Nat Hazards
DOI 10.1007/s11069-013-0835-3

ORIGINAL PAPER

Pushover and nonlinear time history analysis evaluation of a RC building collapsed during the Van (Turkey) earthquake on October 23, 2011

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Received: 22 May 2013 / Accepted: 16 August 2013

Abstract The nonlinear seismic behavior of a collapsed reinforced concrete (RC) residential building in the city of Van in Turkey is investigated by the static pushover and nonlinear time history analyses. The selected RC structure was designed according to the 1975 version of Turkish Earthquake Code (TEC-1975). The building had experienced heavy damage, and it was demolished in the Van earthquake on October 23, 2011. The 2007 version of Turkish Earthquake Code (TEC-2007) is considered for the assessing seismic performance evaluation of the selected RC building. The RC structure presents collapse performance level under the earthquake loads. Besides, the analytical solutions show that different performance levels for the sections are obtained from the pushover and nonlinear time history methods.

Keywords Reinforced concrete structure · Nonlinear static pushover analysis · Nonlinear time history analysis · Performance-based design · Van earthquake

1 Introduction

Many losses in human lives have occurred due to the structures affected by earthquakes in recent years. Major earthquakes have occurred in Turkey and many other countries in recent years. Specifically, serious damages and many losses happened after the 1989 Loma Prieta and 1994 Northridge earthquakes in the United States of America, 1995 Kobe earthquake in Japan, 1992 Erzincan, 1999 Marmara and Duzce and 2011 Van earthquakes

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in Turkey. Therefore, it is needed to investigate performance-based design procedures for the structures recently. There are several procedures for performance assessment in the literature. The most common assessment procedures are explained in four main guidelines/codes: ATC-40, Federal Emergency Management Agency-FEMA 356, Federal Emergency Management Agency-FEMA 440 and TEC-2007. TEC-2007 came into use in 2007. Chapter 7 of TEC-2007 entitled "Assessment and Strengthening of Existing Buildings" sets standards for performance assessment and rehabilitation of existing buildings (Sucuoğlu 2006). The nonlinear seismic performances of structures under earthquake effects are determined by static pushover and time history analyses. Pushover analysis allows for direct evaluation of the performance of the structure at each limit state (Tehranizadeh and Moshref 2011). Time history analysis is the most reliable analysis method among all the nonlinear analysis methodologies. However, static pushover analysis has become important due to its easy application comparing to time history analysis.

Many papers have been published on the topic of performance evaluation of the existing RC buildings (Şengöz 2007; Tuncer et al. 2007; Kalkan and Kunnath 2007; Erdem et al. 2010; Inel et al. 2008; Scawthorn and Johnson 2000; Adalier and Aydingun 2001; Sezen et al. 2003; Yakut et al. 2005; Sadjadi et al. 2007; Duan and Hueste 2012). In the present study, the nonlinear static pushover and time history analyses are used to estimate the expected seismic performance of a residential building, in the Van city of Turkey, that collapsed during the Van earthquake on October 23, 2011. The residential building is typical beam-column RC frame building with no shear walls. The selected building was designed according to TEC-1975 considering both gravity and seismic loads. The 3D pushover and nonlinear time history analysis are performed by using the finite element program SAP 2000. Beam and column elements are modeled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of beams and columns. SAP 2000 provides default or user-defined hinge properties options to model nonlinear behavior of components. In this study, user-defined hinge properties are implemented. Seismic performance evaluation is carried out in accordance with the TEC-2007 that has similarities with FEMA-356 guidelines.

2 Performance levels

As shown in Fig. 1, five points labeled A, B, C, D and E define force–deformation behavior of a plastic hinge. The values assigned to each of these points vary depending on type of element, material properties, longitudinal and transverse steel content, and axial load level on the element (ATC-40; FEMA-356; Inel et al. 2008). Similar to TEC-2007, three limit conditions have been defined for ductile elements on the cross section in ATC and FEMA. These are immediate occupancy (IO), life safety (LS) and collapse prevention (CP). IO defines the beginning of the behavior beyond elasticity; LS defines the limit of the behavior beyond elasticity that the section is capable of safely ensuring the strength; and CP defines the limit of the behavior before collapsing.

The definition of user-defined hinge properties requires moment–curvature analysis of each element. Mander model (Mander et al. 1988) for unconfined and confined concrete and typical steel stress–strain model with strain hardening for steel are implemented in moment–curvature analysis. The points B and C in Fig. 1 are related to yield and ultimate curvatures. The point B is obtained from SAP 2000 using approximate component initial effective stiffness values (cracked section properties) as per TEC-2007; 0.4EI for beams and the below values depending on axial load level for columns:

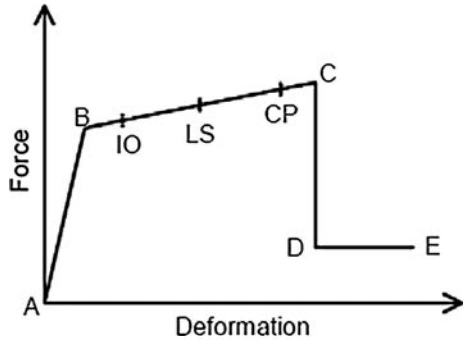


Fig. 1 Force–deformation relationship of a typical plastic hinge (ATC-40, FEMA-356)

$$0.4\mathrm{EI}\ \text{for}\ N/(A_c f_c) \leq 0.1 \tag{1.a}$$

$$0.8\mathrm{EI}\ \text{for}\ N/(A_c f_c) \geq 0.4 \tag{1.b}$$

In Eq. (1), f_c is concrete compressive strength, N is axial load, and A_c is area of section. Linear interpolation is made for the $N/(A_c f_c)$ values between 0.1 and 0.4 (TEC-2007). In this study, moment–curvature analysis is carried out considering section properties and a constant axial load on the structural element.

