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Retracted: Determination of the Earthquake Performance of Reinforced Concrete Structures by Using Linear and Non-Linear Methods

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Abstract

In this paper, the seismic behavior of Reinforced Concrete (RC) health facilities building is investigated by the linear, nonlinear static (pushover) analysis and nonlinear dynamic analysis. The selected reinforced concrete structure was designed according to "Turkish Seismic Code-2007" (TEC-2007). A typical five-story reinforced concrete building is designed. TEC-2007 is utilized for evaluating the seismic performance of the selected reinforced concrete building. Natural earthquake acceleration record selected and adjusted for compatibility with the adopted design spectrum, is used. The aim of this study is to evaluate three different performance analysis methods suggested in TEC-2007. These are the linear (Equivalent Earthquake Load Method) and the nonlinear (Incremental equivalent earthquake Load Method) and Nonlinear Dynamic Analysis procedures of the Code. The linear and nonlinear static evaluation methods are used. Furthermore, the nonlinear dynamic analysis is used for the same building and their results are compared each other. The results from linear and pushover analysis show lower damage ratios for the first story beams and columns than those of the nonlinear dynamic analysis. Besides, the analytical solutions show that different performance levels for the sections are obtained from the pushover and nonlinear dynamic methods.

Keywords: Reinforced concrete structure, Seismic performance evaluation, Linear and non-linear seismic evaluation methods.

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Introduction

Many losses in human lives have occurred due to the structures which have been affected by earthquakes in recent years, including Turkey and many countries in the past 30 years. Especially serious damages and many losses occurred after 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 in Turkey. Therefore, performance based design procedures have been investigated 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 which are ATC-40, FEMA 356, FEMA440 and TEC-2007. TEC-2007 came into use in 2007. The Code also has a new chapter (Chapter 7) where the linear and the non-linear evaluation methods are given for seismic safety evaluation of existing buildings. Chapter 7 is introduced in 2007 and only a limited number of applications are carried out by using its requirements (Celep, 2007). 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). Nonlinear dynamic analysis is the most reliable analysis method among the all nonlinear analysis methodologies. However, static pushover analysis is become important due to its easy application comparing to nonlinear dynamic analysis.

Many papers have been published on the topic of performance evaluation of existing RC buildings (Şengöz, 2007; Tuncer, et al., 2007; Kalkan and Kunnath, 2007; Erdem et al., 2009; 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). Ye et al. (2008) noted the absence of the preferred strong-column, weak-beam damage mechanism in typical RC frames that were damaged in the Wenchuan earthquake. Most building structures in China are normally low- to medium-rise RC frames.

The aim of the present study is that linear, the nonlinear static pushover and nonlinear dynamic analyses are used to estimate the expected seismic performance of a health facilities building. The building is typical beam-column RC frame buildings with no shear walls. The selected building was designed according to TEC-2007 considering both gravity and seismic loads. The pushover and nonlinear dynamic 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. SAP2000 provides default or the 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 recently published TEC-2007 that has similarities with FEMA-356 guidelines.

Performance Levels

As shown in Fig. 1a, 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-273; Inel et al., 2008).

Similar to ATC and FEMA, three limit conditions have been defined for ductile elements on the cross section in TEC-2007. These are Minimum Damage Limit (MN), Safety Limit (GV) and Collapsing Limit (GÇ). Minimum damage limit defines the beginning of the behavior

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beyond elasticity, safety limit defines the limit of the behavior beyond elasticity that the section is capable of safely ensuring the strength, and collapsing limit defines the limit of the behavior before collapsing. This classification does not apply to elements damaged in a brittle condition. Elements that the damages with critical sections do not reach MN are within the Minimum Damage Region, those in-between MN and GV are within Marked Damage Region, those in-between GV and GÇ are in Advanced Damage Region, and those going beyond GÇ are within Collapsing Region (Fig.1b).

The definition of user-defined hinge properties requires moment–curvature analysis of each element. Mander model (Mander et all., 1988) for unconfined and confined concrete and typical steel stress–strain model with strain hardening for steel are implemented in moment–curvature analyses. The points B and C in Fig. 1-a are related to yield and ultimate curvatures. The point B is obtained from SAP2000 using approximate component initial effective stiffness values as per TEC- 2007; 0.4EI for beams and the below values depending on axial load level for columns:

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

$$0.8E\,I \text{ for } N/(A_c\, f_c) \geq 0.4 \quad (1.b)$$

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

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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 analyses. 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:

$$L_p=0{,}08L+6f_yd_b/40\geq 0{,}3f_yd_b \tag{2}$$

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 longitudinal bar and d_b is the diameter of longitudinal reinforcement, respectively.

Description of Investigated Reinforced Concrete Structure

Analytical Model

The selected building is typical RC frame buildings 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.2. Because all the structural drawings and specifications are available, the reinforced-concrete properties of structural members are assumed to be known completely.

A design ground acceleration of 0.4g and soil class Z2 that is similar to class C soil of FEMA-356 is considered in the analyses. The projected concrete class is C25/30 (according to EN 206-1 standard) and projected reinforcing steel class is S420 (according to EN 10080 standard). The reinforced concrete (RC) health facility building has 5 stories, first story is 4.0

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m high and other stories are 3.5 m high (Fig.3). There are 4 bays in X-direction and 3 bays in Y-direction in plan. Floor plan is same for each story and has an area of 333.50 m^2. Slab thicknesses are 18 cm. The dead load is G =6.5 kN/m^2 for all the floors except the top floor where the dead load was considered as G = 8.5 kN/m^2. The live load is Q= 3.5 kN/m^2 for each floor except the top floor where the live load was considered as Q= 2 kN/m^2. The structure is thought to be a health foundation and its live load contribution factor is taken as n = 0.6.

A concrete health facility building was analyzed in detail by performing linear static, pushover and nonlinear dynamic analyses according to the TEC-2007. Three dimensional finite element model of the health building was prepared in SAP2000 structural analysis program shown in Fig. 3.

The 5-story building's first, second and third stories consist of 600 ×600 mm and fourth and fifth stories consists 500 ×500 mm columns. Longitudinal rebars are 20Ø24 for all columns. The dimensions of all beams are 400x500 cm. Beam longitudinal rebars are 4Ø20 on top and 2Ø18 in bottom for the building. Transverse rebars are Ø10/20 cm for columns and beams. Flexural rigidity is calculated for each member. Beams and columns were modeled as frame elements which were connected to each other at the joints.

The vertical loads consist of live and dead loads of slabs, wall loads on beams and dead loads of columns and beams. When determining seismic performance of the selected structure, Seismic Load Reduction Factor is taken as R_a=1. In addition, building importance factor is applied as I = 1. The rigidities of cracked sections are taken instead of the rigidities of uncracked sections. The information level coefficient is taken as 1 for extended information

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level. Predominant mode periods of the building in X and Y directions are 1.251 and 1.190 s, respectively, based on cracked section properties.

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. SAP2000 provides default or the user defined hinge properties options to model nonlinear behavior of components. The default hinge properties of SAP2000 are implemented from FEMA-356 (or ATC-40) (FEMA-356, 2000; ATC-40, 1996). In this study, user-defined hinge properties are implemented.

Linear and Nonlinear Seismic Performance Evaluation of the Building

Performance Evaluation with Linear Elastic Analysis

Equivalent seismic load method shall be implied to buildings not exceeding 25 m and 8 storeys. The torsional irregularity coefficient (η_{bi}) that is calculated in accordance with the elastic linear behavior without considering additional eccentricity should meet the condition $\eta_{bi} < 1.4$ for each floor.

