16th World Conference on Earthquake, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 3050
Registration Code: S-T1459418126

Field Experience toward Validation of Performance-Based Assessment Procedures

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Abstract

Much intellectual capital has been invested in the pursuit of crafting increasingly more refined procedures that collectively comprise the performance based design and assessment methodology. Engineers who are educated at the public's expense operate somewhere between banking and physics in their service to the public, and yet it is usually the banking that wins. If PBD is to turn the table in favor of the engineers, then its bag must be able to exhibit convincing cases that it performs well itself. Field confirmation is a pre-requisite for unconditional adoption. To our knowledge, no major building that has been explicitly designed according to PB anywhere in the world has ever experienced a strong-enough earthquake that would confirm the validity of the design path. Post-de-facto assessment exercises for existing buildings are inconclusive at best.

Every theory must pass the test provided by nature serving engineering as a reliable instrument. Earthquakes are seldom events, and there is as yet little empirical evidence for existing RC building performance well in the nonlinear range except under infrequently occurring conditions or in the lab. Building stock elements are less perfectly known, so there is more uncertainty in modeling them. This study proposes to answer the rhetorical question of "do we have the tools for forecasting building performance under actual earthquakes of real buildings, given their blueprints and their input motion?" In Turkey, cast-in-place, older-type RC buildings were subjected to near-field strong ground motions from three major earthquakes. This occurred over a time span of eleven years. Three identical institutional buildings that had been built to the same design templates were shaken by three different M6.3 or larger earthquakes. The input ground motions recorded in the near-field (less than 10 km R_{jb}) were directly relevant for the buildings. Given the damage information, input motions, design drawings and material properties for all of the buildings; we indulged in replicating analytically the structural damage that occurred in these buildings. Three dimensional (3D) analytical models of the buildings were created using state-of-the-art procedures. Bi-directional excitations have been applied to the models using nonlinear dynamic analysis capability. The results illustrate that the analyses results overestimate the global damage level for all buildings. The overestimation is more significant in one case where the building sustained a pulse-type motion without significant distress. The lackluster performance of the prediction exercise is interpreted as a strong caveat against the irrational exuberance that PBD is the long-awaited nostrum for earthquake engineering.

Keywords: reinforced concrete buildings, nonlinear dynamic analyses, performance based design and assessment

1. Introduction

Many studies on structural performance of reinforced concrete structures have been made through analytical and experimental research but accuracy of the methods used has not been adequately confirmed with the actual data from the field. The analytical results conducted after the experimental study are claimed to predict those obtained at the laboratory conditions; however, many blind prediction tests indicate that the gap between the assumed characteristics of an actual structural system and their idealized counterparts are wide. One example was officially experienced in the blind prediction contest organized during the 15WCEE. In this contest, two full scale, three-dimensional, one-story reinforced concrete structures were subjected to bidirectional earthquake simulations on shaking table and researchers were expected to calculate the accurate reproduction of the displacement response of the structures before the experiments performed. Even most of the predictions for initial period of the structure were irrelevant to that simple mock-up. Although the actual displacement waveform was predicted well even at post-yield state, the maximum displacement values were predicted with an error at a range of 1%-40% [1]. It is obvious that even for a simplified laboratory-scale structure; it is difficult to provide reasonable estimates at post-yield level.

Every theory should be tested by the nature if it is to serve engineering as a reliable predictive instrument. So the question here is: can we forecast the building performance under actual earthquakes of real buildings, given their blueprints and their input motion while even performance of a simple structure at laboratory conditions could hardly be estimated.

The damage and failure mechanisms of structures during earthquakes provide incontrovertible evidence that emphasizes the need for reconsideration of the current procedures in design and assessment of buildings. One such learning process occurred in Turkey over a time span of 11 years. Three identical buildings, built to the same design template, experienced three major earthquakes and underwent different degrees of visible damage, though none collapsed. The provincial branch office of the Turkish Ministry of Public Works and Settlement (MPWR), a ground-plus-four story building constructed in the 1980s in different regions of Turkey, suffered damage to varying degrees of severity during the 13 March 1992 Erzincan, 12 November 1999 Düzce, and the 1 May 2003 Bingöl earthquakes (Fig. 1).

