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A TALE OF THREE CITIES

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ABSTRACT

To our knowledge, no single building has experienced three successive strong ground motions from major earthquakes that occurred nearby. In Turkey this happened in a roundabout way when three identical buildings designed to serve as the provincial directorate offices for the Ministry of Public Works and Resettlement (MPWR) underwent such an experience over a time span of 11 years in three different cities that were hit by major earthquakes. The provincial offices are designed and constructed according to template designs under the Ministry's own supervision. For ease of access and security, the Ministry's strong motion recording stations that are part of the national network are located adjacent to these buildings. This study examines the performance of three of these standard ground-plus-four-story reinforced concrete (RC) frame buildings that were subjected to strong ground motions in different cities of Turkey. Bi-directional nonlinear dynamic analyses of 3D analytical models are performed. The principal focus of these nonlinear analyses is to assess whether the analytical model of the buildings could indicate column-beam damage consistent with that observed at the sites after the earthquakes. Our results illustrate that nonlinear time history analyses are capable of indicating the occurrence of shear failure in captive columns, but they overestimate the global damage level for all buildings.

Introduction

Investigating the response of structures during the earthquakes has been a useful tool to improve methodologies for design and analysis of structures. An example is the study on the structural performance of a RC building: the extensive field and analytical investigation of the Olive View Hospital Medical Treatment and Care Facility which suffered severe damage during the 1971 San Fernando earthquake (Mahin 1976). Observed structural damage was compared with the predictions made through linear and nonlinear dynamic analysis of the mathematical models. Another similar research (Kreger and Sozen 1989) was made on the Imperial County Services Building of El Centro in California which was severely damaged during the October

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15th, 1979, Imperial Valley Earthquake. Measured response of the building was presented and a hypothesis was developed for the prediction of identified column failures. However, research examining the response of identical RC frame buildings subjected to different strong ground motions has not been carried out. This paper investigates the structural performance of typical branch office of MPWR. This is a ground-plus-four-story RC frame building constructed in the 1980s in different regions of Turkey. All buildings suffered damage with varying degrees of severity during the March 13th, 1992 Erzincan, November 12th, 1999 Düzce¸ and May 1st, 2003 Bingöl earthquakes. During these events, three-component strong ground motion data were recorded in one-story buildings adjacent to the main buildings in Bolu and Bingöl and in a one-story building two kilometers away from the main building in Erzincan. The building in Bolu sustained severe damage that we judge to represent a "life safety" level of performance while those in Erzincan and Bingöl sustained lighter damage corresponding to slightly more than "immediate occupancy." After the Düzce earthquake, a careful recording of the damage distribution was performed for the building located in Bolu. The fortuitous combination of known input motions for the buildings and their design drawings permitted us to respond to the obvious question that begs to be answered: given the tools of current computational performance assessment technology, is the damage in each of these buildings within reach of our ability to predict them by proper modeling? In the paper, the observed structural damage is compared with those predicted in bi-directional nonlinear dynamic analyses of 3D analytical models.

Description of the Strong Ground Motions

The strong ground motions used in this study were recorded by stations of the Turkish national strong-motion network. The processed data and the seismological features of the motions have been obtained from (TUBITAK 2009) which is the first systematic compilation and uniform processing on strong motion data recorded by the Turkish national strong motion network with detailed geophysical and geotechnical site measurements for all stations. The station information and important seismological features of the ground motion data used in this paper are given in Table 1.

Table 1. Seismological features of the strong ground motions.

	March 13, 1992 *Erzincan* Earthquake	**November 12, 1999 *Düzce* Earthquake**	**May 1, 2003 *Bingöl* Earthquake**
Station Location	Meteorology Building, *Erzincan*	Ministry of Public Works and Resettlement, *Bolu*	Ministry of Public Works and Resettlement, *Bingöl*
Depth (km)	22.6	10.4	10.0
Rjb* (km)	3.3	8.0	2.2
Fault Type	Strike-slip	Strike-slip	Strike-slip
$V_{s,30}$ (m/s)**	NI****	294	529
M_w***	6.6	7.1	6.3
Longitudinal PGA (g)	0.488	0.754	0.556

