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Araştırma Makalesi / Research Paper

An Evaluation on Seismic Performance of Existing Reinforced Concrete Buildings in Turkey

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ABSTRACT

It is known fact that the earthquake occurred in two decades have been caused serious pecuniary loss and spiritual damages in Turkey. Massive earthquakes revealed that the existing building stocks in urban regions are significantly vulnerable to seismic hazard. It is very important that earthquake safety of existing buildings should be determined as soon as possible and that unsafe ones should be strengthened. Investigations on the damaged buildings show that material strengths are very important parameters on the building performance. This paper aimed to determine the seismic safety of the buildings in risky regions in terms of earthquake and to evaluate general situation. Therefore,120 existing RC residential buildings are chosen from different cities to predict the general trend on the seismicity of buildings. The available material qualities are taken into account in the analyzes of the buildings. Analyzes were carried out by increasing the material strengths and the effect of the material strength on the earthquake performance was examined. Structural Analyzes Program (SAP 2000) was used in modelling of buildings and to obtain seismic performance of residential buildings. Nonlinear elastic procedures given in Turkish Earthquake Code 2007 (TEC) were applied in the analyses of structural systems.

Keywords: Existing RC buildings, seismic performance, nonlinear elastic method

Türkiye'deki Mevcut Betonarme Binaların Deprem Performansları Üzerine Bir Değerlendirme

ÖZ

Türkiye' de son yirmi yıl içerisinde meydana gelen depremlerin neticesinde büyük maddi kayıplar ve manevi zararlar meydana geldiği açık bir durumdur. Büyük depremler neticesinde kentsel bölgelerdeki mevcut binaların deprem riskine karşı oldukça dayanıksız olduğu ortaya çıkmıştır. Dolayısıyla, mevcut betonarme binaların deprem güvenliğinin önceden belirlenmesi ve depreme karşı dayanıksız yapıların güçlendirilmesi hayati önem taşımaktadır. Depremler sonucunda, binaların deprem performanslarının zayıf olmasının en önemli etkenlerinden biri kullanılan beton ve donatı çeliği dayanımlarının zayıf olmasından kaynaklanmaktadır. Bu çalışmada, Türkiye' de deprem açısından riskli bölgelerdeki binaların deprem güvenliği belirlenerek, genel durum değerlendirilmesi amaçlanmıştır. Bundan dolayı, farklı şehirlerden toplam 120 adet mevcut betonarme binalar seçilerek deprem performansları belirlenmiş ve binaların depremselliği hakkında genel bir durum değerlendirilmesi yapılmıştır. Binaların analizlerinde mevcut malzeme kaliteleri dikkate alınmıştır. Aynı zamanda malzeme dayanımları artırılarak analizler gerçekleştirilmiş ve malzeme dayanımının deprem performansı üzerindeki etkisi incelenmiştir. Binaların modellenmesinde ve performanslarının belirlenmesinde SAP2000 (Structural Analyzes Program) bilgisayar programı kullanılmıştır. Binaların analizlerinde, Deprem Bölgelerinde Yapılacak Binalar Hakkında Yönetmelik (DBYBHY)' te belirtilen, mevcut binaların deprem performansının belirlenmesinde önerilen elastik olmayan hesap yöntem esasları dikkate alınmıştır.

Anahtar Kelimeler: Mevcut betonarme binalar, deprem performansı, nonliner yöntem

INTRODUCTION

In the past two decades Turkey has experienced several moderate to large earthquakes. The literature includes many studies related to the earthquakes in Turkey, examining issues such as observed structural damage, sources of damage, performance of structures, structural deficiencies etc. (Sezen et al., 2003; Doğangün, 2004; Özcebe et al., 2004, Yakut et al., 2005; İnel et al., 2008, 2016; Tekeli et al., 2013, 2014, 2017; Yön et al., 2015; Dilmaç, 2017).

After experiences of serious hazards to buildings in consequence of several destructive earthquakes, particularly, those which hit highly dense urban areas, the state of seismic performance of existing building is anxious in terms of life and property safety year after year. The recent earthquakes, i.e., the 1992 Erzincan Earthquake, the 1999 Düzce and Gölcük Earthquake, the 2002 Çay Eartquake, the 2003 Bingöl and Pülümür Earthquake, the 2011 Van Earthquake have strongly pushed our society to recognize the importance of earthquake countermeasures for existing vulnerable buildings. In most countries, one of the major causes of seismic vulnerability related with these buildings is that a large number of the existing RC buildings have been designed by architects and engineers without formal training in the seismic design and construction have been built by inadequately skilled construction workers and inadequate material strength. Besides, it can be considered that the vast majority of the existing buildings have completed their economic life as of the construction year. Particularly, these problems are obviously increased when the concrete quality, the reinforcement strength and the faulty conditions in the choice of the bearing system are taken into consideration

In earthquake prone regions, there are a large number of residential buildings which have very low level of seismic safety. Since most of them have been constructed without receiving any structural engineering attention, there is a need for a general assessment that focuses on selection of buildings which do not have the life safety level. It is very important that the buildings have pre-determined earthquake safety and strengthened buildings with weak earthquake safety. The basis of the calculations that must be observed in order to determine the earthquake safety of RC buildings is given in the Turkish Earthquake Code (TEC). According to principles of nonlinear method of TEC, seismic performances of existing RC buildings should be determined and they should be made strengthen in necessary conditions.

In this study, seismic performance of existing 120 buildings that located in Isparta, Afyon, Burdur, İstanbul and İzmir which are the cities with high risk of earthquake, were determined according to the principles of inelastic method specified in the TEC. Each building was analyzed by taking into consideration the existing material quality and carried out by increasing the material strengths and the effect of the material strength. Structural Analyzes Program (SAP 2000) was used to obtain seismic performance of residential buildings.

