Research Article
Analytical Vulnerability Assessment of the Moderately Damaged RC Building Stock after 1998 Ceyhan, Turkey EQ

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Abstract: Taking 145 lives and causing extensive damages to structures, 1998 Ceyhan earthquake occurred on June 27 with M_s 6.2 in the southern province Adana of Turkey hitting major districts and their vicinities. Most of the observed structural damages were assessed in traditional rural buildings, however many recently built multi-story residential buildings and industrial facilities also suffered from heavy damages or even collapsed. This study is an attempt to investigate a building stock consisting of 22 low -to mid-rise reinforced concrete buildings which experienced moderate damage during this shake. These buildings have been inspected and their detailed site investigations had been conducted by ITU-Building and Earthquake Application and Research Center. In order to perform an analytical study for this building ensemble, nonlinear dynamic and static analyses are carried out utilizing DRAIN-2DX computer program. A set of artificial ground motions is employed during the analyses and the demand parameters in terms of different structural response parameters are obtained. Finally, evaluations are performed in order to designate the boundaries for moderate damage grading in terms of specific structural demand parameters and figure out the failure mechanism of the investigated moderately damaged buildings.

Keywords: Artificial ground motions, Ceyhan earthquake, earthquake damages, nonlinear dynamic analyses, reinforced-concrete buildings

INTRODUCTION

Over the past quarter-century, Turkey was exposed to several moderate to severe destructive earthquakes which resulted in significant amounts of casualties and economic losses. Those events indicated that this poor in stiffness and strength building stock, mostly consisting of low -and mid-Rise reinforced Concrete (RC) structures, have experienced severe damages and even partial or total collapse. The damages to existing RC buildings can be attributed to some deficiencies that increase the damage potential of these buildings:

- Poor reinforcement detailing practice
- Lack of confinement zones
- Heavy and large-span cantilevers
- Indirect supporting preventing the formation of regular structural frames

Therefore, the site-observed damages has emphasized the need for a detailed assessment of the existing stock in order to predict the probable damages during the future earthquakes.

One of the recent major earthquakes of Turkey was the Ceyhan earthquake that struck the southern region of Turkey on 27 June 1998 with an epicenter located between downtown Adana Province and Ceyhan District about 30 km north of the coast of the Mediterranean Sea. Although the focal distance to Ceyhan was 32 km, fault rupture was directed towards Ceyhan, causing heavy damage in the city as a result of the strong motion with a Peak Ground Acceleration (PGA) of 0.28 g (Sucuoğlu *et al.*, 2000). Official estimates placed the death toll at about 145 and the number of injured at about 1500 (Ulusay *et al.*, 2000). Site-assessments demonstrated that remarkable structural damages occurred in traditional rural buildings, however many recently built multi-story residential buildings and industrial facilities also suffered from heavy damages or even collapsed (Wenk *et al.*, 1998). The most affected settlement areas are announced to be Ceyhan and Yüreğir districts with a total of 1025 buildings experiencing total collapse or heavy damages beyond repair. Within these two districts, 51% of the moderately damaged 15801 buildings were located in Yüreğir district and its villages (Tezcan and Boduroğlu, 1998). Amongst them, 22 low- to mid-rise *RC* buildings from downtown Yüreğir for which the detailed investigations are carried out by ITU-Building and Earthquake Application and Research Center teams, are taken into consideration in this study (Fig. 1).

In order to investigate the damage state of these buildings selected to have limited torsional irregularity,

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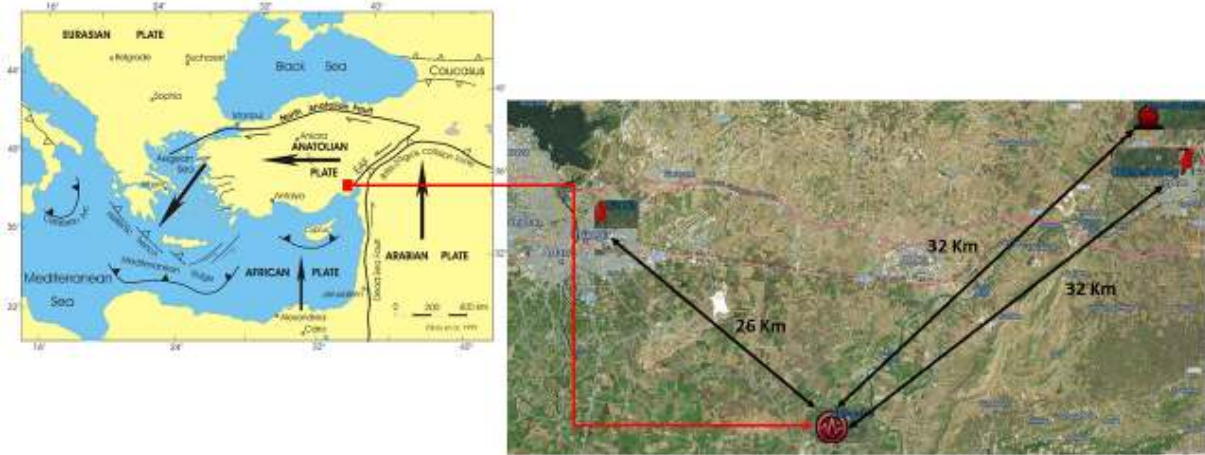
Fig. 1: The epicenter of 1998 Ceyhan earthquake, the settlement of the record station and Yüreğir district

