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Risk-adjusted design basis earthquake's adequacy for use in seismic design codes

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ORIGINAL ARTICLE

Risk-adjusted design basis earthquake's adequacy for use in seismic design codes

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Abstract

In most buildings' seismic design codes, the design basis peak ground acceleration (PGA_{DBE}) is provided by employing a uniform-hazard approach. However, a new trend in updating seismic codes is to adopt a risk-informed method to estimate the PGA_{DBE} so-called risk-adjusted design basis peak ground acceleration (PGA_{RDBE}). An attempt is made here to examine the adequacy of the PGA_{RDBE} to fulfill the assumptions made in seismic codes for converting the maximum considered earthquake's (MCE) intensity to PGA_{DBE}. To this end, the performance of regular intermediate steel moment frames (IMF) is assessed in terms of collapse margin (CMR) and residual drift ratios in the event of MCE and design basis earthquake (DBE), respectively. The PGA_{RDBE}s are computed for Karaj County, Iran. A set of 96 index archetypes of regular IMF is designed considering four design parameters, which include the number of stories (2, 3, 6, 9, 12, and 15), span lengths (4 and 8 meters), occupancies (residential and commercial), and seismic demands (0.15, 0.25, 0.35 and 0.45g). Although PGA_{RDBE} fulfills the acceptance criteria, results show that it does not necessarily meet the implicit assumption made in codes that the code-conforming buildings designed for the PGA_{DBE} have an acceptable CMR if the MCE occurs. This emphasizes that the PGA_{RDBE} should not be used without examining the CMR fulfillment. The results recommend that a lower limit needs to be set on PGA_{RDBE}s, which is found to be 0.35g for regions with high seismicity. This lower limit is set to achieve at least a CMR of 1.5 in the event of MCE. Outcomes also reveal that the code-conforming buildings designed with the proposed PGA_{RDBE} can fulfill both repairability and life-safety performances at the seismic demands of DBE and MCE, respectively. These buildings also have a high chance to be even considered repairable at the seismic demand of MCE. Furthermore, regardless of the employed method for estimating PGA_{DBE}, various relationships between design parameters with different performance indicators such as CMR, residual drift ratio, ductility demand, imposed drift ratio, and building's normalized weight are presented. These relationships can be used to evaluate the buildings' safety factor against collapse and repairability, justification of using IMF in regions with high seismicity, level of structural and nonstructural damage as well as the economic consequence of changes in PGA_{DBE}. Furthermore, they could be incorporated in developing a multi-criteria decision-making tool to decide on the optimum PGA_{DBE} leading to an affordable alternative and acceptable life-safety risk.

Keywords Risk-adjusted design basis earthquake · Collapse margin ratio · Residual drift ratio · Ductility demand · Intermediate steel moment frame

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Article Highlights

- Risk-adjusted design basis earthquake should not be used without ensuring their adequacy to provide intended performance goals assumed in the seismic design codes. A lower limit may need to be set on its values.
- A set of linear relationships between performance indicators and design parameters were introduced. Those can be used to evaluate the impact of changes in design parameters on seismic safety and performance at any desired level of earthquake intensity.
- To evaluate the economic effects of changes in design earthquake intensity (PGA_{DBE}), the building's normalized weight was selected as a basic economic index and a linear relationship between W_N and PGA_{DBE} was provided.

1. Introduction

Among different natural hazards, an earthquake could have major effects on a concentrated area within a short period leading to a significant number of deaths, especially in developing countries (Nadim et al. 2004; Holzer and Savage 2013; Ibrion et al. 2015). The majority of the death tolls in the event of a large earthquake are due to the collapse of buildings (Yamazaki et al. 1996; Tobita et al. 2007; Marano et al. 2010). Therefore, the authorities try to control and restrict earthquake consequences by establishing appropriate regulations and updating design codes based on new achievements. As a minimum acceptable building's performance in the event of a very rare earthquake, seismic design codes try to minimize the number of fatalities by limiting the probability of collapse through prescriptive criteria. Most of the seismic design codes, such as Iranian Standard No. 2800 (2014) and Eurocode 8 (CEN 2004) neglect the vulnerability curves of buildings and consider just one point of the seismic hazard curve to estimate the design basis earthquake ground motion. For instance, in both Standard No. 2800 and Eurocode, a uniform-hazard approach is used to define the design basis earthquake as an earthquake ground motion with a 10% probability of exceedance in 50 years ignoring the occurrence probability of the other earthquakes' intensity. This approach leads to buildings with unknown reliability or probability of failure in their service life. To provide nearly uniform acceptable performance for buildings, both the earthquake hazard and building vulnerability need to be considered in estimating the design basis earthquake's intensity. The uniform-hazard approach fails to estimate the seismic risk by neglecting the vulnerability and earthquake hazard curves. To have buildings with certain and tolerable reliability, a new approach for estimating design basis peak ground accelerations (PGA_{DBE}) needs to be implemented.

A risk-informed approach is just implemented in ASCE/SEI 7 (2022) that takes into account both earthquake hazard and building vulnerability. In this approach, the occurrence probability of all the probable earthquakes is considered by employing the entire hazard curve. Furthermore, the buildings' vulnerability is also considered by utilizing the buildings' collapse fragility function**.** Many studies have been devoted to computing the building's collapse fragility function via different analytical, experimental, and empirical methods (Ibarra and Krawinkler 2005; Black 2011; Gehl et al. 2013; Adam and Ibarra 2014; Ulrich et al. 2014; Gkimprixis et al. 2018; Martins et al. 2018). A comprehensive review of developing building collapse fragility function for risk assessment can be found in Douglas and Gkimprixis (2018). Besides, a thorough review of different methods of the buildings'

collapse capacity assessment is provided in Villaverde (2006). In the same fashion as ASCE/SEI 7 (2022), the general building's collapse fragility function (Luco et al. 2007; Vacareanu et al. 2018) is employed to compute risk-adjusted design basis peak ground accelerations here (PGA_{RDBE}). The effectiveness of this approach is investigated by Gkimprixis et al. (2019).

Since seismic risk assessment depends on the earthquake hazard curve, a specific region needs to be chosen to carry out the study. The authors previously studied buildings' seismic collapse risk in Tehran City (Zaman and Ghayamghamian 2019) to estimate PGA_{RDBE}. So, Karaj is selected here to conduct the study since it is located near the active faults and growing very fast with an almost 2 million population. Karaj is the second city with the most population density in Iran with about 1,200 people per square kilometer (Statistical Center of Iran (SCI) 2013). This concentration of people near the active faults exposes their lives and assets to high seismic risk. Besides, it is located in Tehran's vicinity and has been chosen as a specific city to manage rescue operations when a large magnitude earthquake hits Tehran.