MATERIAL AND METHODS

Nonlinear Elastic Procedure

Analyzes of existing buildings was to collect the information of structure, firstly. Obtained all features of existing buildings is classified with respect to the scope of data and the type of building system in TEC. These levels are "limited", "moderate" and "comprehensive". Generally, structural members can be classified as "ductile" and "brittle" with respect to their mode of failure in determining the damage limits. In TEC, three damage limits and damage states are defined as minimum limit (MN), safety limit (SF) and collapse limit (CL) at the cross section level for ductile members (Figure 1). In procedure, disclosed as first damage state MN that defines the onset of significant inelastic behavior at a critical cross section. Brittle members are not permitted to exceed this limit. A member damage state is determined by its critical cross section with the most severe damage state.

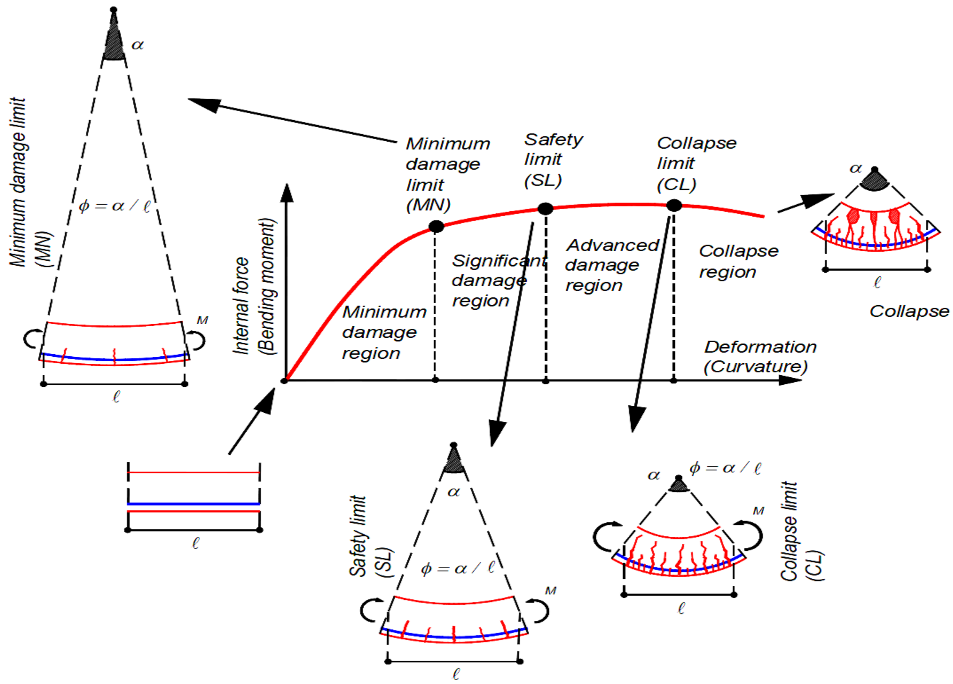

Figure 1. Damage state according to level of damage limit in a ductile member (Celep, 2014)

Nonlinear flexural behavior in frame members are confined to plastic hinges, where the plastic hinge length L_p is assumed as half of the section depth (L_p= h/2). Pre-yield linear behavior of concrete sections is represented by cracked sections, which is 0.40EI_o for beams and varies between (0.40-0.80) EI_o with the axial stress for columns. Strain hardening in the plastic range may be ignored, provided that the plastic deformation vector remains normal to the yield surface. Diagonal braces that represent reinforced masonry infill walls are modelled as elasto-plastic axial tension-compression members.

The objective is to carry out nonlinear static analysis under incrementally increasing lateral forces distributed in accordance with the dominant mode shape in the earthquake excitation direction. Lateral forces are increased until the earthquake displacement demand is reached. Internal member forces and plastic deformations are calculated at the demand level. A capacity diagram is obtained from the incremental analysis which is expressed in the "base shear force - roof displacement" plane. Then the coordinates of this plane is transformed into "modal response acceleration versus modal response displacement" as shown in Figure 2 below.

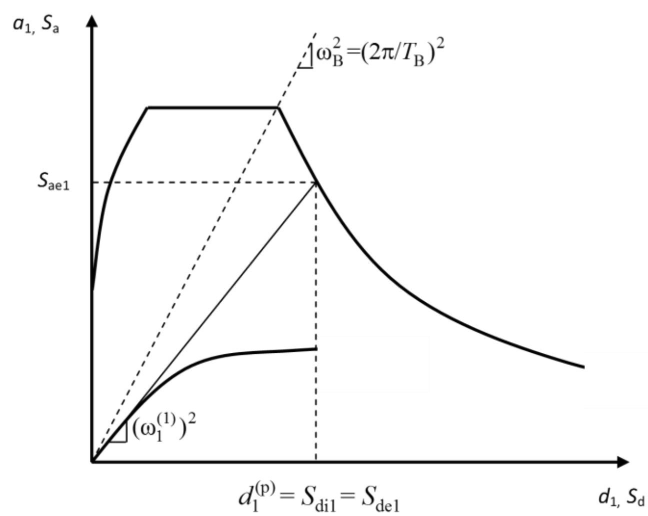

Figure 2. Capacity diagram and the displacement demand in the modal acceleration-displacement plane

The modal displacement demand d_1 is equal to the inelastic displacement demand Sd_{i1}, which is in turn equal to the modal linear elastic displacement demand S_{de1} when $(w_1^{(1)})^2 \leq w_B^2$ as shown in Eq.1.

$$d_1^{(p)} = S_{di1} = S_{de1} \tag{1}$$

where

$$C_{R1} = \frac{1 + (R_{y1} - 1)\, T_B / T_1^{(1)}}{R_{y1}} \geq 1 \tag{2}$$

$$R_{y1} = \frac{S_{ae1}}{a_{y1}} \tag{3}$$

Building displacements, internal deformations and forces can be calculated at the modal displacement demand by appropriate transformations using the first mode properties. The plastic rotations obtained at the member plastic hinge locations are then used for calculating the plastic curvature demands at these critical sections.

$$\phi_p = \frac{\theta_p}{L_p} \tag{4}$$

$$L_p = 0.5\,h \tag{5}$$

(*h: the overall depth of beam or column, L_p: plastic hinge length*)

$$\phi_t = \phi_y + \phi_p \tag{6}$$

Concrete compressive strains and steel tensile strain demands at the plastic regions are calculated from the moment-curvature diagrams at the plastic curvature level in Eq. (4). Moment-curvature diagrams of the critical sections are obtained by using appropriate stress-strain rules for concrete and steel. Finally, the calculated strain demands are compared with the damage limits given below to determine the member damage states in view of Figure 1.

