Proposed Vulnerability Functions To Estimate The Real Damage State Of RC Buildings After Major Turkish Earthquakes

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SUMMARY:
This study focuses on the applicability of fragility relationships, which are employed to predict the seismic vulnerability of existing structures. Since these relationships offer the probability of exceeding a predefined structural response limit in terms of a ground motion intensity parameter, fragility functions are very practical tools to be employed during urban renewal of metropolitan cities with high seismicity. A building ensemble which experienced moderate damage after major Turkish earthquakes is considered herein this paper. Planar structural models for each building are established utilizing DRAIN-2DX computer program and nonlinear dynamic analyses are carried out. The demand parameters are obtained and the capacity is determined in terms of limit states. Finally, fragility relationships recently proposed by various researchers are employed for the building set and compared with the analytical results by means of reflecting the most reliable actual damage state.

Keywords: Earthquake damages, Fragility curves, Reinforced concrete buildings, Damage estimation

1. INTRODUCTION

Fragility curves are used to estimate the probable damage for an individual building as well as building stocks; hence considering the difficulty on making a decision about the whole building stock, they are easily and fastly used in metropolitan cities with high seismicity. Although a fragility curve offers the probability of reaching or exceeding a level of damage under a ground motion intensity parameter, it is observed that the level of damage state can be related to the structural capacity and demand. Other than the structural system properties, however, the post-earthquake damage state of a building is dependent to many parameters including local site conditions; structural material qualities; adequacy of the workmanship and detailing of reinforcement.

Low- and mid-rise reinforced concrete (*RC*) frame structures constitute approximately 75% of the total building stock in Turkey. Most of this building stock is generally composed of non-engineered structures which have not been adequately designed according to the earthquake code regulations. Devastating earthquakes within the last decades proved that structures, especially those constructed prior to the Turkish Earthquake Resistant Design Code of Turkey (TERDC) of 1975, have experienced severe damages and partial or even total collapse, mainly depending on poor structural material quality, inadequate reinforcement detailing, lack of confinement zones, heavy and large-span cantilevers and indirect supporting preventing the formation of regular structural frames. Therefore, the damage to buildings from recent earthquakes occurred in Turkey has emphasized the need for risk assessment of existing building stock to estimate the potential damage from future earthquakes.

In cases of urban renewal, damage or loss estimation and retrofitting decisions, fragility curves offer the probability of exceeding a predefined structural damage limit due to earthquakes in terms of a ground motion intensity parameter: e.g, peak ground acceleration or velocity (*PGA* or *PGV*), elastic spectral acceleration or displacement (S_a or S_d). Existing vulnerability functions can be classified into the four generic groups of empirical, judgmental, analytical and hybrid according to whether the damage data used in their generation derives mainly from observed post-earthquake surveys, expert opinion, analytical simulations or combinations of these (Rosetto and Elnashai, 2003). Analytical fragility relationships can be established either by employing a set of nonlinear dynamic or pushover analyses.

There exist only a few attempts to estimate the seismic vulnerability of Turkish RC buildings by the use of fragility curves (Akkar et al. 2005; Kircil and Polat, 2006). In order to investigate the applicability of these proposed vulnerability functions for a typical building stock in Turkey, nonlinear dynamic analyses are carried out for a set of low- and mid-rise RC frame structures in this study. Time history analyses are carried out utilizing DRAIN-2DX (Prakash et al. 1993) computer program. Planar structural models for these buildings, which experienced moderate damage after major earthquakes occurred in Turkey, are established considering the contribution of non-structural infill walls. 7 simulated ground motions compatible with the design spectrum for 0.40*g* effective acceleration coefficient and *Z2* soil class defined in the TERDC-2007 is employed during the analyses. The demand parameters in terms of maximum inter-story drift ratio, story shear forces and base shear ratio are obtained for the earthquake ensemble consisting artificial records and the capacity is determined in terms of limit states. Finally, fragility relationships recently proposed by various researchers as in DiPasquale and Cakmak, (1989); Erberik and Elnashai, (2004); Kircil and Polat (2006) are employed to the building set and compared with each other and the analytical results as well by means of reflecting the most reliable actual damage state.