After the appropriate material properties are determined, structural element sections are modeled via XTRACT (2004) program. In the section, two concrete models, confined and unconfined concretes, are used. The modeling is finished by inputting reinforced steels into defined section geometry. Thus, moment–curvature relations are determined after analysis.

Plastic hinge length is used to obtain ultimate rotation values from the ultimate curvatures. The plastic hinge length definition given in Eq. (2) is used:

$$L_p = 0.08L + 6\frac{f_y}{40}d_b \geq 0.3 f_y d_b \tag{2}$$

In Eq. (2), L_p is the plastic hinge length, L is the distance from plastic hinge location to location of contraflexure, f_y is yield stress of longitudinal bar and d_b is the diameter of longitudinal reinforcement, respectively.

3 Description of investigated reinforced concrete structure

It can be seen from Fig. 2 that the building collapsed completely in October 23, 2011 Van earthquake. One of the important deficiencies in the existing building stock is the insufficient amount of transverse reinforcement as shown in Fig. 3. In addition, poor workmanship and concrete, and insufficient steel detailing were observed during the investigation of the debris of the building.

3.1 Material properties

After the earthquake, the building was collapsed and concrete core samples and reinforcing bar samples were taken from the ruins and tested in laboratory. The existing properties of the concrete and steel obtained from the tests are given in Tables 1 and 2.

Fig. 2 Views of the collapsed building after Van earthquake

Fig. 3 View of transverse reinforcement of the building

Table 1 Concrete core results

Core no	Diameter (mm)	Height (mm)	Compressive strength (N/mm^2)	Corrected strength (N/mm^2)	Mean strength (N/mm^2)
1	95	100	15.19	13.36	14.88
2	95	95	13.52	11.74	
3	95	55	26.82	19.53	

Table 2 The mechanical properties of steel samples

Diameter (mm)	Yield strength (N/mm^2)	Failure strength (N/mm^2)	Failure strain ε (%)
8	396	538	29.3
8	403	558	18.6
8	446	674	24.0
8	428	645	24.4
12	409	521	20.8
12	418	512	21.0
14	331	467	22.2

3.2 Analytical model

The selected building is typical RC frame building with no shear walls. It has a moment resisting frame structural system consisting of beams and columns. A typical floor plan is shown in Fig. 4. Since the majority of buildings in Van, Turkey, were constructed according to TEC-1975, the selected building was designed according to this code, too. Because all the static projects are available, the reinforced concrete (RC) properties of structural members are assumed to be known completely.

The structure is in Van provincial border and in second-degree seismic zone. A design ground acceleration of 0.4 g and soil class Z3 that is similar to class C soil of FEMA-356 is considered in the analyses. Soil properties are obtained from the tests. The projected concrete class is C16 and the projected reinforcing steel class is S220. In this study, concrete class is taken as value of mean strength in Table 1. The RC residential building has 6 stories, stories 1–4 are 3.2 m and stories 5 and 6 are 3.0 m in height (Fig. 4). Framing of the building is irregular in plan where there are 3 axes in X direction and 4 axes in Y direction. Floor plan is same for each story and has an area of 204 m^2. Slab thickness is 10 cm. The dead load is $G = 3.5$ kN/m^2 for all the floors except the top floor where the dead load was considered as $G = 3$ kN/m^2. The live load is $Q = 2$ kN/m^2 for each floor except the top floor where the live load was considered as zero.

The RC residential building was analyzed in detail by performing both pushover and nonlinear time history analyses according to the TEC-2007. Three-dimensional finite element model of the residential building was prepared in structural analysis program (SAP 2000) shown in Fig. 5.

Column dimensions in a story are 25 × 50, 40 × 20, 20 × 50 and 30 × 60 cm (Fig. 6). The column dimensions in a defined position in the plan are the same in the other stories of the building. Longitudinal rebars are 8Ø14 for all columns. The longitudinal reinforcement ratio of these columns varies between 1.1 and 1.5 %. The dimensions of all

