Seismic Fragility Curves for 1 And 2 Stories R/C Buildings

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Abstract- In this study, the fragility curves were formed for 1 and 2 stories reinforced concrete (RC) residential buildings. The information regarding the structures was taken from projects of the buildings. Nonlinear pushover (NP) analyze was performed to 84 RC buildings which were divided to 2 groups. The buildings built 1998 and later were called Group-A. The building built before 1998 was called Group B. As for nonlinear analysis, a 3D computer model was drawn for each building and nonlinear analyses were applied to these models. Each building was analysed for 2 dimensions (X and Y). Totally 168 NP analysis were performed for each building. 4 damage limits (slight, moderate, heavy and complete) and 5 damage zones (undamaged, slight, medium, extensive and collapse) were determined according to maximum interstorey drift ratio. Fragility curve parameters were obtained at the result of the NP analysis. Probability density functions were calculated with the help of lognormal mean and lognormal standard deviation values of limit states. The fragility curves were generalized for the buildings in Group A and Group B. At the conclusion of this work; 8 fragility curves obtained for 1 and 2 stories reinforced concrete buildings in. Using these 8 curves; damage possibility can be estimated for RC buildings which have same features.

Keywords Fragility curves, regional earthquake risk, low rise reinforced concrete structures

1. Introduction

Seismic evaluation of the existing building stock has become a recognized priority after damage and collapse of many reinforced concrete (RC) structures during recent earthquakes [1]. Turkey is frequently exposed to destructive earthquakes. Besides, it is one of several countries in which earthquakes causing loss of lives occur with the shortest return period [2], [3]. It is known that more than 80% of the land of the country is under high seismic risk [4].

Considerable heavy damages have happened because of the earthquakes during the last 10 years [5]. RC buildings built before the modern codes have either collapsed or sustained extensive damage during the past earthquakes because of low quality concrete, poor confinement of the end regions of columns and beams, weak column–strong beam behavior, short column behavior, inadequate splice lengths and improper hooks of the stirrups [4], [6], [7].].

There are several study about earthquake damages and loss estimation methods for RC buildings. On the other hand, low rise RC structures are not considered as fragile. Especially one and two-storey buildings are usually ignored. However low rise reinforced concrete (RC) structures are not considered as fragile, there has been several collapse case in recent earthquakes. The paper aims to develop useful damage estimation tools for 1 and 2 stories RC structures.

2. Earthquake Hazard for Turkey

Turkey is located on active faults. The country has hazardous earthquake zones. Turkey earthquake zoning map is shown in Figure 1 [8]. Istanbul is the largest city in Turkey, constituting the country's economic, cultural, and historical center. Istanbul is located on the North Anatolian Fault Line. The city and its surrounding areas have been hit by an estimated above 100 earthquakes over the last years according to the Istanbul municipality's disaster coordination center. More than 110,000 deaths, 250,000 hospitalizations and 600,000 destroyed housing units were recorded across turkey as a result of earthquakes in the 20th century [26], [27].

Many earthquakes have been recording every year in the country. Table 1 shows that four and above magnitude earthquakes which were occurred between 30 days in Turkey and its surrounding areas. Likewise; more than 1800 earthquakes were recorded around the country just over the 30 days according to Republic of Turkey, Disaster and Emergency Management Presidency [9]. Table 1 and Figure 2 show that four and above magnitude earthquakes occurred between last 30 days (14 March – 15 April 2015) in Turkey and its surrounding areas.

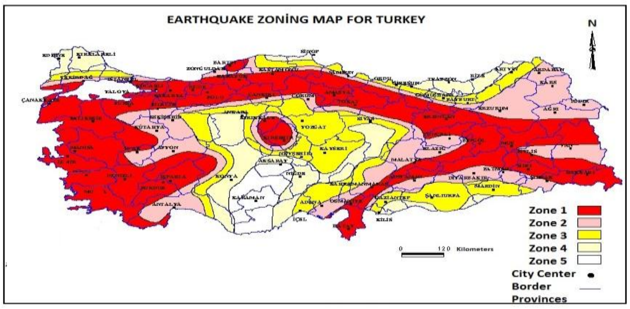

Fig. 1.Earthquake zoning map [8].