2. EARTHQUAKE ENSEMBLE AND INVESTIGATED RC BUILDINGS

2.1. Earthquake Ensemble

Utilizing TARSCTHS computer program (Papageorgiou et al. 2000), which is capable of simulating earthquake motions compatible with a defined spectrum, an earthquake ensemble consisting of 7 acceleration-time histories are generated. According to the TERDC-2007, if a set of 7 strong motions or more are used, then the mean values of the structural responses are permitted to represent the structural demands. Either recorded or simulated earthquake motions should satisfy the following conditions:

- ✓ The duration of strong motion acceleration record shall neither be less than 5 times the first natural vibration period of the building nor less than 15 seconds.
- ✓ Mean value of spectral acceleration for each recorded or simulated acceleration record with 5% damping ratio at zero period (T=0) shall not be less than the spectral acceleration $A_0 \times g$; where A_0 is the effective ground acceleration coefficient and g is the acceleration of gravity.
- ✓ Mean of the spectral acceleration values computed for each recorded or simulated acceleration record with 5% damping ratio within a period range of $0.2T_1 \sim 2T_1$, where T_1 is the dominant vibration period of the structure, shall not be less than 90% of the elastic spectral acceleration $S_{ae}(T)$.

Considering locations and local site conditions of the investigated moderately damaged buildings, the effective ground acceleration of A_0=0.40g for seismic zone-1 and a local soil class of *Z2* with characteristic periods of T_A=0.15s and T_B=0.40s are taken into account. 7 simulated motions each with durations of 25 seconds are generated compatible with the design spectrum defined in the TERDC, which has a probability of exceedance of 10% within 50 years. Table 2.1 summarizes the engineering intensities such as peak ground acceleration PGA; strong motion duration t_{SM} defined as the duration for the accelerations greater than 5% of g; effective duration t_{eff} calculated as the time interval between the 5% to 95% of Arias Intensity curve.

Table 2.1. Characteristics of the Earthquake Ensemble

		Simulated Motions							MEAN
		SimEQ-1	SimEQ-2	SimEQ-3	SimEQ-4	SimEQ-5	SimEQ-6	SimEQ-7	
PGA	(gal)	420.7	435.0	412.8	406.2	413.2	405.3	427.0	417.2
t_{SM}	(s)	17.33	19.79	18.08	18.67	17.69	18.36	19.83	18.54
t_{eff}	(s)	11.27	11.80	12.02	11.91	11.82	11.69	11.95	11.78

Among the values given in Table 2.1, the duration of the strong motion is calculated as 18.54s averagely, which is consistent with condition (i) above. Mean of the PGA is computed as 417.2cm/s^2. As well known, this value corresponds to the zero period spectral acceleration which satisfies the above condition (ii) with a design spectral acceleration of 0.40g (=392.3cm/s^2) for seismic zone-1. Condition (iii) is also satisfied depending on the dominant vibration periods of each building which will be shown in the later chapter. Fig. 2.1 shows the acceleration response spectra for the earthquake ensemble compared with the 'target' design spectrum for a 5% damping ratio. Mean tripartite elastic response spectrum, as well as the corresponding acceleration, velocity and displacement sensitive spectral regions and separating periods with values T_c=0.46s for acceleration to velocity sensitive regions and T_d=3.67s for velocity to displacement sensitive regions, are also shown in the same figure.