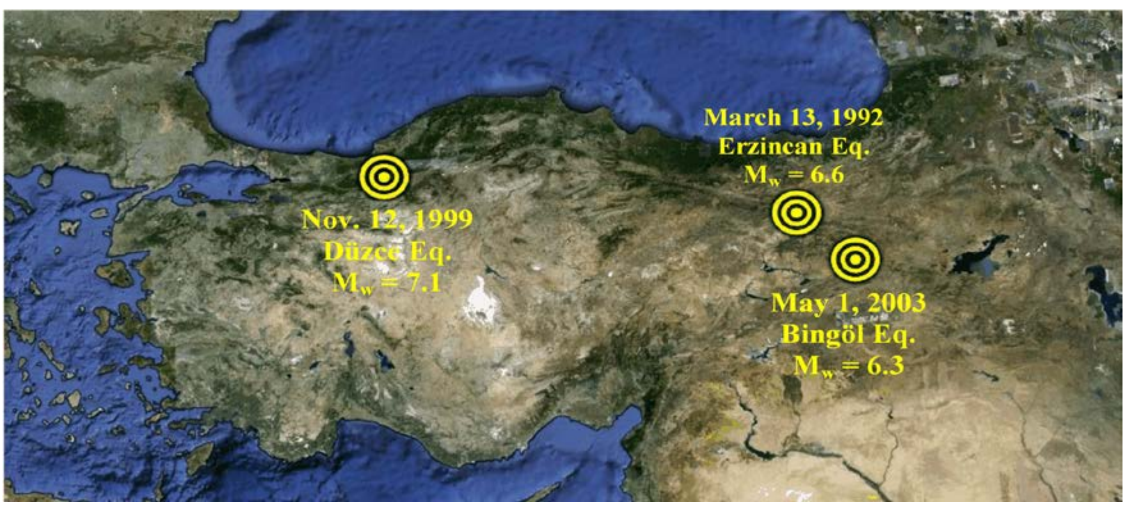


Fig. 1 – Epicenters of the earthquakes (adapted from http://www.google.com)

During these events, three-component strong ground motion data were recorded in a one-story building adjacent to the case-study building in Bolu and Bingöl, and in a one-story meteorological services building about two kilometers away from the case study building in Erzincan. That the ground motions are known for two of the three buildings is a uniquely fortuitous occurrence. The exception was that, the motion in Erzincan was recorded by a station situated about two km from the building. However, the ground composition between the sites is very similar and no tall buildings existed in the vicinity of the recording station to modify the ground significantly.

Thus, in the absence of a better theory it will be assumed here that the record represents the input motion to the building.

After the Düzce earthquake, a careful examination of the damage state was performed for the building in Bolu. A similar exercise was conducted in Bingöl four years later. The Erzincan building was not subjected to an investigation in 1992 as the other two because it seemed practically intact following the earthquake and served as an important critical facility for attending to the needs of the homeless citizens. The Erzincan Building was judged to be in "immediate occupancy" status by its users that included engineers and damage assessors employed by the ministry.

The known input motions for the buildings, their design drawings, material properties and structural damage information provided an opportunity to evaluate the current performance assessment methods. Here, the answer to the question of whether we could predict the seismic damage in RC buildings at the site well by proper modeling will be investigated. The comparisons that will be made between the models and the real buildings are expected to provide a test of the concepts embodied in structural performance assessment procedures in the light of empirical evidence.

2. Description of the Building

The case study building is main part of the typical branch office of the Ministry of Public Works and Resettlement building which is a five-building complex designed and constructed in the 1970s and 1980s, respectively. In all branch office complexes, there are four service buildings in addition to the main building. All buildings are separated by seismic joints in the same compound at all locations (Fig.2b). Here, particular emphasis will be placed on the case-study building that is shown in Fig.2a and will be called for short the "MPWR Building" in this study.

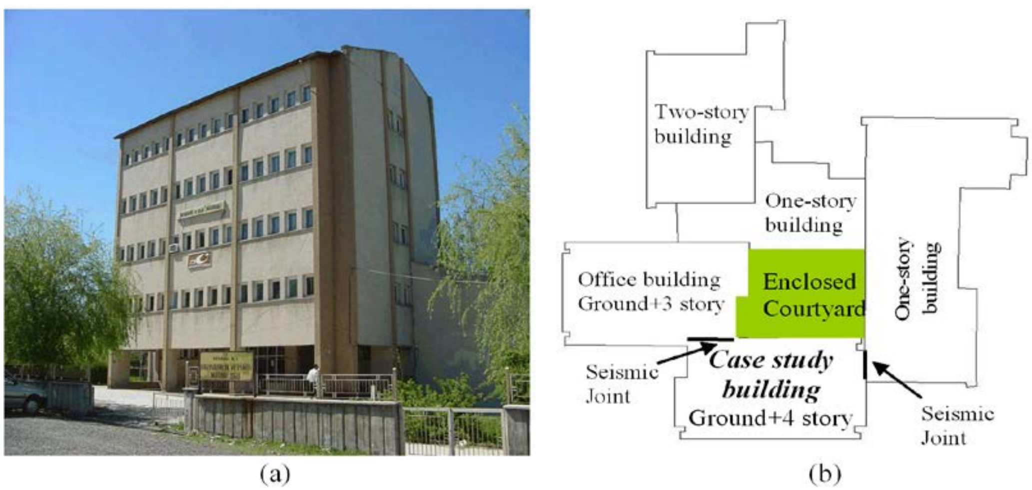


Fig. 2 – a) General view of the case study building b) Plan of the building complex