Transverse PGA (g)	0.412	0.821	0.282
Vertical PGA (g)	0.243	0.204	0.481
Longitudinal PGV (cm/s)	78.2	52.3	34.4
Transverse PGV (cm/s)	108.4	66.0	21.8
Longitudinal PGD (cm)	29.5	12.5	10.2
Transverse PGD (cm)	34.4	10.5	5.1

*Rjb : Joyner-Boore distance, **$V_{s,30}$: Average shear-wave velocity of the upper 30 m. soil layer
***M_w: Moment magnitude, ****NI: Not investigated

Description of the Case Study Building

The case study building is main part of the typical branch office of MPWR which is a five-building complex designed and constructed in 1980's. Other than this building there are four other facility buildings separated by seismic joints in the same compound (Fig. 1). The main building is a ground-plus-four-story RC structure 20 m by 13.2 m in plan. Story height is 3.8 m. for the ground floor and 3.2 m. for the other floors. The building is rectangular in shape with three bays in both perpendicular directions (Fig. 2).

(a) (b)

Figure 1. a) Plan of the building complex b) General view of the case study building

Beams in the exterior frames have an unusual depth of 1.2 meters with 0.3 m width. Dimensions of the beams in the interior frames are 0.3 m by 0.7 m in the longitudinal direction (Fig. 2, Axis A-F) and 0.3 m. by 0.6 m. in the transverse direction (Fig. 2, Axis 1-4). There are eight rectangular columns oriented in the longitudinal direction, five in the transverse direction and three L-shaped columns on the corners. Except for the L-shaped columns, sizes of the columns and their longitudinal reinforcement decrease progressively from the lower to upper stories but dimensions of the beams and amount of the longitudinal reinforcement do not vary with height.

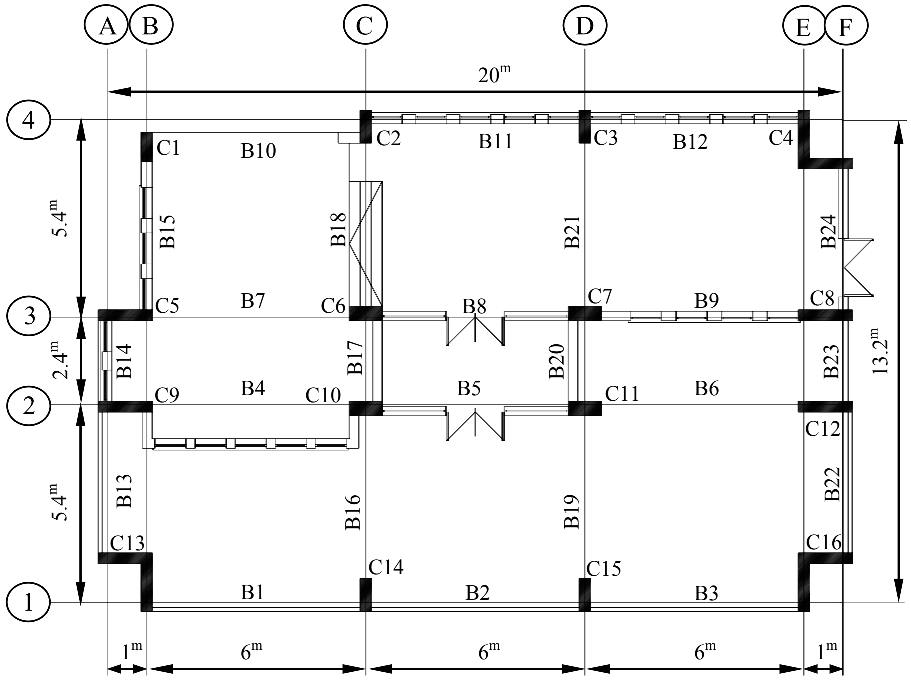


Figure 2. Ground Floor Plan

The slab thickness is 15 cm at each level. The peripheral masonry infill walls are 26 cm in thickness. The infill walls separating office rooms from corridors are 19 cm thick and those separating office rooms from each other are made up of 9 cm masonry. The amount of masonry walls is less at the ground and top floor than the other floors.