analytical studies are performed in terms of nonlinear dynamic and static analyses. Planar structural models are established by considering the contribution of non-structural walls and DRAIN-2DX (Prakash *et al*., 1993) computer program is utilized during the computations. A set of artificial ground motions consisting of 7 earthquakes which are compatible with the design spectrum for 0.40 g effective acceleration coefficient and *Z*2 soil class defined in the Turkish Earthquake Resistant Design Code (TERDC) (2007) is employed during the analyses. The structural demand parameters in terms of top-story displacement ratio and base shear ratio are obtained for the earthquake ensemble and after defining the base shear capacities of each building; the Demand/Capacity Ratios (DCR) are calculated.

In order to demonstrate the damage ranges and the failure mechanisms of the buildings, statistical evaluations are performed oriented to the damage state of the building ensemble. Finally, upper and lower bounds to estimate the probable moderate damage region for shear critical and ductile buildings are proposed.

METHODOLOGY

Analytical modelling of the building ensemble: Twenty two moderately damaged buildings with various story numbers are selected from the archives of ITU-Building and Earthquake Application and Research Center, which were previously investigated by means of their structural layout plans, existing reinforcement details and material qualities during site assessments. Table 1 tabulates some characteristics of these buildings, where T_1 is the dominant vibration period in both directions for the undamaged state and W is the total weight of the building.

In the entire set, number of stories for the buildings varies from 3- to 6- therefore the set consists of low- and mid-rise structures. 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 compressive strength is found to be varying between f_{ck} = 7.0~16.5 MPa with an average value of 11.5 MPa for the ensemble, while the reinforcing steel is S220 (characteristic yield strength f_{yk} = 220 MPa) class for the entire stock. The distribution of the concrete strengths and the number of floors of the ensemble is given in Fig. 2.

The modified version by Ascheim (2005) of DRAIN-2DX computer program is utilized for the analysis, because this version is capable of handling the stiffness and strength degradation through Takeda hysteretic model. Nonlinear behavior of columns and beams is assumed to be represented by a stiffness degrading hysteresis model and element type-7 of DRAIN-2DX program is assigned. For the entire building ensemble, a structural damping of 5% and a strain hardening of 3% are taken into account. The structural model for non-structural partition walls is established as in Al-Chaar and Lamb (2002), where walls are modeled by two diagonal compression struts as shown in Fig. 3 having the effective width of a_w defined by Eq. (1) and element type-9 is assigned for the analytical model of the infill walls:

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

In Eq. (1), *t* is the thickness; r_w is the diagonal length and θ is the diagonal slope angle of the partition wall. Considering the wall height as h_w; story height *H*; the moment of inertia of the neighboring columns as I_c and the modulus of elasticity for the partition walls composed of hallow bricks as E_w = 1000 MPa and

