Earthquakes and Structures, *Vol. 18, No. 6 (2020) 709-718*
DOI: https://doi.org/10.12989/eas.2020.18.6.709

Seismic vulnerability assessment of RC buildings according to the 2007 and 2018 Turkish seismic codes

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(Received February 21, 2020, Revised May 21, 2020, Accepted June 6, 2020)

Abstract. Fragility curves are useful tools to estimate the damage probability of buildings owing to seismic actions. The purpose of this study is to investigate seismic vulnerability of reinforced concrete (RC) buildings, according to the 2007 and 2018 Turkish Seismic Codes, using fragility curves. For the numerical analyses, typical five- and seven-storey RC buildings were selected and incremental dynamic analyses (IDA) were performed. To complete the IDAs, eleven earthquake acceleration records multiplied by various scaling factors from 0.2g to 0.8g were used. To predict nonlinearity, a distributed hinge model that involves material and geometric nonlinearity of the structural members was used. Damages to confined concrete and reinforcement bar of structural members were obtained by considering the unit deformation demands of the 2007 Turkish Seismic Code (TSC-2007) and the 2018 Turkey Building Earthquake Code (TBEC-2018). Vulnerability evaluation of these buildings was performed using fragility curves based on the results of incremental dynamic analyses. Fragility curves were generated in terms of damage levels occurring in confined concrete and reinforcement bar of structural members with a lognormal distribution assumption. The fragility curves show that the probability of damage occurring is more according to TBEC-2018 than according to TSC-2007 for selected buildings.

Keywords: vulnerability assessment, fragility curves, structural damages, TSC-2007, TBEC-2018

1. Introduction

Earthquake is one of the most destructive phenomena which has caused great destruction since the existence of mankind. Researchers have been working to prevent loss of human lives and the collapse of structures. They have produced new solutions which form the basis to develop seismic codes. The aim of the many researchers studying earthquake engineering is to ensure that a structure can demonstrate sufficient performance and strength when encountering an earthquake. However, one of the most important problems to consider is which earthquake meets which performance level? To resolve this problem, seismic vulnerability analysis (SVA) presents satisfying results (Baker 2013). This analysis provides significant scientific guidance to alleviate seismic disaster risk by guiding authorities on prioritization of their limited resources to manage risk reduction (Zhou *et al*. 2020). Fragility theory is an approach for estimating structure performance and seismic risk assessment. This method is a generalized arm of structural reliability that evaluates the vulnerability of a building based on ground motion intensity (Zhang and Hu 2005) which identifies the probability of damage to meet a performance level as a function of the demand on the structure. The steps of fragility analysis are summarized in Fig.1.

Seismic vulnerability evaluation of structures has garnered interest from researchers in recent years. Thus, studies about the seismic vulnerability of structures for reducing earthquake disaster risk are of relevance. Wu *et al*. (2012) conducted a seismic fragility assessment of an RC frame structure designed according to modern Chinese code for the seismic design of buildings. Pavel and Carale (2019) carried out a seismic assessment of typical soft-storey reinforced concrete structures in Romania by performing fragility analysis. Rajeev and Tesfamariam (2012) investigated seismic fragilities for reinforced concrete buildings with an evaluation of irregularities using fragility analysis. Park *et al*. (2009) applied this method and they assessed low-rise unreinforced masonry structures. Çelik and Ellingwood (2010) carried out a study about seismic fragilities for non-ductile reinforced concrete frames. They used fragility curves in their paper. Ortege *et al*. (2019) generated a vulnerability formulation for the seismic vulnerability evaluation of vernacular architecture. Kirçil and Polat (2006) carried out a fragility analysis of mid-rise reinforced concrete frame buildings in Turkey. Kwon and Elnashai (2006) evaluated the effect of material and seismic ground motion uncertainty on the seismic vulnerability curves of reinforced concrete buildings. Erberik (2008) performed a study based on the fragility evaluation of typical mid-rise and low-rise RC buildings in Turkey. Rossetto *et al*. (2016) suggested a new approach for the production of fragility curves. Their approach adapts the capacity spectrum evaluation method and they use inelastic response spectra derived from seismic ground motions to generate fragility curves.

Fragility curves can be generated through empirical- (Rossetto and Elnashai 2003, Shinozuka *et al*. 2000),