In this study, first, it is attempted to evaluate the adequacy of PGA_{DBE} for both design basis peak ground acceleration prescribed in Standard No. 2800 (PGA_{2800}) and those PGA_{RDBE}s proposed here. Second, it is tried to evaluate the economic consequences made by PGA_{DBE}'s variation regardless of the employed method to estimate it. To this end, buildings with regular intermediate steel moment frames as a lateral load-resisting system are selected. Then, a set of 96 index archetypes are constructed by considering four design parameters, namely the number of stories (N_s), span lengths (L_s), occupancies (O_c), and PGA_{DBE}s. The performance indicators, including collapse margin ratio (CMR), imposed drift ratio (Δ_d), ductility demand (μ_d), and residual drift ratio (Δ_r) are calculated for all the building models. The adequacy of the PGA_{2800} and PGA_{RDBE} are explained based on their ability to provide an acceptable level of seismic collapse risk and the calculated CMRs as well. The relation between design parameters and seismic demand (PGA_d) with those of normalized weight (W_N), aspect ratio (A_R), μ_d, Δ_d, and Δ_r are examined. These relationships together could be employed to develop a multi-criteria decision-making framework to ensure that the future proposed updates in seismic codes will be safe and affordable alternatives.

2. Probabilistic Seismic Hazard Analysis

The first step in computing the building collapse risk in Karaj is to calculate the earthquake hazard curves. Probabilistic seismic hazard analysis (PSHA) is a tool to deal with the uncertainties (like magnitude and location) and to estimate different ground motion intensity and their probability of exceedance in a certain period (usually a building's lifetime) by incorporating all the earthquake sources. Karaj is the capital of the Alborz Province, which is situated at the foot of the Alborz rocky mountain range and surrounded by many known active faults, such as Mosha (135 km), North Tehran (130 km), Robat Karim (82 km), Eshtehard (64 km), Taleghan (61 km) Kahrizak (35 km), and Ray (27 km) faults (Hesami et al. 2003).

To conduct PSHA, the studied region is divided into 99 square cells with each side measuring 3 km, and peak ground acceleration (PGA) hazard curves are estimated at the cells' center. These grid points alongside the PGA hazard curves are depicted in Fig. 1. A detailed description of the composed earthquakes catalog, faults' location, seismicity parameters, and ground motion prediction equations (GMPEs) employed to estimate the PGA hazard curves can be found in Zaman and Ghayamghamian (2019). The peak ground accelerations of the maximum considered earthquake (PGA_{MCE}s) also need to be extracted from the calculated PGA hazard curves. According to the Iranian National Building Code (INBC) Part 6 (2019), the maximum considered earthquake (MCE) is defined as an earthquake ground motion with a 2% probability of exceedance (PE) in 50 years. These PGA_{MCE}s are depicted in Fig. 2 and will be used later to measure CMRs.

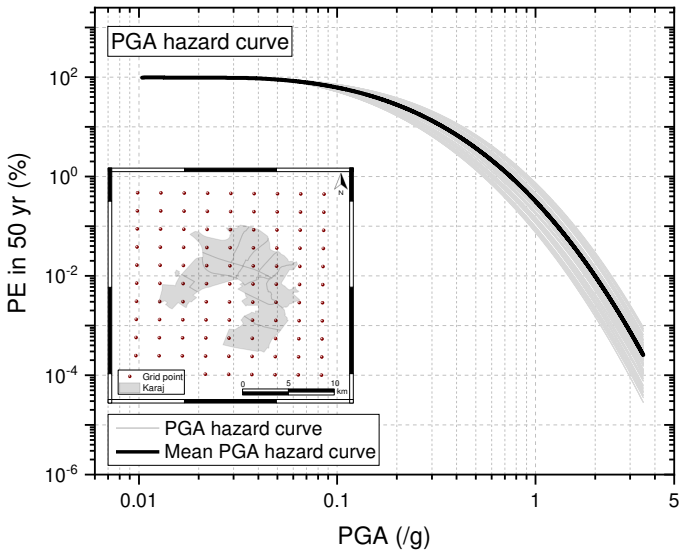


Fig. 1 Karaj's cells center (99 grid points) at which PGA hazard curves are computed. Thick black line is the mean PGA hazard curve of the studied region

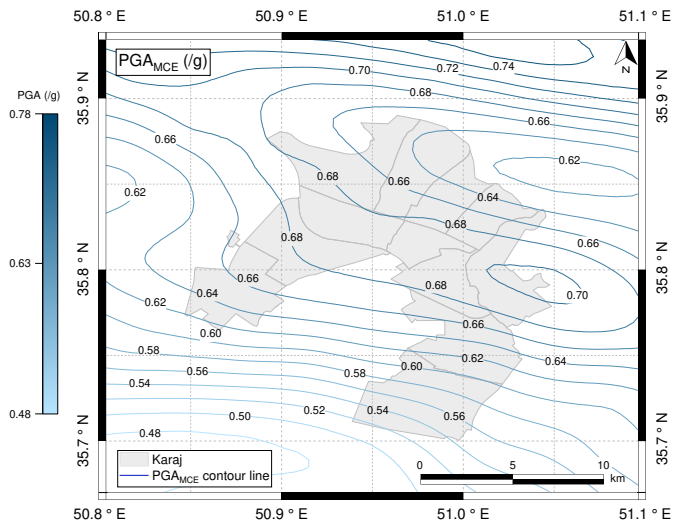


Fig. 2 PGAs with a 2% probability of exceedance in 50 years (PGA_{MCE}) for Karaj

The pseudo-spectral accelerations (PSAs) are also needed in computing the general building's collapse fragility functions. The PSAs can be directly estimated by given GMPEs (Ghasemi et al. 2009; Boore et al. 2014;

Zafarani et al. 2017). However, here, they are indirectly computed as PSA = B.PGA (Standard No. 2800 2014) where B is the shape of the design response spectrum. The advantages of the latter method are: (1) retains the calculated PSAs on the safe side as a requirement of force-based design codes; (2) easily permits any necessary modifications on calculated PSAs due to possible future refinements of design response spectrum shape; (3) makes the calculated PGA_{RDBE}s independent of soil types as discussed by Zaman and Ghayamghamian (2019).

3. General Building's Collapse Fragility Function

Building collapse fragility functions can generally be obtained through four approaches, including empirical (Hisada et al. 2005; Jaiswal et al. 2011), experimental (Gardoni et al. 2002), analytical (Ulrich et al. 2014; Baker 2015), and expert opinion (Porter et al. 2007; Tobita et al. 2007). For code-conforming buildings, there is another approach to simulate the collapse fragility function which is so-called the general building's collapse fragility function (Luco et al. 2007). In this approach, the general building's collapse capacity (x) is assumed to be a random variable whose logarithm has a normal distribution $\Phi([\ln(x) - \mu]/\sigma)$, where Φ is the cumulative distribution function, μ and σ represent the mean collapse capacity and dispersion of the result, respectively. As proposed by FEMA P695 (2009), the σ is assumed to be the total uncertainty (σ_{TOT}), which consist of the record-to-record (σ_{RTR}), design requirements (σ_{DR}), test data (σ_{TD}), and modeling (σ_{MDL}) uncertainties. Luco et al. (2007) proposed a value in the range of 0.6 to 1.0 with the expected value of 0.8 based on the results of ATC-63 (2007) for σ_{TOT}. On the other hand, a value of 0.6 was suggested by ASCE/SEI 7 (2022). Douglas et al. (2013) and Silva et al. (2016) proposed a value of 0.5 through the incremental dynamic analysis (Vamvatsikos and Allin Cornell 2002) conducted by Crowley et al. (2011). Last, Zaman and Ghayamghamian (2019) showed a value of 0.7 is appropriate for buildings with IMF as a lateral load-resisting system, which is used here to construct the general building's collapse fragility function.