It is seen from Table 1 and Table 2 that the torsional irregularity of the building is provided. Seismic Load Reduction Factor (R_a=1) is taken and right side of the equation is multiplied with λ factor. λ Factor is taken as 1.0 in one and two storeys buildings except cellar and 0.85 in others. Information level for structural parameters is supposed to be “extensive” level.

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Onto the results of the calculations regarding all earthquakes applied in any floors, at most 10% of the beams exceed the Significant Damage Zone and all other load-bearing components remain in the Minimum Damage Zone. Such buildings can be agreed to be in the immediate occupancy (IO) Performance Level provided that the brittle damaged components, if any, are strengthened (TEC-2007).

This ratio is exceeded in the X and Y directions of the analyzed structure. Therefore, performance level of immediate occupancy (IO) is not provided under the design earthquake whose probability of exceedance in 50 years is 10% (Fig.4).

For the greatest earthquake, to provide performance level of life safety (life safety is defined in TEC-2007 as marked damage), mostly 30% of the beams and some part of the column can pass to advanced damage level for each analyzed direction. In the structure analyzed, this ratio is not exceeded for X and Y directions. So, performance level of life safety (LS) is provided under the greatest earthquake whose probability of exceedance in 50 years is 2% (Fig.5).

Performance Evaluation with Nonlinear Pushover Analysis

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

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Pushover analysis is an approximate analysis method in which the structure is subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a target displacement is reached. Pushover analysis consists of a series of sequential elastic analysis, superimposed to approximate a force-displacement curve of the overall structure. A predefined lateral load pattern which is distributed along the building height is then applied. The lateral forces are increased until some members yield. The structural model is modified to account for the reduced stiffness of yielded members and lateral forces are again increased until additional members yield. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is plotted with base shear to get the global capacity curve.

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 torsional irregularity coefficient (η_{bi}) that is calculated in accordance with the elastic linear behavior without considering additional eccentricity should meet the condition $\eta_{bi} < 1.4$ 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 analyses, 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 analyses. The vertical loads in nonlinear static pushover analyses are assumed as follows:

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Vertical Load Combination (TEC 2007)

$$G + nQ = G + 0.6Q \tag{3}$$

In Eq. (3), G is total dead load, n is the live load participation factor, 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. It is seen from Fig. 6 that static pushover curvature is obtained by analyzing bearing system under the vertical loads and proportional incremental interval seismic loads. Design earthquake is converted to spectrum curve and modal displacement demand is determined and performance points are determined by TEC-2007 as seen in Fig.7. The plastic hinges are obtained by pushing again the bearing system up to this demand.

The pushover analysis of the selected structure is actualized under design earthquake (10% in 50 year hazard level) and for the collapse prevention earthquake (2% in 50 year hazard level) as proposed in the TEC-2007. Nonlinear static pushover analyses are determined by SAP2000.

Evaluation of the investigated RC building is performed using TEC-2007. Three performance levels, immediate occupancy (IO), life safety (LS), and collapse prevention (CP) are considered as specified in this Code and several other international guidelines such as FEMA-

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356, ATC-40, and FEMA-440. The IO level implies very light damage with minor local yielding and negligible residual drifts, 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 under design earthquake (10% in 50 year hazard level) and for the collapse prevention earthquake (2% in 50 year hazard level) is seen from Figs. 8-9.

It can be seen from the result under design earthquake of the pushover analysis through the X direction (Fig. 10-a) that damage is not occurred in the 16 beams (20%), minimum damage is occurred in the 52 beams (65%), marked damage is occurred in the 12 beams (15%) of total 80. It can be also seen from the result under design earthquake of the pushover analysis through the X direction (Fig. 10-b) that damage is not occurred in the 74 columns (74%), minimum damage is occurred in the 6 columns (6%), marked damage is occurred in the 20 columns (20%), of total 100 columns.

It can be seen from the result under design earthquake of the pushover analysis through the Y direction (Fig. 10-c) that damage is not occurred in the 21 beams (28%), minimum damage is occurred in the 43 beams (57.33%), marked damage is occurred in the 11 beams (14.67%) of total 75 beams. It can be also seen from the result under design earthquake of the pushover analysis through the Y direction (Fig. 10-d) that damage is not occurred in the 63 columns (63%), minimum damage is occurred in the 17 columns (17%), marked damage is occurred in the 20 columns (20%), of total 100 columns.

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

It can be seen from the result under collapse earthquake of the pushover analysis through the X direction (Fig. 11-a) that damage is not occurred in the 16 beams (20%), minimum damage is occurred in the 22 beams (27.5%), marked damage is occurred in the 35 beams (43.75%) and advanced damage is occurred in the 7 beams (8.75%) of total 80. It can be also seen from the result under collapse earthquake of the pushover analysis through the X direction (Fig. 11-b) that damage is not occurred in the 50 columns (50%), minimum damage is occurred in the 24 columns (24%), marked damage is occurred in the 10 columns (10%), advanced damage is occurred in the 4 columns (4%) and 12 columns (12%) of total 100 columns are collapsed.

It can be seen from the result under collapse earthquake of the pushover analysis through the Y direction (Fig. 11-c) that damage is not occurred in the 17 beams (22.67%), minimum damage is occurred in the 32 beams (42.67%), marked damage is occurred in the 20 beams (26.67%) of total 75 beams. It can be also seen from the result under collapse earthquake of the pushover analysis through the Y direction (Fig. 11-d) that damage is not occurred in the 48 columns (48%), minimum damage is occurred in the 18 columns (18%), marked damage is occurred in the 14 columns (20%), and 15 columns (15%) of total 100 columns are collapsed.

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When the graphs are investigated, it is concluded from nonlinear static pushover analysis under collapsed earthquake that according to damage conditions of elements, the building is not provided life safety (LS) rating in the view of life safety level targeted in TEC-2007. According to TEC-2007, the health facility building is not expected to satisfy LS performance levels under collapsed earthquake. The existing building is far from satisfying the expected performance levels.

Performance Evaluation with Nonlinear Dynamic Analysis

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

In addition to the static pushover analyses, in this study, performance evaluation of the selected building also is determined with nonlinear dynamic analysis, comparatively. The Mode Superposition Method considering the Wilson-Θ algorithm is used for solving the dynamic equilibrium equations. 1999 Kocaeli earthquake is the largest natural disasters of the 20th century in Turkey after 1939 Erzincan earthquake. For the Kocaeli earthquake, the official death toll was more than 15 000, with approximately 44 000 people injured and thousands left homeless. For that reason, YPT330 component of Yarimca station records of 1999 Kocaeli Earthquake (Fig. 12) is utilized as ground motion (Peer, 2012). This ground

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motion continued up 35.0 s is applied to the system in a horizontal direction. The dynamic responses of the health facility structure are obtained for a time interval of 0.005 s. The selected ground motion record was scaled to maximum PGA level (0.4g) to produce design earthquake based on the requirements of the Turkish code for seismic design of buildings (TEC-2007).

It is seen from Fig.13 that plastic hinges occurred through X and Y directions as a result of nonlinear dynamic analysis. It can be seen from Fig. 13 that these hinges are concentrated on the first two floors and in the upper floors are dwindle down.

