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FRAGILITY BASED ASSESSMENT OF THE STRUCTURAL DEFICIENCIES IN TURKISH RC FRAME STRUCTURES

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SUMMARY

Low–rise and mid–rise reinforced concrete (RC) structures, which constitute approximately 75% of the total building stock in Turkey, are focused in this fragility–based assessment. The seismic design of three, five, seven and nine story RC frame structures are carried out according to the current earthquake codes and two dimensional analytical models are formed accordingly. The uncertainty in material variability is taken into account in the formation of structural simulations. Frame structures are categorized as poor or superior according to the specific characteristics of construction practice and the observed seismic performance after major earthquakes in Turkey. The demand statistics in terms of maximum inter–story drift ratio are obtained for different sets of ground motion records. The capacity is determined in terms of limit states and the corresponding fragility curves are obtained from the probability of exceeding each limit state for different levels of ground shaking. The results are promising in the sense that the inherent structural deficiencies are reflected in the final fragility functions.

1. INTRODUCTION

The earthquakes that have occurred in Turkey have caused much tragic life and monetary losses within the last ten years. The high population density near or on the fault zones is an indicator of potential future disasters. So, it is necessary to estimate possible earthquake hazard and develop strategies to reduce losses. A fragility based assessment that considers local structural properties is required to prepare such disaster mitigation scenarios. The aim of this study is providing fragility information to inquire effects of ground motion parameters and Turkish construction practice state on structural vulnerability.

2. GENERAL STATE OF TURKISH CONSTRUCTION PRACTICE

The investigation of severely damaged or collapsed RC structures after recent earthquakes in Turkey revealed that most of them do not fulfill code requirements and have both architectural and structural issues. Structural deficiencies can be classified in three groups.

RC structures, often in rural and even in urban areas, have serious *design deficiencies* such as insufficient lateral resistance, lateral and longitudinal irregularities, weak or soft story, short column and weak column–strong beam joints. Insufficient lateral reinforcement and insufficient or wrong splicing of bars are the most frequent *detailing deficiencies*. Finally, low quality concrete and incorrect site applications, due to the lack of supervision and inconsiderate contractors, are among the *constructional deficiencies* facing Turkey [Tankut, 1999].

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3. DESIGN AND ANALYSIS CONSIDERATIONS OF MODELS

The first step in the generation of fragility curves for RC frame structures in Turkey is to construct the analytical models considering the current specifications and local structural characteristics in order to estimate the seismic behavior of these structures.

3.1 Design Considerations

The current spectral shapes in the Turkish Seismic Code, TSC [1998] are based on broadly described geological conditions, ignoring fault distance or magnitude dependencies on spectral ordinates, whereas site–distance–magnitude specific design spectra is more suitable as a tool both for deterministic (scenario earthquakes) and probabilistic seismic hazard assessments [Kalkan and Gülkan, 2004].

For this study, the number of stories is taken as variable and infill walls are not taken into consideration at design and analysis stages. Sixty ground motion records are used both for analysis and design. These ground motions are chosen with different intensity parameters like magnitude, peak ground acceleration (PGA), peak ground velocity (PGV), peak ground displacement (PGD), etc. They are classified in 3 groups, each containing 20 ground motions. Classification of the records based on boundary PGV values of 20 cm/s, 40 cm/s, and 60 cm/s separating ground motion groups, respectively. Design spectra are obtained according to FEMA 356 [2000].

Figure 1 shows acceleration response spectra (R1, R2, R3) and the corresponding design spectra (D1, D2, D3) of three ground motion groups.

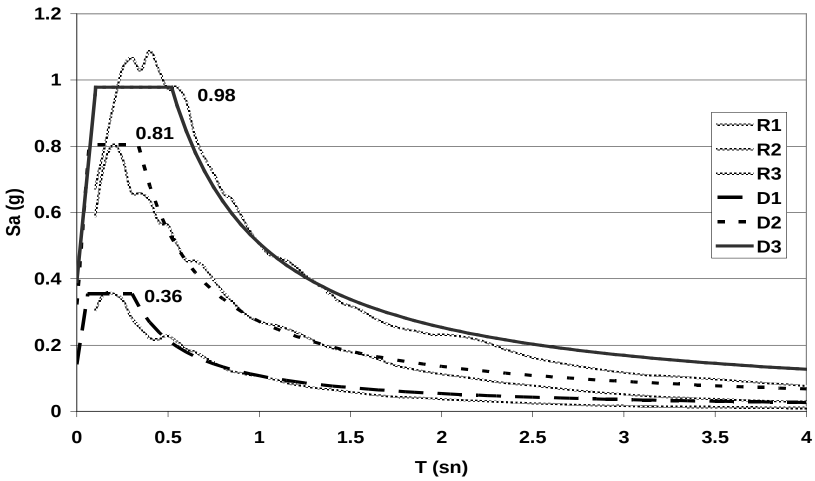

Figure 1: Response and Design Spectra of Ground Motion Groups

3.2 Analytical Models

For analytical models, RC frame structures without infill walls are selected. Low–rise and mid–rise structures, which constitute approximately 75% of the total building stock in Turkey and which are generally occupied with residential purposes, are modeled as 3, 5, 7, and 9–story frames.

