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On performance-based seismic assessment method for medium-rise RC buildings

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Research Paper

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Performance-based seismic assessment method for medium-rise RC buildings

Severe earthquakes registered in Turkey over the last five decades have shown that most of the existing buildings exhibit low resistance to earthquake action. In this study, a simplified version of the performance based rapid seismic assessment method (PERA) is proposed for the analysis of medium-size reinforced-concrete buildings. The influence of a critical storey is also considered when evaluating performance of the entire building. Good agreement is obtained between predictions by the simplified method, the PERA method, and the code based structural performance assessment procedures.

Key words:

earthquake, performance evaluation, RC, Smith method

Prethodno priopćenje

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Metoda za procjenu seizmičkog ponašanja srednje visokih AB građevina

Jaki potresi zabilježeni u Turskoj tijekom posljednjih pet desetljeća pokazuju da je većina postojećih zgrada nedovoljno otporna na njihovo djelovanje. U ovom se radu predlaže pojednostavljena inačica brze metode za procjenu seizmičkog ponašanja građevine (PERA) za analizu srednje visokih armiranobetonskih zgrada. Prilikom procjene ponašanja čitave građevine u obzir se uzima i utjecaj kritičnog kata. Postignuta je dobra podudarnost između predviđanja dobivenih pojednostavljenom metodom, metodom PERA i postupcima za procjenu ponašanja konstrukcija koji se temelje na odgovarajućim propisima.

Ključne riječi:

potres, procjena ponašanja, AB, Smithova metoda

Vorherige Mitteilung

Halil Görgün, Derman Kaya, Mehmet Emin Öncü, Senem Yılmaz Çetin

Eine Methode zur Bewertung des Erdbebenverhaltens von mittelhohen Stahlbeton-Gebäuden

Die starken Erdbeben, die in den letzten fünf Jahrzehnten in der Türkei verzeichnet wurden, zeigen, dass die meisten vorhandenen Gebäude ihren Einflüssen nicht ausreichend standhalten. In dieser Arbeit wird eine vereinfachte Version der Methode für die Bewertung des Erdbebenverhaltens von Gebäuden (PERA) zur Analyse von mittelhohen Stahlbeton-Gebäuden vorgeschlagen. Bei der Bewertung des Verhaltens eines gesamten Gebäudes werden auch die Auswirkungen des kritischen Stockwerks berücksichtigt. Es wurde eine gute Übereinstimmung zwischen den Vorhersagen aufgrund der vereinfachten Methode, der PERA-Methode und der Verfahren zur Bewertung des Verhaltens von Bauwerken auf der Grundlage der einschlägigen Vorschriften erzielt.

Schlüsselwörter:

Erdbeben, Bewertung des Verhaltens, Stahlbeton, Smith-Methode

1. Introduction

Earthquakes registered over the last 50 years, such as Lice/Diyarbakır-Turkey (1975) of 6.6 magnitude, Spitak-Armenia (1988) of 7.0 magnitude, Marmara-Turkey (1999) of 7.4 magnitude, Gujarat-India (2001) of 7.9 magnitude, Sumatra-Indonesia (2004) of 9.1 magnitude, Kashmir-Pakistan (2005) of 7.6 magnitude, Sichuan-China (2008) of 7.9 magnitude, Haiti (2010) of 7.0 magnitude, Tohoku-Japan (2011) of magnitude 9.0, Erciş/Van-Turkey (2011) of 7.2 magnitude, Pakistan (2013) of 7.8 magnitude, and Iran-Iraq (2017) of 7.3 magnitude, clearly show that many buildings situated in seismic regions all around the world, particularly in less developed or underdeveloped countries, are vulnerable to earthquake action. For this reason, the safety of these buildings should be evaluated, taking into account the fact that Turkey is situated on very active fault lines, such as the North Anatolian Fault, the East Anatolian Fault, and that densely populated regions lie along these fault lines. According to the final official assessment by Turkish government, 66 % of national territory is located in the first and second earthquake zones, and about 71 % of the population resides in these regions. Turkey’s Active Fault Line Map was updated 21 years after the previous map (Figure 1). While there are 326 active faults in Turkey, 485 segments can produce magnitudes of 5.5 and higher.

The earthquake resistance of our existing buildings can not be determined quickly according to the principles of Turkish Seismic Code (TSC 2007 [1]) and Regulation on the Determination of Risky Structures (RDRS) [2]. For this reason, with the introduction of the urban regeneration process in Turkey, it has become necessary to use simpler, faster but still reliable methods. The process and procedures for assessing buildings at risk by RDRS method [2] are given in Figure 2. Many methods related to seismic safety assessment are presented in relevant literature.

The first stage methods are faster, and the second stage methods are slower but more reliable. The Federal Emergency Management Agency (FEMA P-154 and FEMA P-155) [3-7] and Sucuoğlu et al. [8] methods are simple evaluation methods based on visual screening.

The aim is to estimate the number of buildings at risk, and their distribution within the city, using a rapid visual screening (RVS) procedure, ranking them in terms of seismic risk from the outside without entering the buildings, using simple observable criteria. The obtained data are processed using the “quick visual evaluation work sheet”. This form is evaluated and earthquake performance is obtained. Such a study and the “street scanning method” were conducted based on a different perspective in [8]. In the risk assessment method, these authors have developed an appropriate risk sequence, using the building parameters that can be observed from the street, i.e. the number of building floors, soft floors, heavy projections, and visible building quality.