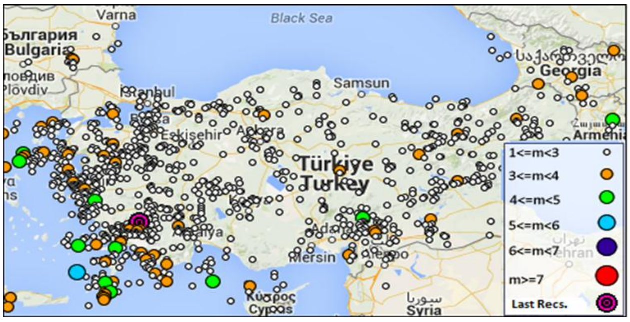

Fig. 2. Earthquake data records for Turkey and its surrounding areas [9].

13 records are shown in table 1. The biggest magnitude earthquake is 5.0 in the table. It was occurred in Aegean Sea, the latitude of 357.295 and longitude of 265.760.

Table 1. Four and above magnitude earthquakes occurred between 14 March – 15 April 2015 in Turkey and its surrounding areas [9].

DATE (GMT)	LATITUDE	LONGITUDE	DEPTH	RMS	MAGNITUDE	LOCATION	
20/06/2015 19:52:47	34.7483	26.3781	7.33	0.77000	4.5	-	AKDENIZ-DOGU BASENI
18/06/2015 01:52:58	35.0610	26.6098	20.93	0.90000	4.1	-	GIRIT
13/06/2015 01:03:15	38.6620	40.1986	25.11	0.56000	4	Turkey	BINGÖL
09/06/2015 21:49:50	35.1771	26.8226	12.74	0.87000	5.1	-	AKDENIZ-DOGU BASENI
09/06/2015 01:09:03	38.7675	23.4230	5.56	0.49000	5	Greece	Greece
08/06/2015 22:20:59	35.3550	26.9141	38.73	0.77000	4.4	ONIKI ADA	ONIKI ADA
03/06/2015 09:35:41	40.7560	49.4020	15	0.80000	4.6	Azerbaijan	Azerbaijan
01/06/2015 14:54:16	37.5080	43.8820	12.69	0.64000	4	Turkey	HAKKARI
30/05/2015 01:56:17	40.2100	21.7920	16.20	0.40000	4	Greece	Greece
29/05/2015 08:02:51	36.8703	27.5928	17.01	0.25000	4.1	GÖKOVA	GÖKOVA
29/05/2015 00:03:48	34.9566	26.7278	15.85	0.82000	4.1	AKDENIZ-	AKDENIZ-
28/05/2015 12:59:21	35.0620	26.7396	6.44	0.68000	4.2	GIRIT	-

The loss estimation is based on the damage states of the structures. There are several models which can be used to quantify the damages, characterization of damage state and estimation of losses after the earthquakes [11]. Fragility analysis is one of the key components in seismic risk assessment and more specifically in regional seismic risk assessment [12].

For instance, Federal Emergency Management Agency (FEMA) has developed HAZUS methodology for Estimating Potential Losses from Disasters. HAZUS uses a systematic approach for probabilistic damage assessment of structures [13], [14]. As part of these procedures, fragility curves are employed in order to estimate the damage of a building after various intensities of ground motion shaking [15]. Fragility curves express the probability of the structure reaching or exceeding various damage states as a function of a specific earthquake intensity measure. The function of fragility curves can be assumed as a cumulative distribution function, such as a normal distribution, lognormal distribution or beta distribution [10], [14], [16], [25].

3. Fragility Curves

Fragility curve is a useful tool for predicting earthquake risk of buildings with similar characteristics such as material, height and design code level [12]. The curves can be formed using one of three kinds of procedures: (a) the experimental procedure, (b) the nonlinear static analysis based procedure, (c) the dynamic analysis based procedure [17]. The principle of the analytical method which is preferred in this study is to analyze the damage state of structures.