The analytical models confirm with the current regulations for different levels of earthquake risk. Hence, the Requirements for Design and Construction of RC Structures in Turkey, TS500 [2000], TSC [1998], and ACI Building Code [2002], are employed during the development stage of the analytical models.

Story height of 3 meters and bay width of 5 meters are assumed in accordance with common practical cases. Using current codes, the design forces and cross sections are calculated. The column cross sections are reduced with respect to increasing story number to be compatible with a frequent application in Turkish construction practice. For structural analysis and reinforcement design, SAP2000 [CSI, 2002] is employed.

Main properties of the analytical models are listed in Table1. The abbreviation used for the models provides information about the number of stories and the design level of the corresponding model. For instance, 3SD1 means that it is a three–story building which has been designed according to the design spectrum representing the first group of ground motions. Remaining columns in the table give information about the beam and column dimensions (in centimeters) of the model and to which stories these dimensions belong. Underneath the model name, first natural period of the model obtained from eigenvalue analysis is presented in parenthesis.

Table 1: Cross Sectional Data and Period Values of the Analytical Models

NO	MODEL	SECTION	STORY	h (cm)	b (cm)	NO	MODEL	SECTION	STORY	h (cm)	b (cm)
1	**3SD1**	BEAM3	1–2–3	45	25	8		COL1	1–2–3	45	45
	(0.65)	COL3	1–2–3	30	30			COL2	4–5	40	40
2	**3SD2**	BEAM3	1–2–3	45	30			COL3	6–7	35	35
	(0.52)	COL3	1–2–3	35	35	9	**7SD3**	BEAM1	1–2–3	55	30
3	**3SD3**	BEAM3	1–2–3	50	30		(0.81)	BEAM2	4–5	50	30
	(0.42)	COL3	1–2–3	40	40			BEAM3	6–7	50	30
4	**5SD1**	BEAM2	1–2–3	45	25			COL1	1–2–3	50	50
	(0.93)	BEAM3	4–5	45	25			COL2	4–5	45	45
		COL2	1–2–3	35	35			COL3	6–7	40	40
		COL3	4–5	30	30	10	**9SD1**	BEAM1	1–2–3	55	30
5	**5SD2**	BEAM2	1–2–3	50	30		(1.19)	BEAM2	4–5–6	50	30
	(0.72)	BEAM3	4–5	50	30			BEAM3	7–8–9	45	25
		COL2	1–2–3	40	40			COL1	1–2–3	45	45
		COL3	4–5	35	35			COL2	4–5–6	40	40
6	**5SD3**	BEAM2	1–2–3	50	30			COL3	7–8–9	35	35
	(0.64)	BEAM3	4–5	50	30	11	**9SD2**	BEAM1	1–2–3	55	30
		COL2	1–2–3	45	45		(1.04)	BEAM2	4–5–6	55	30
		COL3	4–5	40	40			BEAM3	7–8–9	50	30
7	**7SD1**	BEAM1	1–2–3	50	30			COL1	1–2–3	50	50
	(1.06)	BEAM2	4–5	50	30			COL2	4–5–6	45	45
		BEAM3	6–7	45	25			COL3	7–8–9	40	40
		COL1	1–2–3	40	40	12	**9SD3**	BEAM1	1–2–3	60	30
		COL2	4–5	35	35		(0.93)	BEAM2	4–5–6	55	30
		COL3	6–7	30	30			BEAM3	7–8–9	50	30
8	**7SD2**	BEAM1	1–2–3	55	30			COL1	1–2–3	55	55
	(0.89)	BEAM2	4–5	50	30			COL2	4–5–6	50	50
		BEAM3	6–7	50	30			COL3	7–8–9	45	45

Comparatively high first natural period values especially for the ones designed according to D1 spectrum with a maximum acceleration value of 0.36 g are due to the models without infill walls and cross sections determined according to the minimum required quantities to reflect local constructional characteristics. As described previously, most of the structures in Turkey are weak and have inadequate earthquake resistance. So, they are flexible and tend to deform excessively due to lateral loads.

4. FRAGILITY ANALYSIS

Fragility assessment requires definition of risk and determination of the hazard due to the risk. Structural fragility due to the earthquake phenomena is expressed by investigating together the randomness of the earthquake and the uncertainty of the structural response. Accomplishing such an aim is possible by employing fragility curves because this type of assessment enables the visualization of whole range of structural response from slight or no damage to the collapse state with respect to demand parameter.