Table 1: Structural characteristics of the inspected buildings

| Code	Number of stories	H_{total} (m)	f_{ck} (MPa)	f_{yk} (MPa)	W (kN)	$T_{1(x-x)}/T_{1(y-y)}$ (sec)
02-MD-ST3	3	8.25	16.0	220	3950.7	0.423/0.351
03-MD-ST6	6	17.40	16.5	220	10750.7	0.502/0.544
04-MD-ST5	5	13.75	13.3	220	8104.4	0.442/0.445
05-MD-ST3	3	8.85	8.6	220	7676.8	0.439/0.319
06-MD-ST4	4	12.10	10.0	220	6382.8	0.532/0.490
07-MD-ST3	3	8.70	8.3	220	3872.7	0.356/0.379
08-MD-ST5	5	13.85	11.4	220	7859.3	0.196/0.519
09-MD-ST4	4	11.05	12.0	220	5890.4	0.539/0.557
10-MD-ST3	3	8.55	11.9	220	4009.2	0.302/0.285
11-MD-ST3	3	8.10	9.4	220	3556.9	0.393/0.421
12-MD-ST4	4	12.25	13.1	220	5731.8	0.527/0.496
13-MD-ST5	5	13.75	11.5	220	8104.4	0.451/0.454
14-MD-ST5	5	14.25	8.2	220	10587.2	0.921/0.603
15-MD-ST5	5	15.30	11.7	220	7383.9	0.855/0.545
23-MD-ST6	6	15.20	8.3	220	8208.1	0.563/0.546
24-MD-ST4	4	10.80	12.3	220	3442.8	0.443/0.397
29-MD-ST4	4	13.30	12.3	220	8198.4	0.415/0.629
38-MD-ST3	3	9.65	7.0	220	5151.4	0.516/0.535
39-MD-ST5	5	14.75	10.6	220	19050.6	0.626/0.767
40-MD-ST5	5	14.75	10.0	220	15680.1	0.521/0.541
41-MD-ST4	4	12.80	14.5	220	14147.1	0.526/0.507
43-MD-ST4	4	10.65	16.3	220	10684.8	0.261/0.376
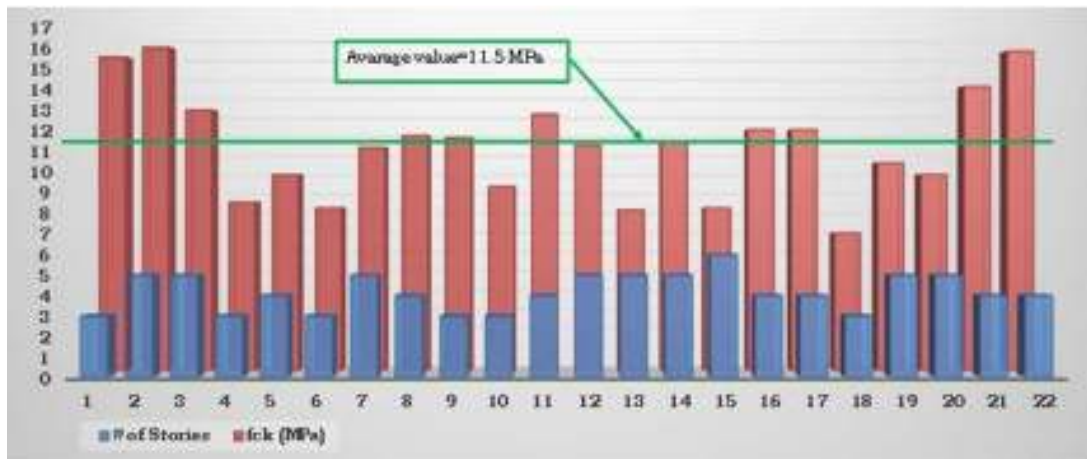
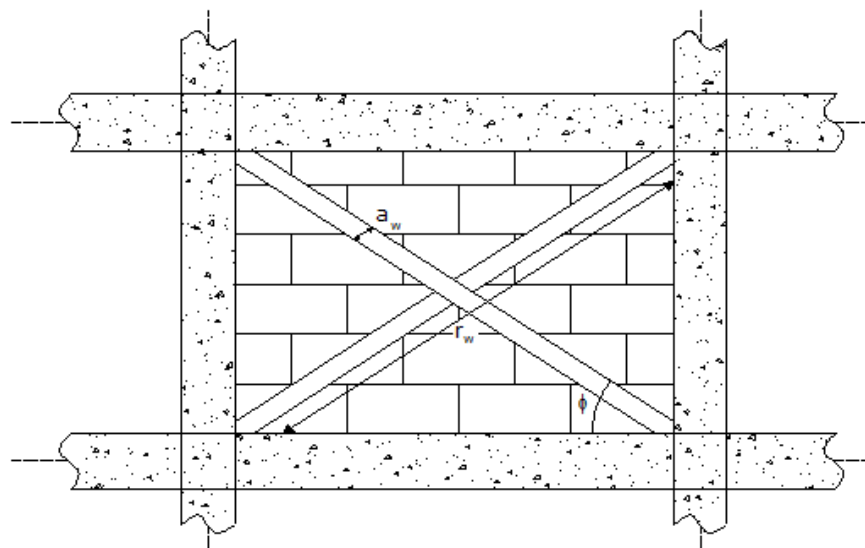Fig. 2: The distribution of the concrete classes and story numbers of the building ensemble

Fig. 3: Modelling of the non-structural partition walls

concrete's modulus of elasticity E_c varying between 22600~27200 MPa for f_{ck} = 7.0~16.5 MPa, respectively. Henceforth, strut widths a_w are computed for each nonstructural wall by Eq. (1).

For the entire building ensemble, the structural system is modeled as planar frames connected to each other with elastic tension/compression link elements representing rigid diaphragm effect and element type-9 is assigned for these elements.

Figure 4 shows the backbone curves employed to represent for the hysteretic behavior of the structural and non-structural elements during the nonlinear dynamic analysis. In the left side of Fig. 4, M_y and M_u represent the yield and ultimate moment capacities 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 partition walls shown in the right side of the same figure, k_1 represents the axial stiffness of the un-cracked section and u_1 and u_2 are the cracking and ultimate deformations, respectively.

Ground motion ensemble: Utilizing TARSCTHS (Papageorgiou *et al.*, 2000) computer program which is capable of simulating earthquake motions compatible with a pre-defined target spectrum, a set of artificial ground motions is achieved by generating 7 acceleration-time histories. According to the TERDC, 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 sec.
- 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 $0.2T_1$~$2T_1$, where T_1 is the dominant vibration period of the structure, shall not be less than 90% of the elastic spectral acceleration $S_{ae}(T)$.