It can be seen from the result of the nonlinear dynamic analysis through the X direction (Fig. 14-a) that damage is not occurred in the 37 columns (37%), minimum damage is occurred in the 35 columns (35%), marked damage is occurred in the 4 columns (4%) and 24 columns (24%) of total 100 columns are collapsed. It can be also seen from the result of the nonlinear dynamic analysis through the Y direction (Fig. 14-b) that damage is not occurred in the 37 columns (37%), minimum damage is occurred in the 35 columns (35%), marked damage is occurred in the 4 column (4%) and 24 columns (24%) of total 100 columns are collapsed. It is seen from Figs.14a-b that collapse damages are occurred especially columns of floors 1-2.

It can be seen from the result of the nonlinear dynamic analysis through the X direction (Fig. 15-a) that damage is not occurred in the 24 beams (30%), minimum damage is occurred in the 45 beams (56.3%), marked damage is occurred in the 19 beams (23.75%), advanced damage is occurred in the 4 beams (5%) of total 80 beams. It is also shown in the result of the nonlinear dynamic analysis through the Y direction (Fig. 15-b) that damage is not occurred in the 75 beams (100%) of total 75 columns.

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When the analysis results are investigated, it is concluded from nonlinear dynamic analysis that according to damage conditions of elements, the building does not provide life safety (LS) rating in TEC-2007. The existing building is far from satisfying the expected performance levels. The performance level of the building is determined as collapse (CO).

The Comparison of Performance Analysis Methods Used

The performances of the first story elements under the design earthquake are compared for linear analysis (LA), pushover analysis (PUSHOVER) and nonlinear dynamic analysis (NDA) in Figs. 16-17. The section damage regions determined with the linear and nonlinear static analysis methods defined in TEC-2007 show similarities with each other. Generally, the result variations of the sections differ as big as one damage region. However, the results obtained from nonlinear dynamic method differ from linear static and pushover analyses. The result variations of the sections obtained from nonlinear dynamic analysis results differ two damage regions than other two methods.

The results from linear and pushover analysis show lower damage ratios for the first story beams and columns than those of the nonlinear time history analysis. Targeted performance level for health facilities building determined in TEC-2007 is LS level. In X-direction, for only nonlinear dynamic analysis 37.5% of the beams are on reach this level. In Y-direction, only for linear analysis, 20% of the beams reach to LS level. For the columns, both in X and Y directions, all of the columns (100%) are reach to LS level and above for nonlinear dynamic analysis (Figs. 16-17). Thus, it is seen from these results that the building performance level is provided in linear and pushover analysis, however, it is collapsed according to nonlinear dynamic analysis.

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Conclusions

This paper investigates the seismic performance of a five-story a reinforced concrete health facility building designed according to the provisions of the TEC-2007. Linear analysis, static pushover and nonlinear dynamic analyses were used to evaluate the seismic performance of the building and these three methods are comparison with each other.

In the linear analysis; the existing structural system of the building does not satisfy the expected performance levels (IO) according to the TEC-2007 under the design earthquake. Performance level of life safety (LS) is provided under the greatest earthquake whose probability of exceedance along 50 years is 2%.

The pushover analysis is a simple way to explore the nonlinear behavior of the buildings. The results obtained in terms of pushover demand, capacity spectrum and plastic hinges gave an insight into the real behavior of structure. Pushover analysis is not only useful for evaluating the seismic performance of the structure, but could also be helpful for selecting seismic details that are more suitable for withstanding the expected inelastic deformations. For the pushover analysis the building is sufficient for IO under design earthquake. According to TEC-2007, the health facility building is not expected to satisfy LS performance levels under collapsed earthquake. The existing building is far from satisfying the expected performance levels.

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 life safety (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.

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References

Adalier, K., and Aydingun, O., (2001). Structural engineering aspects of the June 27, 1998 Adana–Ceyhan (Turkey) earthquake, Engineering Structures; 23(3):343–55.

Applied Technology Council, ATC-40. Seismic evaluation and retrofit of concrete buildings vols. 1–2. California; 1996.

Duan, H., and Hueste, M.B., (2012), Seismic performance of a reinforced concrete frame building in China, Engineering Structures, 41, 77-89.

EN 10080, Steel for the reinforcement of concrete - Weldable reinforcing steel – General, European Committee for Standardization, 2005

EN 206-1, Concrete – Part 1: Specification, performance, production and conformity, European Committee for Standardization, 2000.

Erdem, R.T., Demir, A., Bağcı, M., Kantar, E.,(2010). A Comparative Assessment of Existing Buildings by Turkish Earthquake Code, ATC-40, FEMA-356, FEMA-440, 9th International Congress on Advances in Civil Engineering, Karadeniz Technical University, Trabzon, Turkey.

Federal Emergency Management Agency, FEMA-356. Prestandard and commentary for seismic rehabilitation of buildings. Washington (DC); 2000.

Federal Emergency Management Agency, FEMA-440. Improvement of nonlinear static seismic analysis procedures. Washington (DC); 2005.

Inel, M., Ozmen, H.B. and Bilgin, H., (2008).Re-evaluation of building damage during recent earthquakes in Turkey, Engineering Structures, 30, 412–427.

Kalkan., E and Kunnath, SK., (2007), Assessment of current nonlinear static procedures for seismic evaluation of buildings, Engineering Structures, 29, 305–316.

Mander, J.B., Pristley, M.J.N., Park, R. (1988), Theoretical stress-strain model for confined concrete, Journal of Structural Division (ASCE), 114(8), 1804-1826.

PEER (Pacific Earthquake Engineering Research Centre), http://peer.berkeley.edu/smcat/data, 2012.

Sadjadi, R, Kianoush, M.R., Talebi, S. (2007), Seismic performance of reinforced concrete moment resisting frames, Engineering Structures, 29(9), 2365–80.

SAP 2000, Structural Analysis Program, Computers and Structures Inc., Berkeley, California.

Scawthorn, C., and Johnson, G.S., (2000). Preliminary report: Kocaeli (Izmit) earthquake of 17 August 1999, Engineering Structures; 22(7):727–45.

Sezen, H., Whittaker, A.S., Elwood, K.J., and Mosalam, K.M., (2003). Performance of reinforced concrete buildings during the August 17, 1999 Kocaeli, Turkey earthquake, and seismic design and construction practice in Turkey, Engineering Structures; 25(1):103–14.

Sucuoğlu, H. (2006). The Turkish Seismic Rehabilitation Code, First European Conference on Earthquake, Engineering and Seismology Geneva, Switzerland.

Şengöz A., (2007). Quantitative Evaluation of Assessment Methods in the 2007 Turkish Earthquake Code, Master Thesis, Department of Civil Engineering, METU, Ankara.

Tehranizadeh, M., and Moshref, A., (2011). Performance-based optimization of steel moment resisting frames Scientia Iranica A, 18 (2), 198–204.

Turkish Earthquake Code (TEC-1975) (1975). Specifications for buildings to be built in seismic areas. Ministry of Public Works and Settlement. Ankara, Turkey. [in Turkish]

Turkish Earthquake Code (TEC-1998) (1998). Specifications for buildings to be built in seismic areas. Ministry of Public Works and Settlement. Ankara, Turkey. [in Turkish]

Journal of Performance of Constructed Facilities. Submitted November 28, 2012; accepted March 28, 2013; posted ahead of print April 3, 2013. doi:10.1061/(ASCE)CF.1943-5509.0000462

Turkish Earthquake Code (TEC-2007) (2007). Specifications for buildings to be built in seismic areas. Ministry of Public Works and Settlement. Ankara, Turkey. [in Turkish]

Tuncer, O., Celep, Z., Yılmaz, M.B., (2007). A Comparative Evaluation of the Methods Given in the Turkish Seismic Code, WCCE–ECCE–TCCE Joint Conference: Earthquake & Tsunami.