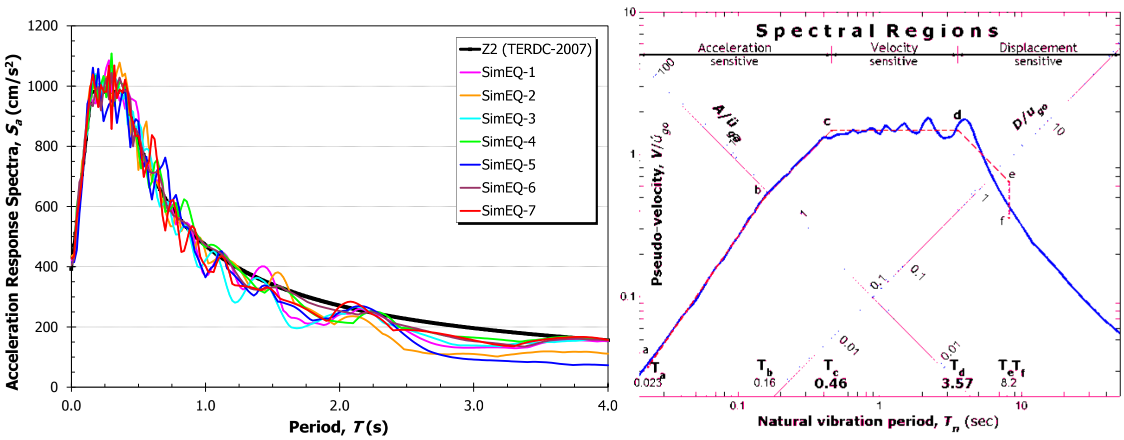

Figure 2.1. Elastic response spectra of ground motions and design spectra in TERDC-2007 (left); Tripartite elastic response spectra of the earthquake ensemble (right)

2.2. General Properties of Building Stock

During devastating earthquakes in Turkey, buildings especially constructed prior to the TERDC of 1975 have experienced severe damages and partial or even total collapse. This demonstrates that these structures having poor structural material quality, inadequate reinforcement detailing, lack of confinement zones, heavy and large-span cantilevers and indirect supporting preventing the formation of regular structural frames are highly vulnerable to seismic actions. Nevertheless, most of the damaged buildings in Turkey are observed to be designed brittle, weak in stiffness and strength and poor in material qualities due to the lack of proper amount of engineering service during the design and site-control stages. Therefore, it is a very important issue in case of urban renewal to replace these buildings with an earthquake resistant building stock using a rapid risk assessment method by means of vulnerability functions.

A set of 17 existing residential RC buildings, which have experienced moderate damages during the major earthquakes in Turkey are considered herein this paper. All buildings with frame structural systems are investigated in details by means of structural material quality, reinforcement amount and detailing of bars and local site conditions. Concrete class is found to be varying from C8~C16 (f_{ck}=8.2~16.5 MPa), while the reinforcing steel is S220 (f_{yk}=220 MPa) class. Below Table 2.2 tabulates some characteristics of the buildings, where T_1 is the dominant vibration period, W is the total weight of the building.

2.3. Analytical Modelling of the Structural Systems

Buildings are modelled utilizing DRAIN-2DX, however the modified version by Ascheim (2005) is preferred, which is capable of handling the stiffness and strength degradation through Takeda hysteretic model. Nonlinear behaviour of columns and beams is assumed to be represented with a stiffness degrading hysteresis model. A single earth-retaining basement RC wall in one of the buildings is modelled considering the pinching effects. For the entire building stock, a structural damping of 5% and a strain-hardening of 3% are taken into account.

Table 2.2. Structural Characteristics of the Inspected Buildings

Code Name	Direction	No of Stories	h_{Total} (m)	f_{ck} (Mpa)	W (kN)	$T_{1(x\text{-}x/y\text{-}y)}$ (s)
GUL	y-y	4	12.00	14.0	17100.0	0.469
L11	x-x / y-y	3	8.25	16.0	3950.7	0.324 / 0.351
P01	x-x / y-y	6	14.5	16.5	10750.7	0.502/0.544
P06	x-x / y-y	6	13.75	13.3	8104.4	0.442/0.445
P20	x-x / y-y	3	8.85	8.6	7676.8	0.439/0.319
P21	x-x / y-y	4	12.1	10.0	6382.8	0.532/0.490
P23	x-x / y-y	3	8.7	8.3	3872.7	0.356/0.379
P24	x-x / y-y	5	11.00	11.4	7859.3	0.196 / 0.519
P25	x-x / y-y	4	11.05	12.0	5890.4	0.539/0.557
P30	x-x / y-y	3	8.55	11.9	4009.2	0.302 / 0.285
P48	x-x / y-y	3	8.10	9.4	3556.9	0.393 / 0.421
P51	x-x / y-y	4	12.25	13.1	5731.8	0.527 / 0.496
P79	x-x / y-y	6	13.75	11.5	8104.4	0.451/0.454
SE05	x-x / y-y	5	14.25	8.2	10587.2	0.921/0.604
SL07	x-x / y-y	5	15.3	11.7	7383.9	0.855/0.545
YDB	x-x / y-y	2	5.60	12.0	4594	0.222/0.233
VKB	x-x / y-y	3	7.95	12.0	5789	0.341/0.258