When the locations and local site conditions of the investigated moderately damaged buildings are considered, the effective ground acceleration of A_0 = 0.40 g for seismic zone-1 and a local soil class of $Z2$ with characteristic periods of T_A = 0.15 sec and T_B = 0.40 sec are taken into account. The artificial 7 strong ground motions, each with durations of 25 sec, are generated compatible with the design spectrum with a return period of 475 years as defined in the TERDC, which has the probability of exceedance of 10% within

Table 2: Characteristic properties of the earthquake ensemble

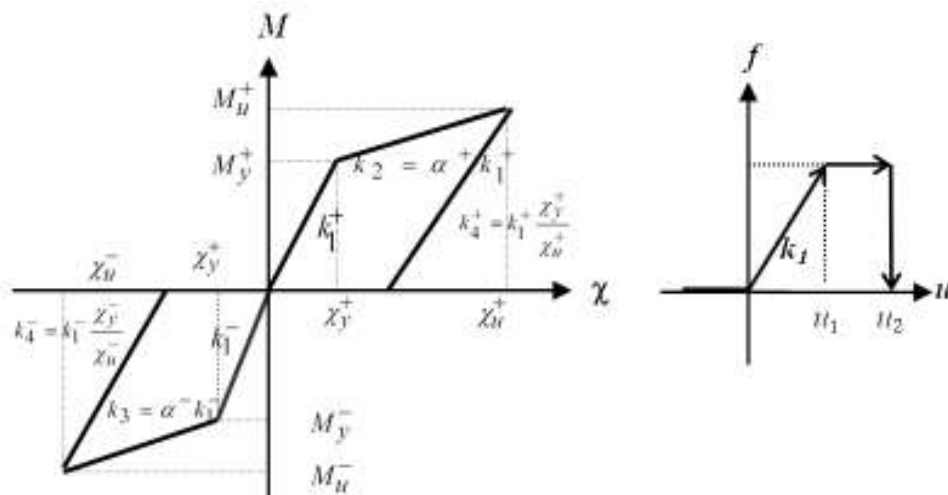|	Artificial strong ground motions							
	SimEQ-1	SimEQ-2	SimEQ-3	SimEQ-4	SimEQ-5	SimEQ-6	SimEQ-7	Mean
PGA (gal)	420.70	435.00	412.80	406.20	413.20	405.30	427.00	417.20
t_{SM} (sec)	17.33	19.79	18.08	18.67	17.69	18.36	19.83	18.54
t_{eff} (sec)	11.27	11.80	12.02	11.91	11.82	11.69	11.95	11.78

Fig. 4: Hysteresis curves for structural (left) and non-structural elements (right)
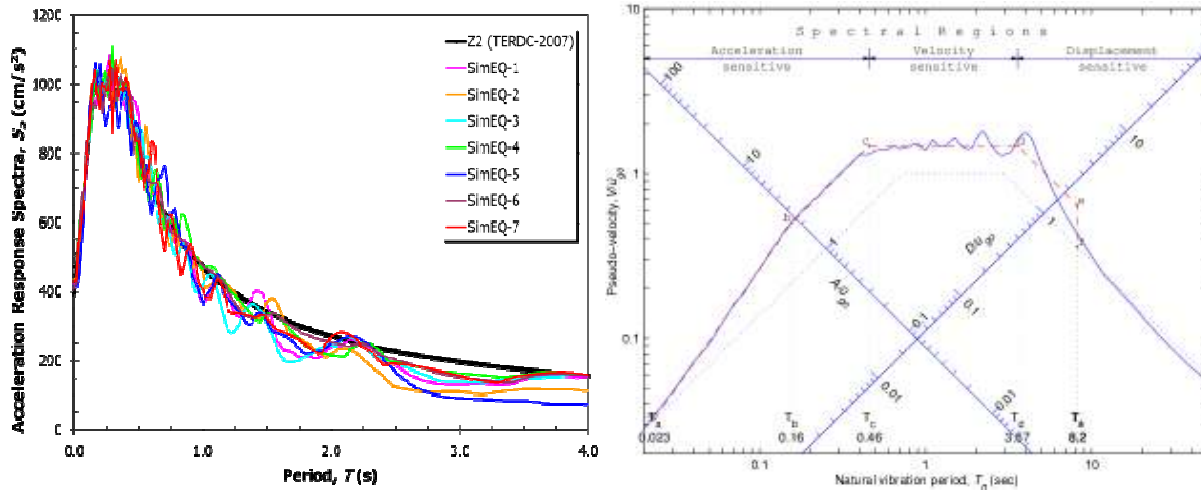
Fig. 5: Elastic response spectra of ground motions and the design spectra in the TERDC (left) and mean tripartite elastic response spectra of the earthquake ensemble (right)

50 years. Table 2 summarizes the engineering parameters; 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.

As it can be observed from Table 2, the duration of the strong motion is calculated as 18.54 sec averagely, which is consistent with condition (i) above. Mean of the *PGA* is computed as 417.2 cm/sec^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 0.40 *g* (= 392.3 cm/sec^2) for seismic zone-1. Condition (iii) is also satisfied depending on the dominant vibration periods of the buildings which are already given in Table 1.