Yakut, A., Gulkan, P., Bakır, B.S., and Yılmaz, M.T., (2005). Re-examination of damage distribution in Adapazari: Structural considerations. Engineering Structures; 27(7):990–1001.

Ye, L., Qu, Z., Ma, Q., Lin, X., Lu, X., Pan, P., (2008). Study on ensuring the strong column weak beam mechanism for RC frames based on the damage analysis in the Wenchuan earthquake. Build Structures; 38(11):52–9.

XTRACT, 2004. Cross Sectional Analysis of Structural Components, Imbsen and Associates Inc., Sacremanto, California.

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TABLES

Table 1. The control of torsional irregularity coefficient for X direction of health facility building.

Story	***Story height (hi) (cm)***	***Displacement (Δimax) (cm)***	***Min. Displacement (Δimin) (cm)***	***Δi-ort***	***nbi<1,4***	***Restriction of TEC-2007***
5	350	24,05	24,03	24,04	1,0004	1,4
4	350	20,66	20,64	20,65	1,0005	1,4
3	350	15,21	15,19	15,20	1,0007	1,4
2	350	10,24	10,21	10,23	1,0015	1,4
1	400	4,79	4,77	4,78	1,0021	1,4

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Table 2. The control of torsional irregularity coefficient for Y direction of health facility building.

Story	***Story height (hi) (cm)***	***Displacement (Δimax) (cm)***	***Min. Displacement (Δimin) (cm)***	***Δi-ort***	***nbi<1,4***	***Restriction of TEC-2007***
5	350	22,78	22,42	22,6	1,0080	1,4
4	350	19,61	19,25	19,43	1,0093	1,4
3	350	14,43	14,14	14,285	1,0102	1,4
2	350	9,79	9,56	9,675	1,0119	1,4
1	400	4,68	4,54	4,61	1,0152	1,4

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FIGURE CAPTIONS

Fig.1. Force-Deformation relationship of a typical plastic hinge (a) ATC-40, FEMA-273, (b) TEC-2007.

Fig. 2. Typical floor plan of the building.

Fig. 3. Three dimensional finite element model of the health facility building

Fig.4. Beams and columns performance levels of (a-b) X direction (c-d) Y direction for design earthquake obtained by linear analysis.

Fig.5. Beams and columns performance levels of (a-b) X direction (c-d) Y direction for collapse prevenient earthquake obtained by linear analysis

Fig.6. Capacity curves for X direction (a) and Y direction (b) by pushover analysis for 5-story buildings

Fig.7. Spectral acceleration, spectral replacement and Modal Capacity curves for X direction (a) and Y direction (b) by pushover analysis for 5-story buildings.

Fig.8. The plastic hinges occurred through the X and Y directions of the building for design earthquake after pushover analysis

Fig.9. The plastic hinges occurred through the X and Y directions of the building for collapse prevention earthquake after pushover analysis.

Fig.10. Beams and columns performance levels of (a-b) X direction (c-d) Y direction for design earthquake obtained by pushover analysis.

Fig.11. Beams and columns performance levels of (a-b) X direction (c-d) Y direction for collapse prevention earthquake by pushover analysis.

Fig. 12. Acceleration time history of Kocaeli earthquake (YPT330), 1999 (PEER, 2012).

Fig.13. The plastic hinges occurred through the X and Y directions of the building after nonlinear dynamic analysis.

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Fig.14. Columns performance levels of (a) X direction (b) Y direction of RC building obtained by nonlinear dynamic analysis.

Fig.15. Beams performance levels of (a) x dimension (b) y dimension of health RC building obtained by nonlinear dynamic analysis.

Fig.16. Comparison chart of the methods (a) X direction and (b) Y direction for the first story beams obtained by the design earthquake.

Fig.17. Comparison chart of the methods (a) X direction and (b) Y direction for the first story columns obtained by the design earthquake.

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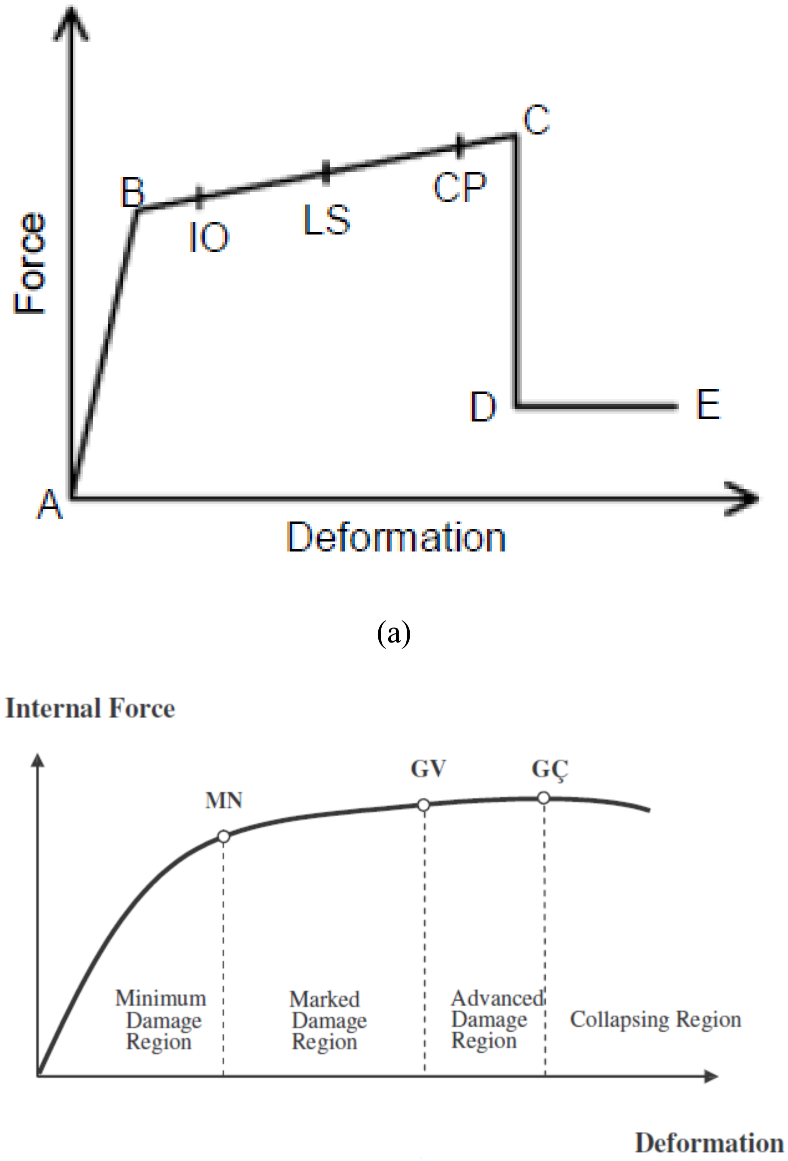

(b)

Fig.1. Force-Deformation relationship of a typical plastic hinge

(a) ATC-40, FEMA-273, (b) TEC-2007.