To evaluate fragility curves, analytical tools are employed. The capacities of the selected models are determined using pushover analysis and the demand statistics are obtained through time history analysis. Material inelasticity and geometric nonlinearity is taken into account during time history analysis. The analysis platform is selected as IDARC–2D [Valles et al., 1996].

4.1 Ground Motion Selection and Characterization

Measuring the intensity of ground motions during earthquakes is a major concern in earthquake engineering because semantically intensity expresses the damaging effect of ground motions on structures. However there is no consensus in the earthquake engineering community on an objective measurement of ground motion intensity. In seismic resistant design the commonly accepted approach is to express the intensity of design ground motions in terms of their PGA and an acceleration response spectrum shape anchored to the PGA at zero period value, which can be adjusted with respect to the local site conditions. Although this is practical for the design of ordinary structures, lessons learned from past earthquakes are that PGA is usually insufficient in explaining the spatial damage distribution during a severe earthquake. Furthermore, the acceleration response spectrum does not reflect the duration of ground motion which is directly related to the accumulation of damage in structures [Sucuoğlu et al., 1999].

PGV indicates the acceleration cycle with maximum energy. PGV and PGA do not necessarily occur during the same ground vibration cycle. According to the Newmark–Hall approach to seismic spectra, PGV primarily influences the seismic spectral response of medium period systems, approximately in the period range $0.5<T<2.0$ seconds [Sucuoğlu et al., 1999]. Thus, ground motions for design and analysis are selected and categorized according to the PGV values.

As clarified above, selection of the major intensity parameter as PGV results in a ground motion data–base with 60 records with different faulting mechanisms, magnitudes from 5.5 to 7.6, PGA, and PGD values. Some of the earthquake records processed are from Imperial Valley (1979), and Northridge (1994), from California; Chi–Chi (1999), from Taiwan; Kocaeli (1999), and Duzce (1999) from Turkey.

4.2 Determination of Building Classes as Poor vs. Superior

Aiming to reflect the Turkish construction state requires classification of the structures according to their condition as poor or superior. Such subclasses define the characteristics and construction qualities of the buildings. The conceptual expressions of those are:

Superior Structures: These structures are designed according to the current codes and have adequate structural capacity. Good material quality, earthquake resistant design and good supervision in the construction stage result in reliable performance levels.

Poor Structures: Unfortunately a considerable fraction of buildings in Turkey fall into this category. They are not designed to resist earthquake loads nor are they even engineered structures. Recent earthquakes in Turkey revealed that these types of structures are extremely vulnerable in seismic action. Most of the deficiencies stated previously are present in these structures.

4.3 Determination of the Material and Structural Parameters of Analytical Models

Surveys in Turkey reveal that there is excessive variation in material properties and construction qualities. To reflect this manifest in the fragility curves, the determination of the material and structural properties is required. Hence, major material parameters that effect structural response directly are designated as concrete strength f_c, steel yield strength f_y, elasticity modulus of concrete E_c and elasticity modulus of steel E_s.

To define material properties, mean characteristic values, coefficient of variations (COV) and distributions are determined. The variables stated above are assumed to have normal distribution and statistical parameters are determined referring to the study of Düzce (1999) and Kocaeli (1999) earthquake databases plus the previous studies related with the material variability [Mosalam et al., 1997; Ghobarah et al., 1998]. The assumed mean and COV values are listed in Table 2. Elasticity modulus of concrete is obtained using Equation (1) and elasticity modulus of steel is assumed as constant [Mosalam et al., 1997; Mirza and MacGregor, 1979] as seen in Table 2.

$$E_{c,mean} = 57\sqrt{1000\ f_{c,mean}} \qquad \text{(in ksi)} \tag{1}$$

Table 2: Material Properties for Structural Subclasses

CLASS	f_c		f_y		E_c		E_s	
	Mean (MPa)	COV	Mean (MPa)	COV	Mean (MPa)	COV	Mean (MPa)	COV
SUPERIOR	20	0.16	480	0.10	21150	0.08	200000	0.03
POOR	10	0.20	250	0.12	16400	0.10	200000	0.05

Besides material variability, mass and damping variability are taken into consideration. Mean values are determined as 46.5 ton and 5%, whereas COV values are taken to be 0.10 and 0.30, respectively.

4.4 Hysteretic Parameters of Analytical Models

Since the hysteretic behavior of structural elements is one of the major aspects of analytical modeling to achieve the required accuracy in the seismic response of building structures, IDARC–2D includes many different types of hysteretic response curves. In this study, the piece–wise linear hysteretic model which incorporates stiffness degradation, strength deterioration, non–symmetric response, and slip–lock is used to simulate the cyclic response of beams and columns. There exist four major parameters that characterize the hysteretic response in the model. These are stiffness degradation parameter α, ductility based strength degradation parameter β_1, hysteretic energy based strength degradation parameter β_2, and slip parameter γ.