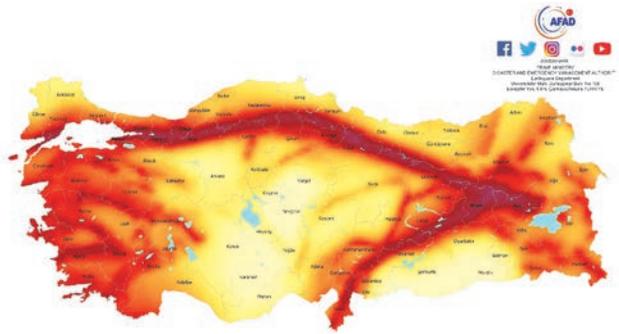


Figure 1. Earthquake hazard map of Turkey [9]

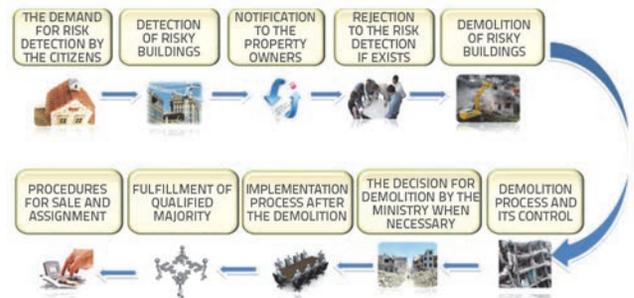


Figure 2. Process and procedures for buildings at risk (Ministry of Environment and Urbanisation of Turkey) [10]

The Japanese Seismic Index Method [11], Hassan and Sozen method [12], Yakut [13], P25 method [14], and the New Zealand Society for Earthquake Engineering method [15], can be used for the analysis in cases when the building density is too high, and when the time and material resources are limited. There are many studies in the literature aiming to adapt the Japanese Seismic Index Method to Turkish buildings [16-20]. Favvata et al. [21], Ni [22] and Ozmen and Inel [23] assessed the inelastic effect of rapid screening parameters on seismic performance of RC buildings. Priestley [24], Chandler and Mendis [25], Lervolino et al. [26], Ruiz-Garcia and Miranda [27] and Jeong et al. [28] proposed seismic evaluation methods taking into account probabilistic approaches. Other researchers compared various assessment methods (Lupo et al. [29] and Kalkan and Kunnath [30]) Also, methods for analysing seismic performance of existing buildings are given in appropriate codes and guidelines (e.g., TSC 2007; NZSEE 2012 Eurocode 8 Part 3; ASCE 41) [1, 15, 31, 32]. The details of these approaches are given in Ilki et al. [33].

In addition to detailed structural analyses that can take considerable time and computing resources, on-site inspection studies are also required when applying these methods for large stocks of vulnerable buildings in developing countries. Ilki et al. [33] proposed a simpler method that minimizes the scope of field investigations. This method involves a simpler approach and is more reliable compared to other methods proposed in the literature. The first vibration mode of earthquake effect is proposed for dominant reinforced concrete frame structures. This method is based on the member tributary area concept and

includes various simplifications and assumptions related to the structural analysis and performance-based evaluation. It makes use of the Muto principles [34] and performance criteria given in TSC 2007 [1].

Some assumptions about the type of elements, diameter and spacing of longitudinal and transverse reinforcing bars, the shear and axial-flexural capacities of columns, concrete quality, geometric ratio, and locations of columns, are considered. Since the seismic safety assessment is based on TSC 2007 [1], potential problems such as non-compliance with the code of conduct, likely to occur if other quick evaluation methods are used, are reduced to minimum [33].

2. PERA procedure

The data collected from recent earthquake clearly show that most of the structures situated in earthquake zones are vulnerable to seismic load [33]. In order to avoid catastrophic consequences of earthquakes, it is necessary to sort the buildings susceptible to seismic action and take the necessary precautions. In this respect, a performance-based quick and low-cost evaluation procedure should be presented and compared to conventional code-based seismic evaluation procedures. Most of these assumptions are mentioned in this method for many RC frame structures in Turkey. However, modifications might be in order for use in other countries. Nevertheless, the methodology of Ilki et al. [33] is useful in all seismic regions where reinforced concrete buildings are widespread.

The seismic performance of RC frame buildings was evaluated according to TSC 2007 performance-based assessment principles in the method of Ilki et al. [33]. TSC 2007 [1] consider irregularities of the building and detailed structural characteristics, together with local soil class and the earthquake zone in which the building is located. The demand/capacity ratios of structural elements should be obtained to determine the building performance. The elastic internal force demands and capacities are first determined, and the demand/capacity ratios are then calculated for each building element. This is followed by determination of member damage levels depending on the demand/capacity ratios and inter-storey drifts. The expected failure modes of structural elements, confinement properties, and levels of axial and shear forces, are taken into account when determining the damage levels. And, finally, the level of seismic performance of the building is determined.