4. Building Collapse Risk Assessment

General building's seismic collapse risk is evaluated through the risk integral in the following form

$$P_c = \int_0^\infty P_c(x).H(x)\,dx \tag{1}$$

Where x, $P_c(x)$, and $H(x)$ stand for PSA, general building's collapse fragility function, and PSA hazard curve, respectively. The computed risk integral needs to be optimized to concur with two acceptance criteria proposed by FEMA P695 (2009). The first condition is the conditional collapse probability in the event of MCE being equal to 10%, and the second one is the probability of collapse being equal to 1% in 50 years. The first criterion is satisfied by replacing the μ with the $\mu + 1.28\sigma$; the second one is also fulfilled by optimizing μ in such a way that Eq. (1) equals 1%. So, the general building's collapse fragility function can be written in the following form

$$P_c(x) = \frac{1}{x.\sigma.\sqrt{2\pi}} e^{-\frac{(\ln(x)-(\mu+1.28\sigma))^2}{2\sigma^2}} \tag{2}$$

where, μ and σ are the mean and standard deviation, respectively.

The so-called risk-adjusted peak ground acceleration (PGA_R) is the one that satisfies the above acceptance criteria. It is assumed that the well-designed buildings have at least a safety margin of 1.5 against collapse. Therefore, two-thirds of the calculated PGA_R is introduced as a risk-adjusted design basis peak ground acceleration (PGA_{RDBE}) with values generally larger than 0.35g as depicted in Fig. 3a. The calculated PGA_{RDBE} varies from 0.35g to 0.45g as moving from the southeast to the northwest of Karaj. When the calculated PGA_{RDBE} is compared with the constant value of 0.35g prescribed by Standard No. 2800 for the entire Karaj, it is found that the PGA_{2800} neither maintains the uniform collapse risk nor stands on the safe side. This evidence suggests the PGA_{2800} needs to be modified by PGA_{RDBE}s. Furthermore, to easily convert the PGA_{2800} to PGA_{RDBE} in Karaj, a risk-adjustment conversion factor (R_A) is defined here as the ratio of the calculated PGA_{RDBE}s to that of PGA_{2800}. Fig. 3b displays the variation of calculated R_A in Karaj.

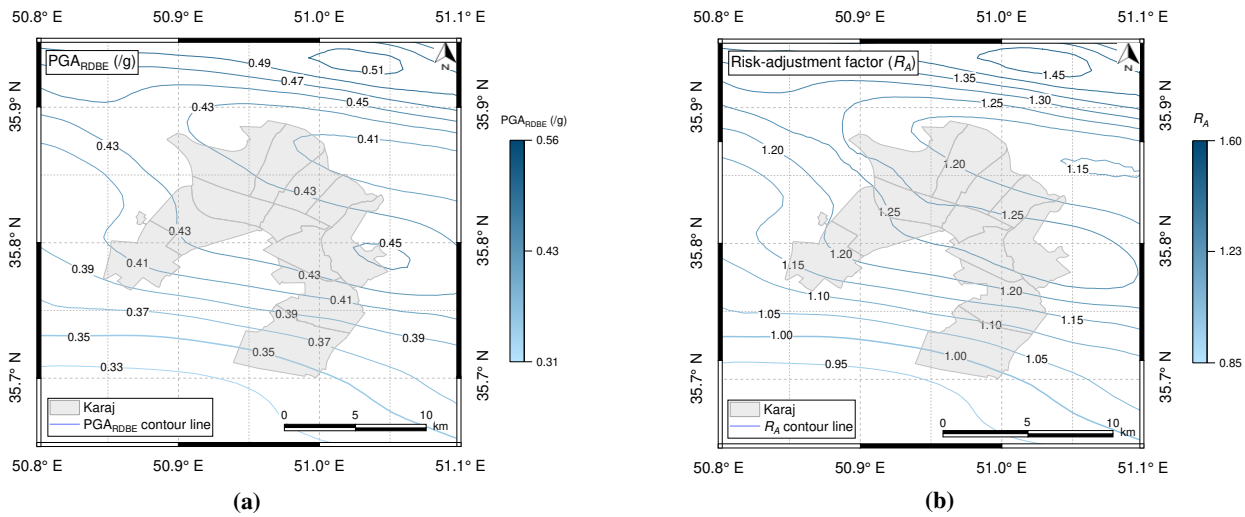


Fig. 3 **(a)** Risk-adjusted design peak ground acceleration (PGA_{RDBE}) **(b)** Risk-adjustment conversion factor (R_A)

5. IMFs' Performance Evaluation

In the previous section, PGA_{RDBE}s that satisfy the collapse risk acceptance criteria were presented. However, before adopting those values as PGA_{DBE} in seismic code, their adequacy to fulfill the other intended performance goals must be ensured. To this end, buildings' seismic performance is evaluated in terms of the CMR, Δ_r, Δ_d, μ_d, and W_N. A set of 96 index archetypes with IMF as a lateral load-resisting system are constructed by considering several design parameters, namely, N_s, L_s, O_c, A_R, and PGA_{DBE}. Then, the effects of design parameters' variation on performance indicators are examined. It should be noted that the performances of the developed buildings are examined for a range of PGA_{DBE} values. Therefore, the results are relevant to be used for the performance assessment of IMFs regardless of the method used to estimate PGA_{DBE} and where the buildings are located.

5.1 Archetype Design Space

Steel moment frame with intermediate ductility is the dominant lateral load-resisting system of steel buildings with ordinary occupancy and usage (including residential and commercial) in Iran. According to Standard No. 2800 (2014), IMF is allowed to be constructed in regions with very high seismicity ($PGA_{DBE} = 0.35$). To evaluate the seismic performance of buildings with IMF, 96 index archetype configurations by assuming different values of key design parameters (N_s, L_s, O_c, A_R, and PGA_{DBE}) as given in Table 1 are constructed to reasonably represent a broad range of design parameters. It is also assumed that all buildings have the same constant height of 3.2 m for all stories, have three bays of equal length, and their floor system behaves like a rigid diaphragm (Fig. 4).