On the other side, fragility curves are cumulative distribution functions that probability of reaching or exceeding a damage state as demand parameters such as story drift ratio (SDR), peak ground acceleration (PGA), spectral acceleration (Sa) or spectral displacement (Sd) [10], [15], [18], [19]. It has been widely accepted that spectral displacement can be closely correlated with seismic damage of structures [18], [19]. Probability density function of a random variable with lognormal distribution is as follows equation-1:

$$f(x) = \frac{1}{x\sigma_Y\sqrt{2\pi}} \exp - \left[-\frac{(\ln x-\mu_Y)^2}{2{\sigma_Y}^2}\right], (0 < x < +\infty) \tag{1}$$

In this distribution; μ_Y is lognormal mean of variable Y and σ_Y is lognormal standard deviation of variable Y. μ_X ve σ_X are associated with μ_Y ve σ_Y by equation-2 and equation-3.

$$\mu_Y = \ln\left[\mu_X / \sqrt{\left(\frac{{\sigma_X}^2}{{\mu_X}^2} + 1\right)}\right] \tag{2}$$

$$\sigma_Y = \sqrt{\ln\left(\frac{{\sigma_X}^2}{{\mu_X}^2} + 1\right)} \tag{3}$$

Probability distribution of earthquake damage is assumed to be lognormal distribution. Thus, the analytical expression of fragility curve for a damage level is written as the follows equation-4

$$Pd \geq \left(d_{s_i} \middle| S_d\right) = \varphi\left(\frac{\ln(S_d) - \overline{S_{d_{s_i}}}}{\beta_{d_{s_i}}}\right) \tag{4}$$

Pd is probability of damage. Sd is modal displacement. d_{s_i} is modal displacement for damage level "i". $\overline{S_{d_{s_i}}}$ is mean modal displacement for damage level "i". $\beta_{d_{s_i}}$ is lognormal standard deviation of modal displacement values for damage level "i". φ is cumulative distribution function.

In this study, analytical parameters of fragility curves were obtained by nonlinear static pushover (NP) analysis. In the newly developed performance-based seismic design, NP analysis is one of the useful methods for determining the patterns and levels of damage for assessing a structure's inelastic behavior [20]. NP analysis uses lateral external static forces at floor levels, in combination with inelastic response spectra [21], [22]. Generally, the first mode response of the structures is considered in NP analysis [23]. In case, fundamental mode of vibration is the predominant response of structures [20]. There is a relatively good statistical correlation observed between the earthquake-induced displacement and the extent of structural damage contributed [24, [12]. NP analysis was performed on 3D computer models of the buildings by CSI SAP2000 computer program.

4. Building Stock

1 and 2 stories 84 RC residential buildings discussed within earthquake damage risk. The buildings were selected by random sampling method from municipal archives. Total 84 structures were divided to 2 groups which are Group A and Group B. the buildings built 1998 and later were called Group-A and the building built before 1998 was called Group B. NP analysis was performed to all structures in 2 directions. Parameters of the buildings in Group A and Group B have given in Table 2.

Concrete and reinforcement bar classifications were made according to Turkish Standards TS500 and TS708. A lot of similarities were observed between the buildings built 1998 and later as shown in table 2. Concrete class was usually observed as C16 and reinforcement bars were usually observed as S420.

As for Group B; concrete class was usually observed as C14 and reinforcement bars were usually observed as S220 for Group A. Column and beam sizes were almost similar with Group A. In addition to table 2 and table 3, the slab thickness are usually 10cm, story highness of the buildings are observed between 2,60 and 2,80m for both 2 groups. Stirrup diameter was 8mm in all columns and beams.