Deformation limits for different sectional damage levels of RC sections are given in Table 1 where ρ_s and ρ_{sm} are the volumetric ratio of the existing and TEC compliant transverse reinforcement, respectively. Based on these damage levels of cross sections, performance level of the story and, the structural performance of the building can be determined.

Table 1. Deformation limits of RC sections in TEC

Cross-sectional damage level	Maximum strain for concrete (ε_c)	Maximum strain for steel (ε_s)
Minimum damage	0.0035	0.010
Safety level	$0.0035+0.01(\rho_s/\rho_{sm})\leq 0.0135$	0.040
Collapse level	$0.004+0.014(\rho_s/\rho_{sm})\leq 0.018$	0.060

Target Performance Levels for Buildings Seismic Performance

The reference design spectrum in the Code has 10% probability of exceeding in 50 years for residential RC buildings. Based on Turkish strong motion data, it is estimated that the spectral ordinates for 50% probability of exceeding in 50 years are half of the reference spectrum whereas the ordinates for 2% probability of exceeding in 50 years are 1.5 times that of the reference spectrum. Accordingly, the target performance levels of residential buildings are expected to provide the level of Life Safety (LS). In TEC, four performance levels were defined to determine building performance of RC buildings. Building earthquake performance level is determined after determining the member damage states, as explained above. The rules for determining building performance are given below for each performance level (TEC (2007), Sucuoğlu, 2007):

Immediate Occupancy (IO): In any story, in the direction of the applied earthquake loads, not more than 10% of beams are in the significant damage state whereas all other structural members are in the minimum damage state.

Life Safety (LS): In any story, in the direction of the applied earthquake loads, not more than 20% of beams and some columns are in the extreme damage state whereas all other structural members are in the minimum or significant damage states. However, shear carried by those columns in the extreme damage state should be less than 20% of the story shear at each story.

Collapse Prevention (CP): In any story, in the direction of the applied earthquake loads, not more than 20% of beams and some columns are in the collapse state whereas all other structural members are in the minimum, significant or extreme damage states. However, shear carried by those columns in the collapse state should be less than 20% of the story shear at each story. Furthermore, such columns should not lead to a stability loss. Occupancy of the building should not be permitted.

Collapse (C): If the building fails to satisfy any of the above performance levels, it is accepted as in the collapse state. Occupancy of the building should not be permitted.

Determining Seismic Performance of Existing Buildings

Many existing RC buildings that located in Isparta, Afyon, Burdur, İstanbul and İzmir that have been selected in order to determine seismic performance and to assess the overall situation in Turkey. All buildings were located within high-hazard zones. Selected buildings are 2, 3, 4 and 5 stories that each of the 120 buildings was analyzed by considering design parameters. Some of selected buildings plan views are given in Figure 3 and the number of building considered are displayed in Figure 4. Some structural features belonging to selected some buildings (located within high-hazard zones) are given on the Table 2. The proportion of reinforcement in columns and beams is taken as minimum percentage (as 1%) in the analyzes.

The performances of the selected 120 different buildings are investigated by considering the detailed rules given in the TEC. The analyses are carried out by considering the design parameters of the building which are obtained from their blueprint drawings. The structural properties and some analysis results of the some selected buildings are given in Table 2. Abbreviated expressions of building names are used such as the buildings in Isparta, Afyon, İzmir, Burdur and İstanbul are coded as ISP, AFY, İZM, BUR and İST, respectively. Where W_b is building weight, A_{ft} is the each area of the floor, A_c is sum of cross-sectional areas of all columns, C_{cn} is characteristic compressive strength of concrete, S_{st} is characteristic yield strength of steel, d^{ep}_{max} is target elasto-plastic displacement of building, T_1 is building's first period, $S_{d(ay)}$ is nonlinear spectral displacement of the first mode of building, R_y is strength reduction factor, μ is ductility of building and Γ_{x1} is additive multiplier for the first mode of building.