The building is a ground-plus-four-story RC structure where the story height is 3.8 m in the ground floor and 3.2 m in the rest. The building is rectangular in shape with three bays in both orthogonal directions. The plan dimensions are about 20 m and 13 m in the longitudinal and transverse directions, respectively. The building consists of columns, beams and slabs for carrying the vertical load. To resist the main portion of lateral load three L-shaped columns exist on the corners which are continuous from the ground floor to the roof. These columns are connected with peripheral deep-beams (Fig.3). Eight rectangular columns (Fig. 3) have their strong axis oriented in the X direction and five rectangular columns in the Y direction of the building. Except for the L-shaped corner columns, sizes of the columns and their longitudinal reinforcement in these members decrease progressively from the lower to upper stories but dimensions of the beams and amount of their longitudinal reinforcement do not vary with height.

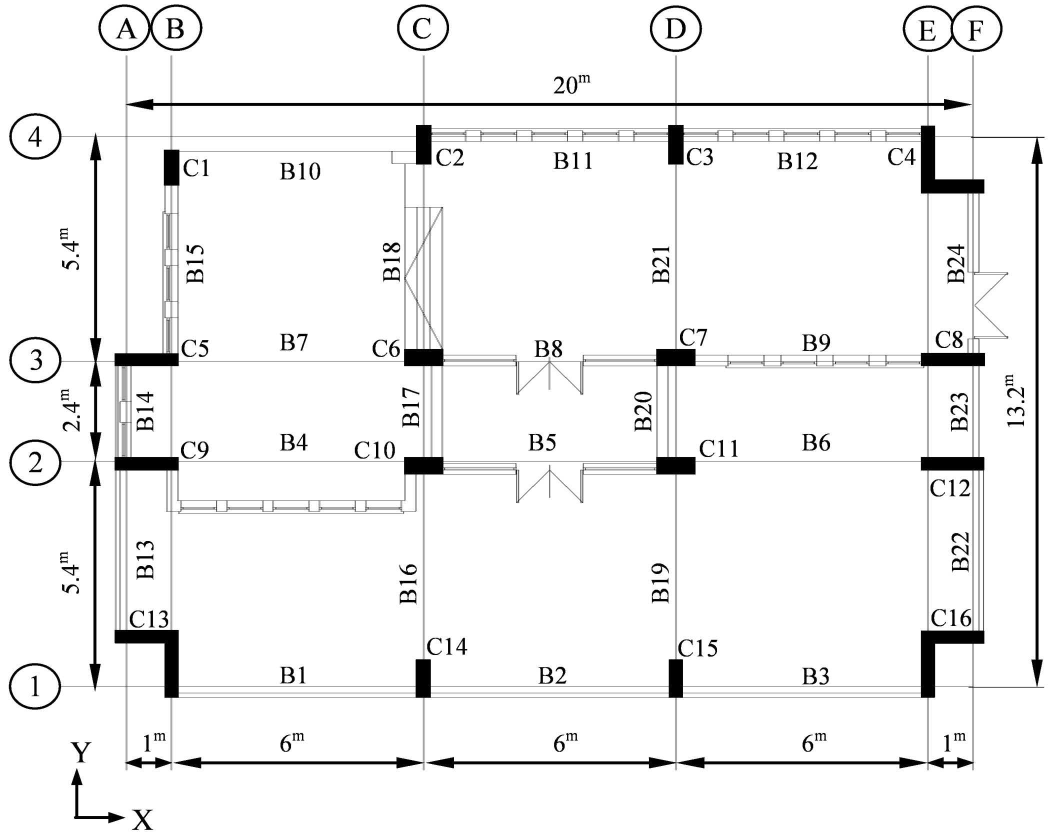


Fig. 3 – Typical floor plan of the MPWR building

The dimensions of the cross sections, longitudinal and transverse bar condition of typical column and beam sections are shown in Fig.4 and 5, respectively. For further information please refer to [2, 3].