5. Modeling and Analysis

In this study; 3D computer models was occurred for buildings to analyze the damage state of structures. As for the modeling issues; columns defined as R/C elements which work for axial load, M2 and M3 moment. Beams also defined as R/C element which working M3 moment. Soil-structure interaction was not considered. All translations and rotations are highlighted for lower ends of the columns at the bottom floor. Otherwise, "Mander model" was used for the stress–strain relation of concrete model (Mander 1988, Ersoy and Ozcebe 2010).

The shear capacity of the section was evaluated based on the code of ATC 32 recommendations. According to ATC – 32, the shear strength of the hoops is determined as:

$$V_S = \frac{A_s f_y d'}{s} \tag{5}$$

Where As is the cross-sectional area of the transverse reinforcement, f_y is the steel yield strength; d' is effective depth and s is the vertical hoops spacing.

The shear strength provided by the concrete is taken as:

$$V_c = 2\left[1 + \frac{P_e}{2000\ A_g}\right]\sqrt{f_c'}\ A_e \tag{6}$$

Where P_e is the axial compressive force on the column, Ag is the gross shaft area and Ae is the effective shear area of column.

3.8 kN/m^2 load was assigned to exterior beams due to brick wall but 2.5 kN/m^2 assigned on the interior beams. The rigid diaphragm effect was modeled using "joint constrains" properties. Plastic hinges were defined on the both upper and lower ends of columns and beams to idealize the nonlinear behavior. Plastic hinge length was considered as half of cross sections.

Nonlinear pushover analyze was applied for each building in two dimensions. Totally, 120 modal capacity curves were obtained at the result of NP analysis. Figure 4 shows that the highest and the lowest capacity curves as an example.

The building B07 (from group B) has the lowest capacity and the building A12 (form goup A) has the higher capacity as shown in figure 3. In addition; general performance of the buildings in group A is higher than the buildings in group B.

After the obtaining modal capacity curves, it was described the damage levels according interstory drift ratio. Table 4 shows that damage levels for low story reinforced concrete frame buildings according to interstory drift ratios.

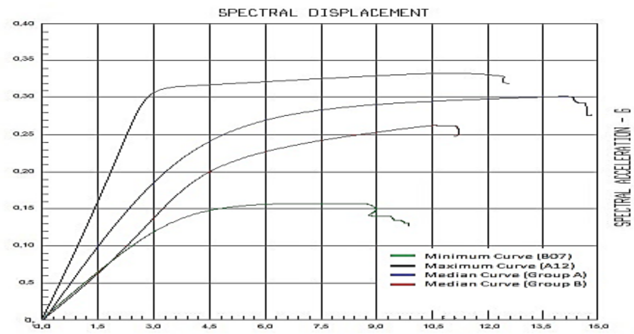

Fig. 3. 3D model and Modal Capacity Curve of Building A01 as an example.