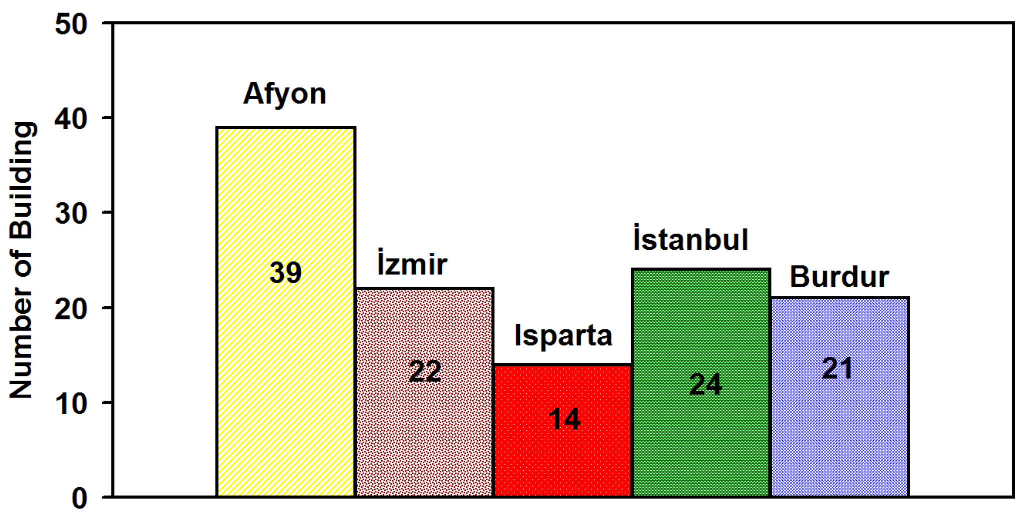

Figure 4. Distribution of building heights

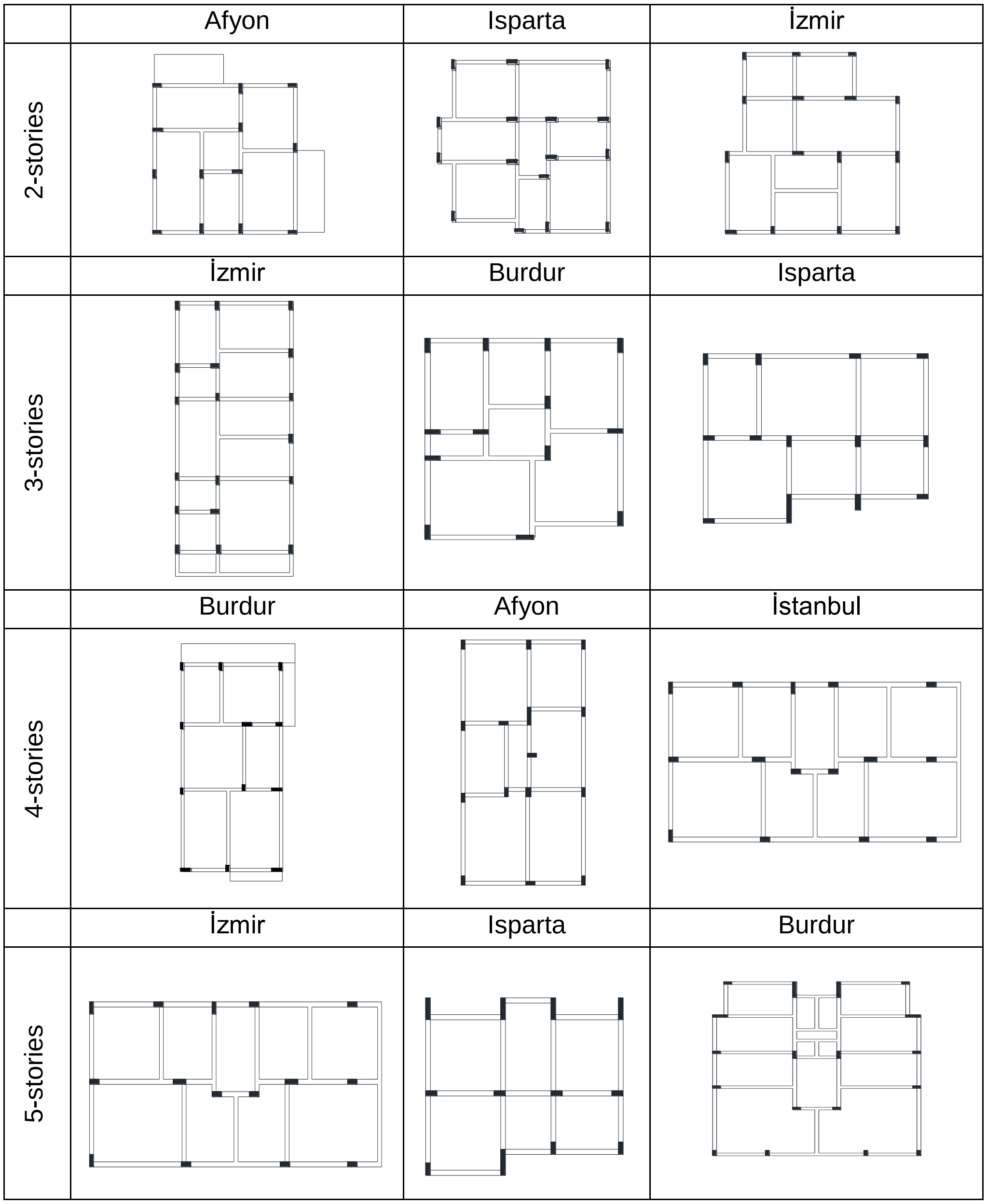

Figure 3. Plan views of some selected buildings.