Fig.2.pdf

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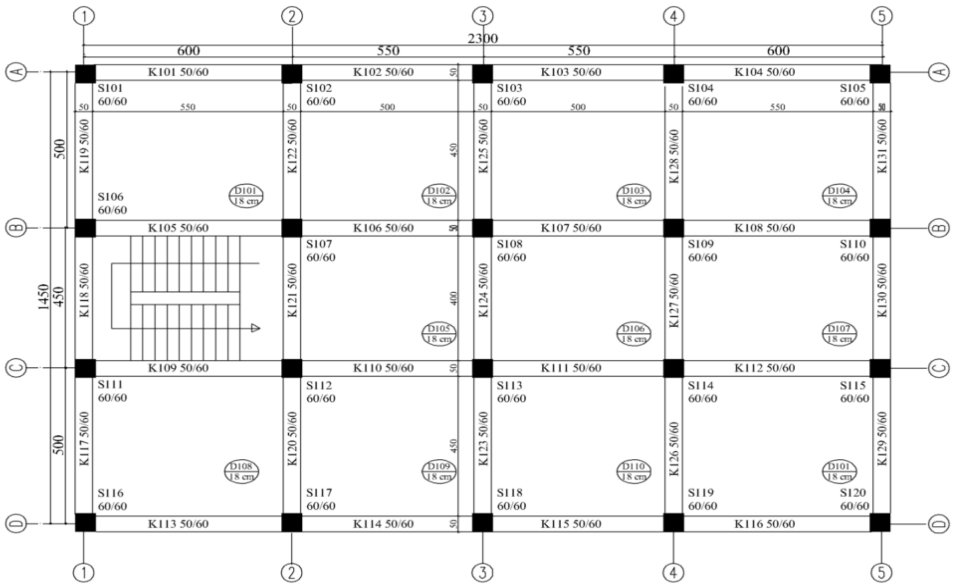

Fig. 2. Typical floor plan of the building.

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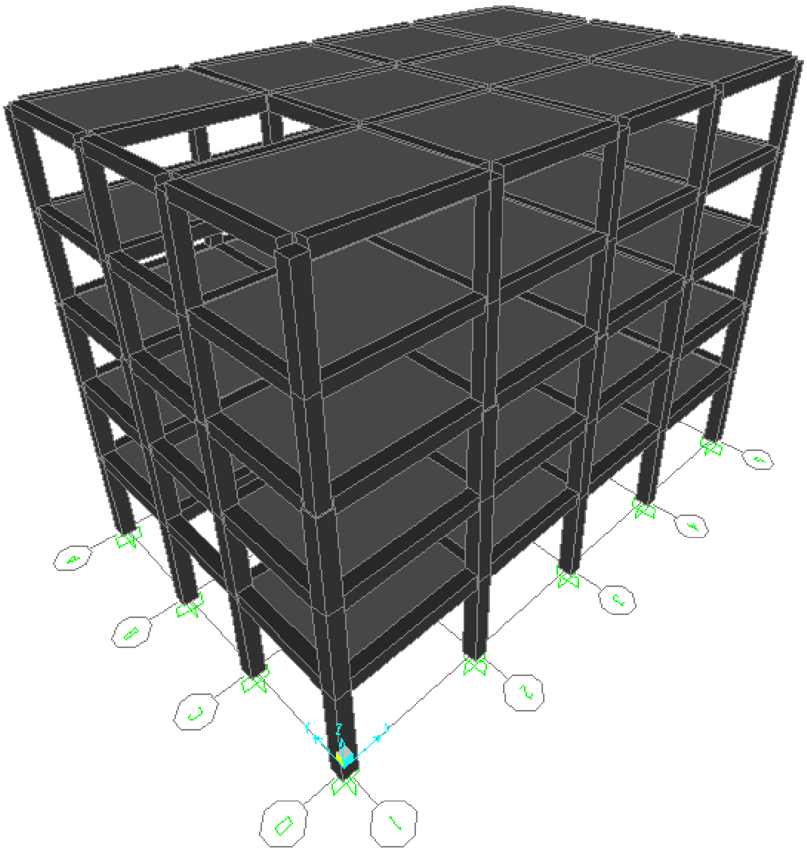

Fig. 3. Three dimensional finite element model of the health facility building

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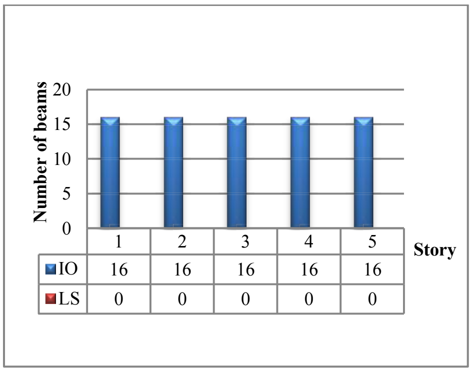

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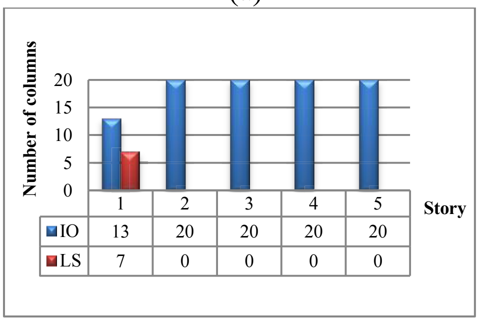

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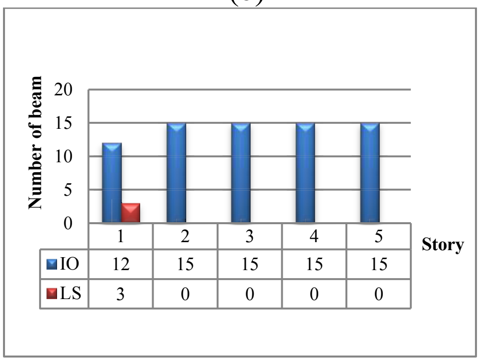

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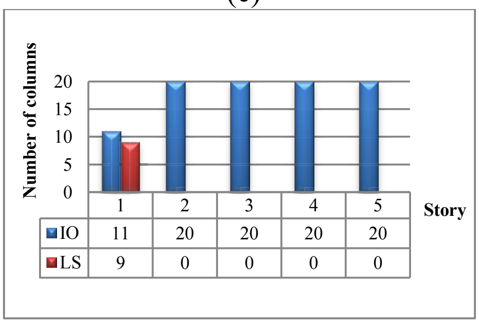

(d)

Fig.4. Beams and columns performance levels of (a-b) X direction (c-d) Y direction for design earthquake obtained by linear analysis.

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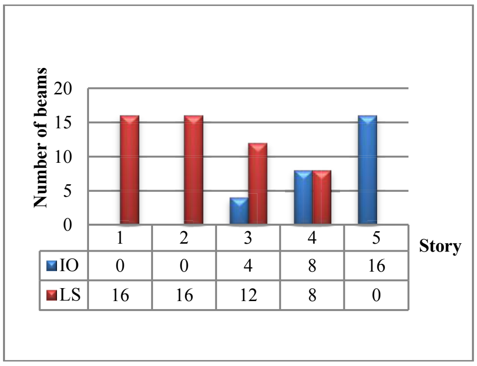

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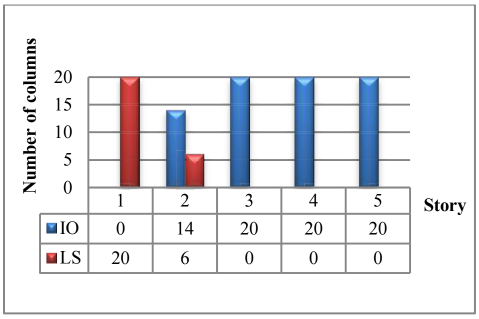