According to its approach, the PERA (Performance Based Rapid Seismic Assessment) method [33] is similar to the method proposed in TSC 2007 [1]. In this method, the duration of site inspections is significantly reduced, and the stages of analysis and evaluation are simplified. This method estimates damage levels of the columns of the ground storey and its inter-storey. The flexural strength and moment capacity of the beams are estimated based on reinforcement rates and beam measurements and observations at typical building structures. In addition, the method assumes the reinforcement configuration of the columns.

Structural irregularities shown in Table 1 as per TSC 2007 [1] are considered through penalty coefficients in the PERA method [33]. The amount of data required is lower compared to the data needed in the above-mentioned rapid and pre-evaluation methods. Thanks to the determination of concrete quality with a limited number of tests, appropriate stirrup spacing and type of reinforcing bars, and use of different modes of comparison, the algorithm used is more realistic compared to classical method.

Table 1. Penalty coefficients for structural irregularities

Irregularity	Penalty coefficient
A-Irregularities in plan	
A1 - Torsional irregularity	0.85
A2 - Floor discontinuities	0.95
A3 - Projections in plan	0.95
B-Irregularities in elevation	
B1 - Inter-storey strength irregularity (weak storey)	0.95
B2 - Inter-storey stiffness irregularity (soft storey)	0.85
B3 - Discontinuity of vertical structural elements	0.95

The PERA method [33] assumes that the ground storey of the building is the critical storey, as defined in the RDRS [2] with regard to seismic loads. The base shear force (V_b) is given in Eq. (1), where A_0 (Table 2) is the effective ground acceleration, W is the building weight ($G + 0.3Q$), G and Q are the dead and live loads, respectively. The spectrum coefficient $S(T)$ is defined in TSC 2007 (Figure 3). The T_A and T_B are corner periods (Table 3). The natural vibration period of the building is estimated by Eq. (2). A coefficient of 0.85 is used in Eq. (1) to take into account the effects of high vibration modes used in the TSC 2007 approach. The ' n ' used in Eq. (2) is the number of stories contributing to the first vibration mode on the investigation side. Detailed calculations should be done to calculate the natural period of construction in a more complex way.

$$V_b = 0,85 \cdot W \cdot S(T) \cdot A_0 \quad (1)$$

$$T = 0,2n \quad (2)$$

Table 2. Effective ground acceleration coefficient (A_0) (TSC 2007) [1]

Earthquake zone*	A_0
1	0.40
2	0.30
3	0.20
4	0.10

*1: High, 2: Moderate, 3: Slight, 4: Low

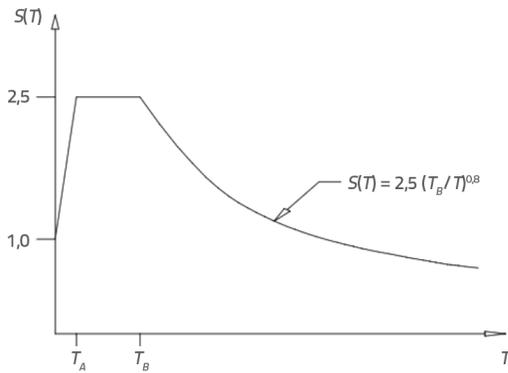


Figure 3. Design response spectrum proposed in TSC 2007 [1]

Table 3. Spectrum characteristic periods (T_A, T_B) (TSC 2007) [1]

Local soil class	T _A [s]	T _B [s]
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

V_{ij} is the column shear force (Eq. 3) where h_i and I_i are heights and the moment of inertia of the ground storey columns, respectively. The γ coefficient is a summation of values and is calculated by Eq. (4) where is the inflection point coefficient.

$$V_{ij} = V_b \left(\frac{I_i}{h_i^3} \right) \quad (3)$$

$$V = V_0 + V_1 + V_2 + V_3 \quad (4)$$

In the PERA method [33], it is assumed that, while coefficients are ignored. γ₀ values are given in Muto (Table 4) [34] for sway loading for wind and seismic loading separately. The stiffness ratio (k̄) is given in Eq. (5). k₁ and k₂ are beam stiffness values, and k_{ci} is the column stiffness. As to the effect of cracking on stiffness, the beam section is assumed to be a rectangular section. Most of these assumptions are made to enable quick assessment of the building, and to speed up and simplify site investigations [33].

$$\bar{k} = \frac{k_2}{k_{ci}} \quad (\text{for exterior columns}) \quad (5)$$

$$\bar{k} = \frac{k_1 + k_2}{k_{ci}} \quad (\text{for interior columns})$$

In the PERA method [33], the first and last (exterior) axes are marked as x₁, x₂ in the x-direction and y₁, y₂ and in the y-direction of the structure. The corner columns are marked as x₁y₁, x₂y₁, etc. (Figure 4).