Assigning element type-7 for stiffness-degrading elements and type-9 for the non-structural and reinforced concrete walls, the analytical model for DRAIN-2DX computer program is established. For each state of the building, the structural system is modelled as planar frames connected to each other with elastic tension/compression link elements representing rigid diaphragm effect. The structural model for non-structural walls is established as in Al-Chaar and Lamb (2002), where walls are modelled as two diagonal compression struts having a width of a_w defined by the following Eqn. 2.1:

$$a_w = 0.175 r_w \left\{ H \left(\frac{E_w t \sin 2\theta}{4 E_c I_c h_w} \right)^{\frac{1}{4}} \right\}^{-0.4} \tag{2.1}$$

Here t is the thickness; r_w is the diagonal length and θ is the diagonal slope angle of the wall. Considering the wall height as h_w; story height H; the moment of inertia of the neighbouring columns as I_c and the modulus of elasticity for infill walls E_w=1000MPa and concrete E_c varies between 24000~27000MPa, strut widths a_w are computed for each non-structural wall.

Fig. 2.2 shows the backbone curves for the hysteretic relations employed to the structural and non-structural elements during the nonlinear dynamic analysis. In the centre figure, M_y and M_u represent the yield and ultimate moments and χ_y and χ_u are the corresponding curvatures. k_1 is the initial stiffness; k_2 and k_3 are the post-yield stiffnesses in the positive and negative directions where α is the strain-hardening ratio and k_4 is the unloading stiffness. In the model for the infill walls, k_1 represents the axial stiffness of the uncracked section, u_1 and u_2 are the cracking and ultimate deformations, respectively.

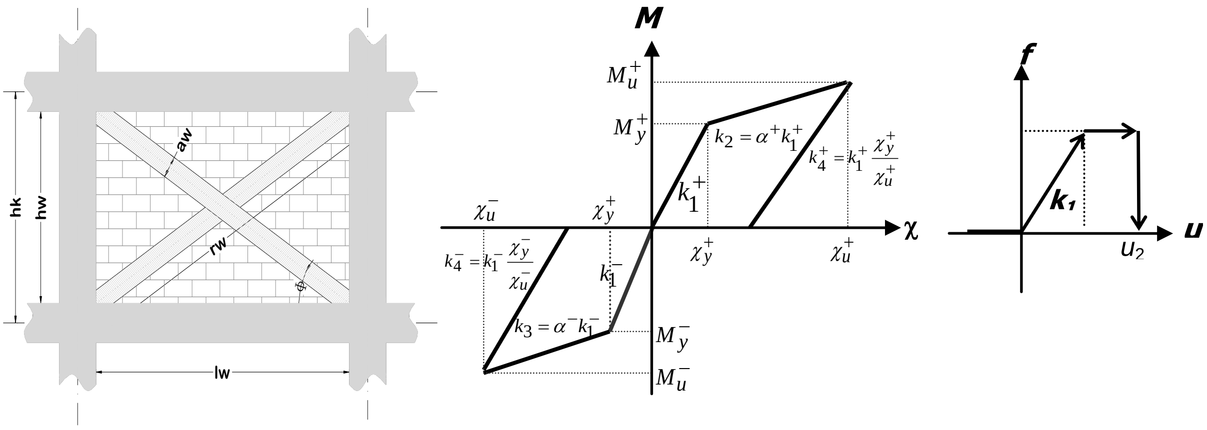

Figure 2.2. Modeling of infill walls (left) and hysteresis curves for structural (center) and non-structural elements (right)

3. NONLINEAR DYNAMIC ANALYSES RESULTS

Initially, modal analyses are carried out for each building and natural vibration periods for both directions are calculated. Substituting the dominant vibration period of each building, comparison of the acceleration response spectra of the simulated motions with the design spectrum given in Fig. 2.1 is realized concerned with the satisfaction of condition (iii) with a value of 90%.