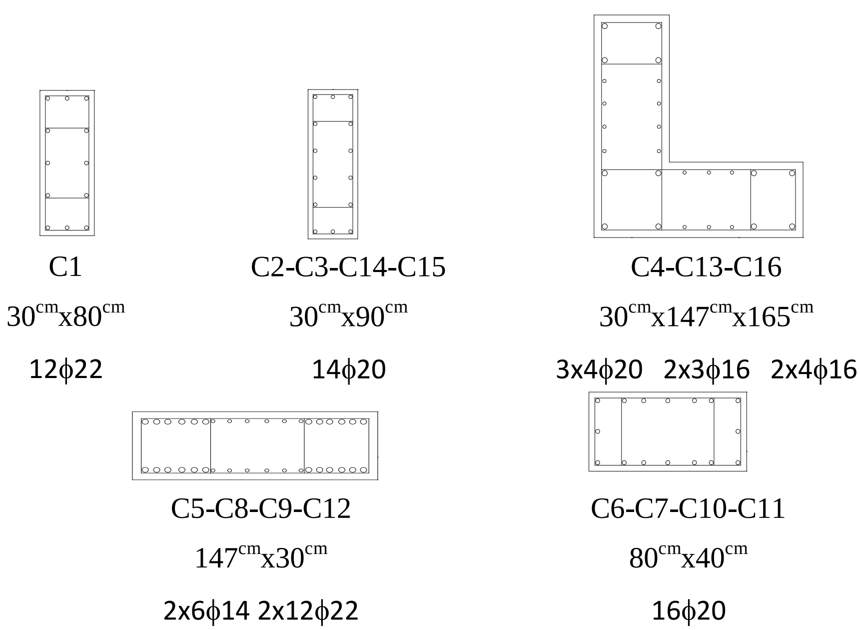
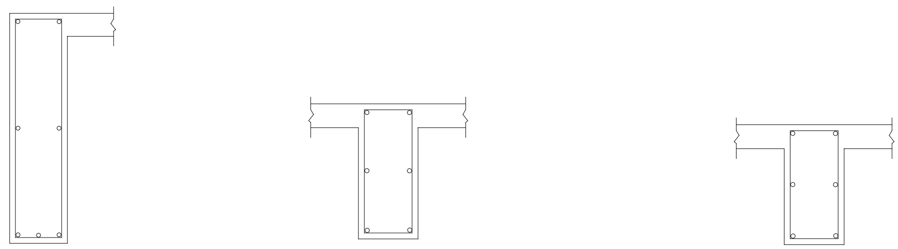


Fig 4. Cross-section dimensions, longitudinal and transverse bar condition of typical column sections



Peripheral Beams
30^{cm} x 120^{cm}
3ϕ14+2ϕ14+2ϕ12
(a)

Interior Beams – Long. (X) Dir.
30^{cm} x 70^{cm}
2ϕ14+2ϕ14+2ϕ12
(b)

Interior Beams – Trans. (Y) Dir.
30^{cm} x 60^{cm}
3ϕ14+2ϕ12+2ϕ12
(c)

Fig.5. Cross-section dimensions, longitudinal and transverse bar condition of (a) peripheral beams, (b) interior beams in X direction and (c) interior beams in Y direction of the building

2.1 Material Properties of the Buildings

The average characteristic compressive strength of the concrete was measured as 20 MPa from the concrete samples taken from the Bolu building. The corresponding values for the buildings in Erzincan and Bingöl were obtained from the technical report prepared by the engineers of MPWR [4]. They reported that measured average characteristic compressive strength of both buildings was 9 MPa. Hence, they applied the same retrofit project to both buildings after the 2003 Bingöl earthquake.

3. Description of the Strong Ground Motions

The strong ground motions used in this study were recorded by the stations of the Turkish national strong-motion network. The processed data and seismological features of the motions have been obtained from a systematic compilation and uniform processing on strong motion data recorded by the Turkish national strong motion network [5]. The detailed geophysical and geotechnical site surveys for all of the stations were available. The station information and other seismological features of the ground motion are summarized in Table 1.

Table 1 – Seismological features of the strong ground motions, M_w: Moment magnitude, R_{jb}: Joyner Boore distance, HRV: Harvard Centroid Moment Tensor, V_{S30}: Average shear-wave velocity of the upper 30 m soil layer

Earthquake	**March 13, 1992** ***Erzincan***	**November 12, 1999** ***Düzce***	**May 1, 2003** ***Bingöl***
R_{jb} (km)	3.3	8.0	2.2
Fault Type	Strike-slip	Strike-slip	Strike-slip
$V_{s,30}$ (m/s)	-	294	529
Soil Type *(Ambraseys et al., 2005)*	Stiff soil	Soft Soil	Rock
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
Longitudinal PGV (cm/s)	78.22	52.33	34.48
Transverse PGV (cm/s)	108.4	66.03	21.87
Longitudinal PGD (cm)	29.5	12.5	10.2
Transverse PGD (cm)	34.4	10.5	5.1

The acceleration response spectra of the ground motions are given in Fig.6 for (a) longitudinal and (b) transverse components of the earthquakes. The significant feature of Bolu record is that contains strong pulse fling which is a characteristic of near-field ground motion [6]. An important note about the Erzincan record is that contains large acceleration pulse causing greatly enhanced ground story drift ratio demands. According to [2], this is the most severe demand a near-source ground motion imposes on structural frames.