Table 2. Structural properties and some analysis results of some buildings

Building ID	n	W_b (kN)	A_{ft} (m^2)	A_c (m^2)	C_{cn} (MPa)	S_{st} (MPa)	d^{ep}_{max} (m)	T_1 (s)	$S_{d(ay)}$ (m)	R_y	μ	Γ_{x1}
AFY-1	4	28692	571	7.20			0.432	0.908	0.09	8.41	3.95	1.32
AFY-2	2	2000	92	2.00			0.077	0.243	0.02	2.90	3.65	1.20
AFY-3	4	7384	193	2.81			0.247	0.616	0.05	6.87	3.50	1.29
AFY-4	2	2732	161	2.18			0.086	0.267	0.02	3.01	3.42	1.20
BUR-1	3	5229	166	2.23			0.174	0.439	0.04	6.28	3.70	1.28
BUR-2	4	6844	137	3.31			0.109	0.335	0.02	3.94	5.20	1.24
BUR-3	3	16065	114	2.13			0.193	0.503	0.03	5.99	4.74	1.25
BUR-4	3	5343	158	3.12			0.172	0.436	0.03	5.37	4.42	1.27
ISP-1	5	25077	413	8.74			0.231	0.679	0.04	4.70	3.91	1.35
ISP-2	5	9540	202	3.84	10	220	0.284	0.690	0.07	5.67	3.00	1.29
ISP-3	3	4714	145	3.30			0.164	0.399	0.04	3.94	3.26	1.33
ISP-4	4	6399	162	3.26			0.194	0.502	0.04	4.67	3.42	1.33
ISP-5	3	4678	154	3.15			0.154	0.413	0.04	2.97	3.31	1.29
İST-1	5	14695	215	5.37			0.294	0.710	0.06	5.80	3.23	1.29
İST-2	2	2064	84	2.19			0.080	0.236	0.02	2.89	3.81	1.27
İST-3	5	14294	214	6.63			0.226	0.554	0.05	5.10	3.55	1.31
İST-4	5	7120	93	2.34			0.303	0.751	0.06	6.23	3.22	1.30
İZM-1	2	3160	107	2.76			0.076	0.244	0.02	2.80	4.19	1.19
İZM-2	2	3841	180	2.85			0.116	0.326	0.02	3.86	4.82	1.21
İZM-3	4	8384	230	3.21			0.212	0.532	0.04	5.56	4.28	1.32
AFY-1	4	28692	571	7.20			0.432	0.923	0.11	4.76	3.06	1.31
AFY-2	2	2000	92	2.00			0.046	0.224	0.03	1.24	1.34	1.20
AFY-3	4	7384	193	2.81			0.240	0.567	0.07	3.96	2.82	1.28
AFY-4	2	2732	161	2.18			0.067	0.246	0.03	2.11	2.66	1.20
BUR-1	3	5229	166	2.23			0.158	0.404	0.05	2.98	2.75	1.27
BUR-2	4	6844	137	3.31			0.097	0.309	0.02	2.71	3.30	1.22
BUR-3	3	16065	114	2.13			0.190	0.464	0.04	3.77	3.76	1.26
BUR-4	3	5343	158	3.12			0.155	0.402	0.04	3.15	3.31	1.27
ISP-1	5	25077	413	8.74			0.285	0.626	0.08	3.73	2.66	1.36
ISP-2	5	9540	202	3.84	20	420	0.274	0.636	0.08	3.46	2.69	1.28
ISP-3	3	4714	145	3.30			0.144	0.367	0.05	2.45	2.32	1.33
ISP-4	4	6399	162	3.26			0.178	0.463	0.07	2.53	2.06	1.29
ISP-5	3	4678	154	3.15			0.133	0.380	0.09	1.84	1.12	1.28
İST-1	5	14695	215	5.37			0.285	0.654	0.07	4.05	3.00	1.28
İST-2	2	2064	84	2.19			0.062	0.217	0.02	1.78	2.63	1.27
İST-3	5	14294	214	6.63			0.215	0.511	0.06	2.96	2.48	1.33
İST-4	5	7120	93	2.34			0.295	0.692	0.09	3.83	2.47	1.28
İZM-1	2	3160	107	2.76			0.053	0.225	0.03	1.48	1.63	1.19
İZM-2	2	3841	180	2.85			0.099	0.301	0.03	2.32	2.84	1.21
İZM-3	4	8384	230	3.21			0.201	0.491	0.05	3.42	2.99	1.30

In the analysis of existing residental RC buildings two different situations were taken into consideration that considering the new design of the buildings, the compressive strength of concrete is 20 MPa, yield strength of steel is 420 MPa and spacing of transverse reinforcement is 100 mm in first case, considering the existing buildings, the compressive strength of concrete is 10 MPa, yield strength of steel is 220 MPa and spacing of transverse reinforcement is 200 mm in second case. Analysis of the 120 buildings was carried out and performance levels were determined for both cases (in both direction).

Pushover Analysis

The pushover analysis consist of application of the building weight that is affected by gravity forces as lateral load (or called seismic load) on a point at roof level of building. Seismic loads were applied in a step-by-step nonlinear static analysis, monotonically. The static nonlinear behavior of building is described by a capacity curve that represents the relationship between the roof displacement and base shear force in pushover analysis. In this study, some of the static pushover curves of the buildings obtained from the elastic analyzes made are given in Figure 5,6,7 and 8. The vertical axis is shown as the global normalized ratio of the base shear force over the seismic load. The horizontal axis plots as global roof displacement that is lateral displacement of building at the roof level normalized by building height.