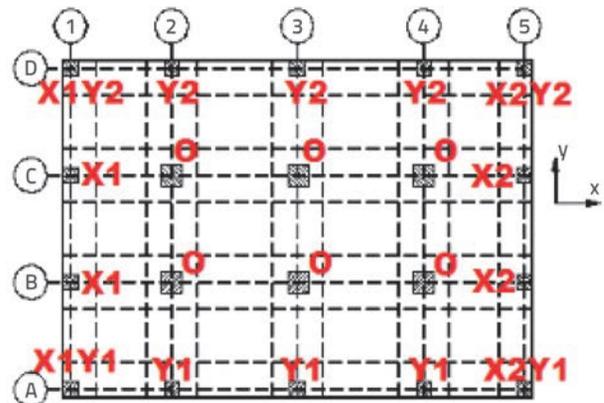


Figure 4. Column marks (corner, edge, interior)

Table 4. Inflection point coefficient (γ₀) depending on stiffness ratio (k̄) of the beams to columns

Number of storey	k̄	γ ₀													
		0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00	2.00	3.00	4.00	5.00
1		0.80	0.75	0.70	0.65	0.65	0.60	0.60	0.60	0.60	0.55	0.55	0.55	0.55	0.55
2		0.95	0.80	0.75	0.70	0.65	0.65	0.65	0.60	0.60	0.60	0.55	0.55	0.55	0.50
3		1.00	0.85	0.80	0.75	0.70	0.70	0.65	0.65	0.65	0.60	0.55	0.55	0.55	0.55
4		1.10	0.90	0.80	0.75	0.70	0.70	0.65	0.65	0.65	0.60	0.55	0.55	0.55	0.55
5		1.20	0.95	0.80	0.75	0.75	0.70	0.70	0.65	0.65	0.65	0.55	0.55	0.55	0.55
6		1.20	0.95	0.85	0.80	0.75	0.70	0.70	0.65	0.65	0.65	0.55	0.55	0.55	0.55
7		1.20	0.95	0.85	0.80	0.75	0.70	0.70	0.65	0.65	0.65	0.55	0.55	0.55	0.55
8		1.20	1.00	0.85	0.80	0.75	0.70	0.70	0.65	0.65	0.65	0.55	0.55	0.55	0.55
9		1.20	1.00	0.85	0.80	0.75	0.70	0.70	0.65	0.65	0.65	0.55	0.55	0.55	0.55

The column shear force demand (V_{ei}) is calculated using Eq. (6). M_{rb} , M_{rt} are column moment capacities at the bottom and top parts of the cross-section, respectively. M_k is the moment demand of a beam. M_{ri} and M_{rj} are moment capacities at the beam ends, Eqs. (7), (8) i (9).

$$V_{ei} = \frac{M_{rbi} + M_{rti}}{L_{ni}} \tag{6}$$

$$M_{vi} = \min(M_{rti}, M_k) \tag{7}$$

$$M_k = M_{ri}/2 \quad (\text{for exterior columns}) \tag{8}$$

$$M_k = (M_{rj} + M_{ri})/2 \quad (\text{for interior columns}) \tag{9}$$

In Eqs (10) and (11), r_{1i} and r_{2i} are demand-capacity ratios of the columns. Damage levels are determined using Table 5. In this table, MN, SL and FL denote the minimum damage, safety and failure limit, respectively. The damage boundary is determined according to the relative storey drift ratio (Table 6).

$$r_{1i} = \frac{V_{ei}}{V_{ri}} \tag{10}$$

$$r_{2i} = \frac{M_{ei}}{M_{ri}} \tag{11}$$

Table 5. Effect/capacity ratios (r_b) define boundary of damage for reinforced concrete columns (TSC 2007) [1]

Ductile columns			Damage boundary		
$\frac{N_k}{A_c f_{cm}}$	Confinement	$\frac{V_e}{bdf_{cm}}$	MN	SL	FL
≤ 0.1	Available	≤ 0.65	3	6	8
≤ 0.1	Available	≥ 1.30	2.5	5	6
$\geq 0.4 \text{ i } \leq 0.7$	Available	≤ 0.65	2	4	6
$\geq 0.4 \text{ i } \leq 0.7$	Available	≥ 1.30	1.5	2.5	3.5
≤ 0.1	NA	≤ 0.65	3.5	5	5
≤ 0.1	NA	≥ 1.30	2.5	3.5	3.5
$\geq 0.4 \text{ and } \leq 0.7$	NA	≤ 0.65	2	3	3
$\geq 0.4 \text{ and } \leq 0.7$	NA	≥ 1.30	1.5	2	2
≥ 0.7	-	-	1	1	1

Table 6. Relative storey drift boundaries (TSC 2007) [1]

Relative storey drift ratio	Damage boundary		
	MN	SL	FL
	0.01	0.03	0.04

3. Proposed method

PERA [33] has been modified in the scope of this study. Thus, the present study is an alternative to the PERA method [33] for reinforced concrete frame buildings.. The proposed

methodology makes use of the approaches presented in the very well-known Smith method [35]. The contra-flexure point occurs in the middle of all members of the frame in Smith method [35], as shown schematically in Figure 5.

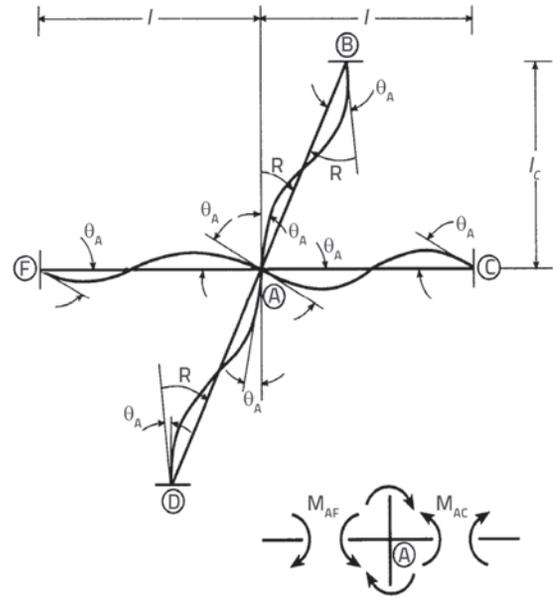


Figure 5. Sub-assembly model for unbraced (sway) frame as per Smith method [33]

