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EVALUATION OF SEISMIC BEHAVIOR OF THE SELECTED BUILDINGS LOCATED CLOSE TO DUZCE RECORDING STATION IN DUZCE, TURKEY WHERE TWO SUBSEQUENT EARTHQUAKES OCCURRED IN 1999

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ABSTRACT

An approach, which refers to displacement-based design, has been developed and proposed for typical reinforced concrete buildings in Turkey. In this study; the proposed methodology will be introduced and applied on two reinforced concrete buildings located at Duzce Recording Station. Earthquake records were obtained at this station during both Marmara Earthquake (August 17$^{th.}$, 1999) and Duzce Earthquake (November 12$^{th.}$, 1999) in Turkey. The buildings and the observed damage to these buildings after both earthquakes were previously investigated. Efficiency of the proposed methodology in assessing the observed damage of the selected buildings located in the near vicinity of the Duzce Recording Station will be examined. The proposed methodology will be implemented for these buildings considering their behavior and their performance after both earthquakes. For the investigated buildings, it is observed that the proposed methodology can be considered as a helpful tool for the estimation of the level of mean drift ratio (MDR) and the corresponding level of damage.

Introduction

In this study the proposed design approach introduced for typical reinforced concrete buildings in Turkey (Ozturk 2003) will be implemented to study its effectiveness in assessing the observed damage of two buildings located next to the Duzce Recording Station. Both Marmara Earthquake (August 17$^{th.}$, 1999) and Duzce Earthquake (November 12$^{th.}$,1999) ground motion records were recorded at this station.

The buildings and the observed damage to these buildings after both earthquakes were previously investigated (Engvall 2002). Findings of the referred investigation will be given and the proposed methodology in this study (Ozturk 2003) will be discussed for these two buildings. It will be assumed that the soil conditions of the Duzce Recording Station and the selected buildings which are located in its vicinity are similar.

The behavior and the observed damage in the selected sample buildings for both August 17$^{th.}$, 1999 and

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November 12th., 1999 events will be discussed in terms of mean drift ratio (MDR). As an example, the nonlinear displacement spectra introduced in Fig. 3 provides help for the estimation of roof displacement of the investigated buildings when they are subjected to NS component of November 12th., 1999 Duzce Record. It has to be mentioned that nonlinear response of buildings in terms of mean drift ratio (MDR) is a helpful tool for evaluation of the level of damage at these buildings. The acceleration-time history and the calculated velocity history for NS component of Duzce record of November 12th, 1999 are shown in Figs. 1, and 2, respectively.

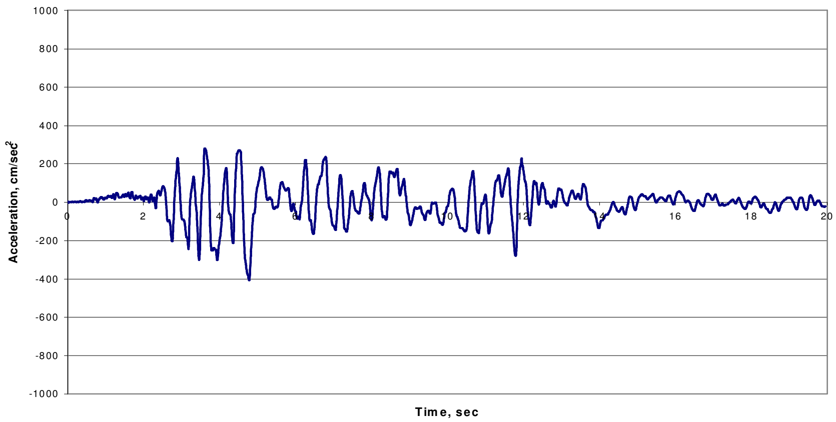

Figure 1. Acceleration-Time History of NS Component of Duzce Record
(Duzce Earthquake, November 12th, 1999)