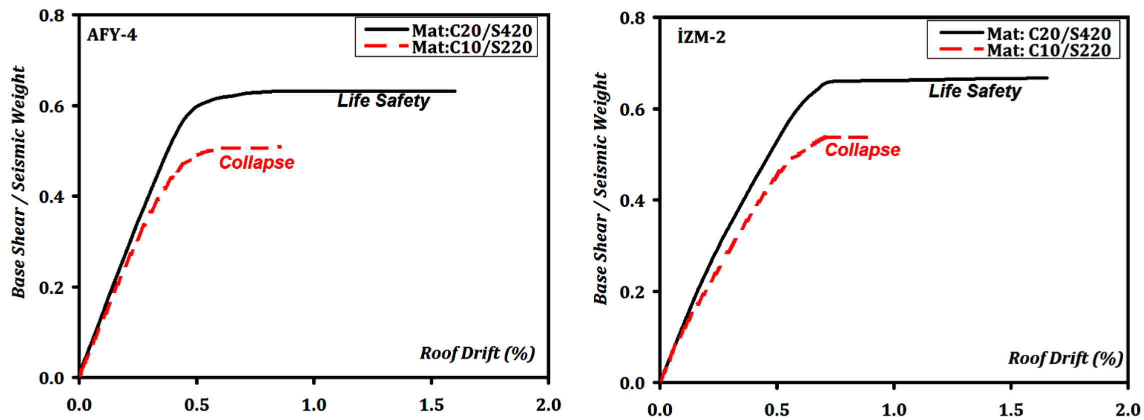

Figure 5. Pushover curve of some selected existing buildings for 2-stories

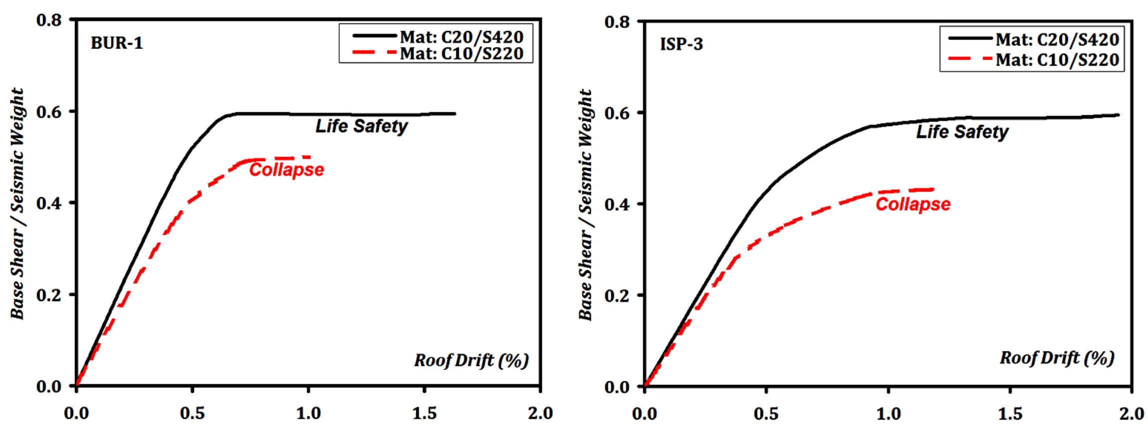

Figure 6. Pushover curve of some selected existing buildings for 3-stories

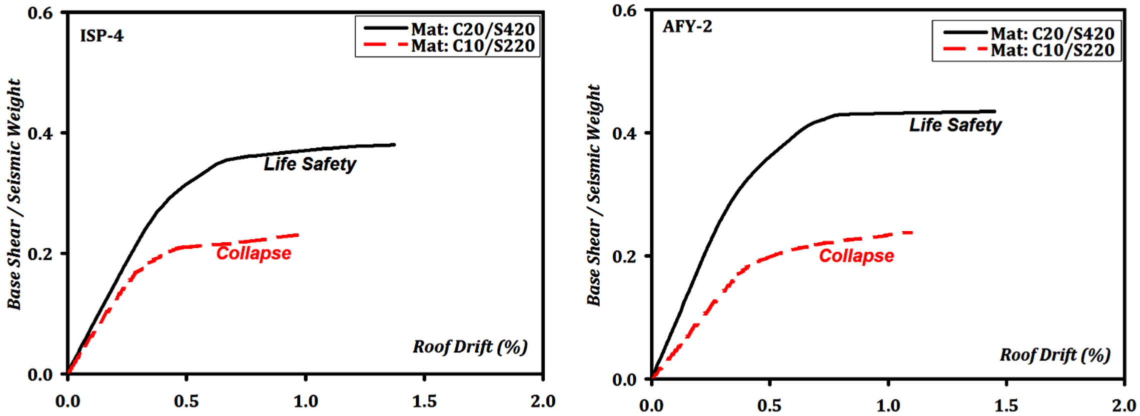

Figure 7. Pushover curve of some selected existing buildings for 4-stories

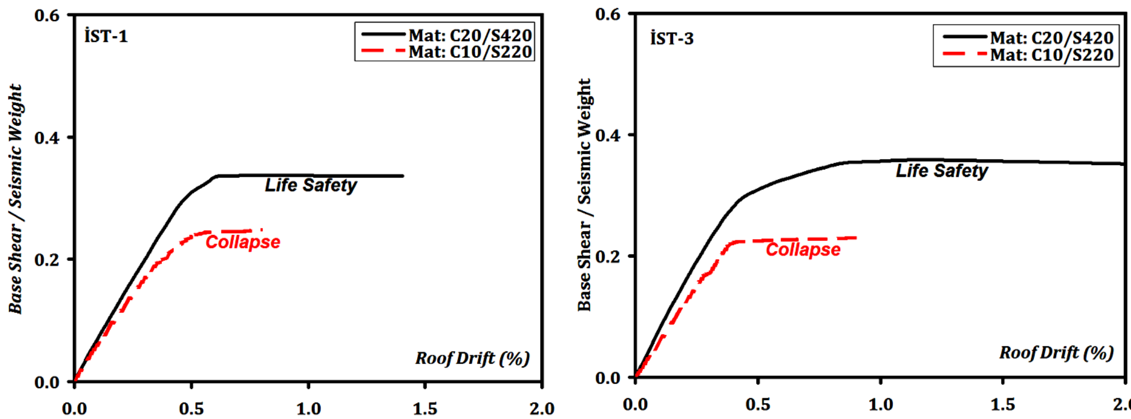

Figure 8. Pushover curve of some selected existing buildings for 5-stories

Damage levels were checked by making plastic hinges assigned to column and beam elements (Figure 9). Seismic performances of buildings were determined by the damage levels of each column and beam element that were individually controlled. Four different analyzes have been performed for two sets of materials for a building. A total of a thousand building solutions were analyzed according to the principle of inelastic method in TEC and seismic performance of buildings were determined. The SAP2000 program is used for pushover analysis.

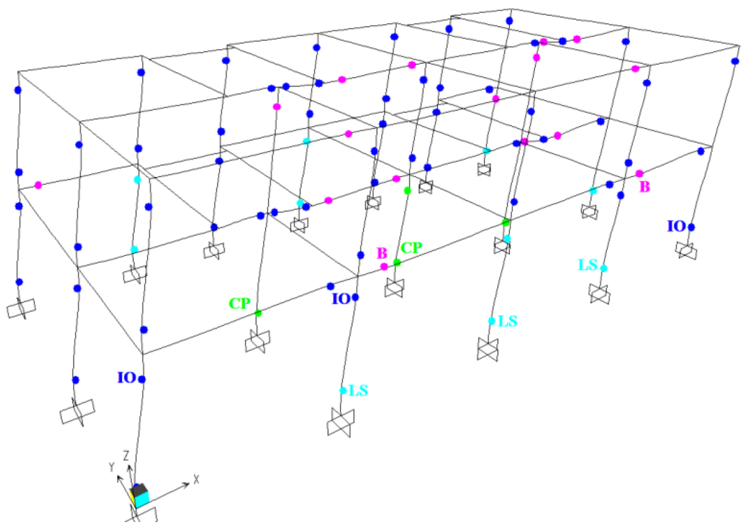

Figure 9. Hinge locations at the columns and beams.

RESULTS OF THE ANALYSIS

The nonlinear behaviors of the structural system elements of the buildings subjected to pushover analysis were determined. Seismic performances of existing buildings have been determined by examining the damage to the column and beam elements. The most important effect on seismic performance of building was the non-ductile brittleness and bending damage that occurred in the columns. The effect of the damages at beams on the performance of the buildings were in the secondary place according to the damages at columns. The performance results of existing buildings selected from Afyon, İzmir, Isparta, Burdur and İstanbul are as shown in Figure 10,11,12,13 and 14, respectively. In addition, Figure 13 are given to examine the general situation of seismic performance of buildings in Turkey.