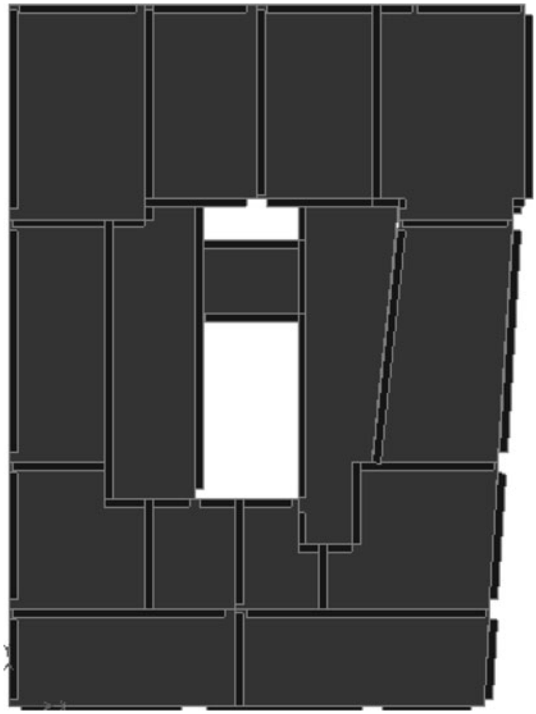


Fig. 4 Typical floor plan of the building

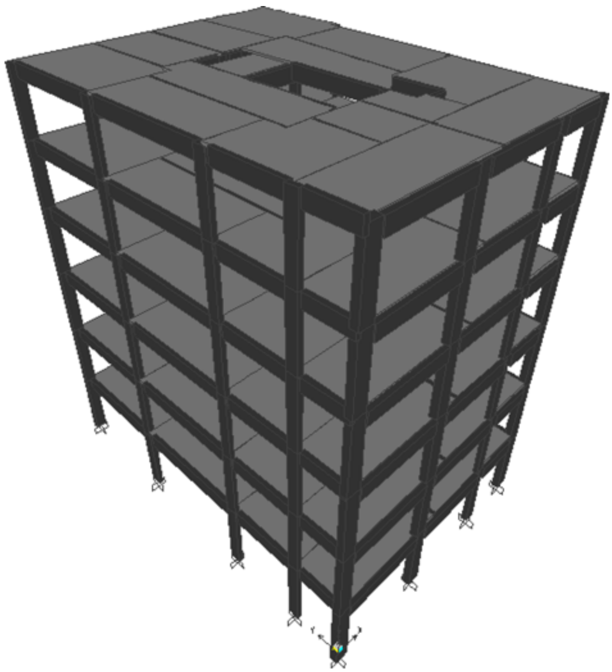


Fig. 5 Three-dimensional finite element model of the residential building

the beams in the building are the same as 20 × 60 cm. Beam longitudinal rebars are 4Ø14 on top and 2Ø14 in bottom for the residential building. Transverse rebars are Ø8/20 cm for columns and beams. Flexural rigidity is calculated for each member. Beams and columns were modeled as frame elements that were connected to each other at the joints. Typical beam and column sections are given in Fig. 7. In Fig. 7, the number before "Ø" is the number of bars and after "Ø" is the diameter of bar in mm.

The vertical loads consist of live and dead loads of slabs, wall loads on beams and dead loads of columns and beams. Predominant mode periods of the building in *X* and *Y* directions are 0.893 and 0.851 s, respectively, based on cracked section properties (Eq. 1).

The collapsed building had poor detailing and no shear walls. The columns of the structure have slender sections through *X* and *Y* directions. Reinforcement of columns do not also provide minimum reinforcement requirement of TEC-1975. In existing RC building, especially with low concrete strength and/or insufficient amount of transverse reinforcement, shear failures of members should be taken into consideration. For this purpose, shear hinges are introduced for beams and columns.

The XTRACT (2004) program is utilized during the preparation of material properties, obtainment of moment–curvature relations of each structural elements and definition of axial load–moment (PM) interaction diagrams for the columns.

Beam and column elements are modeled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of beams and columns. SAP (2000) provides default or the user-defined hinge properties options to model nonlinear behavior of components. The default hinge properties of SAP (2000) are implemented from FEMA-356 (or ATC-40) (FEMA-356 2000; ATC-40 1996). In this study, user-defined hinge properties are implemented.

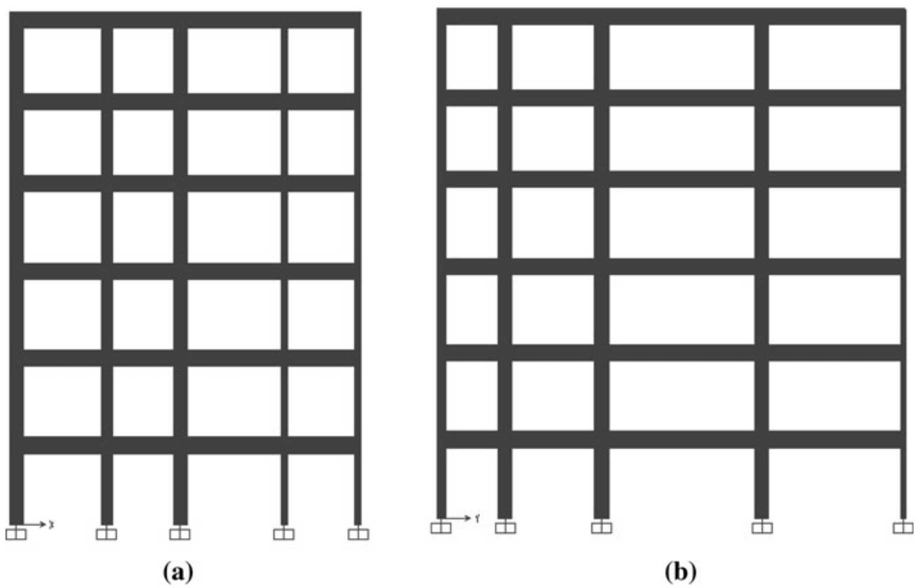
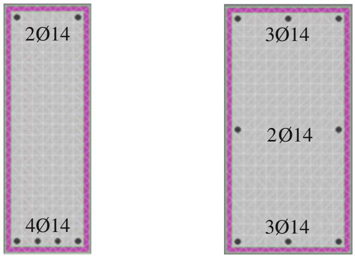


