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Vulnerability of Turkish Low-Rise and Mid-Rise Reinforced Concrete Frame Structures

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In this study, seismic safety of low–rise and mid–rise structures, which constitute approximately 75% of the total building stock in Turkey, is investigated by generating fragility curves. The scope is 3, 5, 7, and 9–story reinforced concrete moment resisting frame structures. The uncertainties in material variability and the specific characteristics of construction practice in Turkey are taken into account in the formation of structural simulations. Two-dimensional analytical models are constructed accordingly and categorized as poor, typical, or superior corresponding to the observed seismic performance of structures after major earthquakes in Turkey. The seismic demand statistics in terms of maximum interstory drift ratio are obtained for different sets of ground motion records by performing nonlinear time history analyses. The capacity is determined in terms of limit states and the corresponding fragility curves are obtained from the probability of reaching or exceeding each limit state for different levels of ground shaking. The generated fragility curves are employed in a preliminary evaluation application. For this purpose, Fatih, a highly populated earthquake–prone district in Istanbul, is selected. This study attempts to be a benchmark for future fragility based studies on earthquake damage and loss estimation in urban areas of Turkey.

Keywords Fragility Curve; Material Variability; Seismic Safety

1. Introduction

Assessment of earthquake hazard requires determination of the risk and evaluation of the structural vulnerability. Vulnerability studies involve consideration of local structural properties and investigation of fragility information accordingly. However, local conditions are usually ignored and vulnerability based assessment studies for structures in different countries are adapted to earthquake hazard estimation projects in Turkey. Unfortunately, differences in country specific structural characteristics cause significant deviations on damage and loss estimations. The aim of this study is to provide fragility information of low–rise and mid–rise reinforced concrete (RC) structures, which constitute majority of the total building stock in Turkey, by using most recent analysis methods and comprehensive database that has been gathered after the devastating earthquakes that occurred within the last decade in Turkey.

2. Design and Analysis Considerations of Building Models

To characterize different levels of seismic hazard, three different ground motion sets are selected. Each set contains 20 ground motion records. Peculiar records due to extreme near–fault (pulse–dominant) wave forms and very soft soil site effects are not included.

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TABLE 1 Statistical properties of ground motion sets

	Set I		Set II		Set III	
	Mean	COV (%)	Mean	COV (%)	Mean	COV (%)
M_w	6.2	7	6.6	6	6.9	6
D (km)	12.3	38	10.4	67	10.4	56
PGA (g)	0.16	35	0.34	31	0.44	38
PGV (cm/s)	11.17	49	29.14	22	48	13

Grouping of the records are based on peak ground velocity (PGV) values which ranges between 0–20, 20–40, and 40–60 (in cm/s). The mean values of moment magnitude (M_w), closest distance to fault (D), peak ground acceleration (PGA), PGV and corresponding coefficient of variation (COV) values as a measure of dispersion are listed in Table 1.

In Turkish Seismic Code, the design spectrum is based on four seismic zones obtained from broadly described geological conditions. However, for vulnerability studies, design spectrum that depends on site–distance–magnitude parameters is more convenient [Kalkan and Gülkan, 2004]. Hence, the median pseudo acceleration spectra are computed for 3 sets of ground motions, each containing 20 records. Then corresponding smooth design spectra are obtained using the procedure in FEMA 356 [ASCE, 2000].

Moment resisting frames are frequently used structural systems in Turkish construction practice. Number of stories is considered as the major parameter in this study. Hence 3, 5, 7, and 9–story planar frame models are developed. Story height of 3 m and bay width of 5 m are assumed in accordance with the common practice. For longitudinal reinforcement design using each spectrum obtained, SAP-2000 is employed. It is worth to mention that SAP-2000 design results agree with requirements in codes used in Turkey due to some basic similarities such as minimum longitudinal reinforcement ratio and loading factors.

This study uses analytical methods to evaluate fragility curve information. The analytical models designed are changed as the imposition of structural, material, and detailing deficiencies in accordance with the Turkish construction practice. Then the generic models are analyzed and response statistics as non-negative scalars are obtained. To calculate the structural response and capacity of buildings under consideration, the analysis program IDARC–2D [Valles *et al*., 1996] is used. A severe limitation due to modeling capabilities of the analysis program is that only flexural failure of structural members is considered. The local shear failure in columns has not been considered explicitly, noting that such an assumption can have an influence, especially on the limit states obtained through pushover analysis.