Table 1 Design parameters used to develop the index archetype design space

Number of stories (N_S)	Span length (L_S) (m)	Occupancy (live load intensity) (kg/m^2)	Aspect ratio (A_R)	PGA_{DBE} [a] (/g)
2, 3, 6, 9, 12, and 15	4 and 8	Residential (200) and Commercial (500)	H_{total}[b]/L_s	0.15, 0.25, 0.35, and 0.45

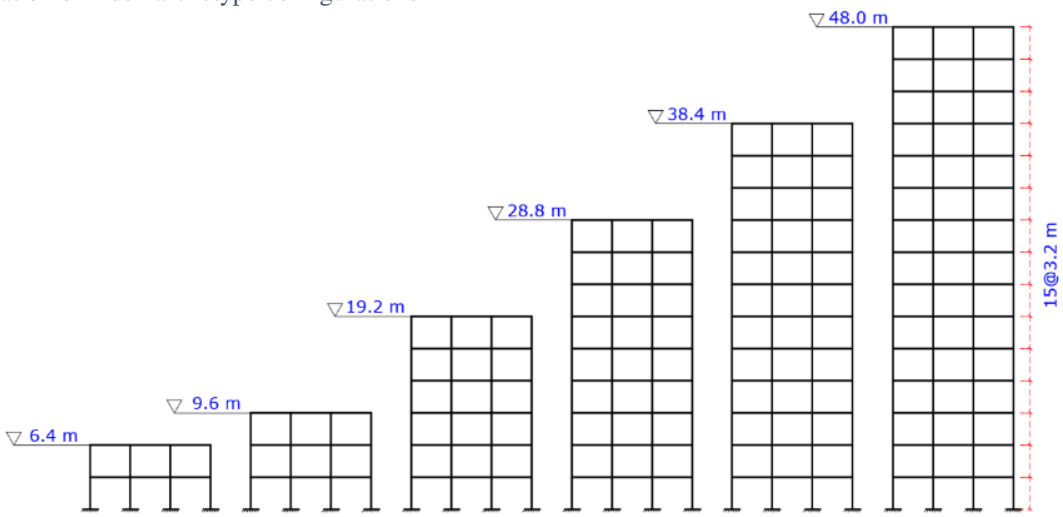
[a] Peak ground acceleration used to design the index archetypes
[b] Total building height (m)

In numerical models, 170 box-shape cross-sections for columns and 110 I-shape ones for beams are defined to be used in the design and optimization process. Structural members' cross-section size is determined through the equivalent lateral force (ELF) procedure, which is the reference method for determining seismic demands (Standard No. 2800 2014). In this procedure, the strength-level base shear (V_u) is calculated by $C_s \times W$, where C_s is the seismic response coefficient, and W is the effective seismic weight. The value of C_s is determined as:

$$C_s = \frac{A.B.I_e}{R_u} \tag{3}$$

where A, B, I_e, R_u are PGA_{DBE}, design response spectrum shape, seismic importance, and strength-level response modification factors, respectively. In Standard No. 2800, for IMFs a value of 5.0 is stipulated for R_u. Structural analyses are performed using CSI ETABS (Computers and Structures Inc. 2016) and then members are designed according to the INBC Part 10 (2013) which is similar to the ANSI/AISC 360 (2010) and ANSI/AISC 341 (2010). To avoid exceeding the minimum code requirements, the optimum sections are used instead of using standard practices in design and construction, which uses the same section for beams at adjacent bays or columns at several successive stories (FEMA P695 2009). The optimum sections are those with minimum weight which satisfy both ultimate and service limit state's requirements. The mild-steel material used for beam and column members is S235JR with the minimum yield stress and ultimate tensile strength of 235 and 360 MPa, respectively.

Fig. 4 Illustration of index archetype configurations



5.2 Nonlinear Model Development

To develop nonlinear numerical models, the OpenSees platform (McKenna et al. 2000) is employed and steel beams and columns are modeled through the elastic beam-column element. Material nonlinearity is incorporated through the flexural concentrated plastic hinge at the beams and columns ends, where the maximum flexural demand occurs. Geometric nonlinearity that may lead to change in structural behavior and loss of structural stability is also considered by updating the stiffness matrix according to the deformed shape. Besides, axial and shear forces are assumed to be force-control actions. Panel zone shear distortion is explicitly utilized through the model proposed by Gupta and Krawinkler (Gupta and Krawinkler 1999). The schematic of the developed nonlinear numerical model is shown in Fig. 5. It should be noted that the masonry infill wall effects are ignored due to their unreliable response as suggested by FEMA P695 (2009). Likewise, the steel beam composite action is neglected since the composite behavior of beams at the protected zones is prevented by design code requirements (ANSI/AISC 341 2010; INBC Part 10 2013). The moment-rotation backbone curves of beams and columns are constructed through the relations proposed by NIST GCR 17-917-45 (2017) for the first-cycle envelope curve of steel components.

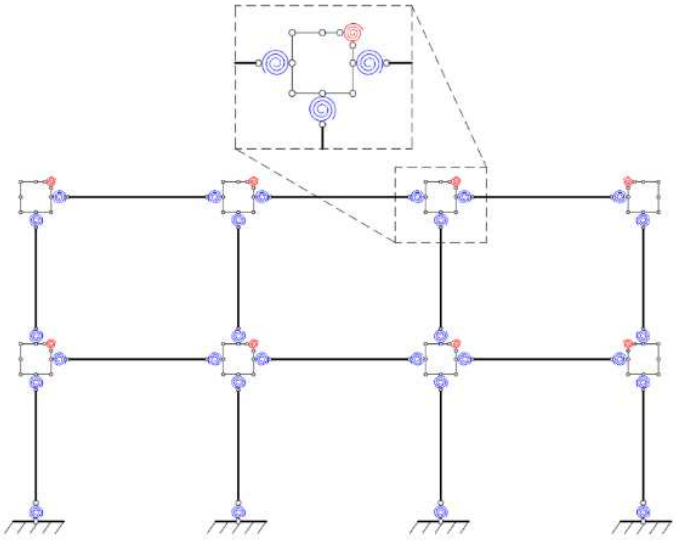


Fig. 5 Schematic representation of the developed numerical nonlinear model

5.3 Nonlinear Analysis

Static nonlinear (pushover) analysis is employed to evaluate the index archetypes' performances in terms of CMR, Δ_r, Δ_d, and μ_d. The advantages of pushover analysis can be summarized as follows: (1) allows easy visualization of system performance at different levels of demand; (2) leads to a unique solution at the considered level of ground motion intensity; (3) needs less computational effort and time than more sophisticated nonlinear history response analysis. Meanwhile, its disadvantages are: (1) ignores the record-to-record variability (σ_{RTR}); (2) is not suitable for buildings with various irregularities; (3) is not appropriate for buildings with significant higher mode effects. To avoid the above disadvantages, only buildings with regular IMFs are considered in compiling the archetype design space. A detailed discussion about the pushover analysis procedures and results accuracy can also be found in FEMA 440 (2005). A triangular load pattern is used in the pushover analysis to compute the global pushover curve (roof displacement against base shear), and then the idealized pushover curve is constructed through the procedure presented in ASCE/SEI 41 (2017).