Fig. 6 **a** Typical X–Z sectional view, **b** Typical Y–Z sectional view



(a) 20x60 cm beam **(b)** 30X60 cm column

Fig. 7 Typical (**a**) beam and (**b**) column sections of collapsed building model (in cm)

3.3 Ground motion

In nonlinear response time history analysis, the selection of acceleration record is an important step. The record of Van earthquake is selected as ground motion. This set provides an opportunity to examine reasons of building damages during past Van earthquake in Turkey. The Van earthquake was a destructive magnitude 7.1-Mw earthquake that struck eastern Turkey near the city of Van on Sunday 23 October 2011. The 2011 Van earthquake is one of the largest natural disasters of Turkey after Kocaeli and Duzce earthquakes in 1999. For the Van earthquake, the official death toll was more than 600, with thousands of people injured and thousands left homeless.

The Muradiye station's acceleration record, which is the nearest station to this building, is used in the nonlinear analyses. The horizontal component (N–S) of the acceleration time history used in the analyses is shown in Fig. 8. Spectrums for the acceleration record and TEC-2007 are shown in Fig. 9.

4 Nonlinear seismic performance evaluation of the building

More realistic and economical structural design is provided by taking into consideration nonlinear material-bearing capacities and the effects of great displacements on geometrical convenience condition. The methods used for these aims are twofold: nonlinear static pushover analysis and nonlinear time history analysis. Nonlinear behavior of the structure can be determined nearly real level with the help of nonlinear time history analysis. However, this method is considerably complex, so it loses practicability. Thus, nonlinear static pushover analysis is more practical than the other.

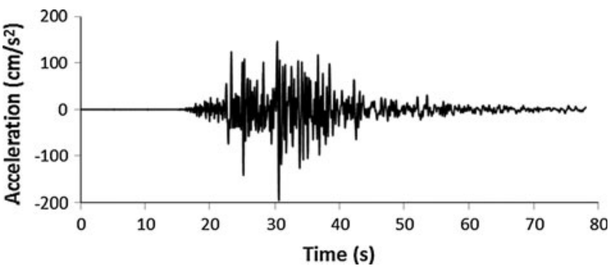


Fig. 8 Acceleration time history of Van earthquake (NS6503), 2011

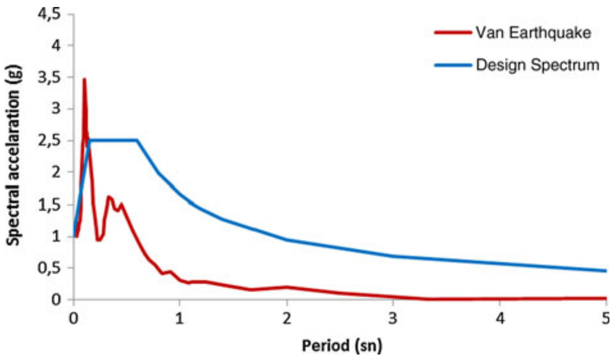


Fig. 9 Spectral acceleration

4.1 Performance evaluation with nonlinear pushover analysis

To be able to use the Incremental Equivalent Seismic Load Method, the number of floors of the building excluding the basement should not be above 8 and the bending irregularity coefficient (η_{bi}) that is calculated in accordance with the elastic linear behavior without considering additional eccentricity should meet the condition of $\eta_{bi} < 1.4$ for each floors. Moreover, in accordance with the earthquake taken into consideration, the ratio of the active mass of the primary (dominant) vibration mode calculated taking the linear elastic behavior as a basis point to the total mass of the building (except for the masses of the basement floors covered by the rigid frames) should be above 0.70 (TEC, 2007). Because the building provides all these conditions, the Incremental Equivalent Seismic Load Method is utilized. Before incremental pushover analysis, a static analysis is done by taking into consideration vertical loads that is harmonic with the masses. This analysis is force controlled, and the results of this study are assumed as initial conditions of incremental pushover analysis. The vertical loads in nonlinear static pushover analysis are assumed as follows:

4.1.1 Vertical load combination (TEC 2007)

$$G + nQ = G + 0.3Q \tag{3}$$

In Eq. (3), G is total dead load, n is the live load participation factor, and Q is total live load stories of building, respectively.