Figure 5 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 = 0.46 sec for acceleration to velocity sensitive regions and T_d = 3.67 sec for velocity to displacement sensitive regions, are also shown in the same Fig. 5.

EVALUATION OF THE ANALYTICAL RESULTS

As it is targeted to figure out the upper and the lower bounds for defining the (probable) moderate damage range to existing *RC* buildings during a future earthquake event, the selected moderately damaged buildings are also analytically investigated. Although each of the 22 buildings experienced this damage rank during a specific shake with a PGA of 0.28 g; a detailed vulnerability analysis is carried out by scaling the PGA values of the 7 artificial motions to 15 different levels.

Starting from 0.1 g and scaling the PGA up to 0.8 g for each ground motion with an increment of 0.05 g, 4620 nonlinear dynamic analyses (7×15 motions, 22 buildings, 2 directions) are achieved for the building ensemble. After the calculation of structural responses for each building subjected to each scaled ground motion, the demand parameters in terms of top story displacement ratio (U_{top}/H) and base shear ratio (V_b/W) are collected from the outputs, which are summarized in Fig. 6. Furthermore, for determining the Demand/Capacity Ratios (DCR), base shear capacities (V_r) are also calculated by conducting Eq. (2):

$$V_r = 0.2 \times \sqrt{f_{ck}} \times A_c \tag{2}$$

for each building, where,
V_r = The shear force resisting capacity
f_{ck} = The characteristic compression strength of concrete
A_c = The effective shear area of the columns that resist the shear force

From Fig. 6, it can be seen that the base shear ratio is approximately varying between 0.9 to 7.9% for the weakest shake with 0.1 g of PGA and is varying between 3.5 to 17.3% for the strongest shake with a PGA of 0.8 g. Additionally, top story drift ratio is varying between 1.2 to 3.9% and 11.7 to 39.1% for the weakest and the strongest shakes, respectively. On the other hand, number 8, 10 and 43 buildings, respectively exhibit distinct behavior from the rest due to their structural system which has locally *RC* shear walls, extremely poor design and average quality, or rather regular and better quality, respectively.