An approach for estimating inflection point of beams and columns is presented in the Smith method [35]. The basic assumptions made in the development of the Smith method [35] are listed as follows (with reference to figures 5 and 6):

- a) Joints (A) and (C) are in a straight line (axial deformation of columns is negligible).
- b) Joints (A) and (D) are in a straight line (floors are infinitely rigid in their own planes).
- c) Rotations at joints (F) and (C) are equal in magnitude.
- d) Rotations at the ends of the columns are equal in magnitude.
- e) Point of contra-flexure occurs at the middle of all members of the frame, as shown in figures 5 and 6.

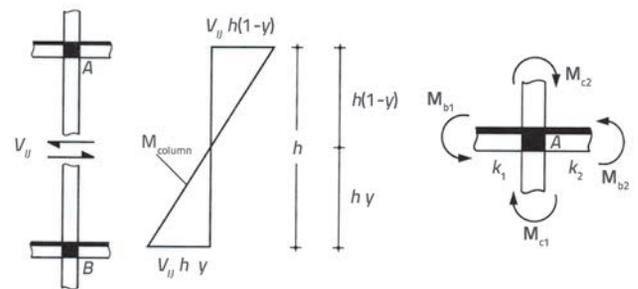


Figure 6. Variation of earthquake moments in beams and columns

In the Smith method [35], the inflection point coefficient is taken as $y = y_0 + y_1 + y_2 + y_3 = 0.50$, giving the location of

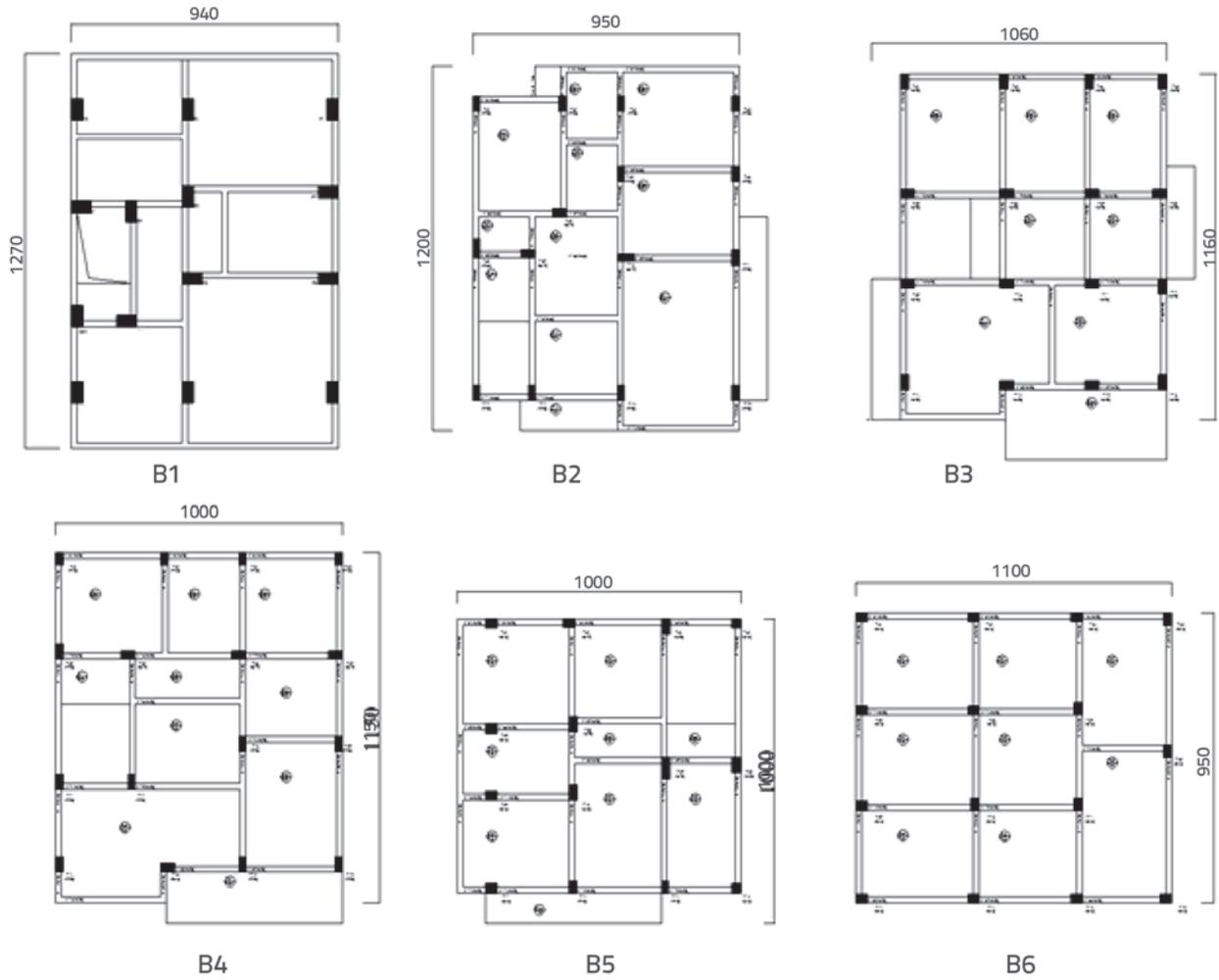


Figure 7. Typical floor plans of existing buildings (adopted from Ozcelik 2014) [36] (dimensions are in cm)