5.4 Evaluation of CMR

In buildings' seismic design codes, it is assumed that the code-conforming buildings designed for the PGA_{DBE} (PSA_{DBE}) have an adequate factor of safety against collapse if MCE occurs. Based on this subjective assumption performance of the buildings at the MCE is not evaluated explicitly. In this study, it is assumed that the code-conforming buildings must have at least a CMR of 1.5 in the event of MCE (ASCE/SEI 7 2022). Therefore, the CMR is employed to examine the available mean safety factor against the collapse of buildings with IMF as a lateral load-resisting system. There are several approaches to estimating the ultimate capacity of structures (Bernal 1987; Challa and Hall 1994; Zareian and Krawinkler 2007; Han et al. 2010; Harrington and Liel 2016). Here, the ultimate capacity of a building, Δ_c, is defined as a displacement at which the lateral strength drops to 80% of its maximum value (FEMA P695 2009; ASCE/SEI 7 2022). Besides, CMR is defined as the ratio of the displacement at which collapse (global instability) is assumed to occur (Δ_c) to the target displacement (Δ_t) imposed by the MCE

ground motion intensity. Δ_t is calculated through the relationships given by the ASCE/SEI 41 (2017) and Δ_y is a displacement at which the building yields. The above terms are also graphically shown in Fig. 6, where V_b, V_y, and μ_d are base shear, building's idealized yield strength, and ductility demand, respectively. In Fig. 7, the computed CMRs for 96 index archetypes are shown. From this figure, it is clear that the safety margin of 1.5 is only satisfied by some of the assumed PGA_{DBE}s, but not all of them (e.g. 0.15g). So, it is necessary to introduce a lower bound on the PGA_{DBE}, which is found to be 0.35g for Karaj. As shown by Eq. 3, PGA_{DBE} is combined with B, I_e, and R_u in the calculation of the C_s. As a result, the estimated lower limiting value is also affected by the values of these coefficients. In other words, the proposed value of 0.35g is consistent with the R_u equals to five and the B value prescribed in Standard No. 2800 (2014) for regions with high and very high seismicity. In other words, to capture the effects of changes in B, I_e, and R_u on the performance indicators (CMR, Δ_r, μ_d, and Δ_d), it is assumed that their values are constant and any changes could be implemented by modifying the value of PGA_{DBE}.

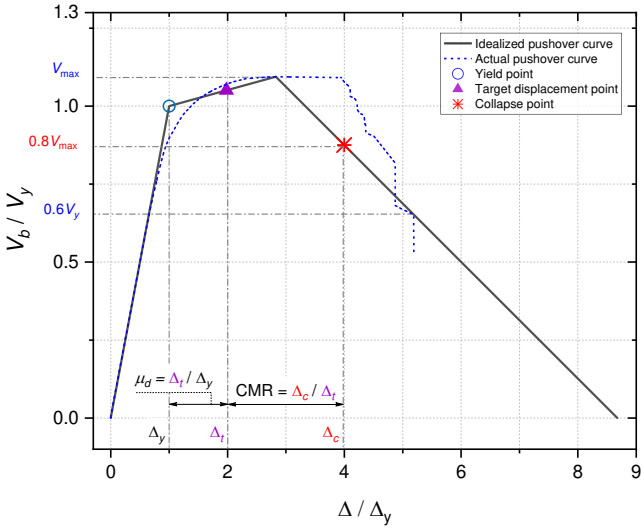


Fig. 6 Illustration of collapse margin ratio (CMR) and terms used in performance assessment

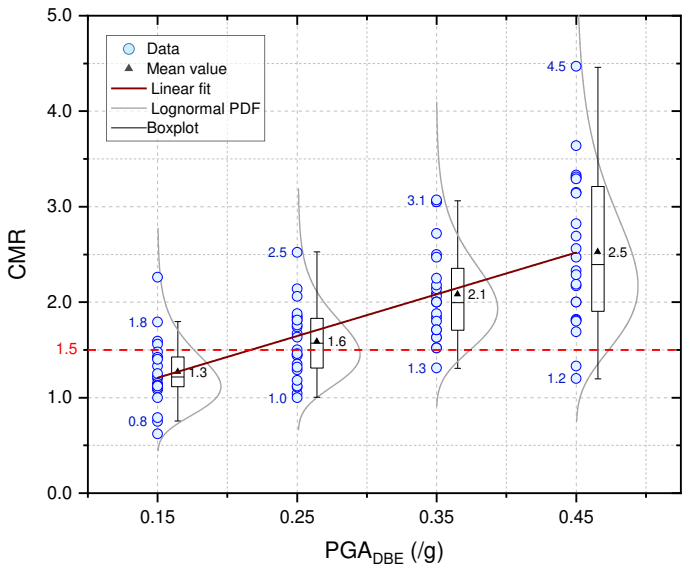


Fig. 7 CMR of 96 index archetypes designed for four levels of PGA_{DBE}

Statistical regression analysis is used to find the relation between CMR and assumed design parameters. According to the statistical t-test analysis, the building's A_R and L_s have minor effects on the CMR and can be ignored from the regression analysis. Therefore, the relation between CMR with design parameters of PGA_{DBE} and O_c is computed and given in Eq. 4, which shows a strong dependence of CMR on PGA_{DBE}. Besides, CMRs' dispersion at each PGA_{DBE} follows a lognormal distribution, whose mean value and standard deviation (SD_{CMR}) are given by Eqs. 4 and 5, respectively.

$$\mathrm{CMR} = \frac{1}{\mathrm{PGA}_d}\left[0.27 + 2.84 \times \mathrm{PGA}_{\mathrm{DBE}} + 0.18 \times O_c\right] \quad \left(\mathrm{R}^2 = 0.51,\ \mathrm{RMSE} = 0.51\right) \tag{4}$$

$$\mathrm{SD}_{\mathrm{CMR}} = 0.15 + 0.34 \times \mathrm{PGA}_{\mathrm{DBE}} \quad \left(\mathrm{R}^2 = 0.70,\ \mathrm{RMSE} = 0.25\right) \tag{5}$$

where PGA_d is the seismic demand (PGA at any desired intensity level) in fraction of g; PGA_{DBE} is the seismic intensity in fraction of g for which the considered building is designed; R^2 and RMSE are the coefficients of determination and root mean square error (residual standard deviation), respectively; O_c is building occupancy (0.0 for residential and 1.0 for commercial buildings). Note that the O_c value for occupancy with a live load intensity in the range of 200 to 500 kg/m^2 could be calculated by linear interpolation between 0.0 and 1.0. The positive sign of the O_c's coefficient indicates that the CMR increases as the O_c (live-to-dead load ratio) increases. This is due to the overstrength created by increasing the live load intensity, which has a lower probability of existence during an earthquake. Eqs. (4) and (5) provide a useful and easy-to-use tool to examine the impact of any seismic demand variation due to any future updates or modifications in design codes. Furthermore, Equation (4) makes it possible to assess the effects of seismic importance (I_e) and response modification (R) factors variations on CMR due to their proportional relationship with the PGA_{DBE}.