Analytical Models

3D nonlinear analytical models of the buildings in Bolu, Erzincan and Bingöl are carried out. In all models, distributed plasticity is utilized through fiber analysis approach (Perform 3D 2005) to simulate the nonlinear and bi-axial flexure behavior of the columns. Shear hinges with V_x and V_y interaction are defined at the column ends to limit the shear strength where the flexure-shear strength is larger. Beam members are composed of elastic elements with effective stiffness. Bi-linear moment-curvature relationship and elastic-perfectly-plastic shear hinges are defined at both ends. The contribution of slip deformation to the yield displacement is taken into account in beams by introducing members with reduced effective stiffness to the model. However, due to high level axial load and aspect ratio, the slip of the reinforcing bars is neglected in the columns (Elwood and Eberhard 2009). The infill walls are also taken into consideration. Eigenvalue and nonlinear dynamic analyses are based on structural stiffness after separation. The in-filled frames are modeled as equivalent diagonally braced frames that are represented by diagonal compression struts. Masses are concentrated at the mass centers of each floor. P-delta effects are included. Following customary practice, T sections are utilized for beam sections and the effective flange width values are considered as 1/5 of clear span length (ASCE 2007) of the beam. Rayleigh damping was utilized in the analytical model with 5 percent damping ratio specified for the first and fourth modes (Chopra 1995). Other assumptions about the material and loading are summarized in Table 2.

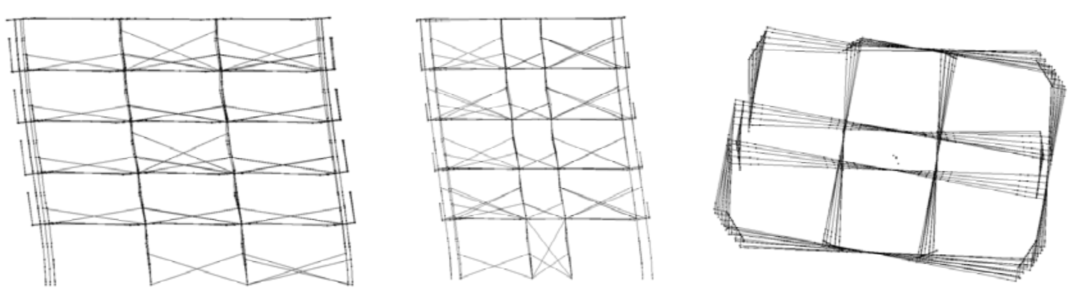
Table 2. Summary of the parameters for the analytical models

	Parameter	**Case Study Building in *Bolu***	**Case Study Building in *Erzincan***	**Case Study Building in *Bingöl***
Material	**Concrete**	f_c = 20 MPa E_c = 21170 MPa	f_c = 9 MPa E_c = 14200 MPa (ACI 2008)	
	Reinforcement Steel	f_y = 220 MPa, E_s = 200000 MPa		
Loading	**Gravity**	DL + 0.3 LL		
	Seismic dead load for mass calculation	DL + 0.3 LL		
	Mass Distribution	At mass centers		
	P-delta effect	Yes		
	Shear deformations	Yes		
	Rayleigh Damping	Yes		
Modeling	**Analysis Program**	Perform 3D		
	Rigid offset at connections	Yes		
	Effective flange width of T-beams	1/5 of the clear span of the beam on both side of the web		
	Element Models	Columns : Fiber section + Shear hinge Beams : Inelastic beam with M-K + Shear hinge Infill Walls :Strut Members		

Eigen Value and Nonlinear Dynamic Analyses

Eigenvalue Analysis

3D analytical models with nonlinear column elements are constructed (Fig. 3). Linear beams with reduced effective stiffness values are taken into account (ASCE 2007). Eigenvalue and bi-directional nonlinear dynamic analyses are conducted (Perform 3D 2005).