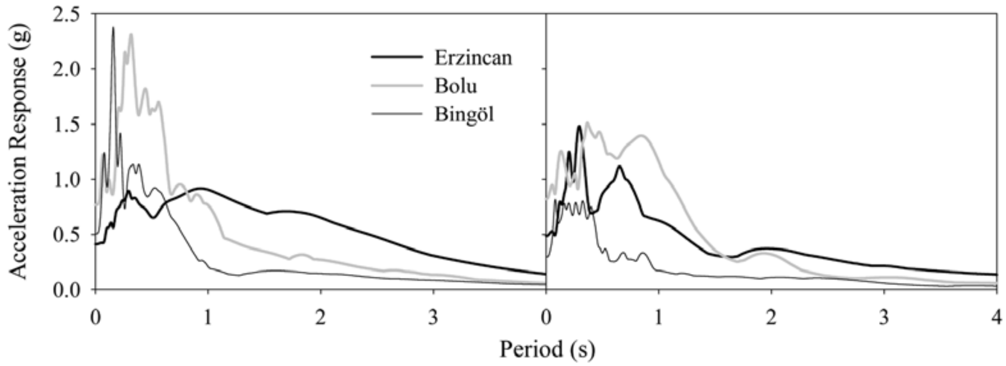


Fig. 6 – Acceleration response spectra (5% damping) graphs of the ground motions for (a) longitudinal and (b) transverse components of the earthquakes.

3. Description of the Analytical Model

3D nonlinear analytical models of the buildings in Erzincan, Bolu and Bingöl were carried out in Perform3D software [7]. In all models, distributed plasticity was utilized through fiber analysis approach in order to simulate the nonlinear and bi-axial flexure behavior of the columns. Beam members were introduced as linear elements with reduced effective stiffness ($0.3E_cI_g$) per [8]. Bi-linear moment-curvature relationship was assigned at both ends of the beam members. Elastic-perfectly-plastic shear hinges were assigned at both ends of the beam and columns representing the shear force-deformation relationship. The contribution of shear reinforcement and concrete to the shear capacity of the beam hinges was calculated by the equation stated in [9].

$$V_n = V_s + V_c = \frac{A_v f_y d}{s} + 0.166\sqrt{f_c'}\, b_w d \tag{1}$$

where A_v is area of the shear reinforcement within a distance s, f_y is the yield strength of the reinforcement, d is the distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement, f_c' is the compressive strength of concrete and b_w is the web width of the section.

The ultimate shear strength values defined for the column hinges were calculated by the formulation given in [10];

$$V_n = V_s + V_c = k\frac{A_v f_y d}{s} + \lambda k \frac{0.5\sqrt{f_c'}}{M/Vd}\sqrt{1+\frac{P}{0.5\sqrt{f_c'}A_g}}\,0.8A_g \tag{2}$$

where λ is taken as 1.0 for normal weight concrete, k is assumed 0.7 in regions of high ductility demand, M/V is the largest ratio of moment to shear under design loadings and not taken greater than 3 nor 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. The longitudinal spacing of transverse reinforcement exceeds half of the component effective depth in strong directions of the columns and beams, so the transverse reinforcement was assumed 50 percent effective in resisting in shear [10]. This assumption might seem to be very conservative and affect overall results but the deficiencies observed in the construction (Fig. 7) engendered us to take this decision.

Fig. 7 –Disengagement of ties (Bolu Building)

The contribution of slip deformation to the yield displacement was 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 was neglected in the columns [11]. The infilled frames were modeled as equivalent diagonally braced frames that are represented by the diagonal compression strut. For the details, the reader is referred to [2, 3].

All other assumptions about material and loading employed in the analytical modeling are summarized in Table 2. The yield strength of the reinforcement steel is 220 Mpa for all buildings. The compressive strength of concrete was measured as 20 Mpa in the case of Bolu and 9 Mpa for the cases of Erzincan and Bingöl.

Table 2 – Summary of the parameters for the analytical models of the case study buildings

	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 *(ACI318, 2008)*	
Loading	**Reinforcement Steel**	f_y = 220 MPa, E_s = 200000 MPa		
	Gravity	DL + 0.3 LL		
Modeling	**Seismic dead load for mass calculation**	DL + 0.3 LL		
	Mass Distribution	At the mass centers		
	P-delta effect	Yes		
	Shear deformations	Yes		
	Rayleigh Damping	with 5 percent damping ratio specified for the first and fourth modes [12]		
	Analysis Program	Perform 3D, 2005		
	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: Elastic beam with Moment-Curvature hinge + Shear hinge Infill Walls : Compressive Strut Members		

The fundamental period of the Bolu Building is calculated as 0.39 s and 0.35 s. in the longitudinal (X) and transverse (Y) direction, respectively. Those calculated for the Erzincan and Bingöl Buildings are the same and calculated as 0.45 s and 0.40 s. in the longitudinal (X) and transverse (Y) direction, respectively. It is to be noted that following the 2003 Bingöl earthquake, the strong motion sensors were located temporarily at the 4th floor (top floor) of the building to record the aftershocks. Using the processed aftershock traces [5] of response recorded both at the fourth floor and ground level, Fourier amplitude spectrum (FAS) analyses were performed to obtain the dominant frequency of the building in both orthogonal directions. The mean fundamental period value was calculated as 0.56 s. for the X and 0.60 s. for the Y direction of the building. These calculated period values are in good agreement with elastic periods of the analytical model. This prediction is also consistent with the period ranges calculated in the studies of [13] and [14].