Table 7. General characteristics of buildings

Building	Number of stories	Average dimensions in plan [m]	Year of construction	Reinforcement type	Stirrup: diameter/spacing [mm]	Confinement zone
B1	4	9.4 x 12.7	1990.	S220	8/200	Yes
B2	4	9.5 x 12.0	1990.	S220	8/200	No
B3	4	10.6 x 11.6	1990.	S220	8/200	No
B4	3	10.0 x 11.5	1990.	S220	8/200	No
B5	3	10.0 x 10.0	1990.	S220	8/200	Yes
B6	3	11.0 x 9.5	1990.	S220	8/200	No
B7	3	8.6 x 10.2	1974.	S220	8/150	No
B8	3	9.5 x 9.0	1974.	S220	8/150	Yes
B9	3	9.0 x 10.5	1974.	S220	8/150	No
B10	3	9.1 x 9.75	1979.	S220	8/150	No
B11	3	9.9 x 8.8	1979.	S220	8/150	No
B12	3	9.8 x 10.25	1979.	S220	8/150	No
B13	3	13.25 x 10.15	2001.	S420	8/300	No
B14	3	14.5 x 9.5	2003.	S420	8/120	Yes
B15	3	9.4 x 14.7	2005.	S420	8/120	Yes

the point of zero moment at the mid height of the column from the base of the column, as shown in figures 5 and 6. This assumption is very important to prevent the possibility of using a wrong value by practicing engineers, and to save time for calculating individual values $\gamma_0 + \gamma_1 + \gamma_2 + \gamma_3$.

4. Results and discussion

The results of the analysis of proposed methods are compared with the TSC 2007 [1] method, RDRS [2] method, and PERA method [33]. The results are obtained from 720 analyses made for 15 existing buildings (numbered B1 to B15) located in Başiskele/Kocaeli, Turkey. B1-B6 buildings were investigated by Ozcelik 2014 [36] and B7-B15 buildings were investigated by Vulas 2014 [37]. Typical floor plans of buildings B1-B6 are shown in Figure 7, and main characteristics of buildings B1-B15 are presented in Table 7. As shown in this table, all buildings are located in Earthquake Zone 1. The Earthquake Zone 1 is defined as a high seismic-risk zone. Number of stories of the buildings varies between three and four. Most of the building were built between 1975 and 2000 in accordance with TSC 1975 [38]. The grade S220 steel was used for reinforcing bars of older buildings, constructed before 2000, while the steel grade S420 was used for the remaining buildings. The buildings were subjected to horizontal load along two principal in-plane axes. The compressive strength values of concrete amounting to 10, 14, and 20 MPa, were assumed. Z2 and Z3 local soil classes were separately considered in the analysis (Table 3).

Table 8. Summation of predictions according to TSC 2007 [1] and RDRS (2013) [2] methods

TSC 2007 [1] \ RDRS (2013)	TSC 2007 [1]		
	Safe	Unsafe	Total
Safe	210	66	276
Unsafe	33	411	444
Total	243	477	720

Table 9. Summation of predictions according to TSC 2007 [1] and PERA [33] methods

TSC 2007 [1] \ PERA (2014)	TSC 2007 [1]		
	Safe	Unsafe	Total
Safe	151	96	247
Unsafe	92	381	473
Total	243	477	720

Table 10. Summation of predictions according to TSC 2007 [1] and proposed method

Proposed method \ TSC 2007 [1]	TSC 2007 [1]		
	Safe	Unsafe	Total
Safe	192	145	337
Unsafe	51	332	383
Total	243	477	720

Table 11. Summation of predictions according to proposed method and other methods

Method	Number of buildings		
	Safe	Unsafe	Total
TSC 2007 [1]	243 (34 %)	477 (66 %)	720 (100 %)
RDRS	276 (38 %)	444 (62 %)	720 (100 %)
PERA	247 (34 %)	473 (66 %)	720 (100 %)
Proposed	337 (47 %)	383 (53 %)	720 (100 %)

Table 12. Summation of predictions according to proposed method and other methods (low and high risk cases are considered separately)

Method	Number of buildings		
	Safe	Unsafe	Total
TSC 2007	243 (100 %)	477 (100 %)	720 (100 %)
RDRS	210 (86 %)	411 (86 %)	621 (86 %)
PERA	151 (62 %)	381 (80 %)	532 (74 %)
Proposed	192 (79 %)	332 (70 %)	524 (73 %)

The results were obtained based on 720 different cases [15 (number of buildings) x 3 (C10, C14, C20) x 2 (Z2, Z3) x 2 (confinement, no confinement) x 4 (+X, -X, +Y, -Y directions)] for 15 buildings. All 720 cases were analysed according to TSC 2007 [1] and the RDRS [2] procedures using the Sta4CAD structural analysis software [39], by Ozcelik [36] for B1-B6 buildings, and by Vulas [37] for B7-B15 buildings. Also, 720 analyses were performed by Ilki et al. [33] using the PERA method. The analysis steps of the PERA [33] procedure were used in this present study by replacing the Muto method [34] with the Smith method [35].