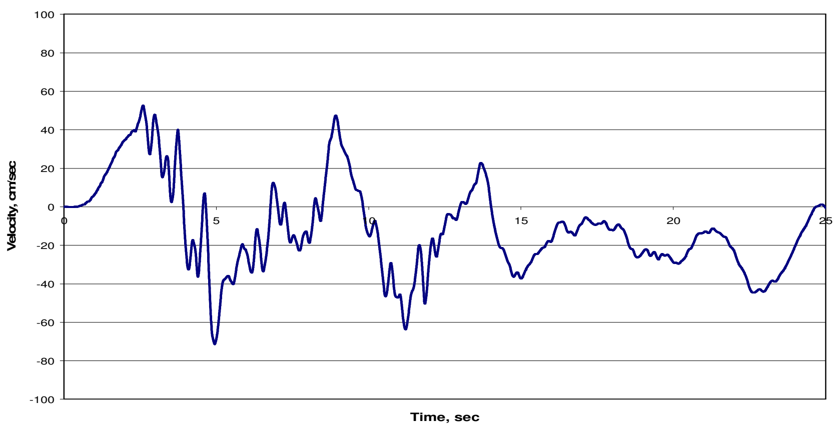

Figure 2. Velocity-Time History of NS Component of Duzce Record
(Duzce Earthquake, November 12th, 1999)

In the nonlinear displacement response spectra for NS component of Duzce Record (November 12th., 1999) used in this study (Fig. 3), it is observed that the maximum displacement demand of a single-degree-of-freedom (SDOF) system and the damage potential of the ground motion increase especially for periods less than T_g. In addition; it is observed that for ground motion records with comparatively higher values of ΔV, there is an increasing requirement for the base shear strength of a SDOF system, if response is to remain within the bound given by Lepage (Lepage 1997).

The idealized displacement response spectrum, S_d (i.e. Fig. 3) is obtained using:

$$S_d = \frac{F_a.\alpha.g.T^2}{(2\pi)^2} \qquad \text{for } T<T_g \qquad (1)$$

$$S_d = \frac{F_a.\alpha.g.T_g}{(2\pi)^2}.T \qquad \text{for } T \geq T_g \tag{2}$$

where;

F_a : acceleration amplification factor (a value of 3.75 for systems with 2% damping factor is representative of a wide range of ground motions);

g : acceleration of gravity;

α : peak ground acceleration expressed as a coefficient of the acceleration of gravity;

T_g : characteristic period for ground motion, it may be defined as the period at which the assumed constant acceleration region ends;