4. Nonlinear Dynamic Analyses

The strong motion sensors had been located with an angle relative to the orthogonal axes of the buildings in the field, so the horizontal components of the ground acceleration were applied with an angle to the analytical models. The orientations of the sensors with respect to the buildings are shown in Fig.8.

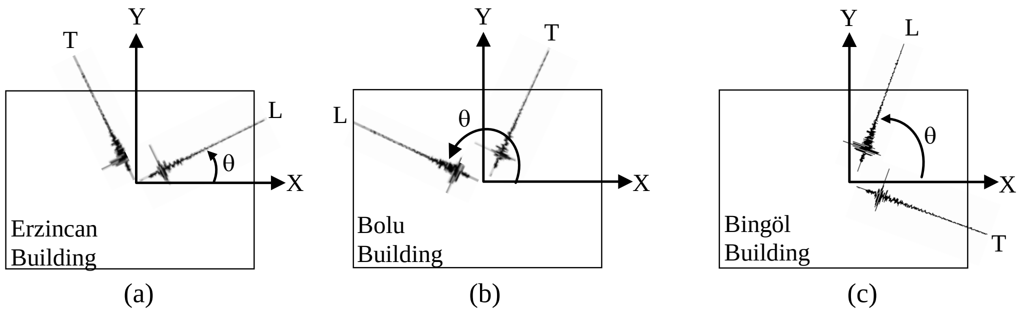


Fig. 8 – Application of orthogonal components of the ground motions to the case study buildings in a) Erzincan, θ=26°, b) Bolu, θ=165° and c) Bingöl, θ=70°

The maximum roof displacements obtained from bi-directional nonlinear NDA are given in Fig.9. The maximum displacement results are 20.4, 13.4 and 7.6 cm corresponding to the global drift ratio (GDR) values of %1.23, 0.81 and 0.46 percent for the analytical models of the buildings in Erzincan, Bolu and Bingöl, respectively.

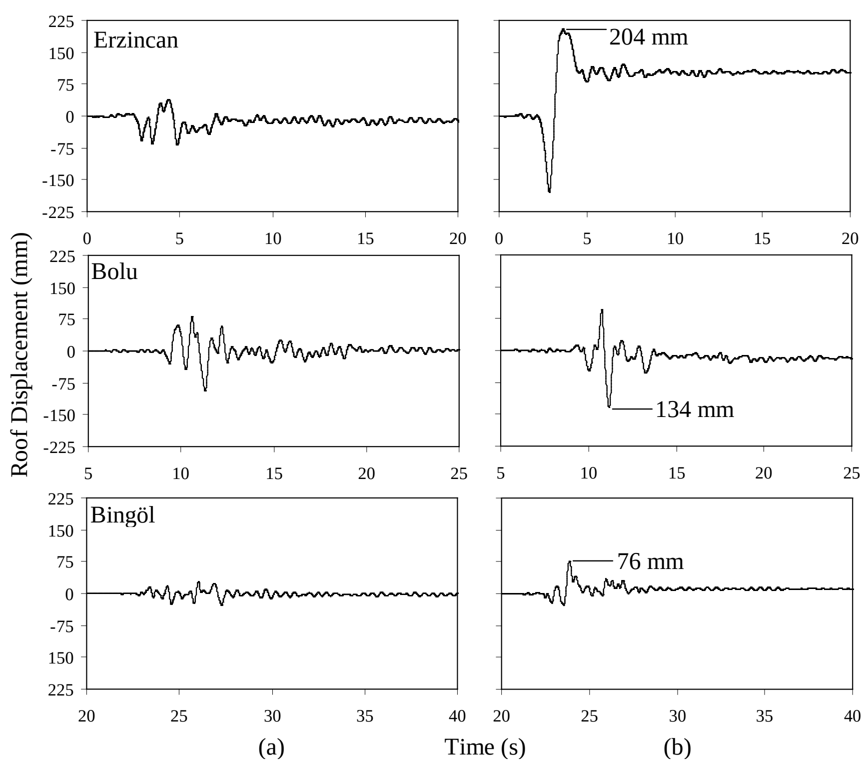


Fig. 9 – Bi-directional nonlinear dynamic displacement results of the buildings in Erzincan, Bolu and Bingöl for (a) X and (b) Y directions of the buildings

The inter-story drift ratio results of bi-directional NDA of the buildings are shown in Table 4. It indicates that the building in Erzincan has the highest and that in Bingöl has the lowest inter-story drift ratios. If the structural damage were directly related to these values, we would expect the building in Erzincan would suffer the most severe damage among the others. However, our analytical results are in diametric contradiction with the damage observed.