3. Characterization of Building Classes

In order to reflect the RC frame construction in Turkey, the building stock considered is classified into three subclasses as poor, typical, and superior according to the inherent characteristics and deficiencies of construction practice and the observed post–earthquake seismic performance. The subclasses can be described in general qualitative terms as the following.

Superior Subclass: The buildings in this subclass are designed according to the current codes and have adequate structural capacity in terms of strength and ductility. Good material quality, earthquake resistant design, and good supervision result in reliable performance levels. This is the desired level of construction practice.

Typical Subclass: It represents the majority of the building stock concerning the RC residential buildings in Turkey. They are generally engineered structures but may violate some fundamental requirements of earthquake resistant design and construction.

Poor Subclass: These buildings are not designed to resist earthquake loads nor are they even engineered structures. Recent earthquakes in Turkey revealed that this type of structures is extremely vulnerable in seismic action. Most of the construction, detailing and design deficiencies are frequently observed in this subclass of RC buildings.

3.1. Material Properties and Hysteretic Parameters of Building Classes

Major material parameters that effect structural response 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). The variables are assumed to have normal distribution and statistical parameters such as mean and COV 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.

In determination of the steel yield strength mean values, it is assumed that Reinforcing Steel Type III (St–III) and Reinforcing Steel Type I (St–I) are used in superior and poor building subclass, respectively. Then, the mean yield strengths are determined by increasing the characteristic values by 15%. Finally, the mean steel yield strength value for typical building subclass is determined by using the actual data based on the tested coupons in METU structural laboratory [Erberik and Sucuoğlu, 2004].

In this study, piece–wise linear hysteretic model of IDARC–2D that incorporates degradation characteristics is used to simulate the cyclic response. For superior building subclass, the structural members are assumed to exhibit no degradation. There is a stable behavior with high energy dissipation characteristics and members exhibit degradation neither in stiffness nor in strength. In case of typical building subclass, the structural members are assumed to exhibit slight–to–moderate degradation. The strength at the maximum displacement slightly decreases with the number of cycles as similar as the area enclosed by the hysteresis loops. For poor building subclass, the structural members are assumed to exhibit severe strength degradation and there is a considerable amount of pinching in the analytical model, which narrows the area enclosed by the loops and reduces the dissipated energy significantly.

4. Fragility Analysis

Fragility represents the probability that the response of the structure exceeds the prescribed limit state for a given hazard intensity. Most common way of obtaining fragility

TABLE 2 Material properties for structural subclasses

	f_c		f_y		E_c		E_s	
Structural Subclass	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
Typical	15	0.18	365	0.11	18950	0.09	200000	0.04
Poor	10	0.20	250	0.12	16400	0.10	200000	0.05

curves is to use analytical models and structural simulations. Detailed models and finite element programs should be employed to obtain the response of a structure and damage distribution. Hence, analytical models and nonlinear time history (NTH) analyses of two-dimensional multidegree of freedom (MDOF) systems are employed in this study.

Among different ground motion parameters, PGV has been selected in this study. According to some recent studies [Akkar and Özen, 2005; Akkar and Bommer, 2007] on selection of ground motion intensity parameter for earthquake hazard estimation studies, PGV correlates well with the earthquake magnitude and ground–motion frequency content and provides useful information about the strong–motion duration that can play a role on the seismic demand of structures.

Assessment of the seismic structural behavior involves uncertainties because of the variability in material qualities and random nature of earthquakes. Hence, sampling methods should be used in order to reflect the whole population in general. Latin Hypercube Sampling (LHS) Method [McKay *et al.*, 1979] is employed in this study to include the material variability and structural uncertainty 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. Considering the advantage of using LHS method, the sample size is chosen as 20.

Realistic and comprehensive limit state determination and thus performance level identification is one of the significant steps of fragility curve construction because these indicators affect resulting fragility curves directly [Erberik and Elnashai, 2004].