The comparison of the results for the 720 different cases obtained using the TSC 2007 [1], the RDRS [2] approach, the PERA method [33], and the proposed method, are given in Tables 8-12. 720 cases have been compared in detail for the four methods.

The results of the analyses using the RDRS [2] are given in Table 8 and are compared with the rigorous analysis by TSC 2007 [1]. 243 out of 720 cases (34 %) unbend to pre-collapse performance level, whereas 477 cases (66 %) correspond to collapse performance level. In another words, according to TSC 2007 [1], 66 % of the examined cases are highly vulnerable to earthquakes.

According to the RDRS [2] method, 276 cases are safe and 444 cases are unsafe, which correspond to 38 % and 62 % of the total of 720 cases, respectively (Table 8 and Table 11). These results show a good agreement between RDRS [2] and TSC 2007 [1] (34 % and 66 %, respectively). Also, if low and high risk cases are considered separately, estimations made according to the RDRS [2] method are again in reasonably good agreement with TSC 2007 [1] predictions. Safe cases estimated by TSC 2007 [1] and RDRS [2] were 243 and 210, respectively (86 % achievement according to TSC 2007 [1]). Similarly, unsafe cases estimated by TSC 2007 [1] and RDRS [2] were 477 and 411, respectively (again 86 % achievement according to TSC 2007 [1]). The ratio of 86 % can be deemed an acceptable ratio of agreement. Also, out of 720 total safe and unsafe cases (life safety or collapse prevention and collapse) found by TSC 2007 [1], the RDRS method [2] simultaneously found 621 low and high risk cases (86 % success with respect to TSC 2007 [1]), as shown in Table 12.

The results of the analyses using the PERA method [33] are given in Table 9 and compared with the rigorous analysis by TSC 2007 [1]. If estimations given in TSC 2007 [1] are accepted as reference values, as given in Table 9, the results of the PERA method [33] show 247 safe and 473 unsafe cases corresponding to 34 % and 66 % of the 720 cases under study, respectively. These values give good, albeit conservative, agreement with TSC 2007 [1] (34 % and 66 %, respectively). In addition, if safe and unsafe cases are considered separately as before, the results of the PERA method [33] once again show good agreement with the TSC 2007 [1] method. Out of 243 safe cases (life safety or collapse prevention) predicted by TSC 2007 [1], 151 were also obtained as safe by the PERA method [33] (62 % success with respect to TSC 2007 [1]). Similarly, out of 477 unsafe (collapse) cases predicted by TSC 2007 [1], the PERA method [33] simultaneously predicted 381 cases as unsafe (80 % success with respect to TSC 2007 [1]). Thus the 62 % and 80 % match was obtained for safe and unsafe cases, respectively. Similarly, out of 720 total safe and unsafe (life safety or collapse prevention and collapse) cases predicted by TSC 2007 [1], the PERA method [33] simultaneously predicted 532 low risk and high risk cases (74 % success with respect to TSC 2007 [1]), as shown in Table 12.

A comparison of building performances for 720 cases between the rigorous TSC 2007 [1] approach and the proposed method is presented in Table 10. The predictions of the proposed method show 337 safe and 383 unsafe cases, which corresponds to 47 % and 53 % of the 720 analysis cases, respectively. These values point to an acceptable level of correspondence between the values of the proposed method and the TSC 2007 [1] (34 % and 66 %, respectively). Again, if safe and unsafe cases are considered separately, predictions of the proposed method are in reasonably good agreement with predictions of the TSC 2007 [1] method. Out of 243 safe cases (life safety or collapse prevention) predicted by the TSC 2007 [1], 192 were also identified as low risk by the proposed method (79 % success with respect to TSC 2007 [1]). Similarly, out of 477 unsafe (collapse) cases predicted by TSC 2007 [1], the proposed method predicted 332 cases as high-risk (70 % success rate with respect to TSC 2007 [1]). The 79 % and 70 % match was established for both safe and unsafe cases, respectively. The effect of concrete cube compressive strength, presence of adequate confinement, and soil conditions, as estimated by the TSC 2007 [1] approach, was also observed. The percentage of unsafe cases generally tends to decrease with an increase in concrete cube compressive strength, as expected. Both existence of adequate confinement and soil condition parameters, for a given concrete strength, seem to have a considerable effect on the classification of performance according to TSC 2007 [1]. Similarly, out of 720 total safe and unsafe (life safety or collapse prevention and collapse) cases predicted by TSC 2007 [1], the proposed method predicted 524 low risk and high risk cases (73 % success rate with respect to TSC 2007 [1]), as shown in Table 12.

Finally, the summary of predictions by TSC 2007 [1], RDRS [2] approach, PERA method [33] and the proposed method, is given in Tables 11-12. Predictions by RDRS [2], PERA method [33] and the proposed method, are again in reasonably good agreement with the predictions by the TSC 2007 [1] method. The 86 % match was obtained using the RDRS [2], the 62 % and 80 % match was obtained by using the PERA method [33], and the 79 % and 70 % match was obtained using the proposed method for safe and unsafe cases, respectively. This can be considered an acceptable ratio of agreement. Similar results were obtained in case of a rigorous structural analysis utilizing slightly different assumptions in TSC 2007 [1].