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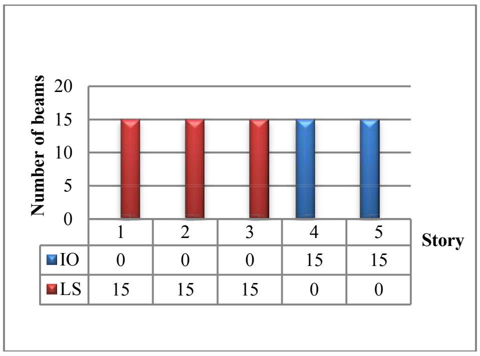

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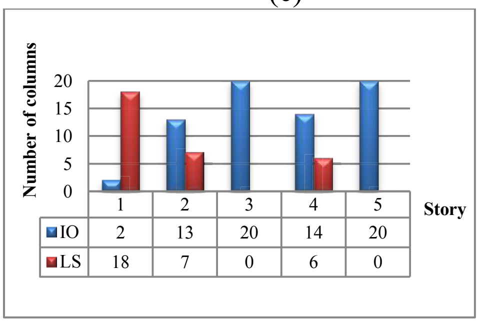

(d)

Fig.5. Beams and columns performance levels of (a-b) X direction (c-d) Y direction for collapse prevenient earthquake obtained by linear analysis

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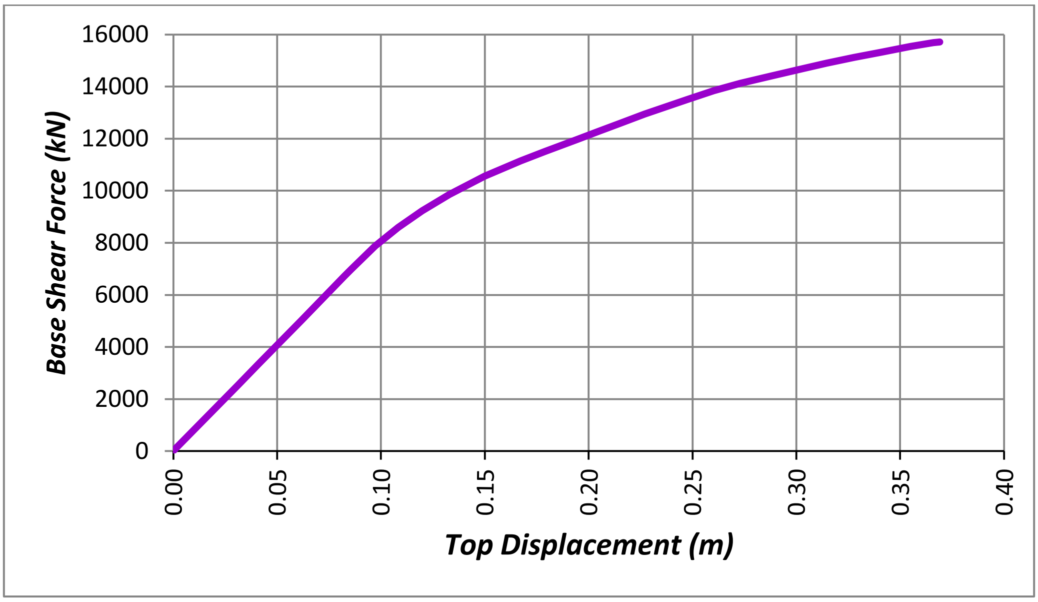

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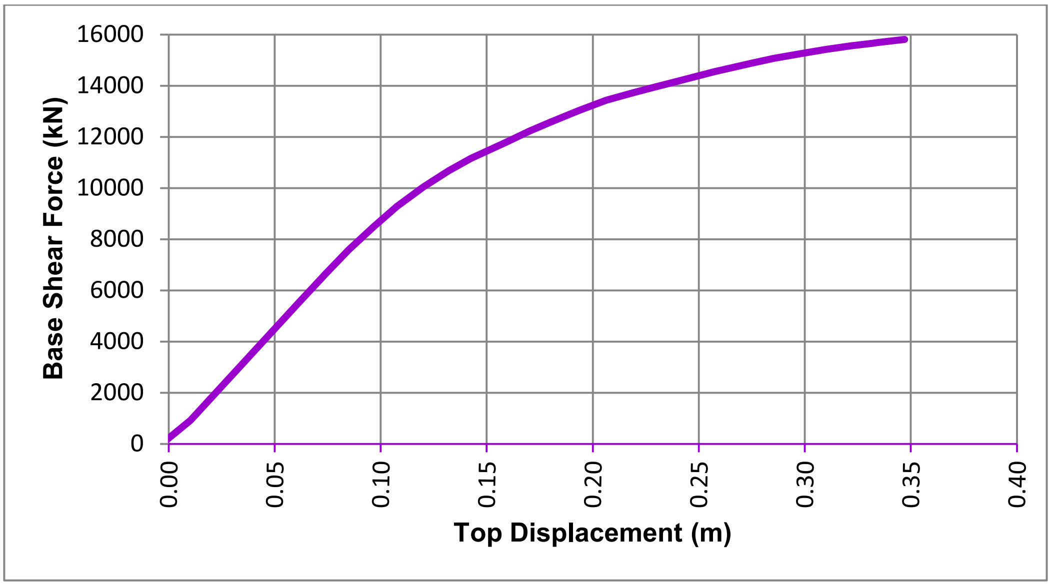

(b)

Fig.6. Capacity curves for X direction (a) and Y direction (b) by pushover analysis for 5-story buildings.

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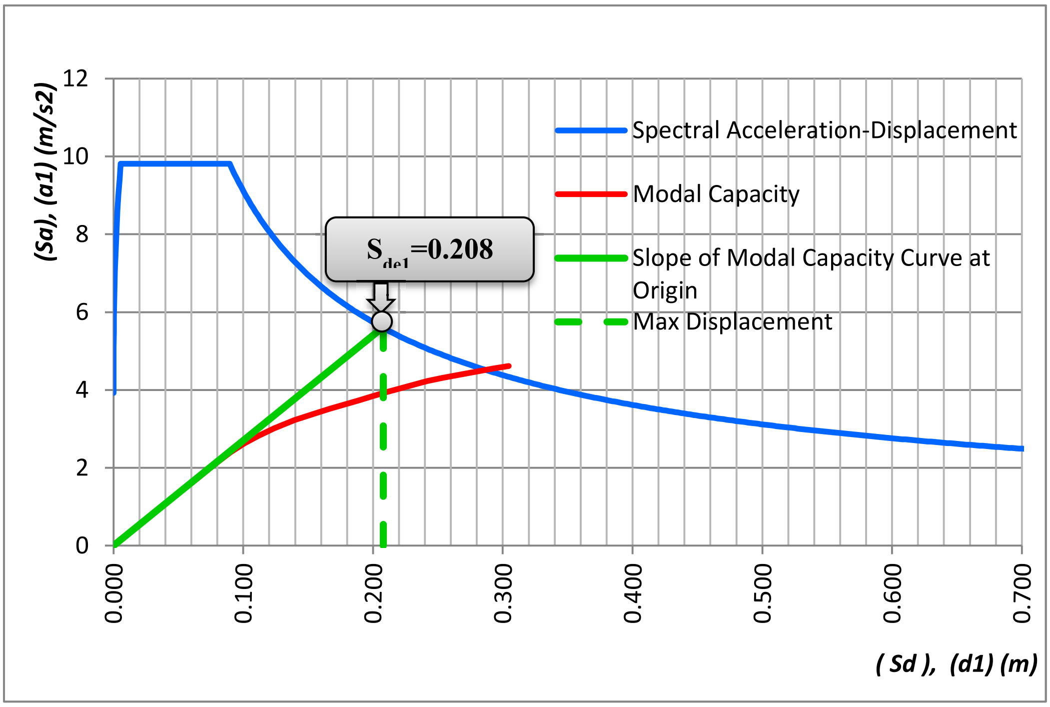