In this study, three limit states are defined as Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). IO limit state should represent slight or no damage whereas LS limit state may imply significant damage, but with an adequate margin against collapse. Finally, CP limit state corresponds a little margin against collapse. In accordance with three limit states, four damage states are introduced as slight or no damage (DS1), significant damage (DS2), severe damage (DS3), and collapse (DS4).

Different criteria are employed in the specification of limit states in terms of maximum interstory drift ratio (MIDR) values. The first criterion is to examine the progressive damage accumulation in the structure by monitoring the local performance stages. Firstly, consecutive yielding in beams induces significant reduction in the overall stiffness. Hence, this abrupt change may be regarded as the onset of significant damage or, IO limit state. Thereafter, yielding in columns, which is an indicator of a reduction in the lateral strength capacity is regarded as the transition from significant to severe damage, or LS limit state. Finally, as damage propagates, due to yielding in many members and even failure in some of them, the collapse mechanism initiates. This transition from severe damage to collapse can be identified as CP limit state. The second criterion in limit state determination is the softening index (SI) which was originally proposed by DiPasquale and Çakmak [1987]. SI takes values between 0 and 1 regarding the amount of stiffness change due to inelastic action. It is observed that in the vicinity of consecutive yielding in beams, SI generally takes values between 0.10 and 0.20 whereas in the vicinity of LS limit state, the SI takes values between 0.45 and 0.55 depending on the structural subclass. Finally, in the vicinity of collapse, the SI ranges between 0.70 and 0.85. The last criterion is the equivalent ductility capacity of the structure, which is obtained by the bilinearization of the pushover curves in accordance with FEMA 356 [ASCE, 2000]. The study conducted by Calvi [1999] specified the ductility values for IO and LS limit states. Accordingly, for buildings under consideration, the ductility values range between 1.1 and 1.8 in transition from slight–to–moderate damage for existing structures whereas in the case of well-designed buildings, the ductility values range between 1.35 and 1.91. Calvi also proposed displacement ductility limits in transition

TABLE 3 MIDR values (%) associated with limit states

Building Class	Maximum Interstory Drift Ratio (%)					
	Immediate Occupancy		Life Safety		Collapse Prevention	
	Lower Limit	Upper Limit	Lower Limit	Upper Limit	Lower Limit	Upper Limit
MRF3–P	0.26	0.34	0.52	0.80	1.19	1.64
MRF3–T	0.35	0.47	1.17	1.75	2.41	3.22
MRF3–S	0.43	0.58	1.07	1.54	2.93	3.89
MRF5–P	0.20	0.26	0.38	0.49	0.85	1.43
MRF5–T	0.26	0.36	0.58	0.95	1.84	2.50
MRF5–S	0.36	0.46	0.86	1.28	2.70	3.47
MRF7–P	0.17	0.22	0.38	0.50	0.61	0.89
MRF7–T	0.18	0.25	0.54	0.72	1.03	1.62
MRF7–S	0.22	0.29	0.51	0.73	1.93	2.76
MRF9–P	0.16	0.21	0.32	0.40	0.54	0.69
MRF9–T	0.16	0.21	0.45	0.57	0.90	1.49
MRF9–S	0.18	0.25	0.48	0.62	1.78	2.68

from moderate–to–severe damage. For existing frame structures, a ductility value between 1.3 and 2.0, and for well-engineered structures a ductility value of 3 to 4 is stated. Regarding limit state in terms of ductility, it is worth considering the study conducted by Booth *et al*. [2004]. In this study, the values specified for ductility corresponding to LS range between 1.5 and 3.0 whereas for CP range between 3.0 and 6.0. The values are then converted to MIDR values that are listed in Table 3 where P, T, and S stand for poor, typical, and superior and MRF3–5–7–9 show the story number.

It should be noted that the first two criteria are based on the observations on the analytical models in this study. The third criterion is used for verification by considering studies on RC buildings both in Turkey and out of Turkey.

Considering the above discussions, the drift values suggested for limit states show a large scatter, especially for ultimate or CP limit state. Therefore, it is more appropriate to consider each limit state as a random variable that is assumed as uniformly distributed within lower and upper limits rather than a single–valued parameter to reflect the uncertainty in structural capacity to the final fragility curves.