Results in terms of top story drift ratios for both directions of the buildings in the ensemble under the effect of the earthquake ground motions scaled from 0.1 to 0.8 g is given in Fig. 7. In the figure, red curve represents the mean distribution for each PGA level
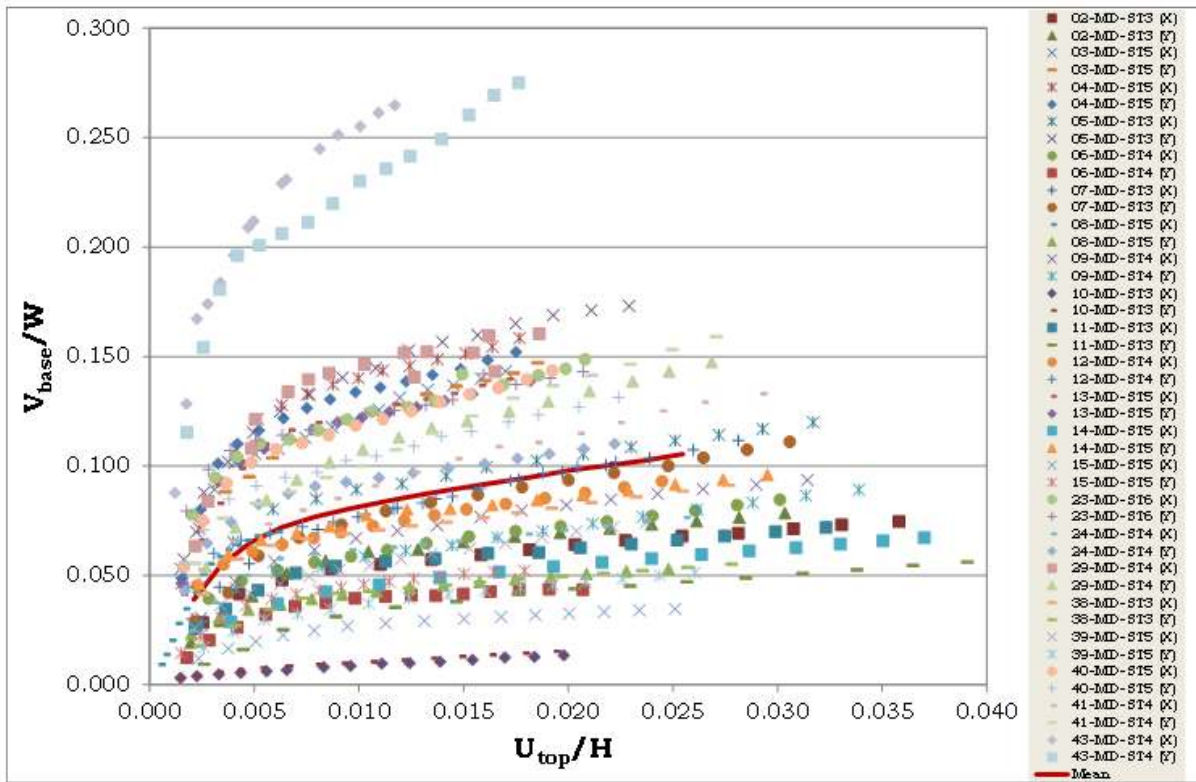
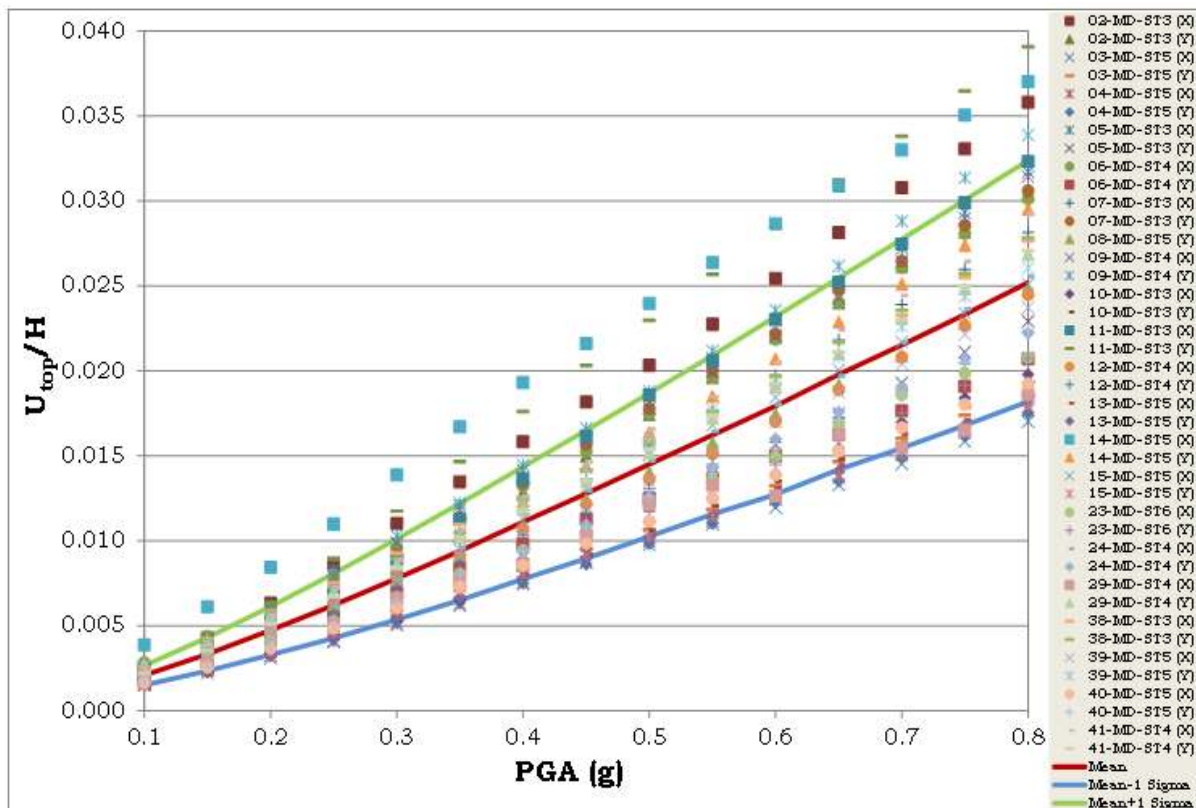Fig. 6: Base shear ratio versus top story drift ratio distributions of the buildings