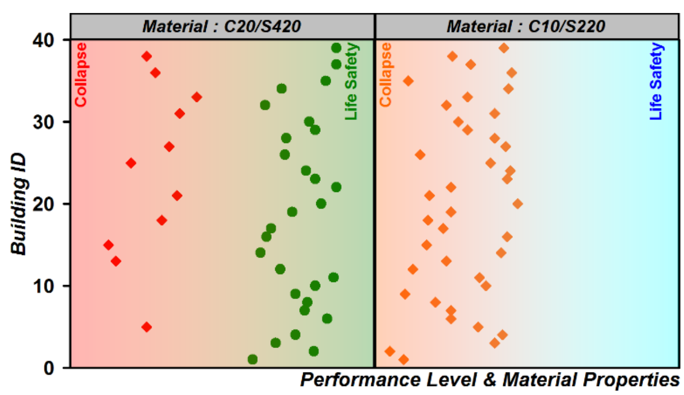

Figure 10. Seismic performance of existing buildings in Afyon

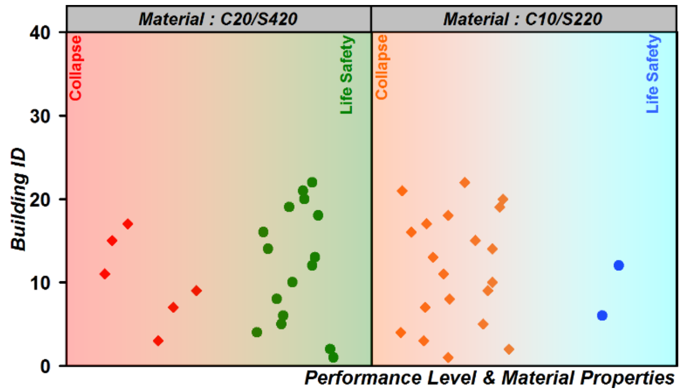

Figure 11. Seismic performance of existing buildings in İzmir

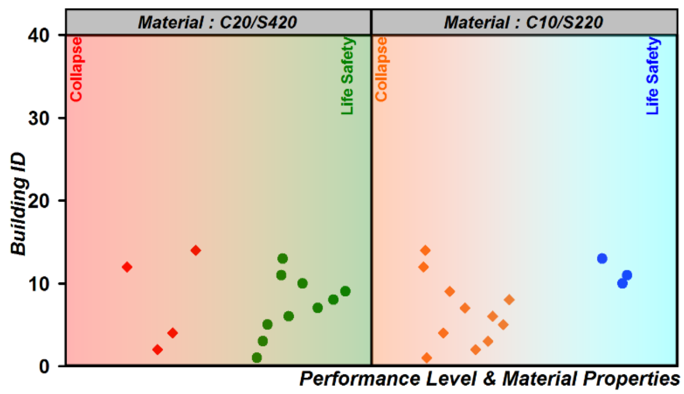

Figure 12. Seismic performance of existing buildings in Isparta

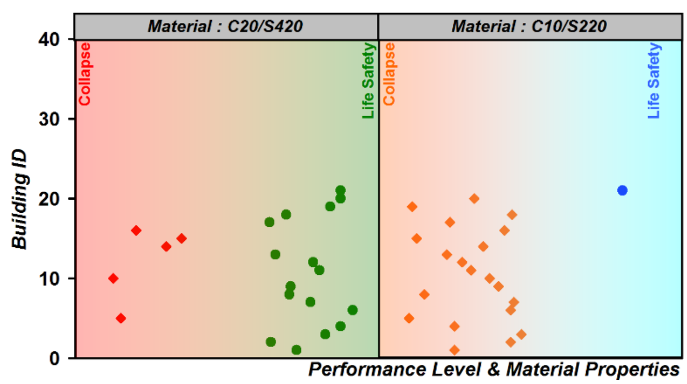

Figure 13. Seismic performance of existing buildings in Burdur

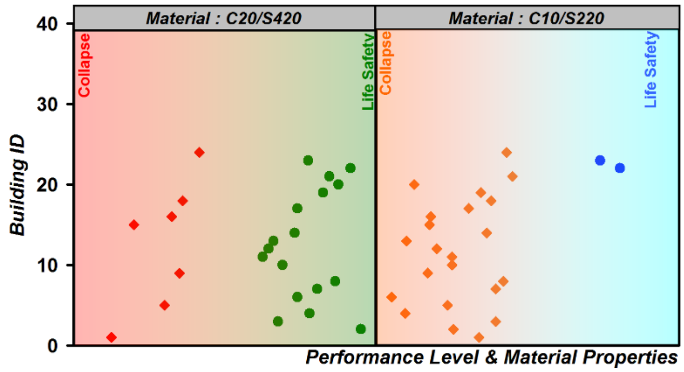

Figure 14. Seismic performance of existing buildings in İstanbul

It is seen that there is a serious difference between the fact that the material group is C10/S220 and that it is C20/C420 when the results of the analytical evaluation are examined. In additionally, the effect of the transverse reinforcement should not be ignored. The percentages of the performance levels of the building according to the different material are as shown in Figure 15 and Figure 16 by comparing the results obtained from the analytical assessment.