To obtain the demand measures, analytical models of structural subclasses are subjected to NTH analyses and the results are obtained in terms of MIDR. Hence, at each PGV value corresponding to a record, 20 MIDR values are obtained as vertical scattered data reflecting the variability of structures. At each PGV value, MIDR is accepted as normally distributed and expressed by a mean and standard deviation value. These statistical parameters can be used to obtain the exceedance probabilities. The mathematical description of the exceedance probability is given in Eq. 1 where, DM and DL stand for demand measure and limit state value, respectively. $PE_{i,j}$ is obtained as probability of structural demand measure exceeds a limit state i at a specific PGV value j.

$$PE_{i,j} = P\left(DM \geq DL_i \middle| PGV_j\right) \tag{1}$$

For every PGV value the exceeding probabilities are obtained and fragility curves are constructed. To visualize plotted data graphically a best line is fitted using lognormal cumulative distribution function. Fragility curves obtained are given in Figs 1–3.

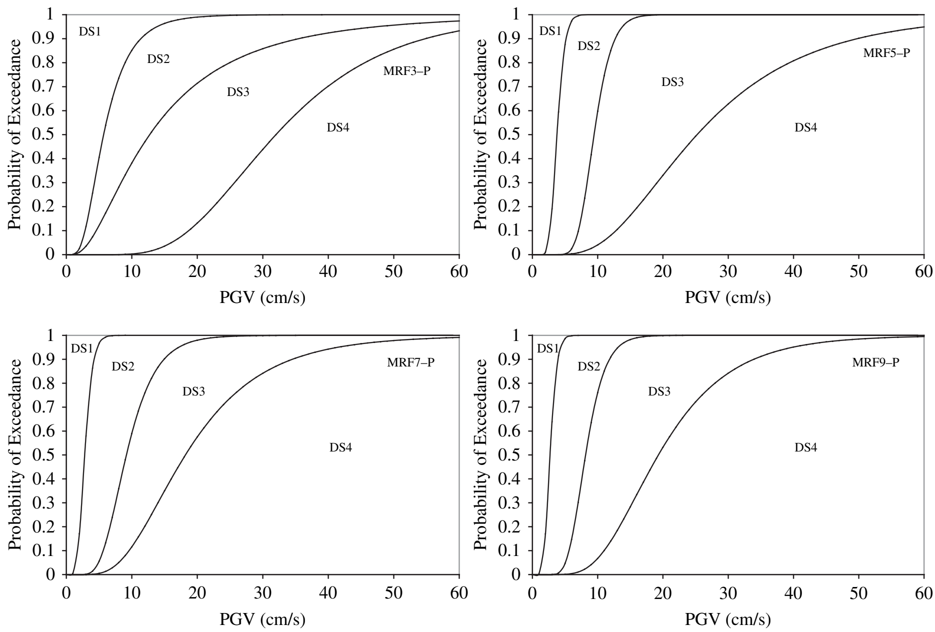

FIGURE 1 Poor subclass fragility curves.