Table 2 Building Parameters of Group A**.**

Building Code	Project Year	Number of Story	Majority Column Size (cm)	Majority Beam Size (cm)	Concrete Material	Bar Mat.	Soil Type	Plan Irregularity	Short Column
A01	2004	1	25x40	25x50	C16	S420	Z2	No	No
A02	2005	1	30x50	25x50	C16	S420	Z3	No	No
A03	2008	1	30x50	25x50	C16	S420	Z3	No	No
A04	2008	1	40x40	30x50	C14	S220	Z3	No	No
A05	1998	1	25x40	25x50	C16	S420	Z4	No	No
A06	1998	1	30x30	30x50	C16	S220	Z3	No	No
A07	1998	1	30x40	30x60	C20	S420	Z3	No	No
A08	1999	1	25x40	30x50	C16	S420	Z2	Yes	No
A09	1999	1	30x30	30x50	C20	S420	Z3	No	No
A10	1999	1	30x40	30x50	C16	S420	Z3	No	No
A11	1999	1	25x40	25x50	C20	S220	Z4	No	No
A12	1999	1	30x50	30x50	C16	S220	Z3	No	No
A13	1999	1	30x50	30x50	C20	S420	Z4	Yes	No
A14	2000	1	25x40	25x50	C16	S420	Z3	No	No
A15	2000	1	30x40	30x50	C16	S420	Z4	No	No
A16	2001	2	25x50	25x50	C16	S420	Z4	No	No
A17	2001	2	30x30	25x50	C16	S420	Z3	No	No
A18	2001	2	30x50	25x50	C16	S420	Z2	No	No
A19	2001	2	30x30	25x50	C20	S420	Z3	No	No
A20	2002	2	25x40	30x50	C16	S420	Z3	No	No
A21	2002	2	30x40	25x50	C16	S420	Z3	No	No
A22	2002	2	30x40	30x50	C16	S420	Z3	No	No
A23	2003	2	25x40	30x50	C16	S420	Z3	No	Yes
A24	2003	2	30x30	25x50	C16	S420	Z3	Yes	No
A25	2003	2	40x40	30x50	C16	S420	Z2	Yes	No
A26	2003	2	30x30	30x50	C16	S420	Z3	No	No
A27	2003	2	25x40	25x50	C16	S420	Z2	No	No
A28	2003	2	25x40	25x50	C18	S420	Z3	No	No
A29	2003	2	30x40	30x50	C16	S420	Z4	No	No
A30	2003	2	30x30	25x50	C14	S420	Z3	No	No
A31	2003	2	40x40	25x50	C16	S420	Z3	No	No
A32	2004	2	25x40	30x60	C16	S420	Z3	No	No
A33	2004	2	40x40	25x50	C16	S420	Z3	No	No
A34	2004	2	25x40	25x50	C20	S420	Z2	No	No
A35	2004	2	25x40	30x60	C16	S420	Z2	No	Yes
A36	2004	2	30x30	25x50	C16	S420	Z3	No	Yes
A37	2004	2	30x30	30x50	C16	S420	Z4	No	No
A38	2005	2	40x40	25x50	C16	S420	Z3	No	No
A39	2005	2	25x40	30x50	C16	S420	Z4	Yes	No
A40	2005	2	25x40	25x50	C20	S420	Z4	No	No
A41	2006	2	40x40	30x60	C20	S420	Z3	Yes	No
A42	2006	2	30x30	30x50	C20	S420	Z3	Yes	No
A43	2006	2	25x30	30x60	C20	S420	Z4	No	No
A44	2006	2	25x40	25x50	C20	S420	Z3	No	No
A45	2007	2	40x40	30x60	C20	S420	Z2	No	No
A46	2008	2	30x30	30x50	C20	S420	Z3	No	No
A47	2008	2	25x30	25x50	C20	S420	Z3	No	Yes
A48	2008	2	25x40	30x50	C20	S420	Z3	Yes	No
A49	2008	2	25x30	25x50	C20	S420	Z2	No	No
A50	2008	2	25x30	25x50	C20	S420	Z2	No	No
A51	2008	2	30x30	25x50	C20	S420	Z3	No	No
A52	2009	2	40x40	30x60	C20	S420	Z4	No	No
A53	2009	2	25x40	25x50	C20	S420	Z4	No	No
A54	2009	2	25x40	30x50	C20	S420	Z3	No	No
A55	2009	2	30x30	25x50	C20	S420	Z3	No	No
A56	2009	2	25x40	30x50	C20	S420	Z3	No	No
A57	2009	2	25x40	25x50	C20	S420	Z2	No	No
A58	2009	2	25x40	30x60	C20	S420	Z3	No	No
A59	2010	2	30x30	30x50	C20	S420	Z4	No	No

A60	2010	2	40x40	25x50	C20	S420	Z3	Yes	No
A61	2010	2	25x40	30x60	C20	S420	Z3	No	No
A62	2010	2	25x40	30x50	C20	S420	Z4	No	No
A63	2011	2	30x30	25x50	C20	S420	Z3	No	No

Table 3 Building Parameters of Group B.