T : period of vibration

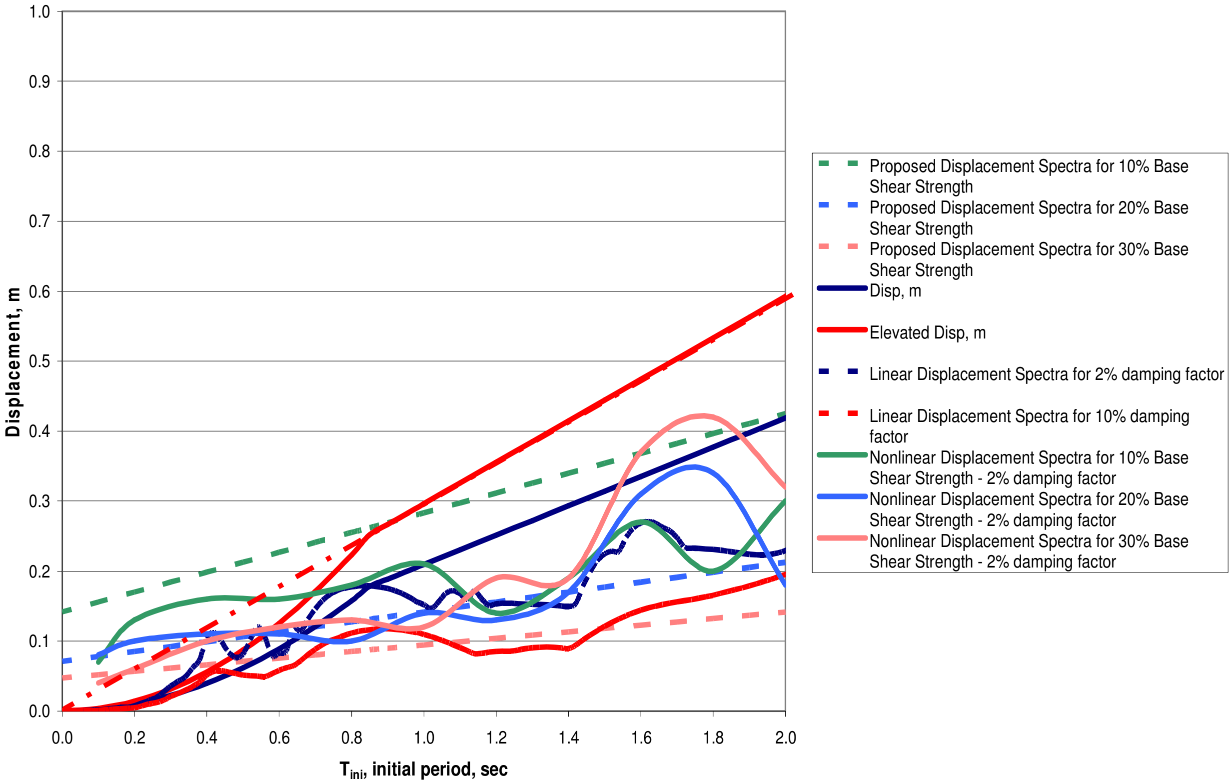

Figure 3. Nonlinear Displacement Spectra for NS Component of Duzce Record
(Duzce Earthquake, November 12th, 1999)

Eqs. 1, and 2 reflect regions of nearly constant acceleration (for $T< T_g$) and nearly constant velocity (for $T> T_g$). These generalizations relate to observations made by Newmark (Newmark 1970). The characteristic period for ground motion, T_g is the period where the energy response spectrum, for a damping factor of 10%, tends to level off. The characteristic period, T_g, closely corresponds to the period where the nearly constant velocity response region starts.

According to Lepage (Lepage 1997), nonlinear-response displacement, D_{max} may be estimated from a linear-response spectrum using:

$$D_{\max} = \frac{F_a.\alpha.g.T_g}{(2\pi)^2}.T \tag{3}$$

Eqs. 1, 2, and 3, which provide idealized nonlinear-response maxima, are based primarily on response spectrum determined for a viscous damping of 2% of the critical. They emphasize that for a given earthquake intensity and frequency content, the period of the system is the most significant parameter. The straight line for the idealized displacement response spectrum in the linear displacement response spectra (i.e. Fig. 3) is extended to the origin for $T<T_g$. In order to evaluate the envelope of the nonlinear displacement spectra maximum from the linear displacement spectra, the slope of the line is multiplied with $\sqrt{2}$. For reinforced concrete systems T_{eff} may be defined as $\sqrt{2}$ times the period that corresponds to uncracked member section properties. The formula is given as below in Eq. 4.

$$T_{eff} = T\sqrt{2} \tag{4}$$

For an idealized nonlinear-response spectra of a system in general (Lepage 1997):

$$D_{\max} = k.T_{eff} \tag{5}$$

where;
k : slope of straight line,
T_{eff} : effective structural period of the system

For the nonlinear displacement spectra provided in Fig. 3, the period range applied is T = 0.1 – 2.0 sec. It is observed that for low values of C_y, an excessive nonlinear displacement demand develops for NS Component of Duzce Record of November 12th, 1999 (Fig. 3). This level of nonlinear displacement demand exceeds the upper bound explained above (Lepage 1997). It is known that high displacement demand emphasizes the destruction potential of the seismic wave for certain combinations of building properties.

Accordingly; in order to avoid excessive levels of nonlinear displacement response which exceeds the upper bound developed by Lepage (Lepage 1997); Eq. 6 is developed and proposed for typical reinforced concrete buildings (Ozturk 2003). It considers maximum ground velocity, V_{ground} and base shear strength coefficient, β for the evaluation of maximum nonlinear displacement response, Δ_m of a structure.

In Fig. 3 the corresponding lines are drawn using Eq. 6 which is developed in order to provide an upper bound to nonlinear displacement response of a structure, Δ_m (Ozturk 2003):

$$\Delta_m = \frac{V_{ground}{}^2}{\pi\beta g}(1+T) \tag{6}$$

where;
V_{ground} : maximum ground velocity,
β : constant used to define base shear strength as a function of weight,
T : period of the structure

Definition of Buildings at Duzce Recording Station

Two reinforced concrete buildings, located in the town of Duzce, are selected (Engvall 2002). The main emphasis was to evaluate the credibility of a drift-based design approach. The strong-motion instrument (SMA-1) is located at Duzce Recording Station, at 110 km from the epicenter of the August 17$^{th.}$, 1999 event and 7.4 km from the epicenter of the November 12$^{th.}$, 1999 event. The longest distance between a structure and the seismograph station is 290 m. The ground motion under each structure is assumed to have the same acceleration-time history as the one recorded at the strong-motion station.