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

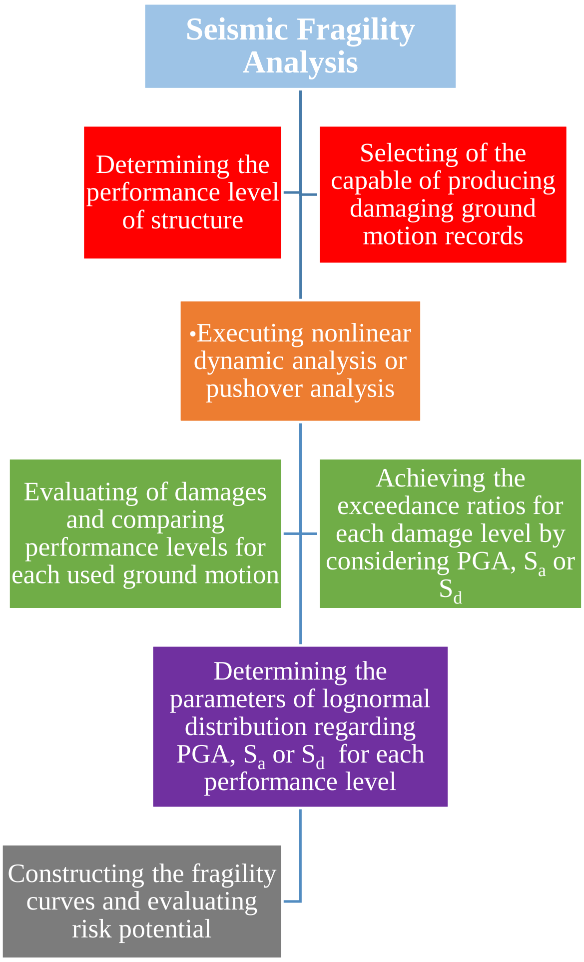


Fig. 1 Flowchart of the proposed method

analytical- (Ellingwood *et al*. 2007, Rossetto and Elnashai 2005), and heuristic- (Earthquake Damage Evaluation Data for California, Applied Technology Council-ATC 1985) based methods. In this study, fragility curves were generated by considering an analytical method. Thus, the vulnerability assessment of selected structures was performed according to fragility curves produced considering the Turkish Seismic Code (TSC-2007) and the Turkish Building Earthquake Code (TBEC-2018). This study differs from other studies in the literature by determining the element-based damage to confined concrete and reinforcement bar to evaluate the fragility of buildings.

2. Construction of fragility curves

Damage from previous earthquakes requires risk assessment to predict future earthquake damage. For this purpose, fragility curves that permit assessment of damage probabilities as a function of ground motion indices (PGA, PGV) or structural parameters (Sa, Sd) have been evaluated as useful tools (Şenel and Kayhan 2010). In this study, fragility curves for each member of the frame were constructed in accordance with confined concrete and reinforcement bar by using incremental dynamic analysis. Element damage was assessed according to unit deformation demand and categorized considering TSC-2007 and TBEC-2018. In this way, exceedance numbers for elements and materials were determined and exceedance ratios for each PGA of the selected records were obtained, while a fragility function analytical relationship was also used. Accordingly, exceedance ratios and a two-parameter lognormal distribution were used, as shown below.

$$P\ (\Delta_D > \delta\ |IM = im\ ) = 1 - \Phi\left[\frac{In\ (\delta) - In\ (\lambda_D)}{\beta_D}\right] \quad (1)$$

$$\lambda_D\ = a.IM^b \quad (2)$$

$$f(x) = \frac{1}{\beta\ \sqrt{2\pi}} \exp\left(\frac{-\ (x-\mu)^2}{2\beta^2}\right) \quad (3)$$

$$f(x) = \frac{1}{\beta\ x\ \sqrt{2\pi}} \exp\left(\frac{-\ (In\ x-\mu)^2}{2\beta^2}\right) \quad (4)$$

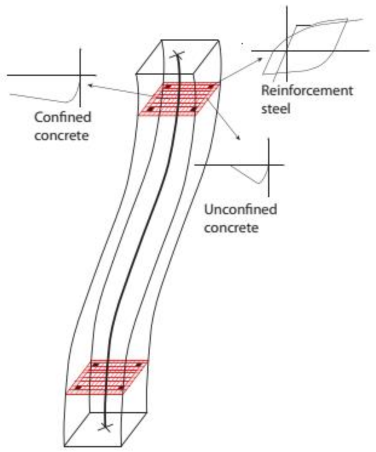
$$with\ \ x > 0$$

The probabilistic relationships between structural responses and ground motion intensity levels are given in Eq. (1). Here, Δ_D describes the seismic demand assumed to be lognormally distributed, while δ and im represent the specified structural demand level and ground motion intensity level, respectively. In addition to this λ_D defines the relationship of the median seismic demand. Also, Φ [..] gives the standard normal probability integral. β_D shows logarithmic standard deviation of the seismic demand. In Eq. (2), a and b are coefficients that can be achieved from regression analyses. Normal and lognormal distributions are given in Eqs. (3) and (4), respectively. Parameters of normal and lognormal distributions are given as mean (μ), standard derivation (β) and variable(x).

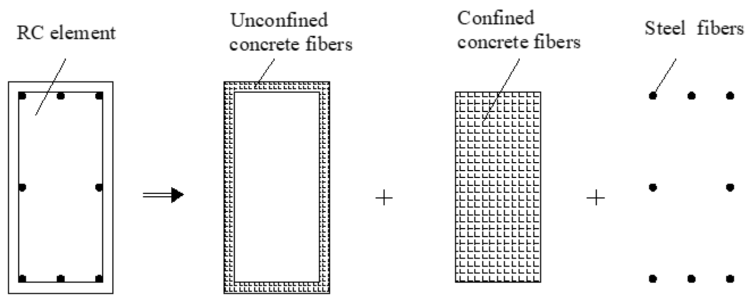
3. Nonlinear modelling approach

A distributed plastic hinge model is more accurate than lumped hinge models, especially when large axial force variations exist (Mwafy and Elnashai 2001). With developing computer technologies, researchers such as Kwon and Kim (2010), Mwafy (2011), Duan and Hueste (2012), Yön and Calayır (2014, 2015), Yön *et al*. (2016), Öncü and Yön (2016), Onat *et al*. (2018), Chaulagain *et al*. (2015), Sadraddin *et al*. (2016), and Khaloo (2016) have made more accurate and sensitive analyses to estimate nonlinear behaviour of reinforced concrete buildings using this model.