5.5 IMFs Usage Justification

To justify the usage of IMFs in different seismic zones, especially those that are to be constructed in regions with very high seismicity (PGA_{DBE} = 0.35), ductility demand (μ_d) needs to be estimated. Next, the estimated ductility demand needs to be compared with the ductility capacity of IMFs stated in the design codes to ensure their stability. In the seismic portion of INBC Part 10 (2013), it is required that the IMFs have at least a rotation capacity of 2% radians at the PGA_{DBE} level of seismic demand, at least 1% of which need to be tolerated in the elastic range and the remaining can be tolerated in the plastic range of behavior. The justification of IMFs usage in regions with very high seismicity is made by providing the relationship between ductility demand as a performance indicator and design parameters via a multiple linear regression analysis as:

$$\mu_d = \mathrm{PGA_d} \times \left[5.99 - 4.57 \times \mathrm{PGA_{DBE}} - 0.29 \times O_c - 0.14 \times L_S - 0.11 \times A_R\right] \quad (6)$$
$$\left(\mathrm{R}^2 = 0.70, \mathrm{RMSE} = 0.25\right)$$

where PGA_d is the PGA of the desired level of earthquake hazard at which building performance needs to be evaluated; PGA_{DBE} is the earthquake intensity for which the considered building is designed; L_s and A_R are the building's span length in units of meter and aspect ratio, respectively.

The use of IMfs is allowed for buildings with normal occupancy (including residential and commercial) and up to 50 meters height from the base level by Iranian Standard No. 2800 (2014), whereas their usage is restricted by ASCE/SEI 7 (2022) in regions with very high seismicity. To ensure the ductility demand will not exceed the ductility capacity of IMFs, ductility demand is calculated at the PGA_{RDBE} as well as PGA_{MCE} levels of demand for residential buildings designed with the PGA_{RDBE} by using Eq. 6 in Karaj. The distribution of calculated ductility demands is shown in Fig. 8. Since all the resulting values are lower than 2.0%, it can be inferred that IMFs have sufficient ductility capacity to be used in the region with high seismicity. It can also be realized that putting a restriction on their usage as stipulated by ASCE/SEI 7 (2022) in regions with very high seismicity might be an unnecessary conservative requirement.

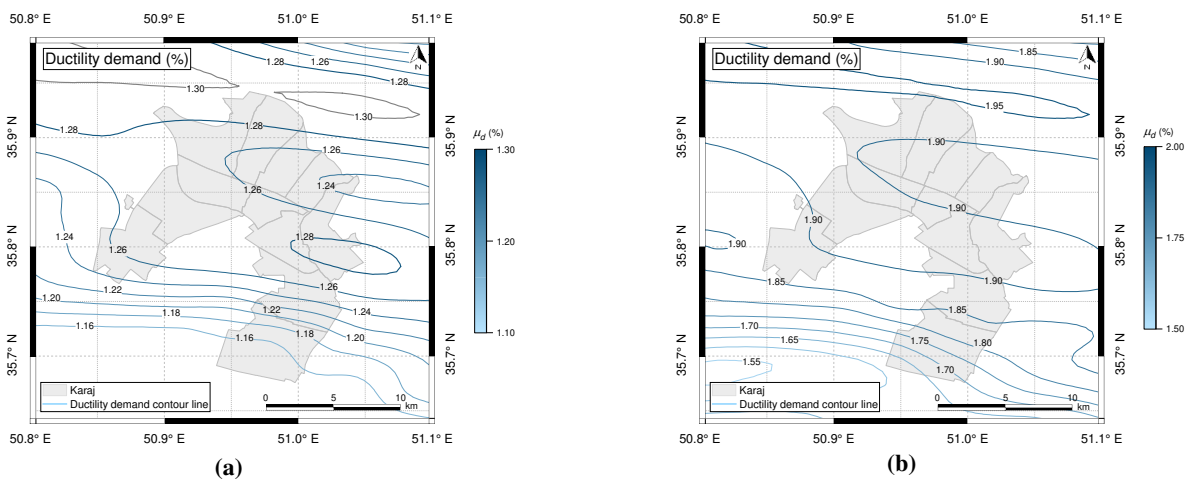


Fig. 8 Ductility demand of residential buildings designed for PGA_{RDBE} and evaluated at the **(a)** PGA_{RDBE} **(b)** PGA_{MCE}

5.6 Damage and Repairability

The structural and many nonstructural (e.g. nonbearing masonry walls) damage could be related to the imposed drift ratio (Δ_d), which is a function of the earthquake ground motion intensity and building's characteristics such as lateral strength, stiffness, geometric properties, and ductility. Moreover, the repairability of buildings at any desired level of demand can be expressed in terms of the residual drift ratio (Δ_r). So, Δ_d and Δ_r are selected as the additional performance indicators and their relationships with the design parameters are calculated through the multiple linear regression as:

$$\Delta_d = \mathrm{PGA}_d \times \left[6.21 - 4.13 \times \mathrm{PGA}_{\mathrm{DBE}} - 0.40 \times O_c - 0.22 \times L_S - 0.21 \times A_R\right] \quad \left(\mathrm{R}^2 = 0.87, \mathrm{RMSE} = 0.16\right) \tag{7}$$

$$\Delta_r = \mathrm{PGA}_d \times \left[4.30 - 4.19 \times \mathrm{PGA}_{\mathrm{DBE}} - 0.34 \times O_c - 0.17 \times L_S - 0.11 \times A_R\right] \quad \left(\mathrm{R}^2 = 0.80, \mathrm{RMSE} = 0.19\right) \tag{8}$$

where PGA_d is the seismic demand of the desired hazard level at which Δ_d and Δ_r are needed to be evaluated; $\mathrm{PGA}_{\mathrm{DBE}}$ is the seismic intensity used to design the considered building; O_c, L_s, and A_R represent the occupancy (live-to-dead load ratio), span length, and building's aspect ratio, respectively. O_c's value should be set to 0.0 for residential and 1.0 for commercial occupancies. For other occupancies with a live load intensity in the range of 200 to 500 kg/m^2, the value of O_c could be computed by linear interpolation between 0.0 and 1.0. Note that the unloading stiffness is assumed to be equal to the elastic stiffness in the derivation of Eq. 8.