Two independent procedures were used in the referred investigation where a building was modeled as a two-dimensional set of linked frames for each orthogonal direction. The first procedure involved modeling the building as a linear SDOF system (Method 1) and the second a nonlinear SDOF system (Method 2).

The dynamic and strength properties were calculated for each building. In addition, the mean drift ratio (MDR) was calculated with all two methods for each direction of each building. From the mean drift ratio the probable damage level of the structure was assumed and compared with the observed damage level to determine the reliability of each procedure. Among the two methodologies that were tested, the nonlinear single-degree-of-freedom (SDOF) system has a better agreement with the observed damage. All two buildings, which are considered, survived the August 17$^{th.}$, 1999 Earthquake with light damage. However, one of the two buildings sustained severe damage after the November 12$^{th.}$, 1999 Earthquake.

Building 1

Building 1 had five stories above ground and one basement. The floor area is calculated to be 111 m^2. The floor plan of the building is given in Fig. 4 and the story heights of the building varied within the range of 2,6 m to 3,4 m. The story weight of the Building is 870 kN and the building weight is 4400 kN.

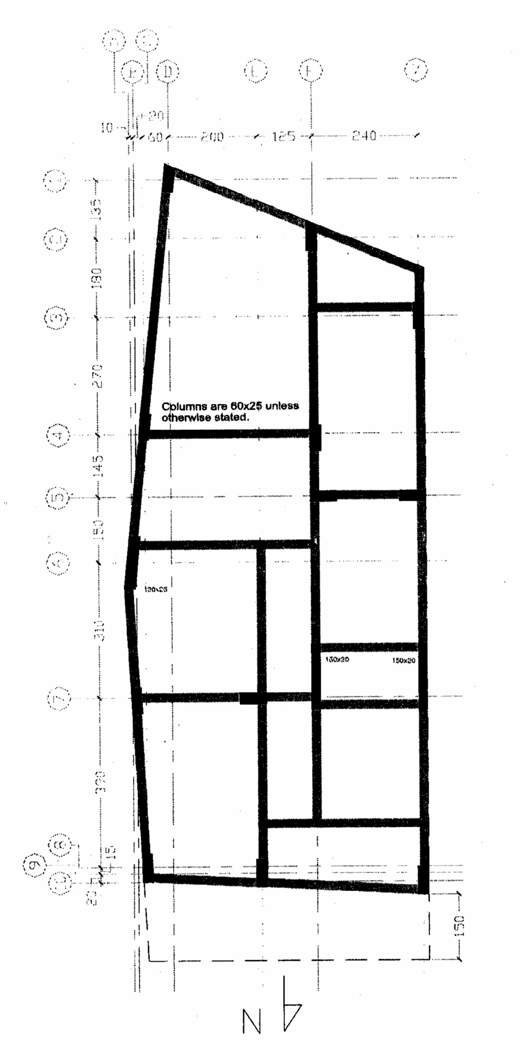

Figure 4. Floor Plan of Building 1 (Dimensions in cm).

Building 1 was standing after August 17th., 1999 Earthquake and was demolished after November 12th., 1999 Earthquake. It is known that the building was habitable after the first earthquake (August 17th., 1999).

Building 2

Building 2 is a five-story, one basement apartment building. This building is considerably larger than Building 1; the floor area is approximately 686 m^2. The floor plan is given in Fig. 5. The story height of this structure is 2.8 m. Unlike the previous structure, it has two separate shearwalls that are U-shaped. The structural frames of the building are actually built, as if they were two separate structures. The dynamic properties of two separate structures are calculated separately. The story weight and total building weight were found to be 5400 kN and 32000 kN, respectively.