This hinge model is based on the monitoring of nonlinear response of reinforcement bar, confined concrete, and cover concrete. This scenario allows us to construct fragility curves for concrete and reinforcement bar separately. To achieve material and geometric nonlinearity of the structural members, the structural element was divided into three types of fibres. The fibre element is mainly concerned with plasticity. Some fibres were used to monitor reinforcement bars; some of fibres were used to



(a) Fibre based modelling (adapted from Rodrigues (2012)



(b) detailing in cross section

Fig. 2 Fibre modelling

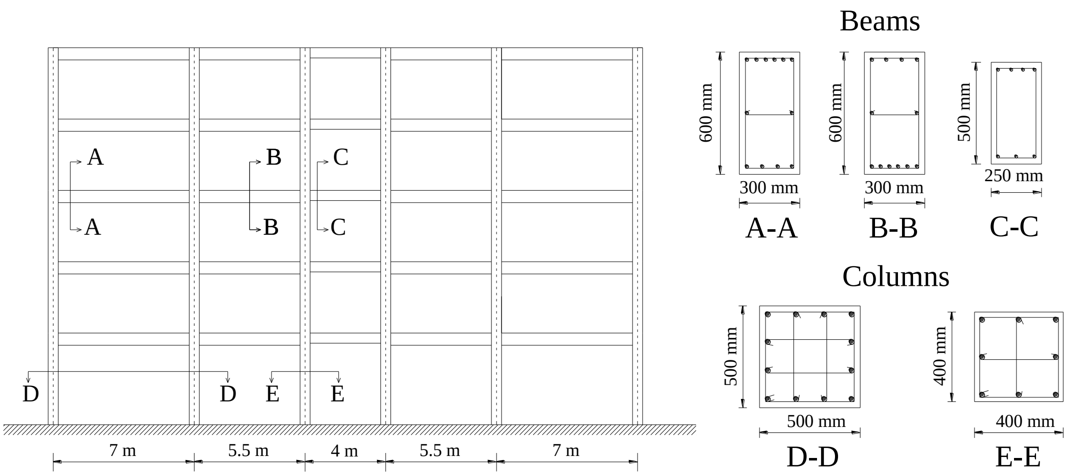


Fig. 3 Five storeys RC building and cross sections of structural members

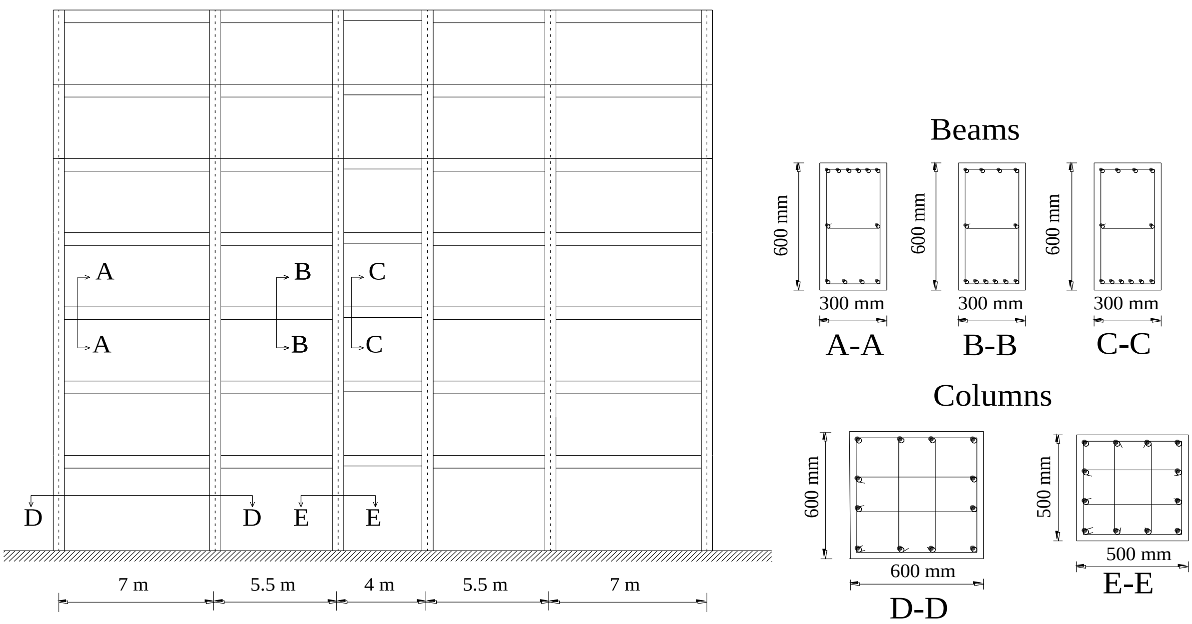
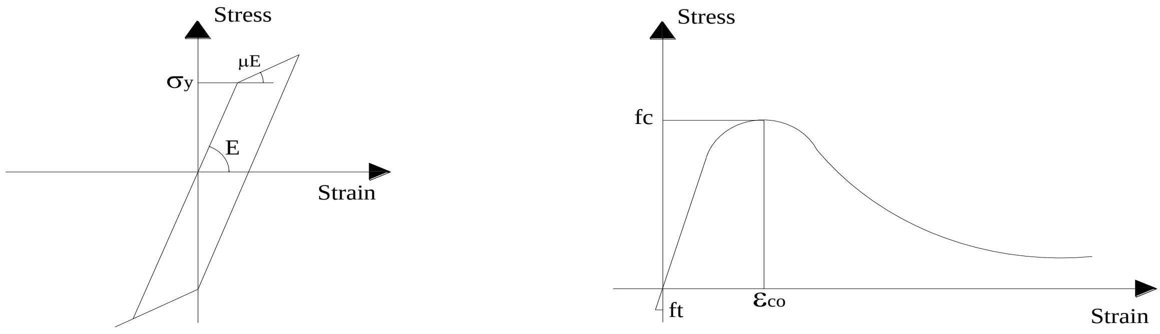


Fig. 4 Seven storeys RC building and cross sections of structural members



(a) Reinforcement bar (b) Concrete

Fig. 5 Material models used for nonlinear dynamic analyses

Table 1 Parameters related to the confinement zones in structural elements

Structural Elements		Longitudinal reinforcement	Transverse reinforcement	Transverse reinforcement spacing (mm)	Length of confinement zone (mm)
Column (600x600 mm)	Confinement zone of column	12Ø20	Ø10	80	800
	Central zone of column			150	
Column (500x500)	Confinement zone of column	12Ø16		80	800
	Central zone of column			150	
Column (400x400)	Confinement zone of column	8Ø16		80	800
	Central zone of column			150	
Beam (300x600 mm)	Confinement zone of beam	Top reinforcement 6Ø12		80	1200
	Central zone of beam	Bottom reinforcement 4Ø12		150	
Beam (250x500)	Confinement zone of beam	Top and bottom reinforcement 4Ø12		80	1000
	Central zone of beam	Bottom reinforcement 4Ø12		150	

define nonlinear behaviour of confined concrete and the other fibres were used to define unconfined concrete behaviour. This plasticity was distributed throughout the cross-section and the length of the element. Also, the stress/strain response was determined for each fibre in a nonlinear range according to the defined materials. Fig. 2 shows fibre modelling of a typical section of a reinforced concrete element, with detail in a cross section.