According to the FEMA P-58-1 (2018), buildings with a residual drift ratio (Δ_r) up to 0.5% could be repairable and those with 1%, although are repairable, may not be economically and practically feasible, meaning those buildings might be at a total economic loss. The repairability of residential buildings designed with the $\mathrm{PGA}_{\mathrm{RDBE}}$s proposed here and PGA_{2800} stipulated by Standard No. 2800 is examined at the MCE level of demand.

In Fig. 9, buildings designed for the $\mathrm{PGA}_{\mathrm{RDBE}}$s (Fig. 9a) display a high chance of repairability as their Δ_rs are in the mid-range of 0.5% to 1.0%. This means that the buildings designed by the $\mathrm{PGA}_{\mathrm{RDBE}}$ although sustaining structural damage could be considered repairable ones if the MCE occurs. Meanwhile, the calculated Δ_rs close to 1.0% (Fig. 9b) show that the buildings designed by the PGA_{2800} have a lower probability of being repairable in comparison with those designed for $\mathrm{PGA}_{\mathrm{RDBE}}$. Furthermore, as a requirement by the seismic design codes, buildings should be repairable at the $\mathrm{PGA}_{\mathrm{DBE}}$ level of demand. Since it is recommended here to replace the uniform-hazard $\mathrm{PGA}_{\mathrm{DBE}}$s in Standard No. 2800 with the uniform-risk $\mathrm{PGA}_{\mathrm{RDBE}}$, it is required to validate the buildings' repairability at this level of demand as well. So, Δ_rs at $\mathrm{PGA}_{\mathrm{RDBE}}$ demand level are also calculated in Karaj and shown in Fig. 10. The calculated values in the studied region are all lower than 0.5%, which confirms the repairability requirement. Furthermore, the comparison of Δ_r's distribution in Figs. 9a and 10 with those in

Fig. 9b reveals a more uniform distribution of repairability chance for buildings designed by the PGA_{RDBE} in comparison to those designed by PGA_{2800}.

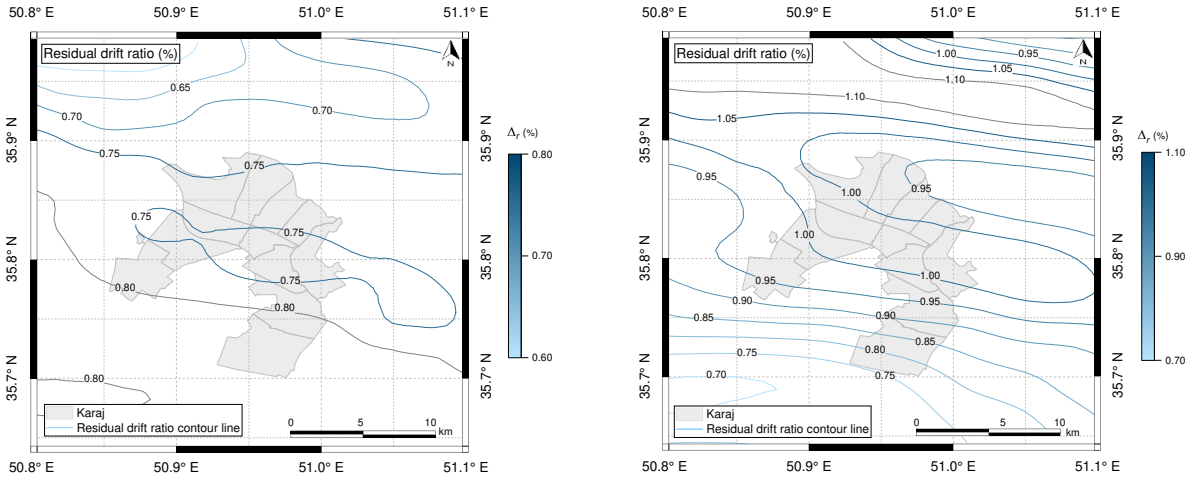
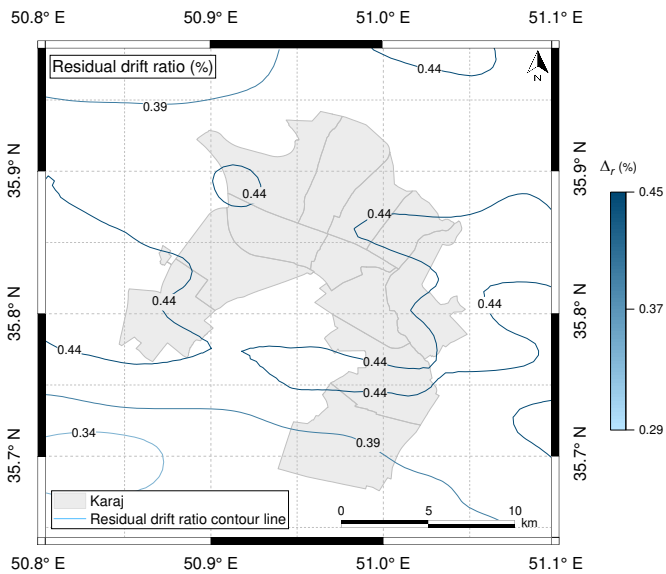


Fig. 9 Residential buildings' residual drift ratio imposed by the MCE level of ground motion intensity (PGA_{MCE}) for those designed with the **(a)** computed PGA_{RDBE} **(b)** PGA_{2800} = 0.35g

Fig. 10 Residual drift ratios (mean values) of residential buildings designed for and evaluated at the PGA_{RDBE}



5.7 Economic Consequence of Changes in the PGA_{DBE}

An increase in the PGA_{DBE} improves the safety margin against collapse; however, it also causes an increase in construction cost that may render the safety enhancement an impossible choice. To find the optimum affordable safety, the relationship between the PGA_{DBE} and building weight as a basic economic index needs to be known. Again, using the results obtained from developed 96 index archetypes, the PGA_{DBE} relation to the building normalized weight (W_N) is investigated, which shows a linear relationship as shown in Fig. 11. This linear relationship is derived by normalizing buildings' weights related to the different levels of PGA_{DBE} (W_i) to that

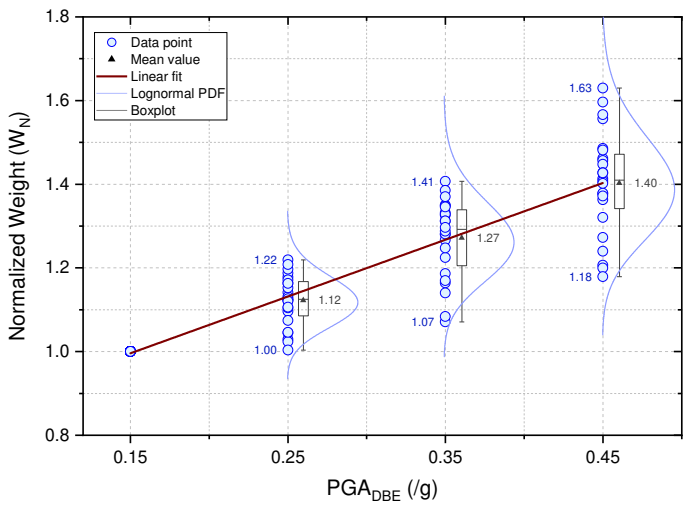
obtained for PGA_{DBE} = 0.15 ($W_{0.15g}$) as given in Eq. 9. Furthermore, W_N's dispersion at each PGA_{DBE} follows a lognormal distribution, whose mean and logarithmic standard deviation can be expressed by Eqs. 9 and 10, respectively.