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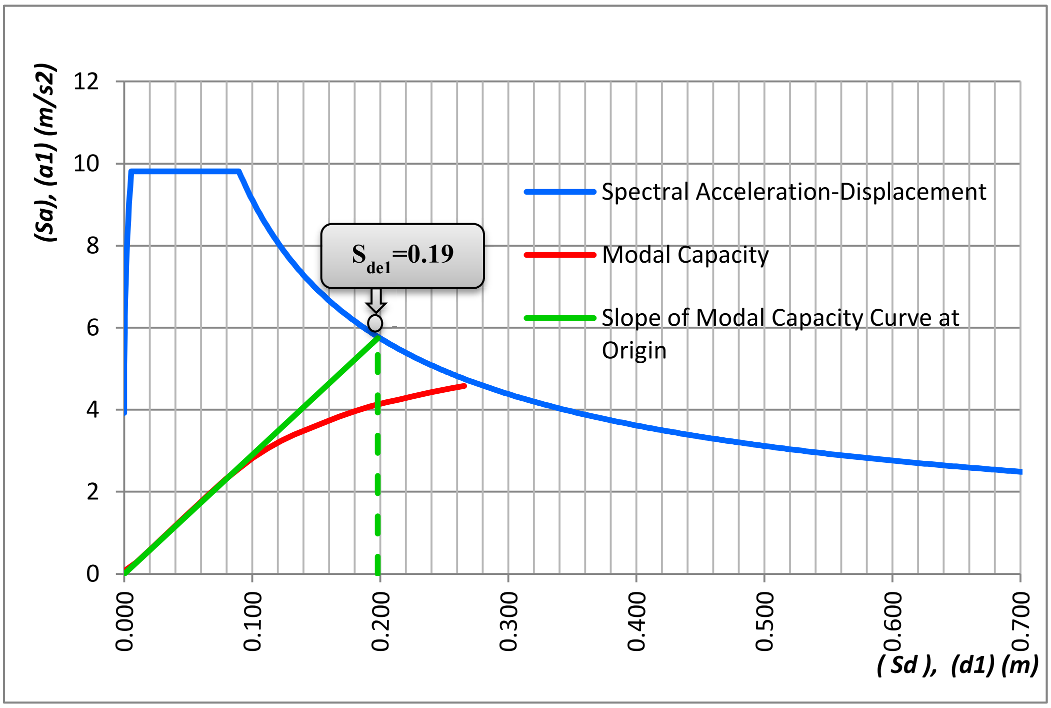

(b)

Fig.7. Spectral acceleration, spectral replacement and Modal Capacity curves for X direction (a) and Y direction (b) by pushover analysis for 5-story buildings.

Fig.8.pdf

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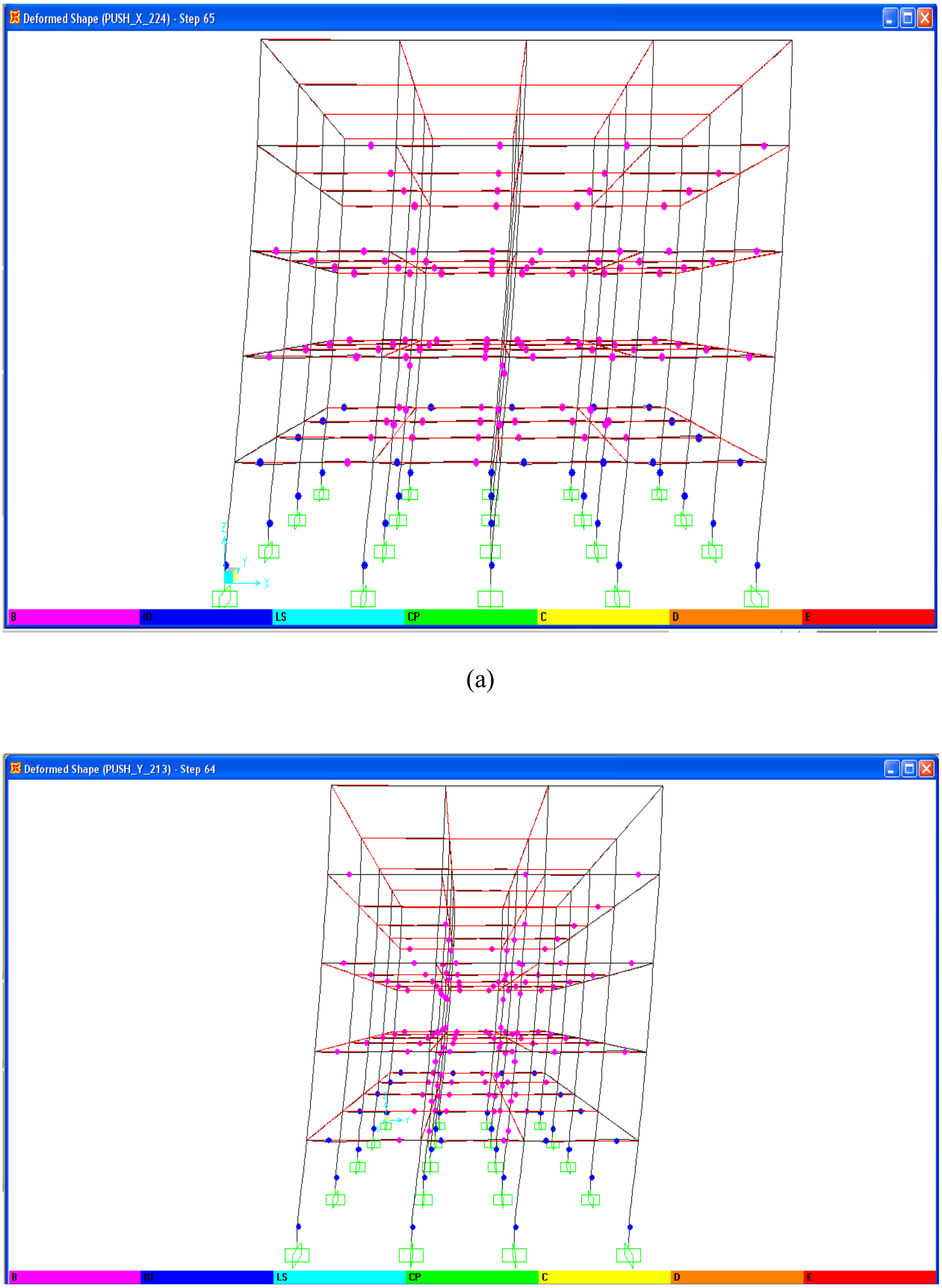

(a)

(b)

Fig.8. The plastic hinges occurred through the X and Y directions of the building for design earthquake after pushover analysis.