4. Numerical Investigation

For numerical modelling, high ductility five- and seven-storey five-bay reinforced concrete frames, which can often be found in Turkey, were selected. The selected models were designed according to the requirements of seismic codes and Requirements for Design and Construction and Reinforced Concrete Structures (Turkish Standard TS500-2000). The total heights of the five- and seven-storey buildings are 18.5 and 25.5 m, respectively. The height of the first storey of the buildings is 4.5 m and the upper storey heights are 3.5 m. Elevation of the selected frame buildings for five storeys and seven storeys together with the cross sections of the structural elements are given in Figs. 3–4, respectively The bases of the buildings were assumed to be rigidly fixed and5% damping was used for both buildings. Also, soil-structure interaction was not considered. Nonlinear analyses were performed in the x direction using the Dead (G)+Live (Q)+Earthquake (E) load combination. In the buildings, infill walls were only accounted for as load (2.0 kN/m^2), dead load was taken as 1.5 kN/m^2 and the live load was selected as 2.0 kN/m^2 required by the Turkish Standard 498 (TS 498-Design Loads for Buildings).

4.1 Materials

For selected buildings, concrete compressive strength was presumed to be 25 MPa and the yielding of reinforcement was supposed to be 420 MPa. The uniaxial confinement concrete model (Mander *et al.* 1988) was

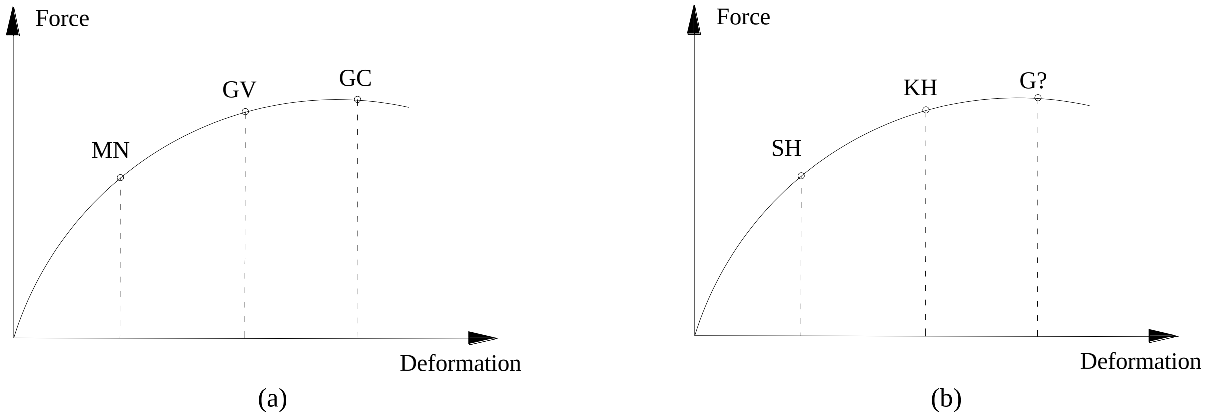


Fig. 6. Performance levels for (a) TSC-2007 and (b)TBEC-2018

Table 2 Performance parameters for (TBEC-2018)

Damage Level	Limit Values	
	Confined Concrete	Steel Bar
Limited Damage (SH) Performance Level	$(\varepsilon_c)^{SH}$=0.0025	$(\varepsilon_s)^{SH}$=0.0075
Controlled Damage (KH) Performance Level	$(\varepsilon_c)^{KH} = 0.75 \ (\varepsilon_c)^{GÖ}$	$(\varepsilon_s)^{KH} = 0.75 \ (\varepsilon_{su})^{GÖ}$
Collapse Prevention (GÖ) Performance Level	$(\varepsilon_c)^{GÖ}= \ 0.0035 + 0.04\sqrt{w_e} \leq 0.018$	$(\varepsilon_s)^{GÖ}$=0.40 ε_{su}

Table 3 Performance parameters for (TSC-2007)

Damage Level	Limit Values for Confined Concrete	Limit Values for Steel Bar
Minimum Damage Limit (MN)	$(\varepsilon_{cu})_{MN}$=0.0035	$(\varepsilon_s)_{MN}$=0.010
Safety Damage Limit (GV)	$(\varepsilon_{cg})_{GV} = 0.0035 + 0.01(\rho_s/\rho_{sm}) \leq 0.0135$	$(\varepsilon_s)_{GV}$=0.040
Collapse Damage Limit (GC)	$(\varepsilon_{cg})_{GC}= \ 0.004 + 0.014(\rho_s/\rho_{sm}) \leq 0.018$	$(\varepsilon_s)_{GC}$=0.060

implemented for the confined concrete fibres, while a bilinear elastic plastic material model, which includes inematic strain hardening was used for the reinforcement bars. These material models are presented in Fig.5. Also, parameters related to the confinement zones in structural elements are expressed in Table 1.

4.2 Structural performance parameters

Both TSC-2007 and TBEC-2018 performance parameters are based on unit deformation demand. These damage limits are given separately for confined concrete and reinforcement bar. However, in TBEC-2018 performance parameters can be procured by considering rotations. In this study, unit deformation demands were considered.