The seismic response characteristics of the RC structures in each subclass heavily depend on the values of the hysteretic model parameters. In this study, the selection of these values is based on the recommended values by IDARC–2D and also on the experimentally observed behavior of the column specimens tested under cyclic loading, taken from the PEER Structural Performance Database (*http://nisee.berkeley.edu/spd/*).

For superior building class, the structural members are assumed to exhibit no degradation. Hence the default values of the parameters (α = 200, β_1 = 0.01, β_2 = 0.01 and γ = 1.0) are employed to simulate the cyclic response of structural members in the superior building subclass.

For poor building subclass, the structural members are assumed to exhibit severe degradation and pinching. Referring to the recommended values and experimental observations, the values selected for the hysteretic response of the structural members in this subclass are α = 5, β_1 = 0.5, β_2 = 0.5 and γ = 0.3.

4.5 Sampling

Assessment of the seismic structural behavior often involves uncertainties due to the variability in material qualities and random nature of earthquakes. Because of these variables and indefinite conditions, to reflect the whole population generally, it is required to execute sampling methods. Latin Hypercube Sampling (LHS) Method [McKay et al., 1979] is employed in this study to include the material variability in the models. LHS enables one to obtain random samples from all the ranges of possible values by providing a constrained sampling approach instead of random sampling. So, it requires less sample size than other alternative sampling methods in order to achieve the required accuracy. Sample size in this study is selected as 20, and each probability density function of normally distributed variable is divided into segments having a probability of 0.05. Since it is applicable to multiple variables, for both subclass of structures, 6 random variables as concrete strength, steel yield strength, elasticity modulus of concrete, elasticity modulus of steel, story mass and damping ratio are considered and sampling matrices are obtained with 6×20 elements as input data.

4.6 Definition of Limit States

Limit states, and thus performance levels, play a significant role in the construction of the fragility curves. Well–defined and realistic limit states are of paramount importance since these values have a direct effect on the fragility curve parameters [Erberik and Elnashai, 2004].

Three performance levels are defined as immediate occupancy (IO), life safety (LS), and collapse prevention (CP). By these boundaries, four damage states as minimum or no damage (DS1), significant damage (DS2), severe damage (DS3) and collapse (DS4) are introduced.

To determine the limit states, nonlinear static (pushover) analysis is used. In this study, limit states are taken deterministic to represent the capacity of the structure. The method based on stiffness reduction index (SI) developed by DiPasquale and Çakmak [1987] is employed. SI is calculated as given in Equation (2). Here, K_i is the secant stiffness of pushover curve at any time and K_o is the initial stiffness.

$$SI = 1 - \frac{K_i}{K_o} \tag{2}$$

For IO and LS performance levels, SI is accepted as 0.2 and 0.5 respectively in accordance with the past studies [DiPasquale and Çakmak, 1987 ; Reinhorn et al., 1992]. CP limit state is determined using the last reasonable point of the pushover curve as the indicator of collapse or unstability. Eventually, for collapse prevention state, SI takes values usually between 0.75 and 0.9.

According to the performance criteria stated above, inter–story drift ratios of the first story that is accepted as the critical one, are calculated as capacity parameter. Relevant inter–story drift ratios at defined performance levels for different structural subclasses are shown in Table 3. Comparing these values with the ones obtained in previous studies reveals that relatively reasonable results have been obtained [Rossetto and Elnashai, 2003; Erberik and Elnashai, 2004; Dymiotis et al., 1999; Kwon and Elnashai, 2004; Ghobarah, 2004].

Table 3: Inter–story Drift Ratios for Performance Levels

PERFORMANCE	Inter–story Drift Ratio (%)					
LEVEL	IO	LS	CP	IO	LS	CP
CLASS	SUPERIOR			POOR		
3 STORY	0.61	1.34	5.53	0.47	0.99	2.49
5 STORY	0.53	1.13	4.17	0.32	0.6	1.81
7 STORY	0.34	0.62	3.32	0.24	0.43	1.87
9 STORY	0.28	0.49	2.58	0.2	0.33	1.2

4.7 Evaluation of Fragility Curves

Analytical models for superior and poor structural subclasses with story number of 3, 5, 7, and 9 are subjected to time history analyses and the results are obtained in terms of maximum inter–story drift ratio (MIDR). PGV and MIDR are used to get the hazard vs. demand relationship. At all PGV levels, 20 MIDR values of structural simulations are obtained using IDARC–2D [Valles et al., 1996].

At each PGV value, MIDR are accepted as normally distributed and expressed by mean and standard deviation. The probability of exceeding each limit state is calculated. Obtained conditional probabilities are plotted with respect to PGV values as the demand parameter. Finally, to visualize plotted data graphically lognormal distribution is fitted to these data points.