In this calculation, cracked section bending rigidities of columns and beams are determined by analyzing bearing system under the vertical loads that is harmonic with masses according to Eq. 1 and these are utilized in the incremental loading as linear method. The plastic hinge places are assumed and defined on the two ends of the column and beams elements constituting the bearing system. Static pushover curvature is obtained by analyzing bearing system under the vertical loads and proportional incremental interval seismic loads. This pushover curvature is converted to capacity diagram. Design earthquake is converted to spectrum curve and modal displacement demand is determined. The plastic hinges are obtained by pushing again the bearing system up to this demand. The design earthquake, for which the possibility to be exceeded in 50 years is 10 %, is considered in the analysis. Nonlinear static pushover analysis is determined by SAP 2000.

The capacity curves of the building for *X* and *Y* directions are obtained by static pushover analysis, and performance points are determined by TEC-2007 as seen in Fig. 10.

Evaluation of the investigated residential RC building is performed using TEC-2007. Three performance levels, IO, LS and CP are considered as specified in this code and several other international guidelines such as FEMA-356, ATC-40 and FEMA-440. The IO level implies very light damage with minor local yielding and negligible residual drifts. LS defines the limit of the behavior beyond elasticity that the section is capable of safely ensuring the strength, while the CP level is associated with extensive inelastic distortion of structural members with little residual strength and stiffness (Fig. 1).

A design performance level is a statement of the desired structural behavior of a building. After determination of damage regions of sections, the performance level of the building is controlled. The hinges through the *X* and *Y* directions of the structure after pushover analysis can be seen in Fig. 11.

In each floor, the ratio of the beams that are not providing targeted performance level to total beam number in this floor and the ratio of the shear forces of the columns that are not providing targeted performance level to total floor shear force are determined. For any floor, if these ratios exceed the targeted performance level ratio, it is concluded that the building is not sufficient for this performance level. Displacement demand estimated for earthquakes with probability of exceedance of 10 % in 50 years are compared for IO, LS and CP displacement capacities.

It can be seen from the result of the pushover analysis through the *X* direction (Fig. 12a) that there was no damage in 72 columns (57.14 %), minimum damage occurred in 21

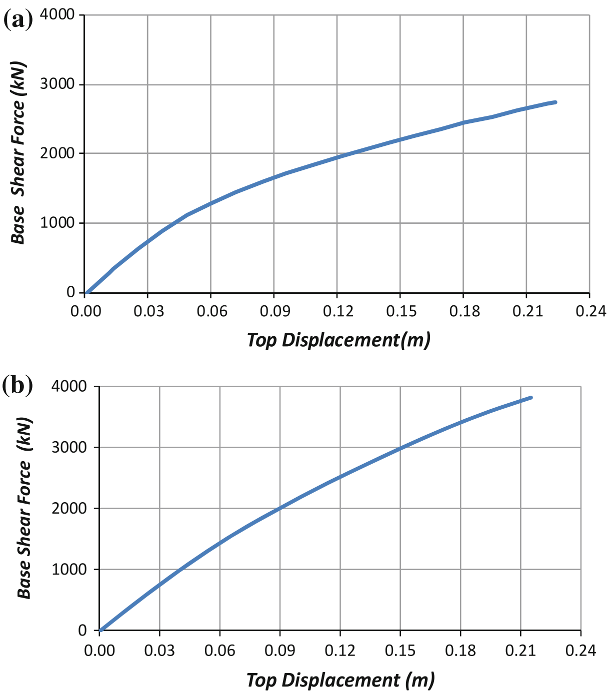


Fig. 10 Capacity curves for (**a**) *X* direction and (**b**) *Y* direction by pushover analysis for 6-story buildings

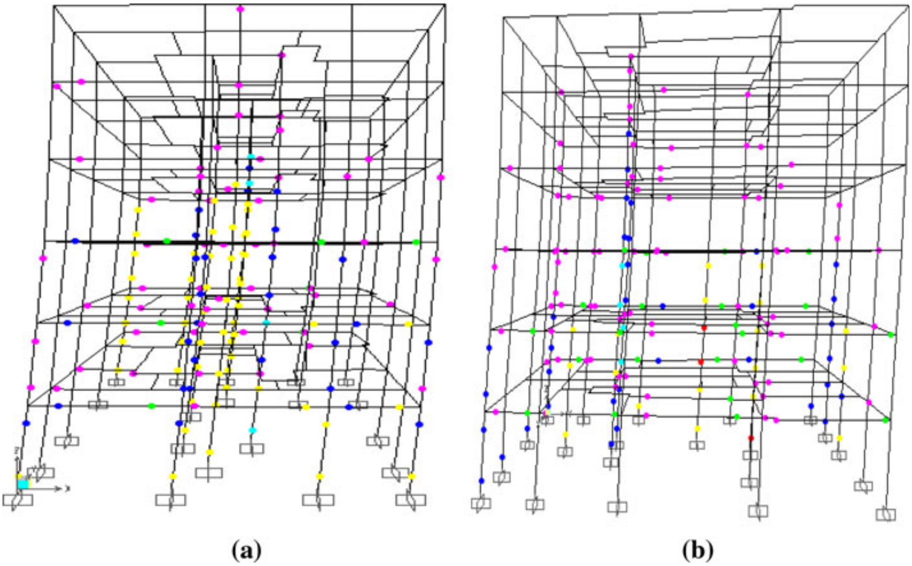


Fig. 11 The plastic hinges occurred through (**a**) *X* direction and (**b**) *Y* direction of the building after pushover analysis