In TBEC-2018, for ductile elements, three damage limits are defined at the cross-sectional level. These limits are limited damage (SH), controlled damage (KH), and pre-collapse damage (GÖ) limits. Limited damage refers to a limited amount of inelastic behaviour in the relevant section, controlled damage refers to the inelastic behaviour in which cross-sectional strength can be achieved safely, and pre-collapse damage refers to the advanced inelastic behaviour in the section. This classification does not apply to brittle damaged elements.

In terms of TSC-2007, these damage limits are defined in a similar way to TBEC-2018. For ductile elements, three boundary conditions are defined at the cross-sectional level. These are minimum damage limit (MN), safety limit (GV), and collapse limit (GC).

The minimum damage limit is the beginning of inelastic behaviour in the relevant section; the safety limit is the limit of inelastic behaviour in which the section can safely provide strength; and the collapse limit defines the limit of pre-collapse behaviour of the section. This classification does not apply to brittle damaged elements. The performance levels for the two seismic codes are shown in Fig. 6.

However, unit deformation demands for concrete and steel are different for the two seismic codes. The limit values for rectangular section elements are shown in Table 2 and Table 3 for TBEC-2018 and TSC-2007, respectively. In Table 2, the first term in the pre-collapse damage (GÖ) limit (0.0035) expresses the unit deformation of unconfined concrete. In this table, w_{we} is the reinforcement ratio of active confinement reinforcement, ε_c gives total unit deformation of confined concrete, while ε_s represents

Table 4 Selected earthquake acceleration records for dynamic analyses

Number	Earthquakes	Station	Direction	Date	Magnitude	PGA (g)
1	Kocaeli	Düzce	E-W	August 17, 1999	7.4	0.381
2	Kocaeli	Sakarya	E-W	August 17, 1999	7.4	0.415
3	Düzce	Bolu	E-W	November 12, 1999	7.2	0.821
4	Düzce	Düzce	E-W	November 12, 1999	7.2	0.524
5	Van	Van-Muradiye	N-S	October 23, 2011	6.7	0.182
6	Van	Van	E-W	November 9, 2011	5.6	0.251
7	Erzincan	Erzincan	E-W	March 13, 1992	6.1	0.480
8	Bingöl	Bingöl	N-S	May 1, 2003	6.1	0.556
9	Sultandağı	Afyon	N-S	February 3, 2002	6.1	0.116
10	Dinar	Afton-Dinar	E-W	October 1, 1995	6.0	0.336
11	Ceyhan	Adana-Ceyhan	E-W	June 27, 1998	5.9	0.279

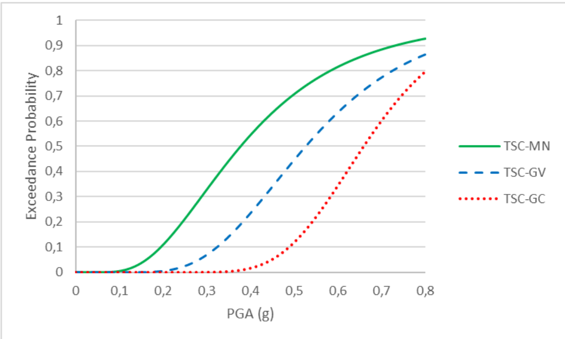


Fig. 7. Seismic fragility curves for confined concrete of beams in five-storey RC frame (TSC-2007)

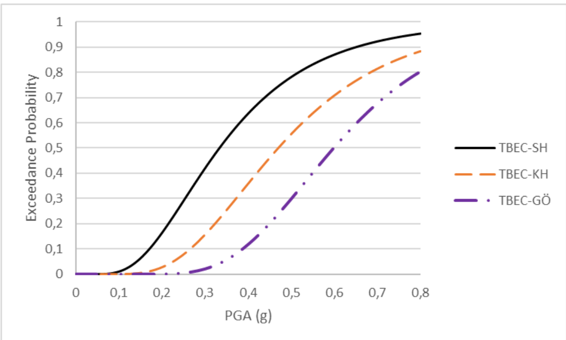


Fig. 8. Seismic fragility curves for confined concrete of beams in the five-storey RC frame (TBEC-2018)

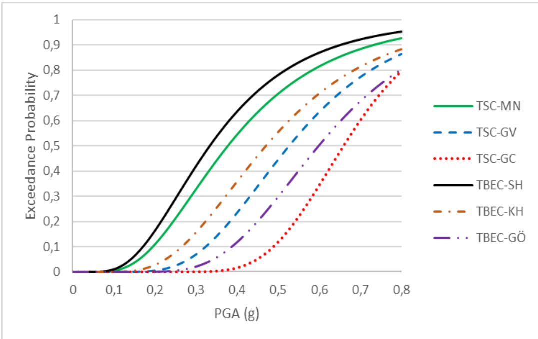


Fig. 9. Comparison of TSC-2007 and TBEC-2018 using seismic fragility curves for confined concrete of beams in the five-storey RC frame

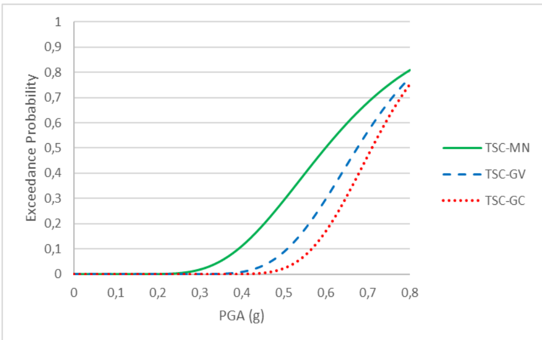


Fig. 10. Seismic fragility curves for confined concrete of columns in the five-storey RC frame (TSC-2007)

deformation of the reinforcement bar unit. ε_{su} shows unit elongation corresponding to maximum strength.