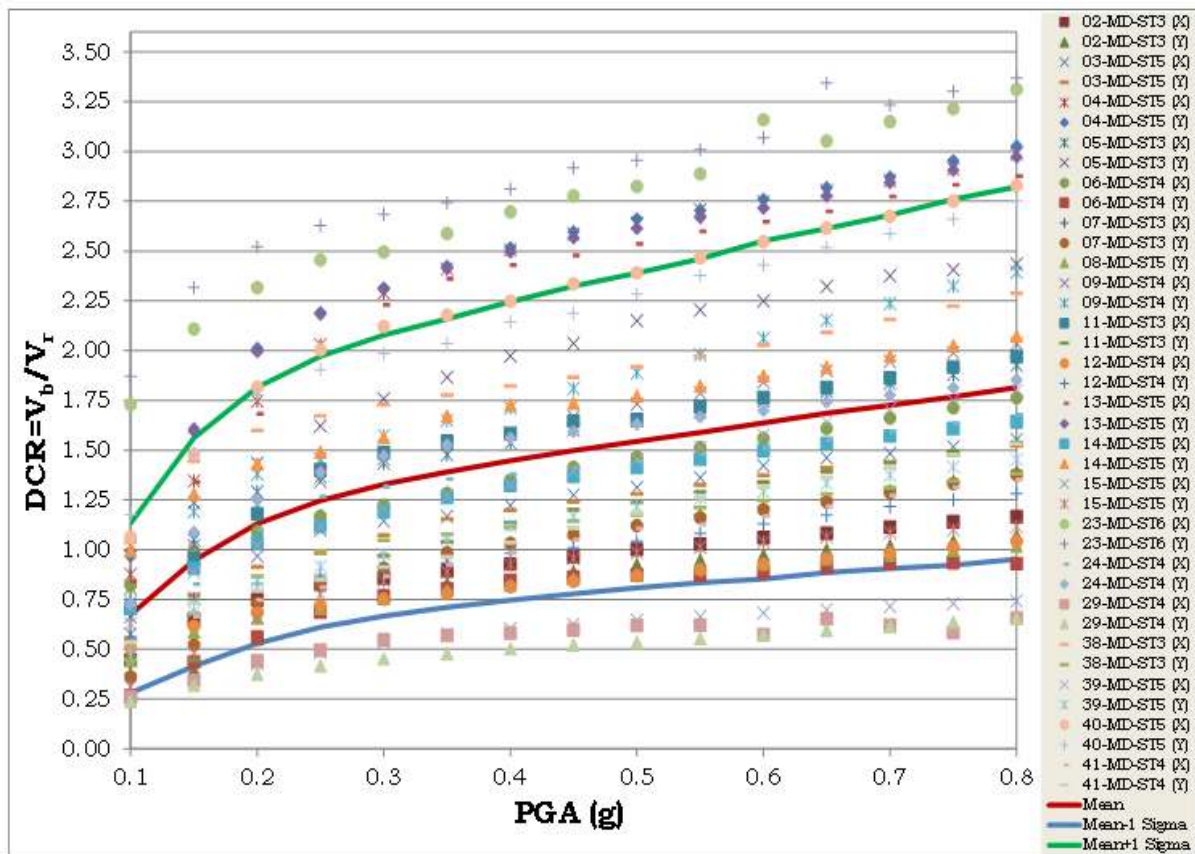Fig. 7: Top story drift ratio distributions of the buildings versus PGA

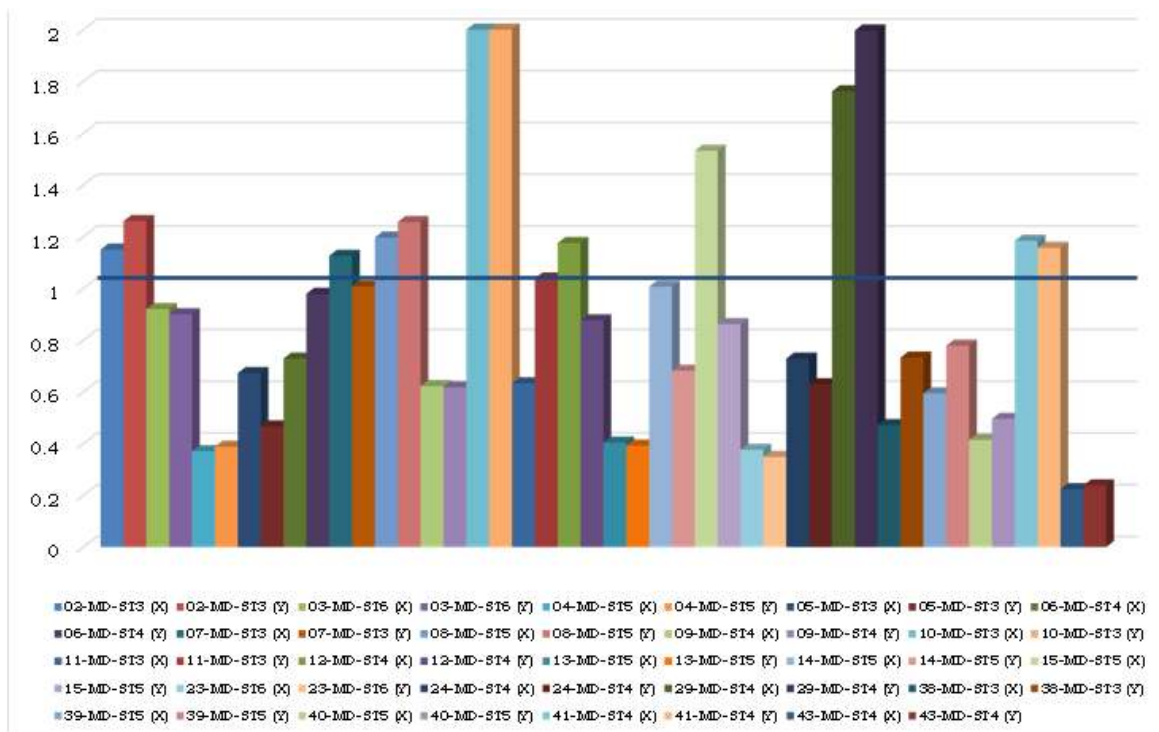Fig. 8: DCR distributions of the buildings versus PGA

Fig. 9: V_r/V_y distributions of the buildings

Table 3: Top story drift ratio and demand/capacity ratio ranges for moderate damage level

Damage level	Drift ratio	Demand/capacity ratio
Moderate damage	0.93-1.69%	0.78-2.40

while blue and green curves designate the values that have 10% probability of being smaller and greater than mean value, respectively. During the statistical studies that are performed by the aim of determining this specific damage level boundaries, the distributions of the building ensemble are investigated in details.