Later, nonlinear dynamic analyses under the effect of earthquake ensemble are carried out for the entire building stock. The top story displacements (U_{top}) versus base shear (V_b) demands for each building in each direction are obtained. P48 coded building in X-X direction under the effect of SimEQ-4 ground motion is given below as an example in Fig. 3.1. The code limit for story drift Δ_i and base shear demand V_b are also shown in the figure with dashed lines. In most of the buildings, base shear demands slightly exceeded the code limits. Furthermore, for entire building set, top story displacements significantly exceeded the code limits. These results can be explained with poor concrete quality causing increments in the displacement demands, however decrease in the base shear depending on the low structural resistance.

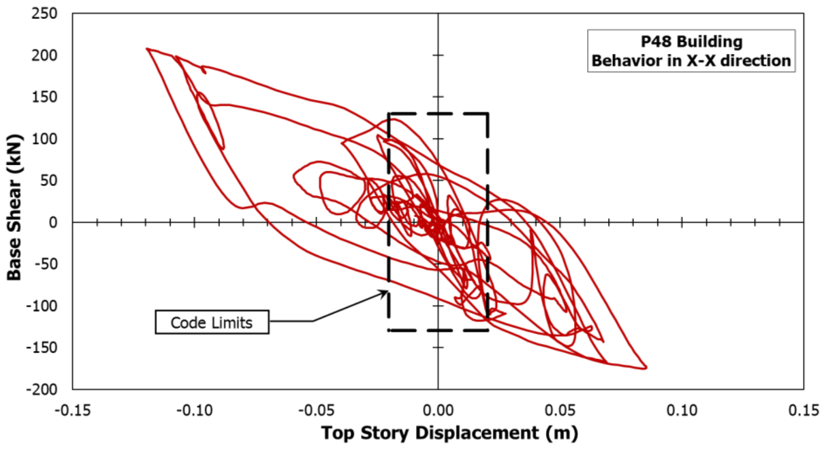

Figure 3.1. Top story displacement-base shear variation of P48 building with code limits

Subjected to 7 simulated ground motions, a number of 231 time history analyses are carried out and the envelopes for story displacement and drifts are obtained. The below Table 3.1 tabulates the computed demands for story drifts Δ_i, base shear V_b and the overturning moment M_0 for each direction of each building in the ensemble .

Table 3.1. Structural Characteristics of the Inspected Buildings

Code Name	W (kN)	$U_{top(\text{x-x})}$	$U_{top(\text{y-y})}$	$V_{base(\text{x-x})}$ (kN)	$V_{base(\text{y-y})}$ (kN)	$M_{0(\text{x-x})}$ (kNm)	$M_{0(\text{y-y})}$ (kNm)
GUL	17100.0	-	0.124	-	1649.4	-	19574.1
L11	3950.7	0.131	0.101	234.5	242.4	2966.4	3695.3
P01	10750.7	0.126	0.139	1206.3	1239.8	19080.4	17809.6
P06	8104.4	0.099	0.098	1059.2	1004.8	12875.3	12279.9
P20	7676.8	0.124	0.076	733.3	1034.8	6889.7	10174.1
P21	6382.8	0.166	0.146	636.2	527.9	5515.7	6359.1
P23	3872.7	0.102	0.117	310	315	3539.8	3333.9
P24	7859.3	0.031	0.15	380.8	335.7	5077.6	6370.4
P25	5890.4	0.152	0.16	428.8	374.1	5345.2	4976.1
P30	4009.2	0.066	0.063	32.3	35.2	24451.9	5766.9
P48	3556.9	0.11	0.143	205.2	144.4	3233.7	2404.8
P51	5731.8	0.147	0.142	429.9	512.3	5094.2	5918.8
P79	8104.4	0.104	0.103	957.8	930.4	12133	11781.3
SE05	10587.2	0.238	0.175	543	807.7	6142.6	9882.3
SL07	7383.9	0.195	0.145	223.8	367.1	4499.1	7791.9
YDB	4594	0.044	0.041	480.5	485.6	7007.8	5916.2
VKB	5789	0.063	0.047	1144.8	1161.4	8426.8	9741.5

4. COMPARISON OF THE RESULTS WITH PROPOSED FUNCTIONS

Although various researchers focused on establisihing vulnerability functions in order to estimate the probability of structural damage due to earthquakes, there exist only few attempts to estimate the seismic vulnerability of Turkish RC buildings by use of fragility curves. In order to investigate the applicability of proposed vulnerability functions for typical building stock in Turkey, some of these fragility curves are employed for the building set and compared with the analytical results.