columns (16.67 %), marked damage occurred in 5 columns (3.97 %) and advanced damage occurred in 28 columns (22.22 %) of the total 126 columns. It can also be seen from the result of the pushover analysis through the *Y* direction (Fig. 12b) that there was no damage in 88 columns (69.84 %), minimum damage occurred in 5 columns (3.97 %), marked damage occurred in 16 columns (12.70 %), advanced damage occurred in 13 columns (10.32 %) and 4 columns (3.18 %) of the total 126 columns that collapsed.

It is shown in the result of the pushover analysis through the *X* direction (Fig. 13a) that minimum damage occurred in 83 beams (69.17 %), marked damage occurred in 26 beams (21.67 %) and advanced damage occurred in 11 beams (9.17 %) of the total 120 beams. It is also shown in the result of the pushover analysis through the *Y* direction (Fig. 13b) that there was no damage in 76 beams (63.33 %), minimum damage occurred in 28 beams

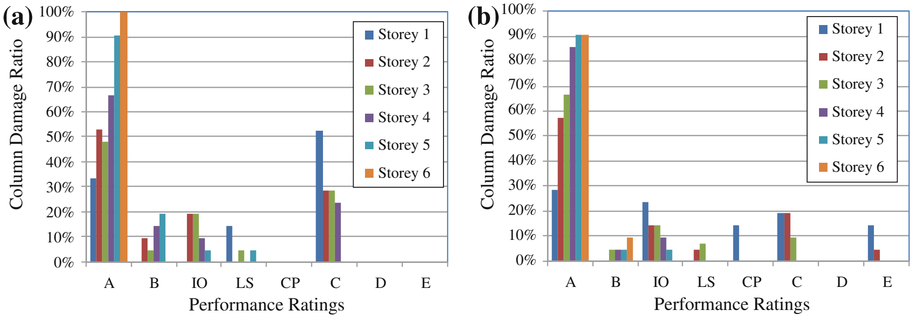


Fig. 12 Columns performance levels of (**a**) *X* direction and (**b**) *Y* direction of residential RC building obtained by pushover analysis

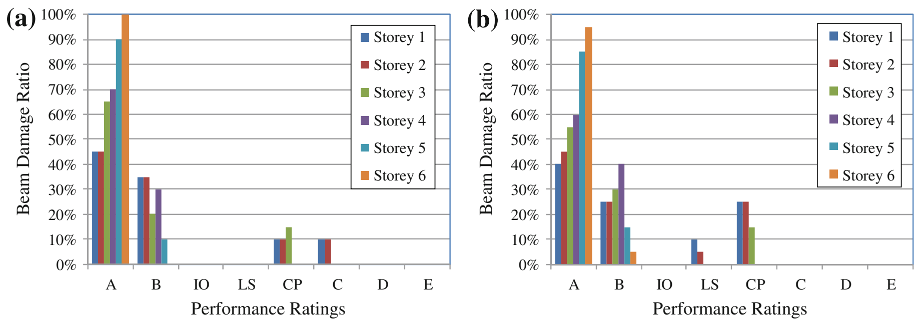


Fig. 13 Beams performance levels of (**a**) *X* direction and (**b**) *Y* direction of residential RC building obtained by pushover analysis

(23.33 %), marked damage occurred in 3 beams (2.5 %) and advanced damage occurred in 13 beams (10.83 %) of the total 120 beams.

When the graphs are investigated, it is concluded from nonlinear static pushover analysis that according to damage conditions of elements, the building does not provide LS rating in the view of LS level targeted in TEC-2007. According to TEC-2007, the residential building is expected to satisfy LS performance levels under design earthquake. The existing residential building is far from satisfying the expected performance levels.

It should also be noted that considerably small displacement drift capacities for IO and LS performance levels in residential building are associated with the existence of weak column–strong beam mechanism and insufficient amount of transverse reinforcement. These columns fail in shear before global yielding. Because of this behavior, the building became incapable of satisfying IO and LS performance levels before yielding.

4.2 Performance evaluation with nonlinear time history analysis

It is assumed that nonlinear time history analysis defines structure behavior ideally because of the seismic loads directly applied to structure (Li 1996). The aim of nonlinear time history analysis is to integrate the equations of the motion of the system step by step by taking into consideration the nonlinear behavior of bearing system. It is calculated for each time increment that displacement, plastic deformation and internal forces occurred in the system, and maximum values of them were observed during earthquake.

In addition to the static pushover analysis, in this study, performance evaluation of the selected building is also determined with nonlinear time history analysis, comparatively. Because the building is in Van provincial border, horizontal component of Van earthquake (Fig. 8) is taking into consideration. The responses of the structure are computed using the Newmark's method.

It is seen from Fig. 14 that plastic hinges occurred through *X* and *Y* directions as a result of nonlinear time history analysis. 4It also shows that these hinges are concentrated on the first two floors and the upper floors are dwindled down. Because at the downstairs collapsing mechanism has occurred, the structure does not act its mission and it is collapsed completely.