Figure 2 and Figure 3 show the fragility curves of 3, 5, 7, and 9–story analytical models for poor and superior structural subclasses, respectively.

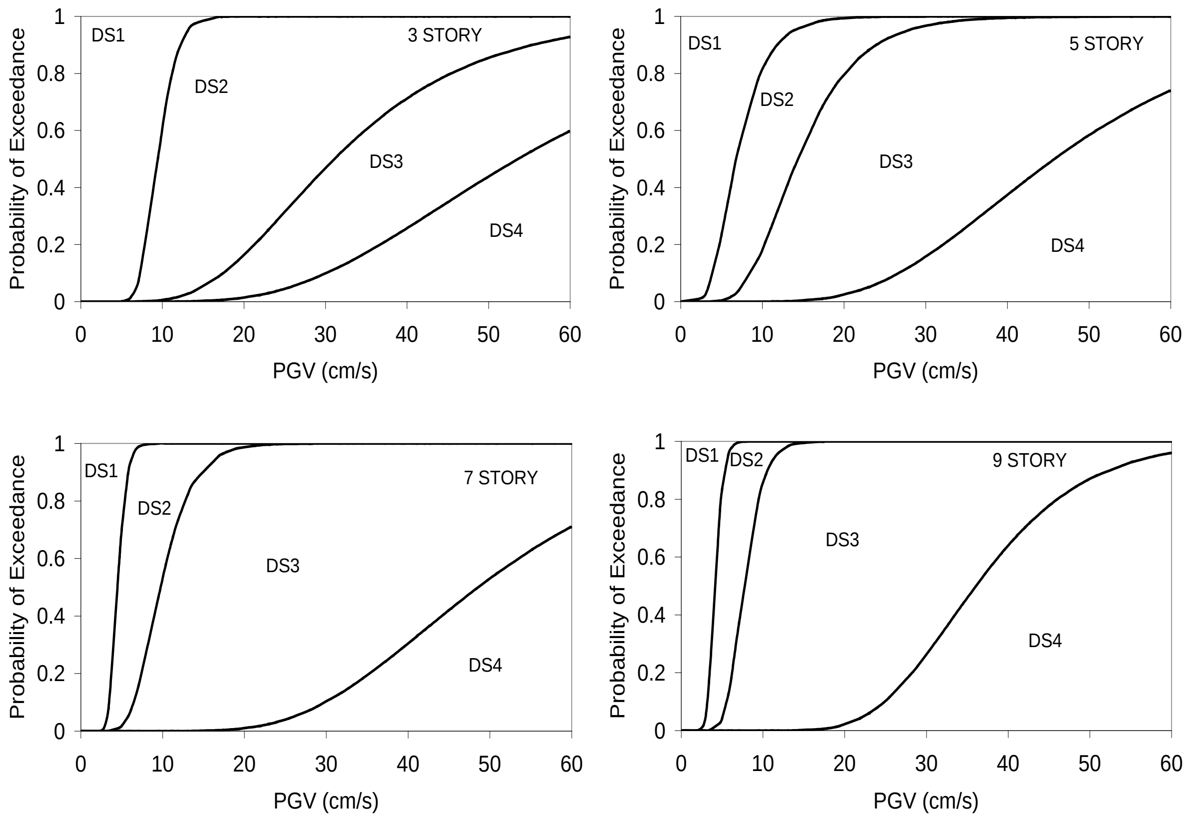

Figure 2: Poor Subclass Fragility Curves

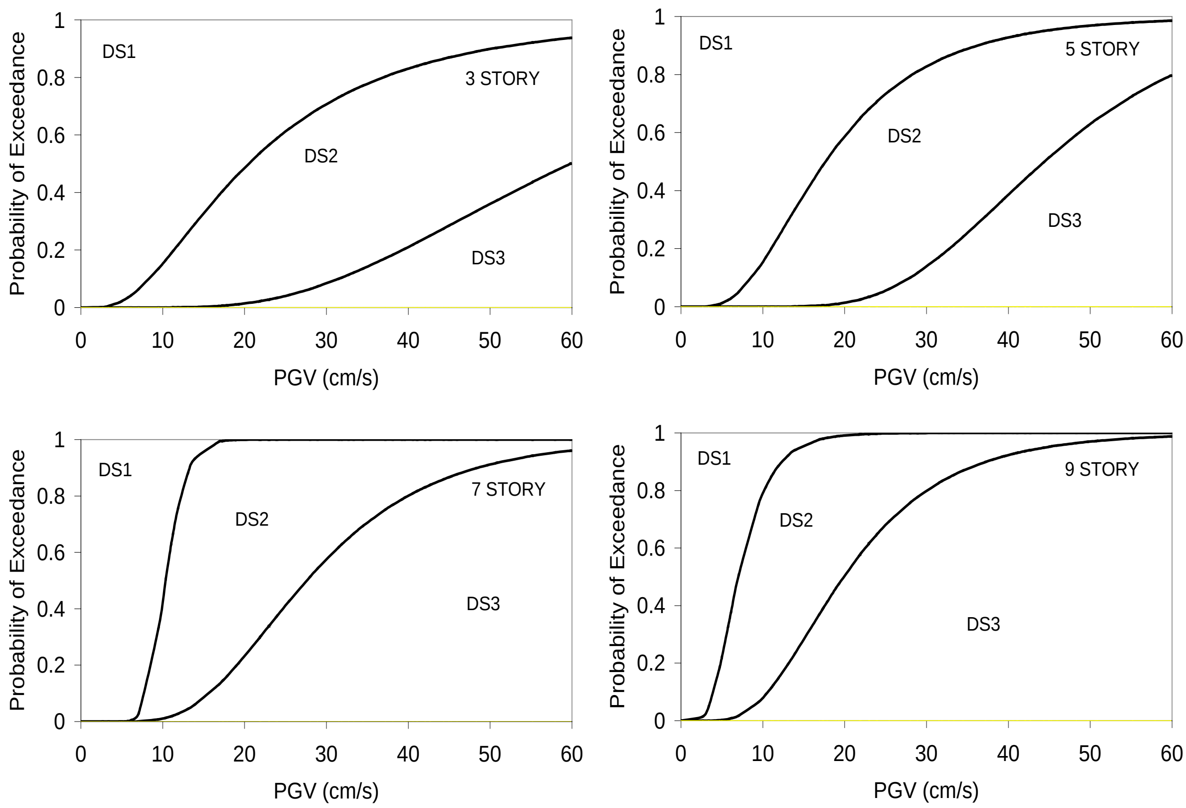

Figure 3: Superior Subclass Fragility Curves

For superior subclass structures, it is observed that, collapse prevention limit state does not exist. Since these structures are well designed and code requirements are fully satisfied, the probability of collapse is found negligibly small. Observation of poor subclass structures show that, as the number of stories increase, first and second damage limits get closer. The reason of that is the rapid evaluation of damage after initiation. In other words, the tolerance of structural resistance between no damage state and severe damage state is low and the structure reaches the collapse state rapidly.

5. CASE STUDY

Final phase of the study is devoted to the comparison of damage state probabilities of the considered RC frame structures for specific levels of hazard intensity, with an emphasis on the subclass definitions and number of stories. The PGV values that represent the hazard intensity are selected as 40 and 60 cm/s. Although the selected values are arbitrary, they confirm with the published work for a scenario earthquake of M = 7.5 in Istanbul [JICA, 2002]. According to this report, the PGV values in the western Marmara Coast, at a distance of 10 to 15 km from the major Marmara Sea segment of the North Anatolian fault, are expected to vary between 50 and 60 cm/s during the scenario earthquake.

Figure 4 and Figure 5 show the damage state probabilities of 3, 5, 7, and 9–story structures comparing superior and poor subclasses for PGV values of 40 and 60 cm/s, respectively. The damage state probabilities shift from low to high levels of damage in the case of poor structures compared with the superior counterparts. Especially for high PGV values, this distinction is much more pronounced. Overall, the inherent characteristics of considered RC structures which were reflected in fragility curves (degrading behavior, rapid evaluation of damage after initiation, etc.) can also be observed through damage state probabilities.

As seen from Figure 6 and Figure 7 the damage state probability increases with the story number for both superior and poor structural subclasses. So, structures with more number of stories but of same subclass subjected to same ground motion intensity (in terms of PGV) seem to be more vulnerable in seismic action. Such kind of a trend has been observed before by other researchers [Aydoğan, 2003; Akkar et al., 2005]. This result verifies that the local structural characteristics have been successfully embedded into the analytical models.