4.1. Damage Indices Defined by DiPasquale and Cakmak, (1989)

As a primarily study that concerns with damage assessment of buildings, DiPasquale and Cakmak mentioned about several indices that can be proposed as measures of structural damage which are functions of the fundamental periods $(T_0)_i$ estimated during an earthquake, as well as of the initial fundamental period $(T_0)_{initial}$ and the final one $(T_0)_{final}$. The functional form of the indices may depend upon phenomenological aspects of damage at the local level, upon analytical considerations and upon the analysis of data recorded from damaged structures. They proved the possibility to compute averages of the local stiffness degradation using the vibrational parameters of the body, in particular $(T_0)_{final}$. According to their study by comparing the fundamental period $(T_0)_{initial}$ and $(T_0)_{final}$ before and after the earthquake, a measure of the structure's global stiffness degradation can be obtained. The indices thus defined is called final softening and indicated with δ_f as given in Eqn. 4.1:

$$\delta_f = 1 - \frac{(T_0)^2{}_{initial}}{(T_0)^2{}_{final}} \tag{4.1}$$

By the aim of demonstrating the applicability of this criterion in limit state determination, modal analyses are renewed for the building ensemble after nonlinear dynamic analyses and vibration periods for the damaged states of structures are recalculated. Table 4.1 summarizes the results for building set for each direction. Depending on the site-observed damages of the related structures and the damage indices values, it can be regarded that the degradation of the low-amplitude characteristics of existing structures can in fact be quantified by monitoring the periods of the vibration.

Table 4.1. Damage Index of the Inspected Buildings

Code Name	$T_{1x\text{-}x}$ (s)	$T_{1y\text{-}y}$ (s)	$T_{1x\text{-}x}$* (s)	$T_{1y\text{-}y}$* (s)	$\delta_{fx\text{-}x}$	$\delta_{fy\text{-}y}$
GUL	-	0.469	-	1.753	-	0.928
L11	0.324	0.351	2.389	2.517	0.982	0.981
P01	0.502	0.544	1.7	1.788	0.913	0.907
P06	0.442	0.445	1.233	1.22	0.871	0.867
P20	0.439	0.319	1.721	1.248	0.935	0.935
P21	0.532	0.49	2.096	2.075	0.936	0.944
P23	0.356	0.379	1.673	1.729	0.955	0.952
P24	0.196	0.519	0.697	2.166	0.921	0.943
P25	0.539	0.557	2.255	2.61	0.943	0.954
P30	0.302	0.285	1.382	1.244	0.952	0.948
P48	0.393	0.421	2.542	2.899	0.976	0.979
P51	0.527	0.496	2.484	2.085	0.955	0.943
P79	0.451	0.454	1.351	1.298	0.889	0.878
SE05	0.921	0.604	4.136	2.68	0.950	0.949
SL07	0.855	0.545	3.082	2.379	0.923	0.948
YDB	0.222	0.233	1.241	1.133	0.968	0.958
VKB	0.341	0.258	0.897	0.806	0.855	0.898

4.2. Empirical Functions Proposed by Rosetto and Elnashai, (2003)

Empirical fragility curves for European-type RC building populations are derived in their study based on a data bank of 99 post-earthquake damage distributions observed in 19 earthquakes and concerning o total of 340,000 RC structures. The heterogeneous observational data are reinterpreted in terms of a damage scale, namely homogenised reinforced concrete (HRC-scale), which is calibrated experimentally. The feasibility of using observation-based data for the generation of vulnerability functions for different strong ground motion parameters is investigated.

The functions most commonly used in existing relationships are cumulative normal and log-normal distributions. However, in their study, by following many trials a relationship in the form of Eqn. 4.2 is found to yield the optimum fit for all considered ground motion parameters.

$$P(d \geq DI_{HRC} | GM) = 1 - \exp(-\alpha . GM^{\beta}) \tag{4.2}$$

Using the parameters α and β, which are defining the mean curves according to the equation, empirical fragility relations developed for six different HRC damage states for a ground motion parameter, PGA. Fig. 4.1 illustrates the comparison of the building set results with the proposed vulnerability functions.