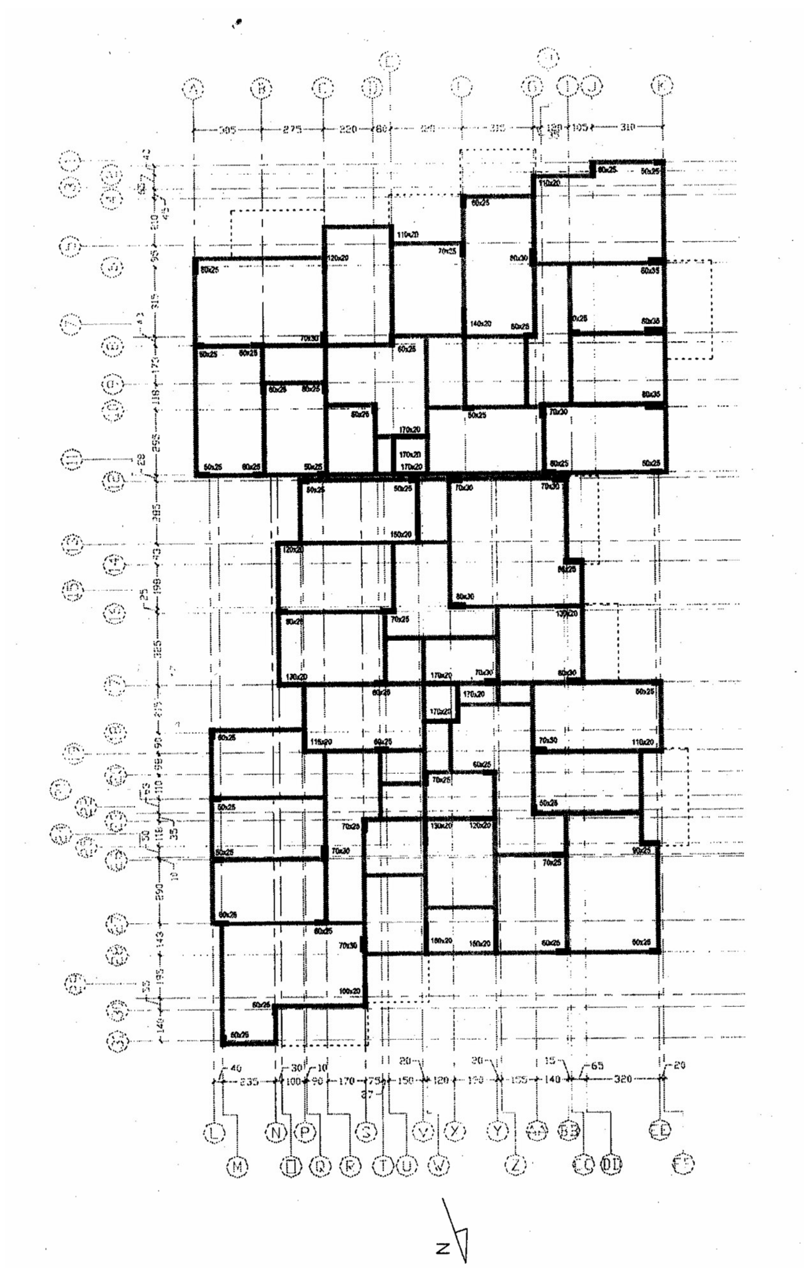

Figure 5. Floor Plan of Building 2 (Dimensions in cm).

This structure was undamaged in August 17th., 1999 Earthquake and virtually undamaged in November 12th., 1999 Earthquake. Only small 'X' shaped shear cracks were noticed in the ground floor of each of the two U-shaped shear walls.

Comparison of Observed Damage and Calculated Response of the Investigated Buildings

The basic dynamic properties such as period, participation factor, and effective weight coefficient were calculated for both structures and in each direction. In addition, the base shear strength coefficient of each building in each direction was computed from limit analysis and the combined shear capacity of the concrete columns, concrete walls, and masonry walls (Engvall 2002). These values are presented in Table 1.

Table 1. Base Shear Strength Coefficients (Engvall 2002)

Base Shear Coefficient	Building 1 NS	Building 1 EW	Building 2 NS	Building 2 EW
Shear Failure Base Shear Coefficient	1.29		0.97	
Limit Failure Base Shear Coefficient	0.22	0.13	0.15	0.14

The responses of the structural frames were evaluated using a SDOF linear system (Method 1) and a SDOF nonlinear system (Method 2) for each building and in each direction. The mean drift ratio (MDR) determined by these two methods were compared with a critical drift ratio to determine an approximate level of damage in the structure. Finally, the observed damage was compared with the calculated damage to determine the relative representation of each method. The period was determined assuming uncracked cross-sections for each building and in each direction. The period calculated for each structure is given in Table 2.

Table 2. Calculated Periods (Engvall 2002)

Period	Building 1 NS	Building 1 EW	Building 2 NS	Building 2 EW
Period (in sec) with Masonry walls	0.25	0.45	0.40	0.35
Period (in sec) without 1st story Masonry walls	0.30	0.55	0.40	0.35
Period (sec) with No Masonry Walls	0.44* 0.46(S) 0.53(N)	0.68* 0.44(S) 0.46(N)	0.50*	0.45*

* calculated by RISA 2D, assuming Flexible girders

Because Building 2 includes two separate structures, each building was modeled separately. The top period in Table 2 corresponds to the south side structure of the building and the bottom period corresponds to the north side structure of the building. The structural analysis program, RISA 2D was used to calculate the period of each structure.