Table 4 – Performance level of the buildings based on deformation limits of [15]

	Nonlinear Dynamic Analyses		
	Erzincan	**Bolu**	**Bingöl**
Max. Inter-story drift (%)	1.49	1.01	0.56
Performance Level [15]	*Damage Control (DC)*	*Damage Control (DC)*	*Immediate Occupancy (IO)*

It is not convenient or fair to arrive at a general conclusion by using these results; however, in all analyses, the performance level estimation for the building in Bingöl is similar to that of real case. The analyses give variable estimates for the performance level of the building in Bolu. DC-LS levels are acceptable judgments compared to the observations made after the earthquake.

The most surprising results were calculated for the building in Erzincan. The NDA, which are expected to give the most accurate results, estimates a performance level worse than the case in Bolu. In addition, it estimates approximately the same performance level (DC) both for the building in Erzincan and Bolu which is not consistent with the real case where the observed damage was much higher for the building in Bolu than that in Erzincan.

5. Comparison of Analytical Results with the Observations

The results of bi-directional nonlinear dynamic analyses are compared with the observations made after the earthquakes.

5.1 The Bolu Building: heaviest damage

The column C3 at first and second floor of the building experiences shear failure and severe buckling due to the captive column effect. Bi-axial shear response of the columns at the ground and first floor of the building is shown in Fig.10.

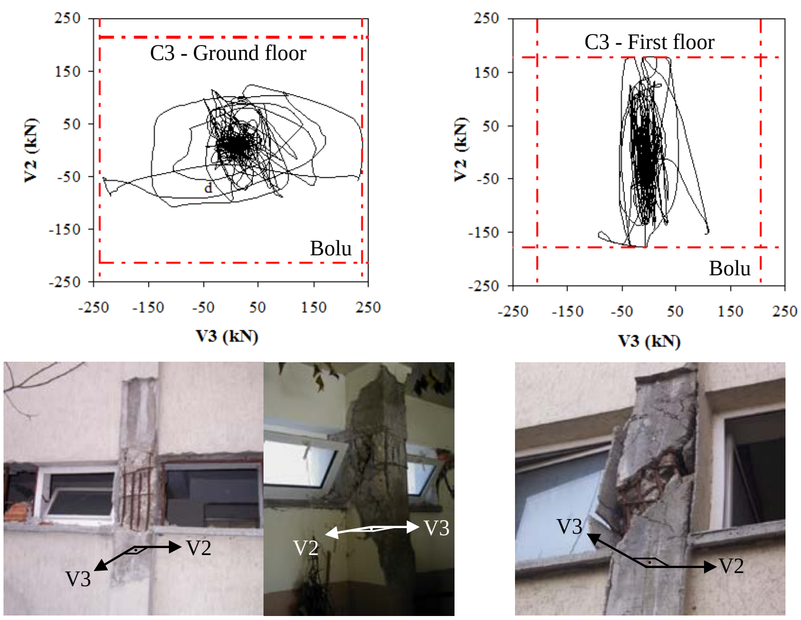


Fig. 10 – Bi-axial shear behavior of the of the column C3 at the (a) ground floor and (b) first floor (dashed lines)

Observed damage in the columns is due to shear which is obvious by the extensive diagonal cracks. The assessment results indicate that the Eq. (2) underestimates the shear strength of the columns under dynamic loading. In the analytical model, columns reach their shear capacity even where only minor diagonal cracks occur. Flexural cracks were observed in most of the beams of the first three stories but no shear failure was observed which is in agreement with the analytical results. All beams in the X direction were calculated to be in IO performance level for the first three floors. However, for some of the beams in the Y direction, results are calculated between IO-LS.

5.2 The Bingöl Building: Intermediate

Diagonal shear cracks of the columns were reported to be larger than those of the building in Erzincan; however, no numeric data was described about the crack width. Shear damage in column C3 at first floor of the building is shown in Fig.11.