$$W_N = 0.79 + 1.36 \times \mathrm{PGA}_{\mathrm{DBE}} \quad \left(\mathrm{R}^2 = 0.78,\ \mathrm{RMSE} = 0.08\right) \tag{9}$$

$$\mathrm{SD}_{W_N} = 0.014 + 0.17 \times \mathrm{PGA}_{\mathrm{DBE}} \quad \left(\mathrm{PGA}_{\mathrm{DBE}} \geq 0.25,\ \mathrm{R}^2 = 0.99,\ \mathrm{RMSE} = 0.003\right) \tag{10}$$

where W_N is the normalized weight ($W_N = W_i/W_{0.15g}$). This equation shows that changing the PGA_{DBE} from 0.35g to 0.40g leads to a 5% increase in the building's structural weight, which has a minor economic impact on the individual scale.

Fig. 11 Normalized weight versus PGA_{DBE}



6. Conclusion and Discussions

Many design codes such as the Iranian Standard No. 2800 use a uniform-hazard procedure to determine the design basis earthquake. These need to be updated by taking a risk-informed approach to make them compatible with the acceptable and tolerable level of risk. The risk-adjusted design basis earthquake is presented here by limiting the building's collapse risk in its expected lifespan and in case the MCE occurs. PGA_{RDBE}s in Karaj are calculated by optimizing the building collapse risk integral so that the acceptance criteria are satisfied. The calculated PGA_{RDBE}s are found to be in the range of 0.35g to 0.45g, while the PGA_{DBE} prescribed by Standard No. 2800 gives a constant value of 0.35g for the whole region. It is also shown that the PGA_{2800} neither provides uniform collapse risk nor stands on the safe side. Based on these results, it is recommended to modify PGA_{DBE} in Standard No. 2800 with those PGA_{RDBE}s given here. Risk-adjustment factors (R_A) are defined to easily convert the PGA_{2800} of uniform-hazard to PGA_{RDBE} (Fig. 3b).

The above results show that the PGA_{RDBE} provides a good alternative to modifying PGA_{DBE} in the seismic design codes. Meanwhile, it is found that further investigations are necessary to confirm its compliance with other code requirements. The adequacy of calculated PGA_{RDBE}s is examined at different demand levels by nonlinear

analysis of 96 index building archetypes with IMF developed here. The various relationships between calculated performance indicators (CMR, Δ_r, μ_d, and Δ_d) with the assumed design parameters (PGA_{DBE}, N_s, L_s, O_c, and A_R) are established using multiple linear regression analysis, which accelerates the calculation of performance indicators at any desired demand levels. The performance indicators are calculated for a ranged PGA from 0.15g up to 0.45g regardless of the employed method to calculate them (either be a uniform-hazard or uniform-risk).

Two intended performance goals, namely CMR and repairability, are evaluated at two demand levels of PGA_{MCE} and PGA_{DBE}. To examine the implicit assumption of code-conforming buildings having at least a safety margin of 1.5 against collapse if the MCE occurs, the CMRs are calculated using Eq. 4 for the buildings in the studied region. The calculated CMRs show that the PGA_{RDBE} may fail to provide CMRs larger than 1.5. This emphasizes that the direct application of PGA_{RDBE} without ensuring its adequacy may provide misleading results. To prevent undesirable results (CMR<1.5), the calculated PGA_{RDBE}s are modified so that the CMRs larger than 1.5 are guaranteed for almost all buildings in the studied region. This is carried out by defining a lower limit for the calculated PGA_{RDBE}s in the studied area to be 0.35g (Fig. 7).

The repairability requirement of buildings designed with the calculated PGA_{RDBE}s is also examined by computing residual drift ratios using Eq. 8 as shown in Figs. 9 and 10. Buildings designed for and evaluated at the PGA_{RDBE} are repairable by showing a residual drift ratio lower than 0.5%. Furthermore, they have a high chance to be considered repairable at the PGA_{MCE} demand level with drift ratios in the mid-range of 0.5% to 1.0%. Furthermore, ductility demand maps for residential buildings designed for PGA_{RDBE} and evaluated at the PGA_{RDBE} as well as the PGA_{MCE} are produced for those located in the studied area (Fig. 8). The values of calculated ductility demand lower than 2% confirm that the IMFs could be utilized in regions with very high seismicity without concern about the imposed ductility demand.

From an economic point of view, although replacing the PGA_{DBE} with PGA_{RDBE}s in seismic codes enhances safety, it is necessary to check whether this increase in PGA_{DBE} is also affordable or not. So, by selecting the building's normalized weight as a basic economic index, the relationship between this parameter and the PGA_{DBE} is found and proposed to be used (Eq. 9) as a tool for checking the economic consequences of an increase in PGA_{DBE}. From this relation, it is found that an increase of PGA_{DBE} from 0.35g to 0.40g leads to a 5% increase in the building's structural weight, which has a minor economic impact on the individual scale.

Finally, to design buildings beyond the intended performance goals assumed in the seismic design codes, the performance-based design procedure has been proposed as a suitable alternative. From the practical point of view, this approach needs iterative, complicated, and time-consuming nonlinear analyses that make it a difficult-to-use choice in traditional design. Using the proposed relationships between performance indicators (CMR, Δ_r, Δ_d, μ_d) and design parameters for buildings with IMF not only overcome those disadvantages but also provide a simple straightforward tool to achieve desired performances at required seismic demand levels in the preliminary design stages.

Declarations

Ethics approval and consent to participate

Not applicable.

Consent for publication

I (Mohammad Zaman) aware of the submission and have agreed for the paper "Risk-adjusted design basis earthquake's adequacy for use in seismic design codes" to be submitted to "Journal of Seismology".

I (Mohammad Reza Ghayamghamian) aware of the submission and have agreed for the paper "Risk-adjusted design basis earthquake's adequacy for use in seismic design codes" to be submitted to "Journal of Seismology".

Availability of data and material

Some or all data, models, or codes that support the findings of this study are available from the corresponding author upon reasonable request.

Competing interests

The authors have no competing interests to declare that are relevant to the content of this article.

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Authors' contributions

Mohammad Zaman: Conceptualization, Methodology, Software, Validation, Writing original draft , Reviewing, and Editing.

Mohammad Reza Ghayamghamian: Conceptualization, Methodology, Data curation, Validation, Supervision, Writing, Reviewing and Editing.

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