The result of the nonlinear time history analysis through the *X* direction seen in Fig. 15a shows that there was no damage in 53 columns (42.06 %), minimum damage occurred in the 21 columns (16.67 %) and marked damage occurred in 10 columns (7.94 %). In

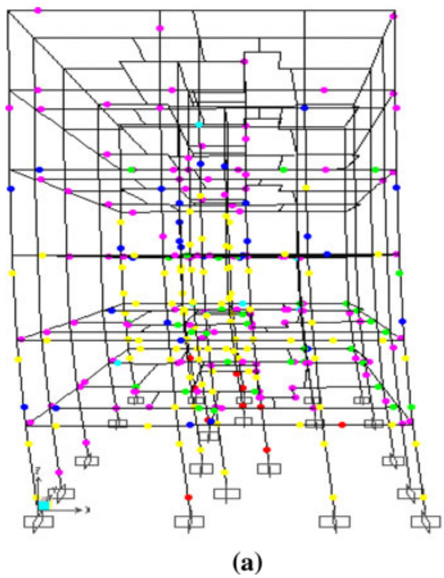


Fig. 14 The plastic hinges occurred through the (**a**) *X* direction and (**b**) *Y* direction of the building after nonlinear time history analysis

addition, advanced damage occurred in 36 columns (28.57 %) and 6 columns (4.76 %) of the total 126 columns that collapsed. It can also be seen from the result of the nonlinear time history analysis through the *Y* direction (Fig. 15b) that there was no damage in 41 columns (30.95 %), minimum damage occurred in 3 columns (7.14 %), marked damage occurred in 9 columns (0.79 %), advanced damage occurred in 50 columns (39.68 %) and 23 columns (18.25 %) of the total 126 columns that collapsed. It is seen from Fig. 15a–b that collapse damages occurred especially in columns of floors 1–3. Thus, the building collapsed completely.

It can be seen from the result of the nonlinear time history analysis through the *X* direction (Fig. 16a) that there was no damage in 30 beams (25 %), minimum damage occurred in 37 beams (30.83 %), marked damage occurred in 7 beams (5.83 %), advanced damage occurred in 45 beams (37.5 %) and 1 beam (0.83 %) of the total 120 beams that

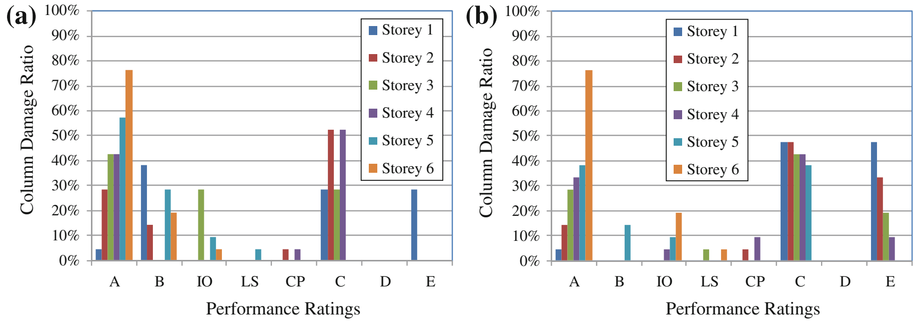


Fig. 15 Columns performance levels of (**a**) *X* direction and (**b**) *Y* direction of residential RC building obtained by nonlinear time history analysis

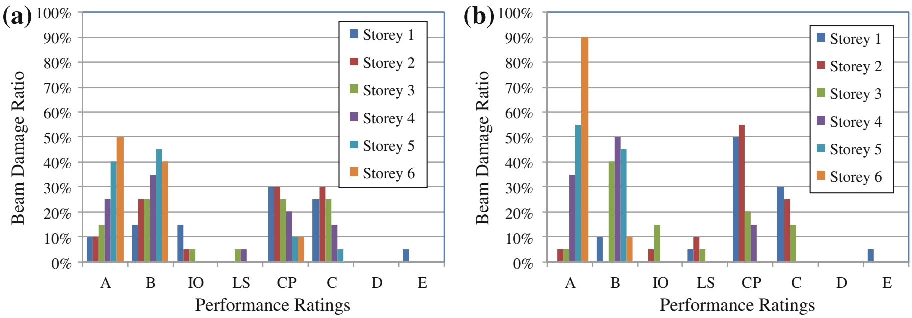


Fig. 16 Beams performance levels of (**a**) *X* direction (**b**) *Y* direction of residential RC building obtained by nonlinear time history analysis

collapsed. It is also shown in the result of the nonlinear time history analysis through the *Y* direction (Fig. 16b) that there was no damage in 37 beams (30.83 %), minimum damage occurred in 27 beams (22.5 %), marked damage occurred in 10 beams (8.33 %), advanced damage occurred in 45 beams (37.5 %) and 1 beam (0.83 %) of the total 120 beams that collapsed.