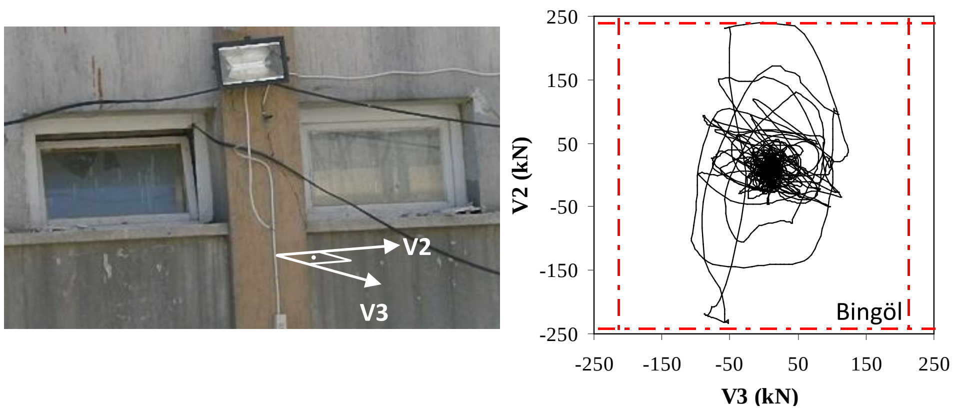


Fig.11 - Bi-axial shear behavior of the column C3 at the ground floor (dashed lines represent the shear strength calculated by Equation 2)

Hairline flexural cracks were observed in most of the beams of first three stories. All beams were calculated to be in IO performance level for the first three floors which is consistent with the observed damage.

5.2 The Erzincan Building: least severe damage

Minor shear cracks were observed at the ground and first floor levels after the earthquake. Analytical results appear to underestimate the shear capacity of the columns. The columns with diagonal shear cracks were calculated to fail which is not consistent with the observations. Beams of the ground and first floors were calculated to be in the range of IO-LS performance levels. Plastic rotation values were calculated higher for the beams in the Y direction. The ground and first floor beams had larger flexural cracks relatively to those of upper stories; however, all beams were reported to be in the IO level.

6. Concluding Remarks

It is an obvious truth that every theory must pass the test provided by nature before serving engineering as a reliable instrument. While performance assessment procedures are being developed and refined, their corroboration with field observations is necessary. Earthquakes are seldom events, and there is as yet little empirical evidence for existing RC building performance well in the nonlinear range except under infrequently occurring conditions or in the lab. Building stock elements are less perfectly known so there is more uncertainty in modeling them. This research answers the rhetorical question of “do we have the means of forecasting building performance under actual earthquakes of real buildings, given their blue prints and their input motion?”

Prior to this study, to our best knowledge, no cast-in-place RC building has ever been subjected to near-field strong ground motions from three major earthquakes. This happened in an indirect way in Turkey over a time span of eleven years. Three identical buildings belonging to MPWR that had been built to the same design templates experienced 1992 Erzincan earthquake in Erzincan, 1999 Düzce earthquake in Bolu and 2003 Bingöl earthquake in Bingöl, respectively. The ground motion sensor stations were fortuitously nearby in an adjacent single story building in Bolu and Bingöl. The station in Erzincan was about 2 km away from the case study building but we assume that the record applies to the building there. The buildings sustained varying degrees of damage during the earthquakes. After the Düzce earthquake a careful examination of the damage distribution was performed for the building in Bolu. A similar exercise was conducted for the building in Bingöl four years later. The Erzincan building seemed to be intact following the earthquake and served as an important facility for processing applications from homeless citizens seeking re-housing.

Given that the damage information, input motions, design drawings and material properties of the buildings are all known, the following question comes to mind: Could we have predicted the structural damage that occurred in these buildings by proper modeling using the tools of current computational performance assessment procedures? For any procedure to qualify as a scientific instrument, it needs to be reasonably able to predict events. In order to that, the MPWR buildings were analyzed. The description of the buildings, input ground motions and observed structural damage were described in detail. Then, the analytical models were employed to perform nonlinear analyses. The principal purpose of these nonlinear analyses was to assess whether the analytical model of the buildings could indicate framing damage consistent with that observed at the sites after the earthquakes using the current performance procedures.

Our analytical results, though uncertainties may exist in our structural models, seem to illustrate that nonlinear time history analyses are capable of capturing the occurrence of shear failure in captive columns, but they overestimate the global damage level for all buildings, especially where the building has sustained a pulse type motion but did not display any significant distress. While current methodologies and established guidelines provide reasonable estimates for the performance of the structures, there is still much uncertainty in the dynamic response of the RC structures subjected to a suite of differing strong ground motions.

While performance assessment procedures are being developed and refined at a ferocious pace, their corroboration with field observations is a meagerly populated bag. Earthquakes are seldom events, and there is as yet little empirical evidence for existing RC building performance well in the nonlinear range except under laboratory conditions. Building stock elements are less perfectly known so there is more uncertainty in modeling them. For this reason we contend that the demand and capacity predictions must be understood better under dynamic loading effects, and performance limits and criteria must be refined through further systematic research before they are incorporated into routine engineering practice.

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