(a) (b) (c)

Figure 3. a) 1st Mode b) 2nd Mode c) 3rd Mode

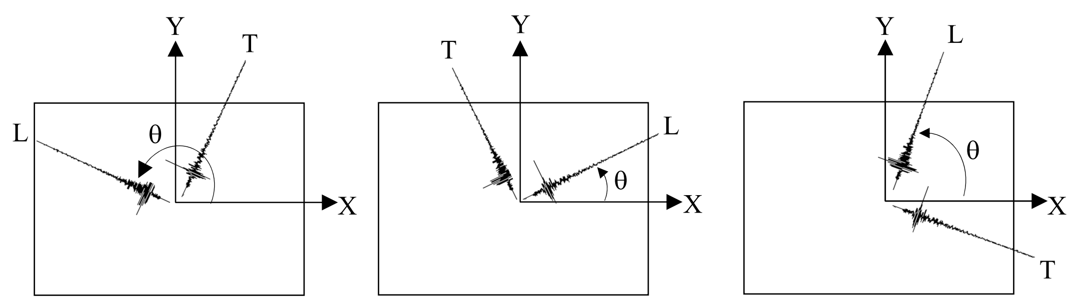
Regarding to the free vibration analysis, the first five modes are considered. The sum of modal masses for the first five modes is more than ninety percent of the total. The first and second modes are in the orthogonal directions of the building and the third mode is torsion (Fig. 3).

Table 3. Eigen value analysis results for the models conducted according to the recommendations in ASCE/SEI 41-06 of the case study

Building in	**Bolu**			**Erzincan and Bingöl**		
Mode	**Period**	**MPF (%)**	**MPF (%)**	**Period**	**MPF (%)**	**MPF (%)**
	(sec)	**Long. Dir.**	**Trans. Dir.**	**(sec)**	**Long. Dir.**	**Trans. Dir.**
1	0.39	78.4	0.8	0.45	78.3	0.4
2	0.35	0.7	79.2	0.40	0.3	78.6
3	0.28	0.2	0.1	0.32	0.2	0.3
4	0.12	1.3	12.9	0.14	1.5	13.1
5	0.11	12.5	1.7	0.13	12.5	1.9

Nonlinear Dynamic Analysis

As the strong motion sensors had been located with an angle relative to the orthogonal axes of the buildings in the field, the horizontal components of the ground acceleration are applied with an angle to the analytical models. The orientation of the sensors with respect to the buildings is shown in Fig. 4.



(a) (b) (c)

Figure 4. Application of longitudinal and transverse components of the ground motions to the case study buildings in a) Bolu, θ=165° b) Erzincan, θ=26° c) Bingöl, θ=70°

Bi-directional nonlinear dynamic displacement results are shown in Fig. 5. The maximum global drift ratio which is defined as the floor displacement divided by the floor height from the base are 0.96, 1.32, and 0.47 for the analytical models of the buildings in Bolu, Erzincan and Bingöl, respectively.