Besides an overall comparison between TSC 2007 [1] and the proposed method, the effect of significant variables, including concrete quality, confinement, and soil conditions, on the performance of the proposed method was evaluated by Ilki et al. [33]. It was reported that highly successful prediction rates were obtained relative to TSC 2007 [1] results.

The 86 %, 74 % and 73 % match obtained using the RDRS [2], the PERA method [33] and the proposed method, respectively for safe and unsafe cases, can be considered an acceptable rate of agreement. A good agreement was obtained between the predictions of the simplified method, the PERA method and code based structural performance assessment procedures. as shown in Table 12.

Figure 8 shows the effect of several parameters such as concrete compressive strength values (C10, C14, and C20), shear reinforcement (confinement, no confinement), and soil conditions (stiff, soft), on TSC 2007 [1] predictions. In this figure, the horizontal x-axis corresponds to concrete compressive strength, the vertical y-axis corresponds to percentages of unsafe cases in each parameter group. The percentage of unsafe cases generally decreases with an increase in concrete compressive strength.

In addition to an overall comparison between TSC 2007 [1] and the proposed method, the effect of the concrete compressive strength, presence of sufficient confinement, and soil conditions, on the performance of the proposed method is evaluated in Figure 9.

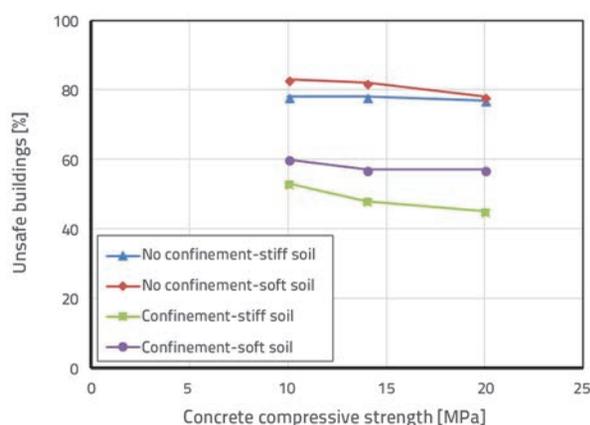


Figure 8. Effect of several parameters on TSC 2007 [1] predictions

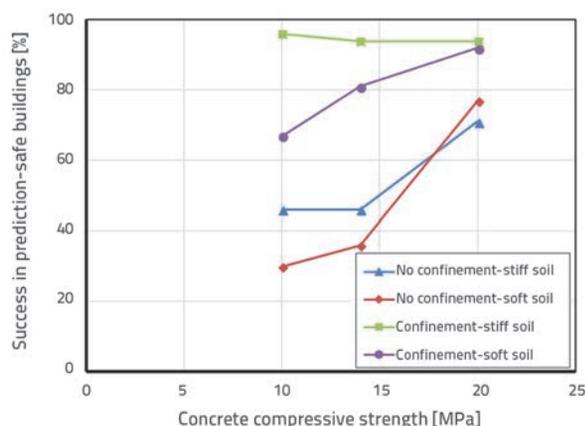


Figure 9. Comparison of proposed method with TSC 2007 [1]

In this figure, the confinement status (confinement, no confinement) and soil conditions (stiff, soft), are defined as data series corresponding to four parameter groups. The vertical axis

now corresponds to the percentage of the analysis results for the safe case according to TSC 2007 [1]. These results were predicted successfully by the proposed method. According to Figure 9, highly successful prediction rates relative to the TSC 2007 [1] results were obtained for two out of four parameter groups.

5. Conclusions

A simplified version of the PERA method for rapid evaluation of seismic safety of the existing medium-rise reinforced concrete frame structures is proposed in the paper. The performance based rapid seismic assessment method (PERA) is modified in this study. The analysis steps of the PERA procedure were used by replacing the Muto method with the Smith method. The integrity of the presented method is validated for common medium-rise reinforced concrete frame structures for which the first mode of vibration is dominant. The proposed method exhibits an acceptable level of agreement for both safe and unsafe cases. A good agreement is obtained between the proposed method, the PERA method and conventional detailed seismic safety assessment analyses carried out for 720 different cases representing typical medium-rise reinforced concrete frame buildings in Turkey. A comparison of building performance for 720 cases using the rigorous TSC 2007 [1] approach and the proposed method is also presented. The predictions of the proposed method result in 337 safe and 383 unsafe cases, or 47 % and 53 % of the 720 cases, respectively. These values point to an acceptable level of correspondence between the values of the proposed method and TSC 2007 [1] (34 % and 66 %, respectively). It should be noted that the pace of application of the proposed method is remarkably higher compared to conventional structural performance assessment methods. Not only the accuracy of the presented method is acceptable for structures with limited structural irregularities, but also the pace of implementation is considerably higher in comparison with current conventional structural performance assessment methods. Further study is required to evaluate and improve reliability of the proposed rapid seismic assessment method for structures with significant irregularity defined in conventional structural performance assessment methods, and structures with shear walls in two principal directions.

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