In this table, ε_{cu} gives the ultimate strain of unconfined concrete while ε_{cg} demonstrates ultimate strain of confined concrete. In addition to this, ε_s indicates unit deformation of reinforcement bar. ρ_s and ρ_{sm} describe volumetric ratio of spiral reinforcement existing in the cross section and arranged as 'special seismic ties' existence of transverse rebar in the cross section as a volumetric ratio is necessary.

4.3 Seismic ground motions

To carry out IDA, records of eleven seismic events which occurred in Turkey were selected because the TBEC-2018 requires at least this number. In TSC-2007, just three were required. These selected seismic acceleration records were multiplied by various scaling factors from 0.2g to 0.8g. IDA analyses were performed using the SeismoStruct V7 (2014) structural analysis program which can simulate the inelastic response of structural systems subjected to static and dynamic loads. Properties of the records are presented in Table 4. The seismic records were acquired from the Disaster and Emergency Management Authority (AFAD).

4.4 Nonlinear dynamic analysis and fragility curves

Damage assessment of the buildings were performed by using strain-based damage limits. Forty-four dynamic analyses were performed for each selected building. After analyses, damages to confined concrete and reinforcement

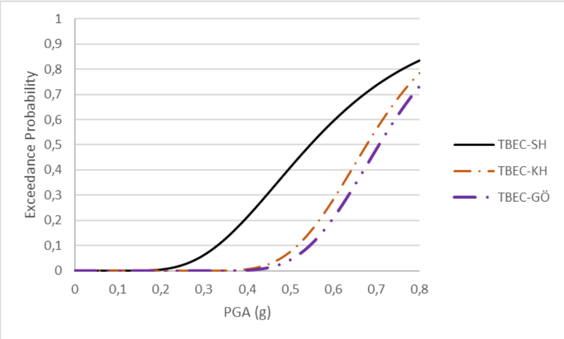


Fig. 11. Seismic fragility curves for confined concrete of columns in the five-storey RC frame (TBEC-2018)

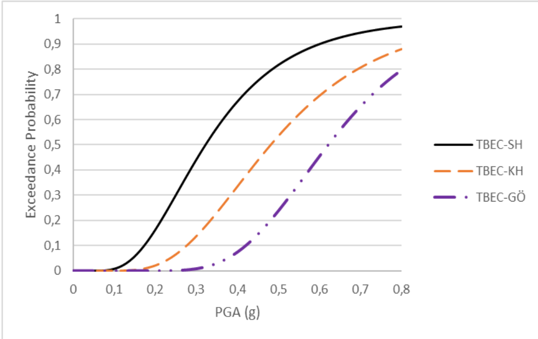


Fig. 14. Seismic fragility curves for confined concrete of beams in the seven-storey RC frame (TBEC-2018)

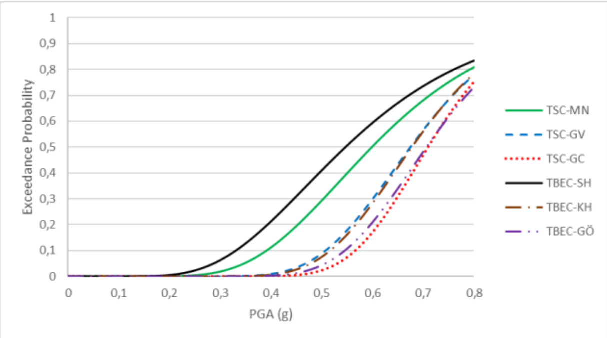


Fig. 12. Comparison of TSC-2007 and TBEC-2018 using seismic fragility curves for confined concrete of columns in the five-storey RC frame

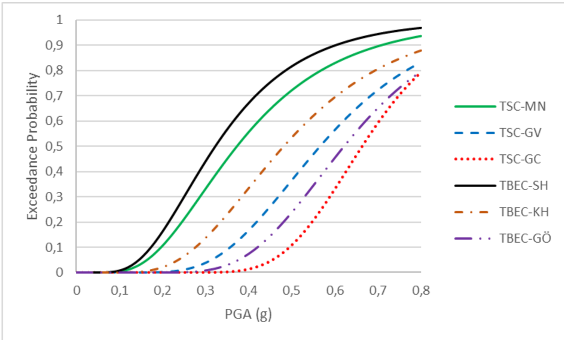


Fig. 15. Comparison of TSC-2007 and TBEC-2018 using seismic fragility curves for confined concrete of beams in the seven-storey RC frame

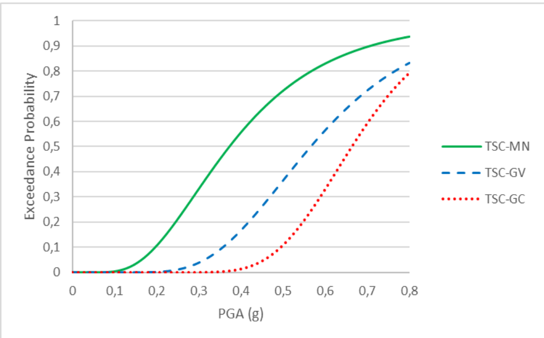


Fig. 13. Seismic fragility curves for confined concrete of beams in the seven-storey RC frame (TSC-2007)

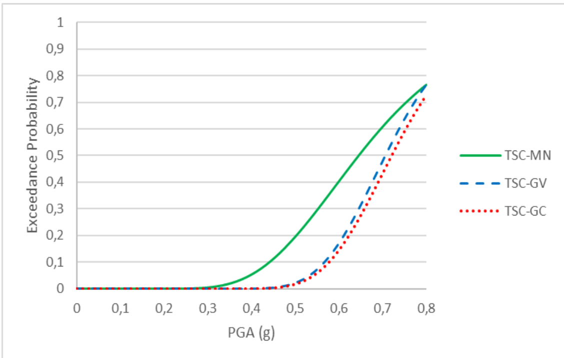


Fig. 16. Seismic fragility curves for confined concrete of columns in the seven-storey RC frame (TSC-2007)

bar of the structural members were procured according to the unit deformation demands of TSC-2007 and TBEC-2018. The fragility curves were drawn according to the lognormal distributions of the damages obtained from different acceleration amplitudes. To create the curves, the EasyFit program (Schittkowski 2002) was used.