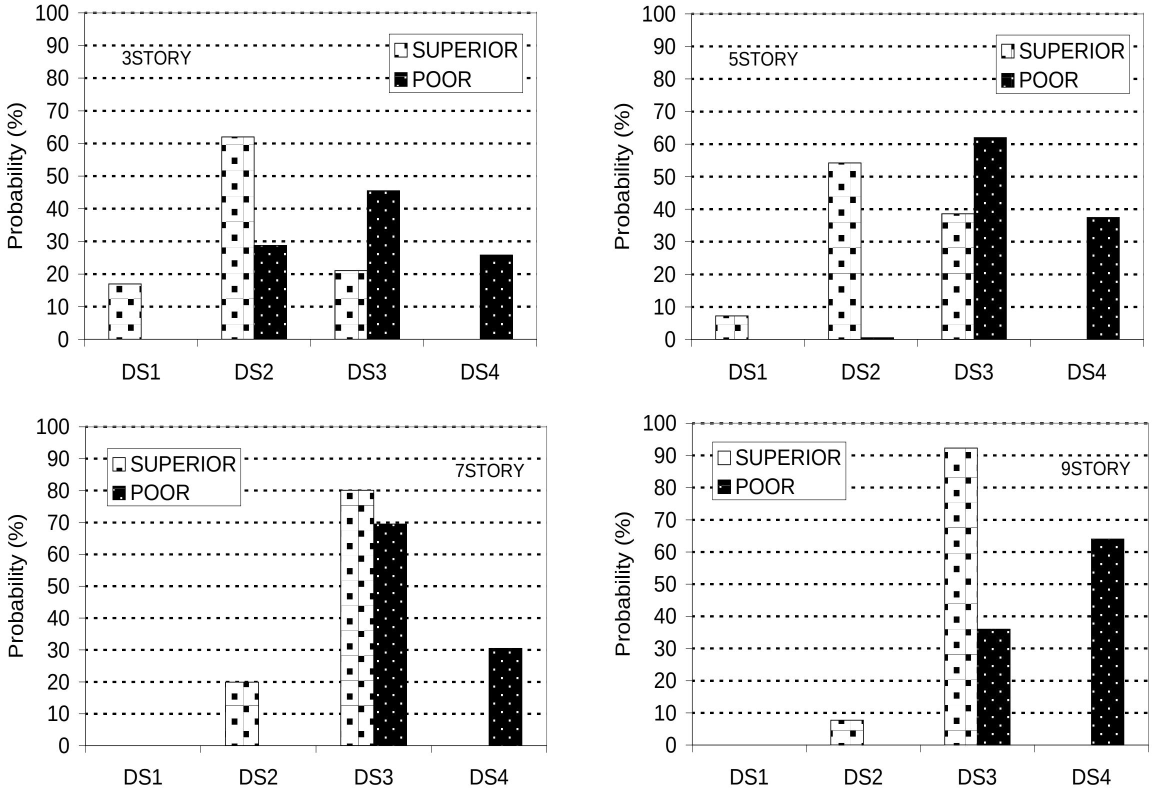

Figure 4: Damage State Probability of Superior and Poor Subclasses for PGV = 40 cm/s

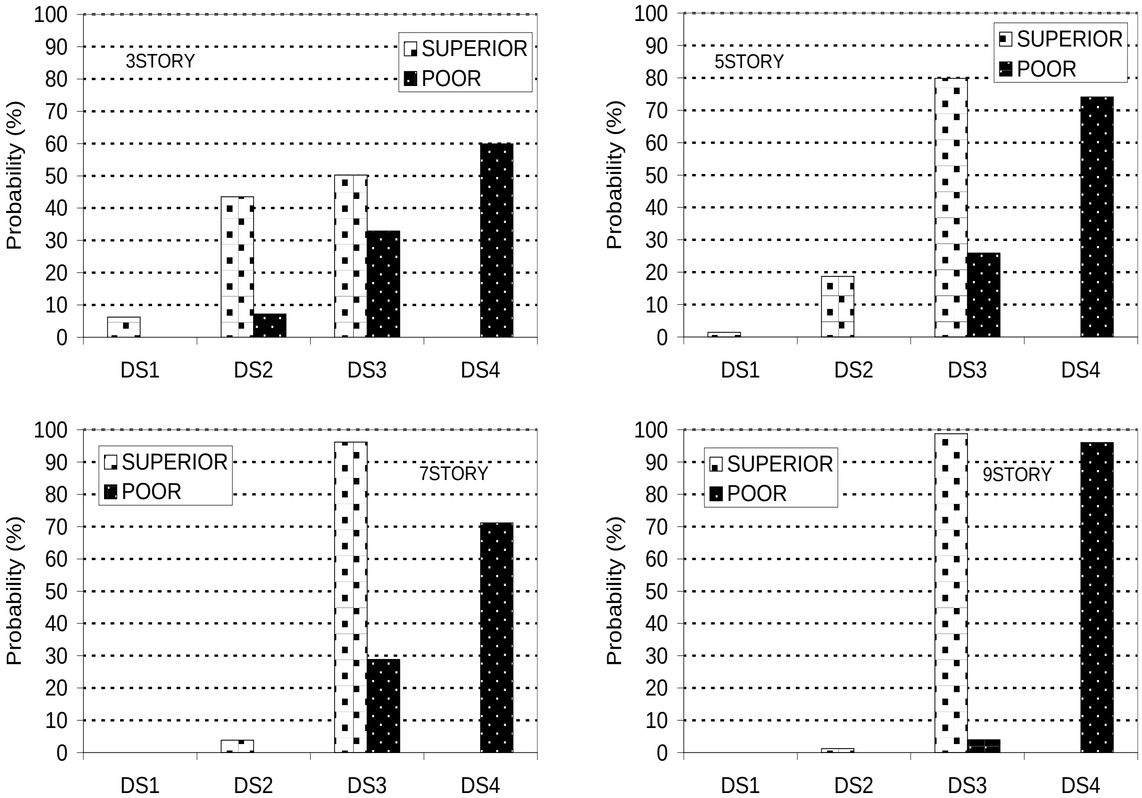

Figure 5: Damage State Probability of Superior and Poor Subclasses for PGV = 60 cm/s