Building Code	Project Year	Number of Story	Majority Column Size (cm)	Majority Beam Size (cm)	Concrete Material	Bar Mat.	Soil Type	Plan Irregularity	Short Column
B01	1973	2	30x30	25x50	C16	S220	Z2	Yes	No
B02	1974	2	25x40	25x50	C16	S220	Z3	No	No
B03	1977	2	30x30	25x50	C16	S220	Z3	No	No
B04	1978	2	40x40	30x50	C16	S220	Z3	No	No
B05	1984	1	25x40	25x50	C14	S220	Z3	No	No
B06	1985	2	30x30	25x50	C14	S220	Z3	No	Yes
B07	1986	1	25x40	30x60	C14	S220	Z4	Yes	Yes
B08	1986	2	25x40	25x50	C14	S220	Z3	No	No
B09	1987	2	25x40	25x50	C16	S220	Z3	No	No
B10	1988	2	30x30	25x50	C16	S220	Z3	No	No
B11	1989	2	25x40	25x50	C16	S220	Z2	No	No
B12	1989	2	30x50	25x50	C16	S220	Z3	No	No
B13	1993	2	25x40	25x50	C14	S220	Z3	Yes	No
B14	1995	2	30x30	25x50	C16	S420	Z3	No	No
B15	1995	2	25x40	30x50	C16	S220	Z3	No	No
B16	1996	1	30x50	30x60	C14	S220	Z4	No	No
B17	1996	2	25x40	25x50	C16	S220	Z2	No	No
B18	1997	1	30x30	25x50	C20	S420	Z4	No	Yes
B19	1997	1	30x50	25x50	C16	S220	Z3	Yes	No
B20	1997	2	30x40	25x50	C16	S220	Z3	No	No
B21	1997	2	25x50	25x50	C20	S420	Z2	No	No

Table 4. Damage levels according to FEMA HAZUS MH MR 5.

Design Code	Slight Damage (%)	Medium Damage (%)	Heavy Damage (%)	Collapse (%)
Low Code	0,005	0,008	0,02	0,05
Moderate Code	0,005	0,0087	0,0233	0,06

The buildings in Group A were assumed as “moderate code” and the buildings in Group B “were assumed as “low code” in the table.

At the final step of calculations; lognormal mean modal displacement and lognormal standard deviation of modal displacement values were utilized as parameters of fragility curves. All parameters of fragility curves are shown in table 5 for Group A and Group B.

Table 5. Parameters of Fragılıty Curves.

Groups	Sd1 (Slight)		Sd2 (Moderate)		Sd3 (Heavy)		Sd4 (Collapse)	
	LM	LS	LM	LS	LM	LS	LM	LS
A	1,09	0,32	1,66	0,39	2,24	0,39	2,94	0,39
B	0,83	0,37	1,30	0,39	2,31	0,46	3,08	0,40

4 fragility curves were calculated for each group using the parameters in table 5. Figure 5 and Figure 6 show the fragility curves of slight, moderate, heavy and complete damage level for Group A and Group B.

In the literature; the best value for the damage estimation is 50 percent corresponding to the mid-point of fragility curve. Then the Group A buildings would be situated in undamaged zone with more than % 50 probabilities for 3cm modal displacement. Similarly, there is more than %50 probabilities for the buildings to be situated in slight damage zone between 3 and 5,30cm. There is %51 possibility to be in moderate damage zone between 5,30 and 13cm spectral displacement. As for heavy damage zone, this value changed between 13cm and 19cm. Finally, the buildings have more than %50 possibilities being in collapse zone for above 19cm spectral displacement value.

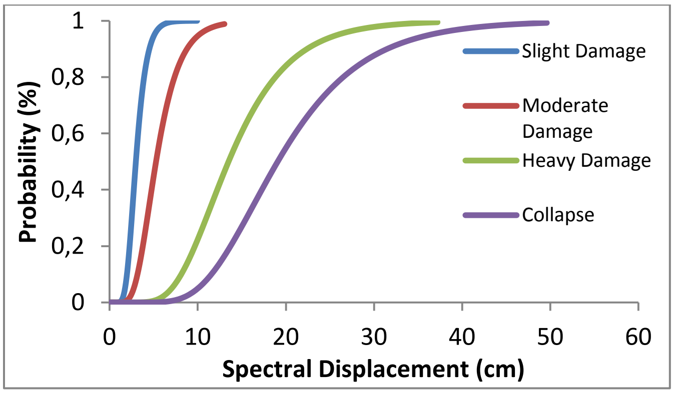

Fig. 4. Fragility curves for Group A.

