 9 illustrated the variations of structural reliability indices. Structural safety assessment according to target indices showed that the total failure probability (P_f) and TRI (β) of the structures, designed according to the new edition of Iranian regulations with 50 years lifetime, vary from 0.00123 to 0.00072 and 3.0282 to 3.1865, respectively. According to ISO 2394, the values of $\beta \geq 3$ were proposed for structures with 50 years lifetime [46]. Therefore, it can be stated that, in accordance with the reliability criterion, all studied structures were safe against the probable seismic hazard.

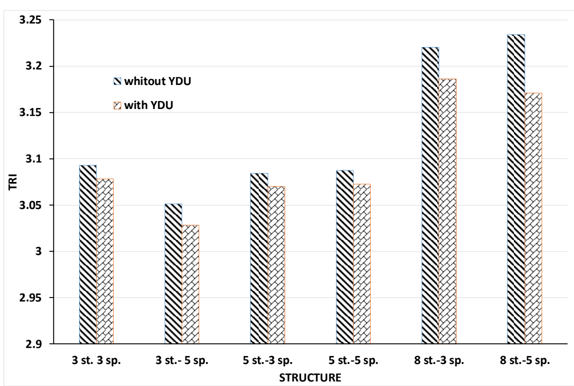

Fig. 9. Efficacy of considering YD uncertainty on total reliability index.

4. Conclusion

Regarding the role of reliability theory in the structural design methods and its ability to consider various uncertainties in the evaluation of the seismic performance of structures, this study investigated the impact of YD uncertainty of structures on TRI of six mid-rise RC buildings with intermediate moment resisting frame systems (3, 5 and 8 stories with three and five-spans). In order to find the YD, considering that the bilinear static pushover capacity curves were not be taken into account the strength degradation in the high displacements; In this paper proposed the method for finding the distribution of YD, using the bilinear linearization of the dynamic pushover curve. Parameters obtained from this method provided a better and more accurate expression of the seismic response of the structure in comparison with static pushover. Dynamic analysis input had also been selected using the CMS method. Based on the observations, Results showed that:

1- With increasing building height, the effect of YD uncertainty on the TRI increased; so that variation of TRI increased from 0.47% in the 3-story structure to 1.96% in the 8-story structure.

2- The effect of YD uncertainty on the TRI of the studied structures was less than 2% and hadn't a significant impact on seismic evaluations; hence, it could be ignored in seismic evaluations of mid-rise RC structures.

3- Increasing the height of structures and their period, the total failure probability of structures decreased, and the value of the TRI increased.

4- It was also observed that the TRI for these structures vary from 3.0282 to 3.1865 and are larger than the minimum limits of international regulations. So these structures are safe in probable earthquakes in 50 years lifetime.

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