When the analysis results are investigated, it is concluded from nonlinear time history analysis that according to damage conditions of elements, the building does not provide LS

Fig. 17 Comparison chart of the methods (**a**) *X* direction and (**b**) *Y* direction for the first-story beams

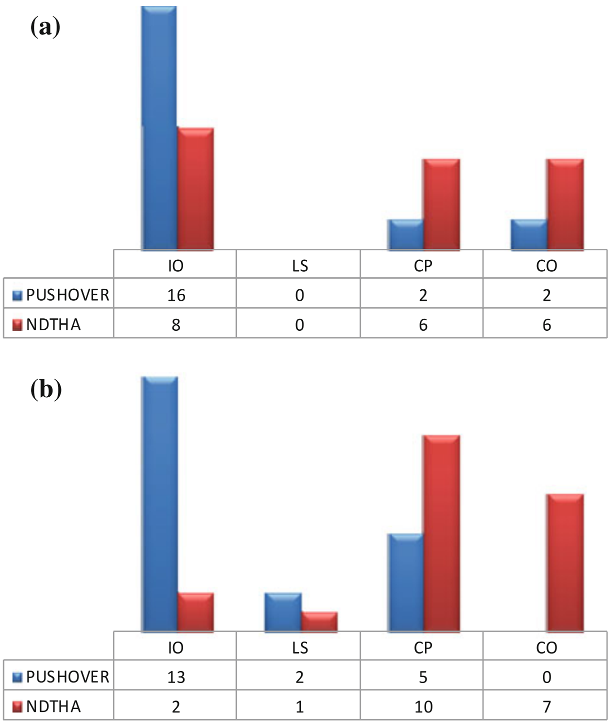
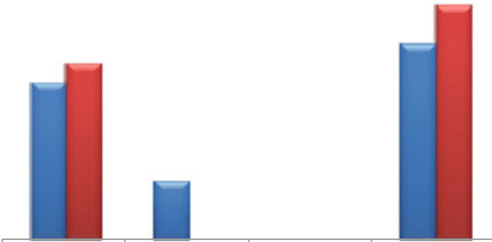


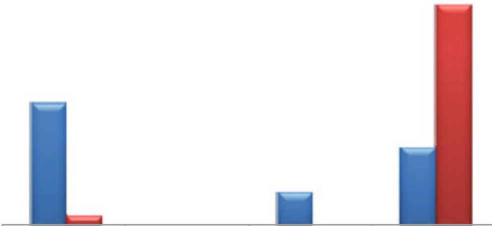
Fig. 18 Comparison chart of the methods (**a**) *X* direction and (**b**) *Y* direction for the first-story columns

(a)



	IO	LS	CP	CO
PUSHOVER	8	3	0	10
NDTHA	9	0	0	12

(b)



	IO	LS	CP	CO
PUSHOVER	11	0	3	7
NDTHA	1	0	0	20

rating in TEC-2007. The existing residential building is far from satisfying the expected performance levels. The performance level of the building is determined as collapse (CO).

4.3 The comparison of two performance analysis methods

The performances of the first-story elements under the earthquake are compared for pushover analysis and nonlinear dynamic time history analysis (NDTHA) in Figs. 17 and 18. The results from pushover analysis show lower damage ratios for the first-story beams and columns than those of the nonlinear time history analysis. The damage ratio indicates relatively high dispersion between the results from NDTHA. The average differences between the damage levels of these two methods are about 50 %.

If it is mentioned that the other results were obtained from this example, for the analysis of this residential structural system presented its numerical properties (Fig. 5), the reduction in the required CPU time using pushover analysis is dramatic, while the results are generally precise with the aforementioned exceptions.

5 Conclusions

This paper investigates the seismic performance of a six-story collapsed RC building designed according to the provisions of TEC-1975. Static pushover and nonlinear time history analyses were used to evaluate the seismic performance of the building collapsed during the Van earthquake on October 23, 2011. Performance evaluation is performed using the current Turkish Earthquake Code, TEC-2007. The performance levels, IO, LS, CP and CO are considered as specified in this code and several other international guidelines such as FEMA-356 and ATC-40. Pushover analysis, time history analysis and criteria of TEC-2007 were used to determine global displacements of the building corresponding to the performance levels considered above. Displacement demand estimates for earthquake with probability of exceedance of 10 % in 50 years are compared for IO, LS and CP displacement capacities.

The existing structural system of the residential building does not satisfy the expected performance levels (LS) according to the TEC-2007. The building designed according to TEC-1975 presents CO performance level through two-direction results according to both nonlinear static pushover analysis and nonlinear time history analysis under the Van earthquake loads. Structural irregularities affect seismic performance of building. It is investigated in situ after the earthquake that insufficient reinforcement and detailing, poor workmanship and low concrete quality can result in this performance level of the structure.

It is concluded from nonlinear dynamic analysis of the structure to the scaled ground motion that according to damage conditions of elements, the building does not provide LS rating in TEC-2007. The building is far from satisfying the expected performance levels.

In addition to these, the results from linear analysis and pushover analysis show lower damage ratios for the first-story beams and columns than those of the nonlinear dynamic analysis.

The reduction in the required CPU time using pushover analysis is dramatic, while the results are generally precise with the aforementioned exceptions.

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