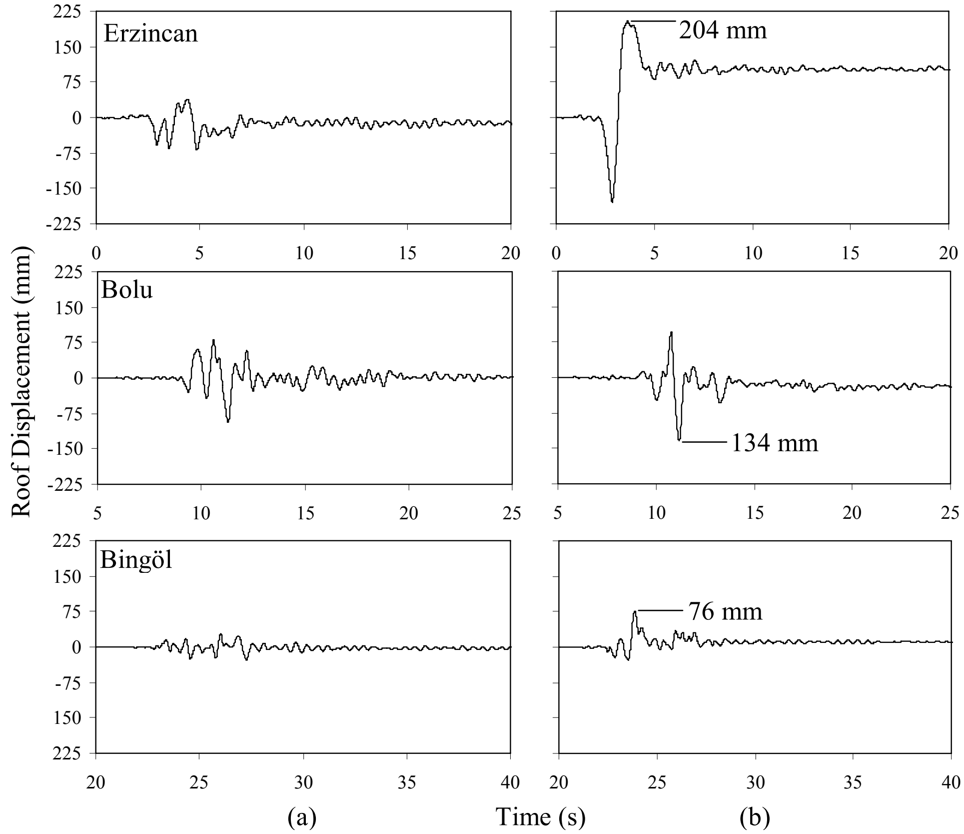


Figure 5. Bi-directional nonlinear dynamic displacement results of the building in Erzincan, Bolu and Bingöl for their (a) longitudinal and (b) transverse directions

The structural performance levels of the buildings are determined based on inter-story drift ratios as prescribed in ATC-40 (ATC 1996) and shown in Table 4. The performance level of the building in Bolu is calculated to be slightly higher than immediate occupancy (IO). The building in Erzincan is at damage control (DC) and the structure in Bingöl is at IO level.

Table 4. The performance levels calculated according to ATC-40

Building in	**Bolu**	**Erzincan**	**Bingöl**
Max. inter-story drift (%)	1.01	1.49	0.56
Performance Level	Lowest limit of DC*	DC	IO**

*Damage Control **Immediate Occupancy

Observed Structural Damage

The building in Bolu sustained severe damage that we judge to represent a “life safety” level of performance while those in Erzincan and Bingöl sustained lighter damage corresponding to slightly more than “immediate occupancy.” In this paper, only the building located in Bolu will be discussed due to space limitations.

After the Düzce earthquake, a careful recording of the damage distribution was performed for the building in Bolu. The structural damage was concentrated in the lowest three stories. Damage consisted essentially of diagonal shear cracks in the columns, shear failure in the captive columns (Fig. 6) and infill wall failures. Flexural cracks were observed in almost all beams of the first three floors. Crushing of concrete, buckling of longitudinal steel and disengagement of ties was noticed after a careful investigation (Cagnan 2001).

Figure 6. Captive column effect and buckling of longitudinal steel

Comparison of Analytical Results with the Observations

Results regarding to the bi-directional dynamic analysis of numerical model performed for the building in Bolu are compared with the observations made after the earthquake. Structural members are evaluated according to ASCE/SEI 41-06 at member level. The performance of beams and shear-critical columns is evaluated by means of plastic rotation and shear demand, respectively. The infill walls are also assessed according to their strength values. Column C3 at the first and the second floor of the building experiences shear failure and severe buckling due to captive column effect. Bi-axial shear response of the column at the first floor of the building is shown in Fig. 7. The shear capacity of the column is indicated by the dashed lines. Observed damage in the columns is due to shear which is obvious by the diagonal cracks. The shear capacity of the columns is calculated according to the formulation specified in ASCE/SEI 41-06:

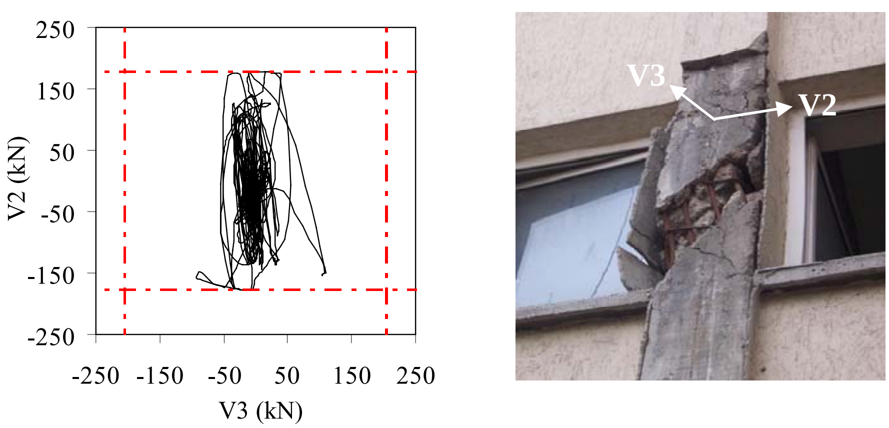


Figure 7. The bi-axial shear behavior (captive column effect) of the column C3 at the first floor