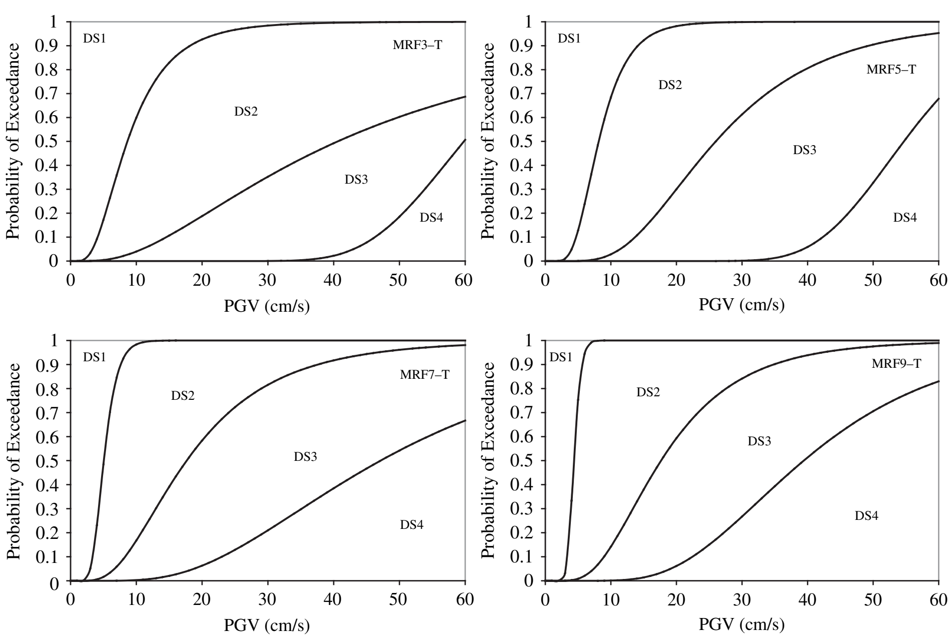

FIGURE 2 Typical subclass fragility curves.

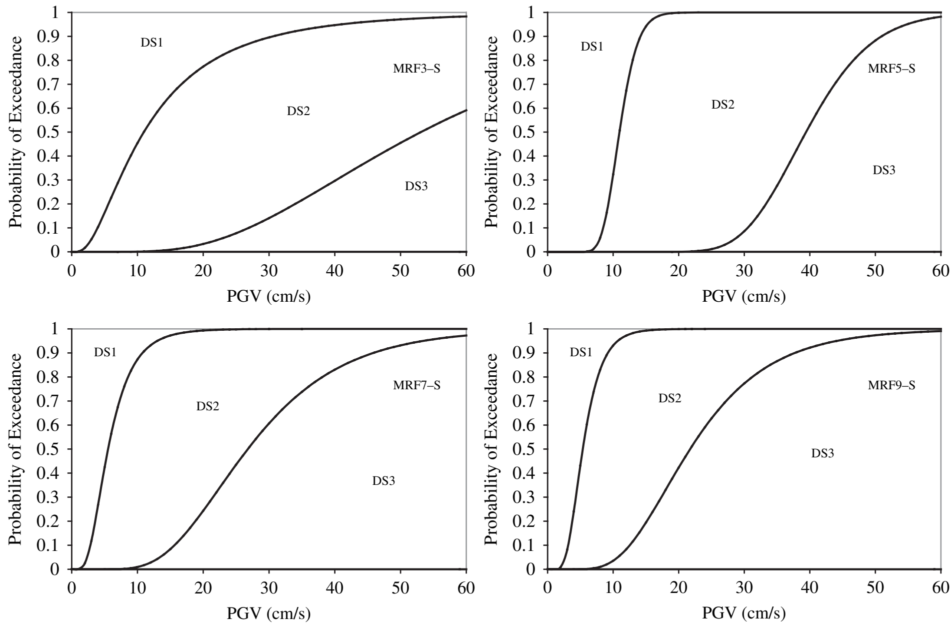

FIGURE 3 Superior subclass fragility curves.

According to poor subclass fragility curves, as the number of stories increase, first and second damage limits get closer because the tolerance of structural resistance between no damage state and severe damage state is low and the structure reaches the collapse state rapidly. This trend is consistent when compared to the observations by Booth *et al*. [2004] who stated that there is a little margin between low damage and high damage for Turkish RC frame structures with typical structural deficiencies.

As observed in fragility curves obtained for superior subclass, CP limit state does not exist. Since these structures are well designed and code requirements are fully satisfied, the probability of collapse damage state is found negligibly small within the PGV range considered. In most of the cases, seismic demand cannot exceed the ultimate limit state (capacity) even for high values of hazard intensity.

5. Preliminary Evaluation Application Using Fragility Information

To illustrate the use of the generated fragility information as a preliminary evaluation stage of a regional damage estimation study, an application is conducted. For this purpose, the study region is selected as Fatih, a highly populated earthquake–prone district in Istanbul. The building database in Fatih district has been already gathered for another project regarding the evaluation of seismic safety of existing building stock in Istanbul Metropolitan Area. In the project, a multi–level seismic evaluation method that is composed of walkdown survey, preliminary evaluation and detailed evaluation is employed for the existing buildings that have been developed within the scope of NATO Science for Peace project [Özcebe *et al*., 2003; Yakut *et al*., 2003].

The building inventory in Fatih provides information about 17,108 RC frame structures. Since the generated set of fragility curves is limited to RC moment resisting frames with 3, 5, 7, and 9 stories, the corresponding building data is extracted from the inventory.