5. Results

Damages to structural members were determined for confined concrete and reinforcement bar. After that, damages to each column and beam were proportioned to the total damage of the selected structures. Fragility curves were generated for confined concrete and reinforcement bar for columns and beams.

5.1 Fragility curves for confined concrete

The procured fragility curves are presented in Fig. 7 and Fig. 8 for confined concrete of beams of the five-storey RC frame in accordance with TSC-2007 and TBEC-2018, respectively. In Fig. 9, the fragility curves are compared with each other for the two seismic codes. These figures show that the probability of the occurrence of damage increases in line with the damage limit. Comparison of these curves shows, as in Fig. 9, that the damage probability for TBEC-2018 is higher than TSC-2007, particularly for beams.

Developed fragility curves are illustrated in Figs. 10 and 11 for confined concrete of columns in the five-storey RC frame according to the codes. A comparison of the fragility

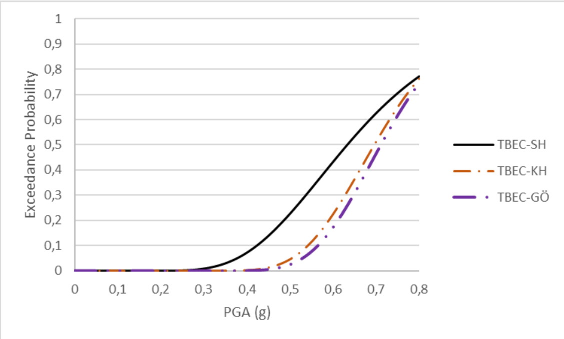


Fig. 17. Seismic fragility curves for confined concrete of columns in the seven-storey RC frame (TBEC-2018)

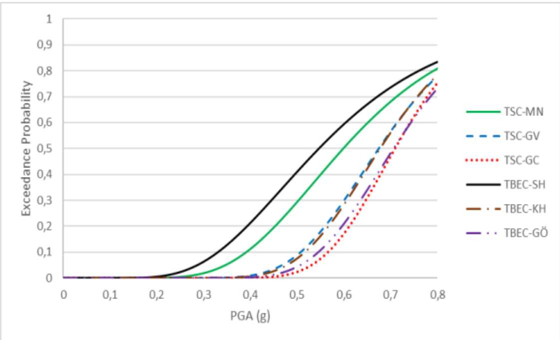


Fig. 18. Comparison of TSC-2007 and TBEC-2018 using seismic fragility curves for confined concrete of columns in the seven-storey RC frame

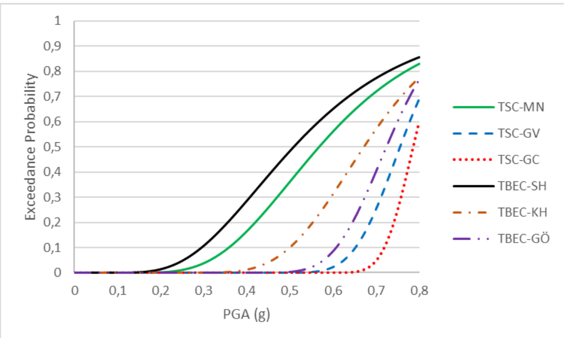


Fig. 19. Comparison of TSC-2007 and TBEC-2018 using seismic fragility curves for reinforcement bar of beams in the five-storey RC frame

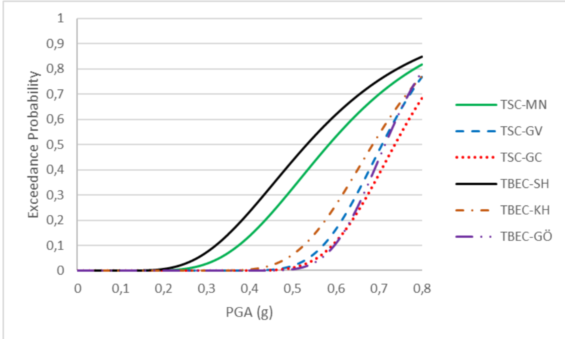


Fig. 20. Comparison of TSC-2007 and TBEC-2018 using seismic fragility curves for reinforcement bar of beams in the seven-storey RC frame

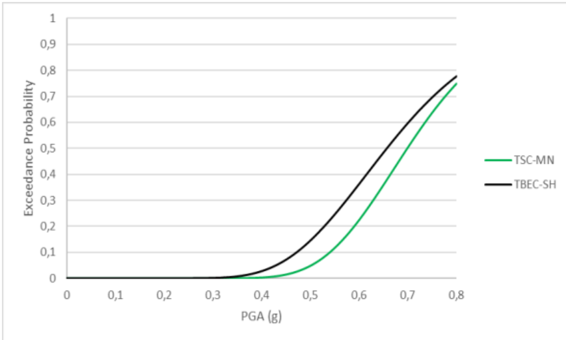


Fig. 21. Comparison of TSC-2007 and TBEC-2018 using fragility curves for reinforcement bar of columns in the five-storey RC frame