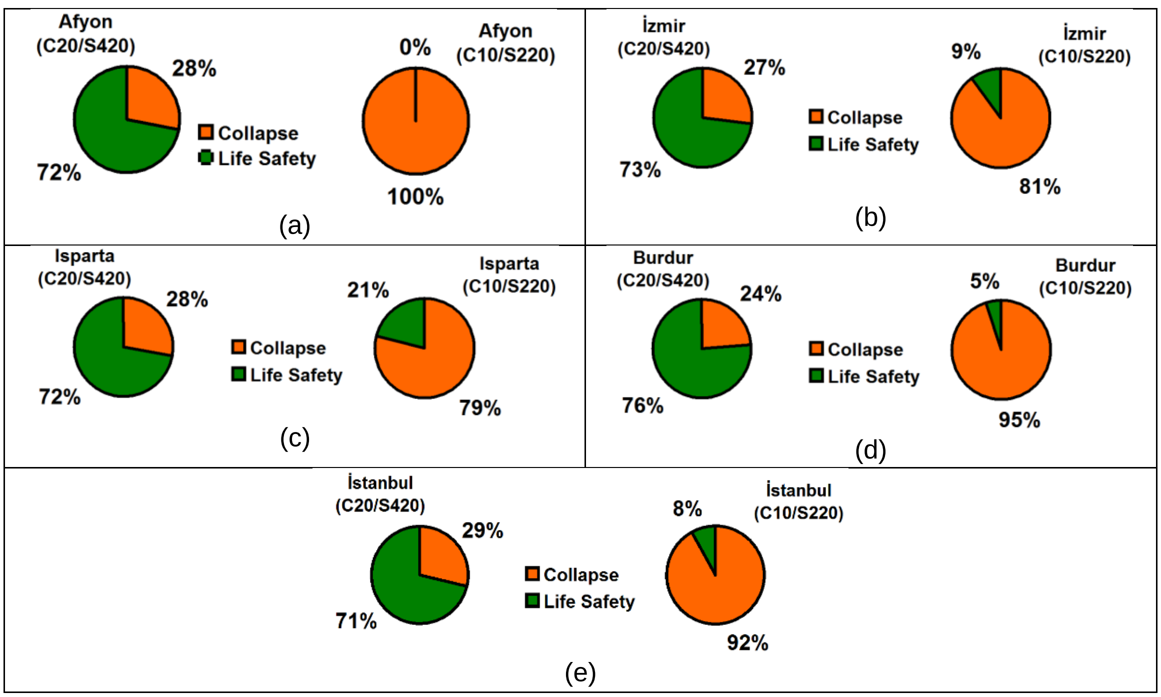

Figure 15. The percentage of seismic performance of buildings based on analytical assessments

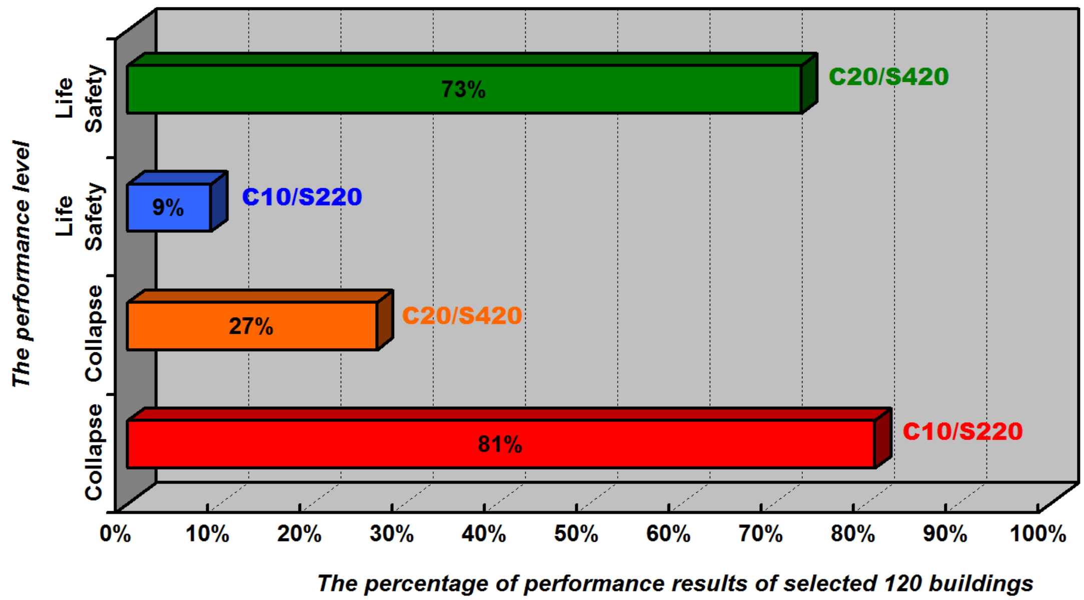

Figure 16. The percentage of general state on the seismic performance of selected building

As can be seen in Figure 15 and Figure 16, the class of materials and transverse reinforcement are quite influential in the seismic performances of buildings. If the material is C10/S220 almost all the buildings provide the collapse level. However, the great contribution of the material on the seismic performance of buildings clearly appears to be due to the C20/S420.

CONCLUSİON

The typical construction type is residential RC building in Turkey. Therefore, it is crucial to understand to earthquake response of each structure. The paper presents seismic assessment of 120 existing RC residential buildings according to principles of nonlinear elastics that specified in the TEC.

It was observed that compressive strength of concrete and yield strength of steel was effective on the lateral load-carrying capacity of RC buildings. In two, three, four and five stories RC buildings for C10/S220 and C20/S420, the effect of strenght of material had considerably increased the lateral load-carrying capacity of the buildings approximately between 15-25 percent. The proportional change in the lateral load-carrying capacity seems to cause a change in building performance.

Seismic performance analyzes of all buildings were occured for concrete strength 20 MPa, steel yield strength 420 MPa, confinement spacing value 100 mm, and concrete strength 10 MPa, steel yield strength 220 MPa, confinement spacing value 200 mm. It has been determined that the buildings provide an average of 73 percent of the level of life safety seismic performance for C20/S420 and an average of 90 percent of the level of collapse performance for C10/S220.

It is understood that the influence of the material strength on the building performance is incredibly effective. Most of the inspected buildings are around fifteen and twenty years old, so the materials used to build the buildings are probably not good. It is clear that demonstrates the importance of using ready-mixed concrete in building reinforced concrete buildings and a majority of these buildings do not provide a level of life safety performance and that measures must be taken when considering the effect of concrete and reinforcement strength on building performance.

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