TABLE 4 Number of buildings based on VS

VS	3P	3T	3S	5P	5T	5S	7P	7T	7S
$0.9 < VS \leq 1.0$	37	0	0	1548	0	0	879	0	0
$0.8 < VS \leq 0.9$	0	0	0	163	15	0	0	2	0
$0.7 < VS \leq 0.8$	0	31	0	20	0	0	0	484	0
$0.6 < VS \leq 0.7$	0	11	0	0	2611	0	0	69	0
$0.5 < VS \leq 0.6$	0	85	7	0	1816	10	0	2	0
$0.4 < VS \leq 0.5$	0	215	109	0	0	30	0	0	0
$0.3 < VS \leq 0.4$	0	6	197	0	113	16	0	0	0
$0.2 < VS \leq 0.3$	0	0	40	0	0	0	0	0	0
$0.1 < VS \leq 0.2$	0	0	0	0	0	0	0	0	0
$0 < VS \leq 0.1$	0	0	0	0	0	0	0	0	0

There exist 8,516 buildings in the scope of this study. It is worth to mention that there are 73 3–story, 6,342 5–story, and 1,436 7–story buildings. However, there is no 9–story building. Walkdown evaluation method is the first and the simplest level in seismic vulnerability analysis. The method does not require any analysis and its goal is to determine the priority levels of buildings that require immediate intervention [Sucuoğlu and Yazgan, 2003]. In walkdown evaluation method, a base score is evaluated for each building, which is used to assign subclasses for buildings. Accordingly, buildings with a score greater than 100 are assumed to be superior, whereas buildings with a score between 50 and 100 are assumed to be typical and buildings with a score less than 50 are assumed to be poor. Hence, 409 superior, 5,460 typical, and 2,647 poor buildings are found.

There exist three ingredients in damage estimation analysis, which is used as an alternative preliminary evaluation method for RC frame structures in this study; seismic hazard identification, building inventory, and the associated fragility information. For seismic hazard identification, other studies regarding seismic risk evaluation in Istanbul Metropolitan Area [Japan International Co–operation Agency and Istanbul Metropolitan Municipality, 2002] are referred. PGV is employed as the hazard parameter and a scenario earthquake with a return period of 72 years is selected. Then, the probability of being in damage states DS1, DS2, DS3, and DS4 are found using the on–site PGV values and, a single–valued vulnerability score (VS) is obtained by multiplying the damage state probabilities by the corresponding damage state multipliers. In this study, damage state multipliers are taken as 0 for DS1, 0.33 for DS2, 0.67 for DS3, and 1 for DS4. The VS of all the buildings in the Fatih inventory is calculated. The distribution of VS values is given in Table 4. Then it is possible to draw a line as seen in Table 4 to decide about the relative seismic safety of buildings such that the ones below the line are assumed as safe whereas the others are assumed as unsafe and transferred to final stage of evaluation.

6. Conclusions

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. Based on the assumptions and limitations of this study and the fragility curves obtained:

- In this study, the main parameters affecting structural fragility are considered as number of stories, structural deficiencies; which are quantified as superior, typical, or poor subclass, and ground motion intensity level.
- Probabilistic limit states are essential in fragility studies, in which the single–valued (deterministic) limit states cannot be obtained with much confidence and the quantification of limit states directly affect the resulting fragility.
- The generated fragility curves are novel in the sense that such curves have been developed for Turkish RC building inventory with detailed analysis and tools (NTH analysis, MDOF models, material variability, and probabilistic limit states).
- Structural damage shifts from low to high levels with decreasing structural subclass quality. Especially for high PGV values, this distinction is much more pronounced. Besides, structural damage seems to increase with the number of stories for superior, typical, and poor building subclasses. As the number of stories increase, first and second damage limits get closer because the tolerance of structural resistance between no damage state and severe damage state is low and the structure reaches the collapse state rapidly. Overall, the inherent characteristics of considered RC buildings (degrading behavior, rapid evolution of damage after initiation, etc.) are reflected in generated fragility curves, and in turn, damage state probabilities.
- The generated fragility information can be employed as an alternative tool for conventional methods for the vulnerability or seismic safety evaluation analyses of actual RC frame structures and also in loss estimation studies.

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