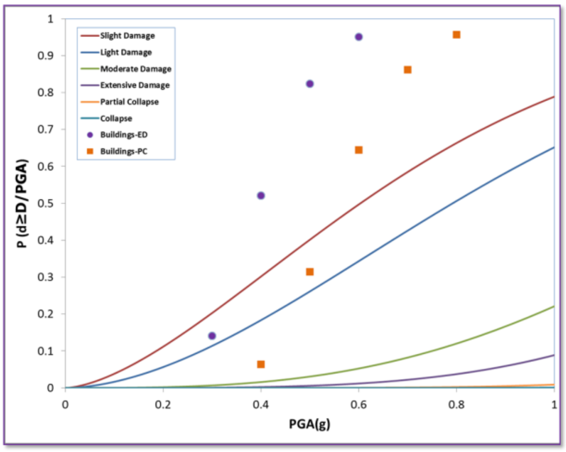

Figure 4.1. Comparison of the analyses results with the proposed curves for different damage states

It can be observed from the figure that, existing highly vulnerable structures to seismic actions in Turkey experiences higher damages under the effect of average-values of PGA.

4.3. Analytical Functions Proposed by Kircil and Polat, (2006)

Analytical fragility curves for mid-rise RC frame buildings in Istanbul, which have been designed according to the 1975 version of the TERDC, are developed based on numerical simulation with respect to the number of stories of the buildings. For this purpose, 3-, 4-, 5- and 6-storey representative buildings are designed according to the Turkish seismic code. Incremental dynamic analyses are performed using 12 artificial ground motions and the yielding and collapse capacities of each sample building are determined. Based on those capacities, vulnerability functions are developed in terms of different ground motion parameters such as *PGA*, S_a, S_d and inter-story drift ratio (ISD %) with lognormal distribution assumption.

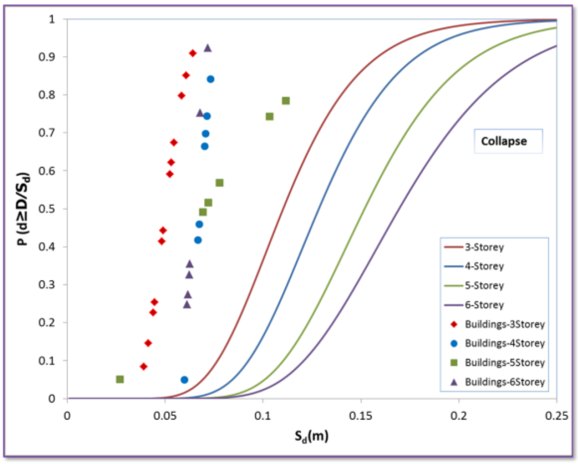

Figure 4.2. Comparison of the analyses results with the proposed curves for different storey numbers

Fig. 4.2 represents the comparison of analytically derived fragility functions for different storey numbers and nonlinear dynamic analyses results of buildings having similar heights. It can be easily seen that, distribution of the analyses results for each building in the ensemble significantly differ from the proposed functions for the buildings.

5. CONCLUSION

This paper reviews existing vulnerability relationships for RC structures with a view to their adequacy by means of reflecting the most reliable actual damage state for existing building stock in Turkey. Initially, nonlinear dynamic analyses are realized under the effect of an earthquake ensemble consisting of 7 artificial strong motions for 17 moderately damaged existing frame systems. Later, curves proposed by various researchers are searched and three of these fragility functions are employed to the nonlinear dynamic analysis solutions in order to investigate the capabilities of good estimation.

One of the main observations of this research is; existing buildings which reflect the general characteristics of Turkish-type of structures and which are moderately damaged during the destructive earthquakes, have significantly high values of displacement demands due to their weaknesses in stiffness, however low values of base shear, depending on the poor concrete quality and their low structural strengths. Secondly, poor estimation is achieved during the comparisons of the proposed vulnerability functions compared to the analytical results of the building set. To figure out this bias, increasing the amount of buildings in the analysis set during the future studies seems to be necessary.

Additionally, from the results of this investigation, it is evident that the derivation of vulnerability curves by taking into account different structural parameters such as base shear resistance capacity is required due to the fact that the building stock in Turkey deemed unsafe since there is a lack of control mechanism during the construction stages for most of the structures constructed prior to the National Construction Control and Supervising Law.

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