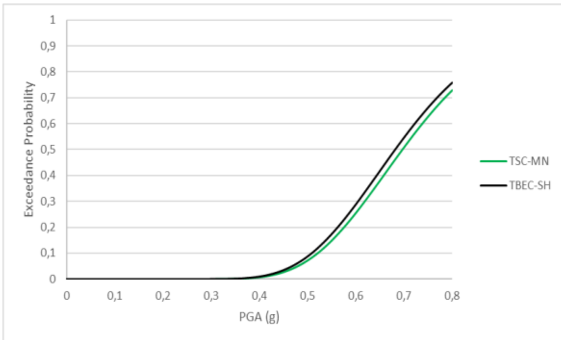


Fig. 22. Comparison of TSC-2007 and TBEC-2018 using fragility curves for reinforcement bar of columns in the seven-storey RC frame

curves is shown in Fig. 12. The fragility curves are close to each other – unlike the beams – while damage probability increases in line with the damage limit. Fig. 12 shows fragility curves for GV and GC in which the damage limits of TSC-2007 almost overlap with the curves for KH and GÖ with the TBEC-2018 damage limits. However, the fragility curve of the SH damage limit for TBEC-2018 is on the MN damage limit of TSC-2007.

Figs. 13 and 14 show the comparison of performance levels for confined concrete of beams in the seven-storey RC frame in accordance with the two seismic codes. The probability occurrence of damage increases in line with the performance level, in parallel with the five-storey frame. Fig. 15 demonstrates a comparison of performance levels of the fragility curves for TSC-2007 and TBEC-2018. According to this comparison, the probability of damage for TBEC-2018 is higher than for TSC-2007, which is similar to the situation in the five-storey building.

Figs. 16 and 17 indicate the fragility curves for confined concrete of columns in the seven-storey building, considering the codes. Fig. 18 shows a comparison of fragility curves for the seismic codes. On inspection, this figure shows that fragility curves for the KH damage limit of TBEC-2018 and the GV damage limit of TSC-2007 are very nearly along the same line. However, the damage probability fragility curve for SH and GÖ damage limit for TBEC-2018 is higher than the MN and GC damage limits of TSC-2007.

5.2 Fragility curves for reinforcement bar

The damage probability of reinforcement bar is less than that of confined concrete because the unit deformation of reinforcement bar is higher than that of confined concrete.

Hence, only a comparison of TSC-2007 and TBEC-2018 are presented. Figs. 19 and 20 illustrate a comparison of the damage limits of reinforcement bar of beams in the five-storey and seven-storey RC buildings, considering the two seismic codes, respectively. According to the produced fragility curves, the damage probability is more for TBEC-2018 than for TSC-2007. In addition, the fragility curve of reinforcement bar for GV and GC, according to TSC-2007, is separate from the fragility curve of KH and GÖ for TBEC-2018. This situation arises because the unit deformation values given for TSC-2007 are more than those given for TBEC-2018.

Comparison of the fragility curves is shown in Fig. 20 for reinforcement bar of beams in the seven-storey RC frame. The fragility curves are close to each other, similar to the fragility curves of confined concrete in columns. The damage limits curves for GV and GC are close to the curves for KH and GÖ. However, the damage probability for the SH performance level is more than that of the MN performance level.

Comparisons of the fragility curves are shown in Figs. 21–22 for reinforcement bar of columns in the five-storey and seven-storey RC frames, respectively. Damage did not occur in the reinforcement bar of columns for GV and GC of TSC-2007 and KH and GÖ of TBEC-2018 due to the columns of the selected buildings being stronger than the beams. Hence, adequate data was not attained, and fragility curves were developed only for MN and SH damage limits. In accordance with the results, the damage probability for TBEC-2018 is more than that for TSC-2007 for the five-storey building. In terms of the seven-storey building, the fragility curves are close to each other because the column dimensions of the seven-storey building are stronger than those of the five-storey building.

6. Conclusions

In this paper, a seismic vulnerability assessment of reinforced concrete buildings for the 2007 Turkish Seismic Code (TSC-2007) and 2018 Turkish Building Earthquake Code (TBEC-2018) was carried out. To realize this evaluation, typical five- and seven-storey five-bay reinforced concrete frame buildings were selected and analysed using incremental dynamic analysis (IDA). For dynamic analysis, eleven seismic acceleration records of seismic events which occurred in Turkey were multiplied by various scaling factors from 0.2g to 0.8g. To predict nonlinear behaviour, a distributed plastic hinge model was used. Confined concrete and reinforcement bar damages for beams and columns were procured by considering the unit deformation demands of TSC-2007 and TBEC-2018. A vulnerability evaluation of the selected buildings was performed using fragility curves based on confined concrete and reinforcement bar damages obtained from incremental dynamic analyses. The procured results are summarized below:

- Fragility curves show a good estimation for confined concrete of beams for both seismic codes for the investigated buildings. According to the results, the probability of the occurrence of damage for confined concrete for TBEC-2018 is more than for TSC-2007.
- In terms of columns, fragility curves are close for controlled damage limit (KH) and pre-collapse damage limit (GÖ) of TBEC-2018 while a similar situation is true for safety limit (GV) and collapse limit (GC) for TSC-2007. However, the damage probability for TBEC-2018 is higher than for TSC-2007.
- According to the fragility curves, damage risk for TBEC-2018 is higher than for TSC-2007 for reinforcement bar of beams. This was seen for all damage limits.
- Because of selected columns are stronger than beam, reinforcement bar damage did not occur in columns for controlled damage (KH) and pre-collapse damage (GÖ) for TBEC-2018. In this situation, damage was seen for safety limit (GV) and collapse limit (GC) in TSC-2007. Hence, fragility curves could not account for this performance level. However, the probability of occurrence of limited damage (SH) is more than minimum damage (MN).

Consequently, this paper demonstrates, using fragility curves, that the possibility of damage in TBEC-2018 is higher than in TSC-2007. This situation shows that TBEC-2018 provides more safety than TSC-2007. It is suggested that in order to achieve more general inferences, more case studies should be implemented in future studies.

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