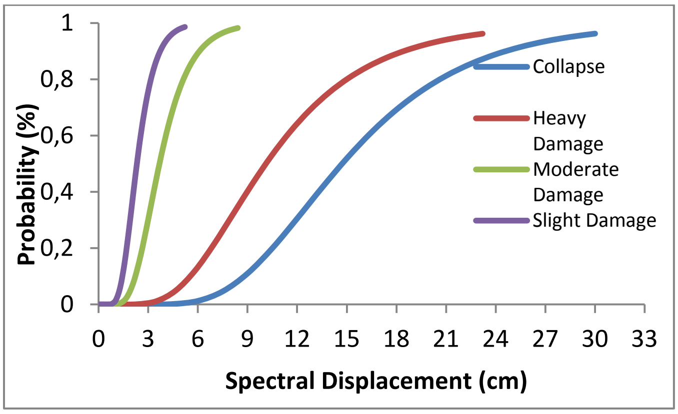

Fig.5. Fragility curves for Group B.

The Group B buildings would be situated in undamaged zone with more than % 50 probabilities for 2,30cm modal displacement. Then, there is more than %50 probabilities for the buildings to be situated in slight damage zone between 2,30 and 3,65cm. There is %51 possibility to be in moderate damage zone between 3,65 and 10cm spectral displacement. As for extensive damage zone, these values changed between 10cm and 15cm. Finally, the buildings have more than %50 possibilities being in collapse zone for above 15cm spectral displacement value.

6. Conclusions

This paper aims at using numerical simulation to develop fragility curves for 1 and 2 stories RC frame structures. Fragility curves were formed for 84 RC residential buildings. The buildings were divided into 2 groups according to year of Turkish Earthquake Code 1998. A 3D computer model was drawn for each building and analysis was applied to these models. 4 damage limits (slight, moderate, extensive and complete) and 5 damage zones (undamaged, slight, medium, heavy and collapse) were determined according to interstory drift ratios of the buildings. Cumulative probability functions were calculated with the help of lognormal mean and lognormal standard deviation values of limit states. Then, fragility curves that show probability of the damages were generalized.

According to pushover results; the buildings in Group B have lower lateral capacity compared to the buildings in Group A. However modal capacity curves give information about the current status of the buildings, analyses need long time. Therefore, regional studies and rapid risk assessment methods are required. Therewith, a general assessment can be made by the results of the fragility curves.

In this study, it has been observed that fragility curves were close to each other for slight and moderate levels. The Group A buildings would be situated in undamaged zone with more than % 50 probabilities in the range of 0-3cm modal displacement. Similarly, there is more than %50 probabilities for the Group A buildings to be situated in slight damage zone in the range of 3,00-5,30cm, moderate damage zone in the range of 5,30-13cm, heavy damage zone in the range of 13-19cm and collapse zone above 19cm.

Likewise, The Group B group buildings would be situated in undamaged zone with more than % 50 probabilities in the range of 0-2,30cm modal displacement. There is more than %50 probabilities for the Group B buildings to be situated in slight damage zone in the range of 2,30-3,65cm, moderate damage zone in the range of 3,65-10cm, extensive damage zone in the range of 20-15cm and collapse zone above 15cm.

In conclusion; the buildings in Group A have more ductile behavior as compared to Group B. The difference between fragility curves of Group A and Group B was compared for midpoint of the curves. The buildings in Group A have higher displacement capacity than the buildings in Group B. The difference is approximately %30 for slight, heavy and collapse damage states. Nevertheless, the difference is increased to approximately %45 for moderate damage state.

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