If there were a large number of masonry walls in the building (such as in Building 2), they would effect the period of the building by increasing the stiffness and reducing the calculated period of the same building without masonry walls. In addition, masonry walls prior to their cracking contribute to the base shear strength of the structures which is generally not considered in the analysis of structures. The amount of masonry walls and shearwalls are provided in Table 3 as well as the column index, CI and the wall index, WI values of Hassan Index (Hassan and Sozen 1997).

Table 3. Data for the Hassan Index (Engvall 2002)

Definition	Building 1 NS	Building 1 EW	Building 2 NS	Building 2 EW
Floor Area (m^2)	111		686	
Area of Concrete Columns (m^2)	2.10		9.60	
Area of Concrete Walls (m^2)	0.60	0.00	3.03	2.58
Area of Masonry Walls (m^2)	2.76	0.94	6.26	13.59
Column Index, CI	0.95		0.70	
Wall Index, WI	0.79	0.08	0.53	0.57
Weight, (kN)	4400		32000	

The mean drift ratio (MDR) and base shear coefficient were used to define the response of each structure to the earthquake ground motions. The mean drift ratio is a measure of the average story drift and is determined by dividing the calculated roof drift by the total height of the structure. If a structure has sustained a drift of 0.2 % or more there may be damage to the masonry walls. If the structure has sustained a drift of 0.7 % or more there is intolerable damage to the masonry walls. It is assumed that concrete columns without special transverse reinforcement can withstand a drift of approximately 1.0 % before there is intolerable damage; since they are more flexible than masonry walls (Koru 2002).

The governing base shear coefficient for any structure is determined from the flexural strength and shear strength. It has to be noted that the yield base shear strength coefficient cannot control the behavior of the structure, if the drift is not large enough to cause a displacement between the top and the bottom of a column. If a structure has masonry or reinforced concrete walls, it is unlikely that the drift will be large enough to cause the yield base shear coefficient to govern unless the walls fail. Once the walls fail, the yield base shear strength coefficient becomes the governing coefficient (Engvall 2002).

The calculated MDR is used to determine a probable level of damage and this level of damage can be compared to observed level of damage. For this purpose, Eq. 6 which was developed for assessment of maximum nonlinear displacement response, Δ_m for typical reinforced concrete buildings in Turkey (Ozturk 2003) can provide an insight on determination of the level of damage to the investigated buildings.

Conclusions

The base shear coefficients to be considered are based on flexural strength of the buildings rather than the base shear coefficient based on shear strength; since for the application of Displacement-based design and MDR calculation the use of limit failure base shear coefficient is considered. The results of two methods (linear SDOF and nonlinear SDOF) are tabulated in Table 4, considering both August 17th., 1999 and November 12th., 1999 Earthquakes for the investigated buildings (Engvall 2002). The buildings are examined in both EW and NS directions.

The mean drift ratio (MDR) values for the investigated buildings are given in Table 4. It is examined that in means of the observed damage, the nonlinear SDOF system has a higher probability of correct structural response calculations compared to the linear SDOF system. Hence; Eq. 6, which was developed for the assessment of maximum nonlinear displacement, Δ_m of typical reinforced concrete buildings in Turkey (Ozturk 2003), can be considered as a helpful tool for the estimation of level of MDR and the corresponding level of damage for the investigated buildings. It has to be noted that even though the linear SDOF (Method 1) MDR values and their correctness reliability are included in Table 4, the main emphasis of this study is to test the reliability of nonlinear behavior of the selected buildings in means of nonlinear SDOF systems.

Table 4. Calculated Mean Drift Ratio (MDR) (Engvall 2002)

MDR			Method 1	Method 2
August 17th, 1999 (Marmara Earthquake)	Building 1	North-South	0.2 %	0.4 %
		East-West	2.5 %	0.9 %
	Building 2	North-South	1.1 %	0.5 %
		East-West	2.3 %	0.4 %
November 12th, 1999 (Duzce Earthquake)	Building 1	North-South	0.2 %	0.8 %
		East-West	4.5 %	1.7 %
	Building 2	North-South	0.8 %	0.7 %
		East-West	4.2 %	0.5 %

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