$$V_n = V_s + V_c = k\frac{A_v f_y d}{s} + \lambda k \frac{0.5\sqrt{f_c'}}{M/Vd}\sqrt{1+\frac{P}{0.5\sqrt{f_c'}A_g}}0.8A_g \tag{1}$$

where λ is taken as 1.0 for normal weight concrete, k is assumed 0.7 in regions of high ductility demand, f'_c is the specified compressive strength of concrete, M/V is the largest ratio of moment to shear under design loadings and not taken greater than 3 or less than 2, d is the effective depth, P is the axial compressive force and A_g is the gross sectional area of the column. A_v is the area of shear reinforcement within a distance s, f_y is the specified yield strength of reinforcement.

The assessment which is made according to ASCE/SEI 41-06 at member level shows that the Eq. 1 which is used to calculate the shear strength of the columns, underestimates the shear capacity of the columns subjected to bi-axial dynamic loading. In the mathematical model, the columns reach their shear strength even where only minor diagonal cracks occur. The columns are in immediate occupancy performance level (IO) for flexure which is consistent with observations. The analytical results match the observed data with 86%, 93% and 70% success ratio for the infill walls at the ground, first and second floors, respectively.

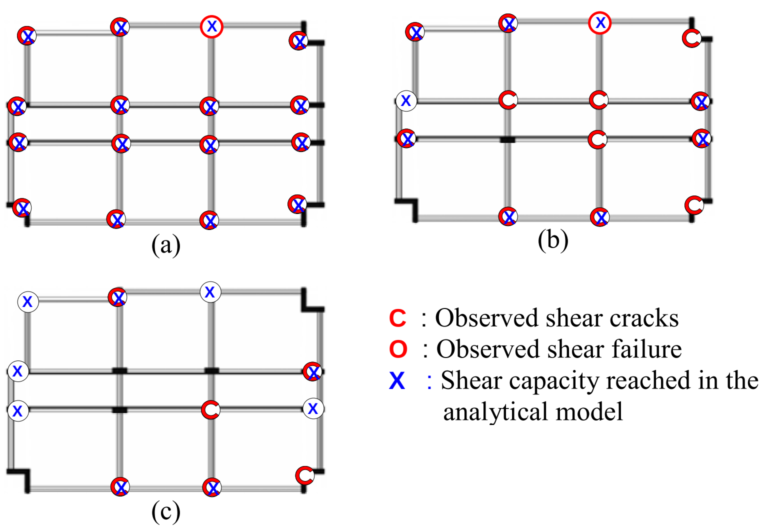


Figure 8. Shear performance of the columns at (a) ground (b) first and (c) second floor

Conclusion

Earthquake damage and loss influences the need for reconsideration of the current design and evaluation procedures. In this concept, past major earthquakes teach new lessons. Prior to this research to our knowledge, no single building has experienced three successive major earthquakes. This happened in a roundabout way in Turkey. In different regions of Turkey, three identical buildings designed to serve as the provincial directorate offices for the Ministry of Public Works and Resettlement (MPWR) underwent such an experience. In this paper, performance of these ground-plus-four storey buildings is investigated according to the existing procedures. The strong motion recording stations that are part of the national network are located adjacent to these buildings and provide reliable input data for the simulation of the motion and assessment of the building. The evaluation process is performed through bi-directional dynamic analyses of the buildings. The main purpose of these nonlinear analyses is to assess whether the analytical model of the buildings could indicate column-beam damage consistent with that observed at the sites after the earthquakes. Our results illustrate that nonlinear time history analyses are capable of indicating the occurrence of shear failure in captive columns, but they overestimate the global damage level for all buildings, especially where the building sustained a pulse type motion but did not show any significant distress. It is concluded that although current methodologies and guidelines give reasonable estimates for the performance of the structures, the performance limits and criteria need to be further refined.

Acknowledgements

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