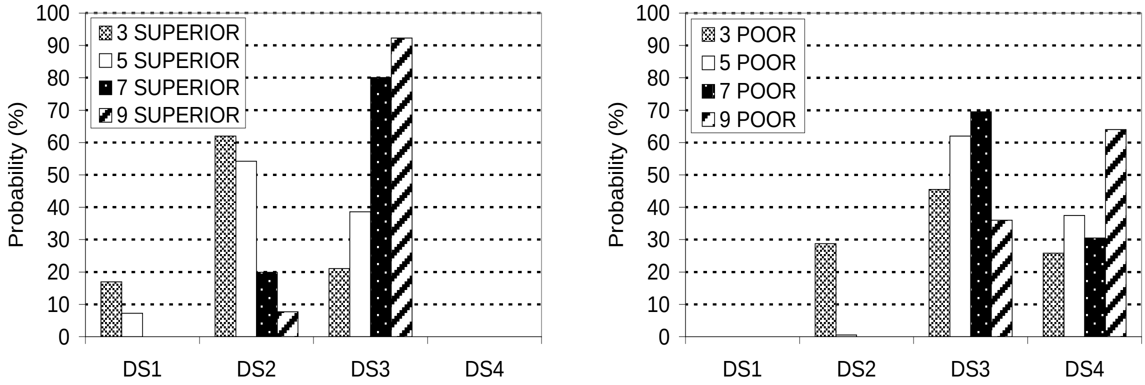

Figure 6: Story Based Damage State Probability for PGV = 40 cm/s

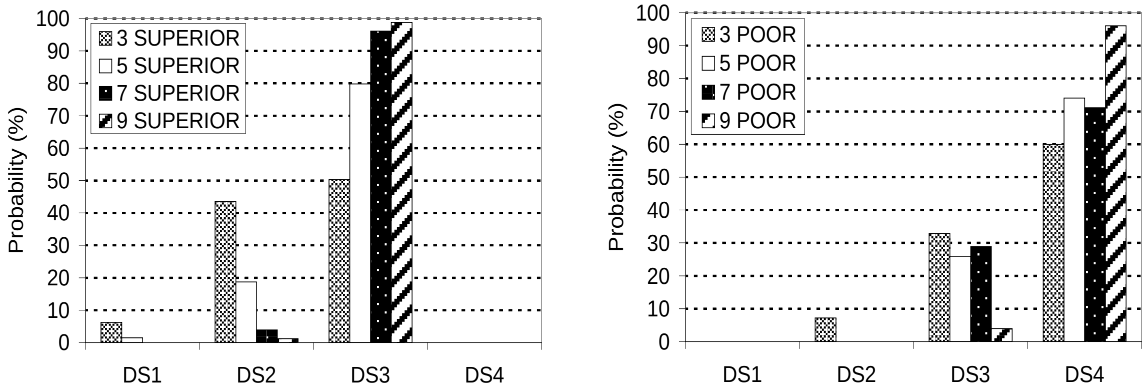

Figure 7: Story Based Damage State Probability for PGV = 60 cm/s

6. RESULTS & CONCLUSION

Determination of structural fragility considering local construction practice and building inventory is vital for assessing earthquake hazard and managing a disaster. As a consequence, seismic vulnerability of low–rise and mid–rise RC frame structures, which constitute approximately 75% of the total building stock in Turkey and which are generally occupied with residential purposes are examined in this study through fragility analysis.

Fragility curves are generated using planar analytical models and inelastic time history analyses. The curves are expressed in terms of different structural categories with respect to the dominant structural parameters influencing their seismic performance. These parameters are number of stories and structural deficiencies encountered in both design and construction stages. Hence the seismic performances of RC frame buildings are assessed in terms of number of stories and superior/poor building subclasses, using fragility as a tool.

The damage state probabilities reflect the inherent characteristics of the considered building class. Number of stories seems to be a critical parameter for seismic vulnerability of the considered buildings. Furthermore, structural deficiencies that are typical to Turkish construction practice lead to poor seismic performance. The structures in the poor building subclass exhibit either low damage or high damage, with little margin in between. Hence rapid increase in damage accumulation due to insufficient stiffness and strength characteristics are reflected both in fragility curves and damage state probabilities.

Overall, the typical characteristics of Turkish low–rise and mid–rise RC frame buildings are reflected in the generated fragility curves. Therefore, the curves can be employed for estimation of damage and losses in risk scenarios involving earthquake prone regions in Turkey.

7. ACKNOWLEDGEMENT

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