As a result, it is established that the distributions correspond to each acceleration value have goodness of fit with lognormal distribution. Since the aforementioned 43-MD-ST4 building's results obviously differ from the ensemble, the results of the building are ignored during the evaluation of statistical parameters such as mean and standard deviation values. Furthermore, the analytical results for *x-x* direction of 08-MD-ST5 building also are not taken into consideration in consequence of conspicuous behavior of the building under the effect of the earthquakes. This finding is comprehensible when we consider the structural system of the building which has locally RC shear walls in the *x-x* direction at basement level.

Within the scope of representing the failure mechanism of the building stock, a parameter called Demand/Capacity Ratio (DCR) is defined in this research considering the base shear demands under the effect of ground motions and the obtained base shear capacities. Figure 8 illustrates the results in terms of DCR parameter for each structural direction of the buildings under the effect of scaled ground motions. Similarly, the colored curves represent the same values explained in the previous paragraph.

In accordance with the statistical works done for top story drift ratio parameter, the results of the buildings signified above are not considered during the determination of mean and standard deviation parameters for DCR parameter as well. Furthermore, although 10-MD-ST3 building's shear capacity does not vary among the other buildings in the ensemble, base shear demands are found to be in smaller ranks. For this reason, the results of this building are also ignored during the calculation of statistical parameters.

As shown in Fig. 1, the location of Yüreğir district in which the buildings are settled, is approximately 26 km away from the epicenter of the earthquake. The record station is 32 km away from the epicenter and it can be assumed that the peak ground acceleration values might be similar at a distance of 26 km. However it is more likely that the PGA values might be amplified due to the local site conditions, since the district settles near Seyhan River and is very close to this river's reservoir lake. Within this context, it can be suggested that the PGA value that had effected the building stock might vary between the ranges of 0.25~0.45 g. In another aspect of a similar research conducted by Booth *et al*. (2004), where a building database composed of Turkish low-rise and mid-rise RC frame structures which have experienced moderate damages during 1999 Kocaeli Earthquake, it is argued that the PGA values might be between the ranges 0.35~0.55 g to cause moderate damage for the investigated buildings.

The obtained ranges for top story drift ratio and demand/capacity ratio parameters that represent the moderate damage level are tabulated in Table 3. The top story drift ratio range is definitely consistent with the interval given in Booth *et al*. (2004).

In order to figure out the failure mechanisms of the structures in the building database, nonlinear static pushover analyses are performed for each direction of the building assuming that lateral load distribution is the first mode loading pattern and the structural capacities are calculated. Using the bi-linearization technique given in FEMA-356 (2000) for the obtained capacity curves, yielding forces (V_y) are determined for each structure in the ensemble. As it is expected, flexural failure is observed in the buildings that have V_r/V_y ratio greater than or equal to 1, while shear failure is observed in the buildings that have V_r/V_y ratio smaller than 1 when the analyses results are taken into consideration by means of the stated intervals in Table 3. The V_r/V_y distributions of the shear critical and ductile buildings in the stock is shown in Fig. 9.

Depending on the analyses results of the building ensemble, if we consider the lower bound of the "Moderate Damage" zone which is equal to 0.93% in terms of top story drift ratio, it is observed that 64% of the building stock is exposed to shear failure even under the effect of slight to moderate intensity earthquakes.

CONCLUSION

The significant amounts of casualties, injures and economic losses during the devastating earthquakes of Turkey over the last quarter-century are regrettably far too much when compared to those for similar intensity earthquakes recorded elsewhere in the world. Site-observed damages after the earthquakes indicate that main structural design and construction mistakes cause partial or total collapse of RC buildings in Turkey. Therefore a detailed and a fast estimation of the building stock in order to predict the probable damage during future earthquakes is required by means of organizing the emergency management services efficiently.

Within the scope of this research, 22 RC buildings which were moderately damaged during the 27 June 1998 Adana-Ceyhan Earthquake (M_s 6.2) are investigated analytically by means of the site-observed devastations. Nonlinear dynamic and static analyses are carried out utilizing DRAIN-2DX computer program and a set of artificial ground motions is employed during nonlinear dynamic time history analyses by scaling the *PGA* values of the earthquake ensemble with an increment of 0.05 g from 0.1 to 0.8 g.

Consequently, suggesting that the PGA value that had effected the building stock might vary between the ranges of 0.25~0.45 g, the boundaries for top story drift ratio and demand/capacity ratio parameters that represent the moderate damage level are designated by the implementation of statistical works over the distributions of the analyses results and these bounds are tabulated in Table 3.

As a result, when a fast estimation is necessitated in order to define the damage rank of a building and/or a building stock subjected to an earthquake, the boundaries of the "Moderate Damage" zone based on DCR and drift ratio parameters can be used for shear critical and ductile systems, respectively.

ACKNOWLEDGMENT

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