Fig.9.pdf

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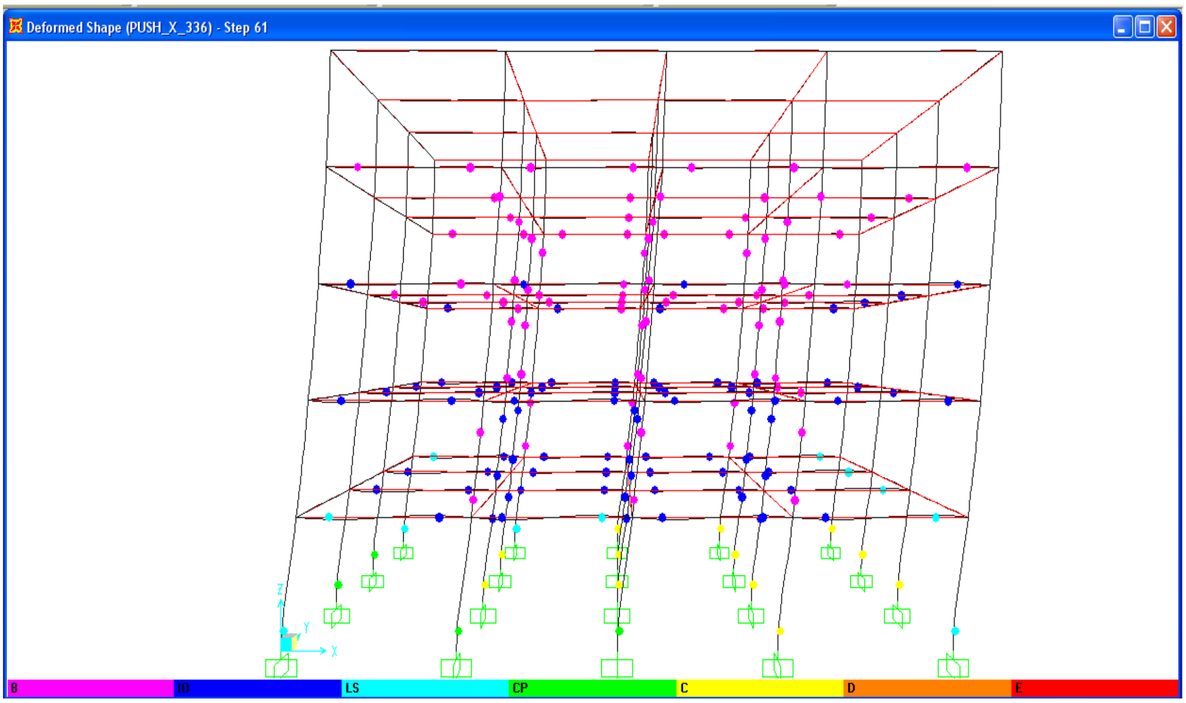

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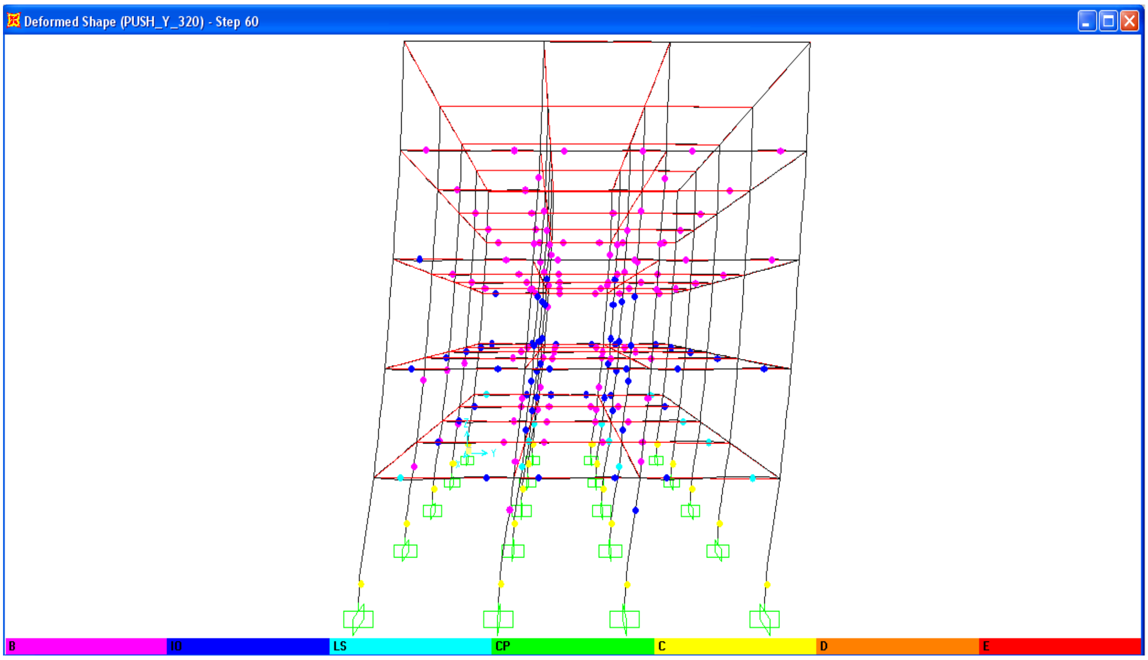

(b)

Fig.9. The plastic hinges occurred through the X and Y directions of the building for collapse prevention earthquake after pushover analysis.

Fig.10.pdf

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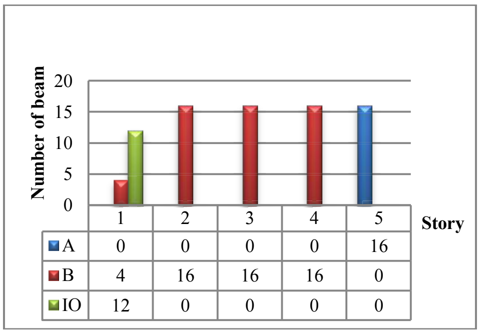

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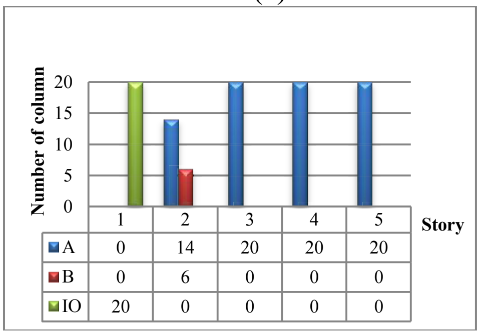

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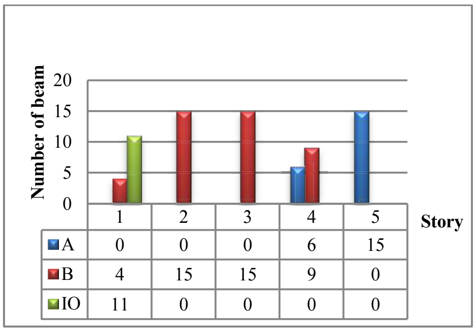

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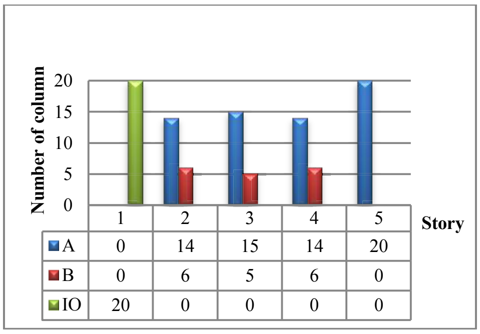

(d)

Fig.10. Beams and columns performance levels of (a-b) X direction (c-d) Y direction for design earthquake obtained by pushover analysis.

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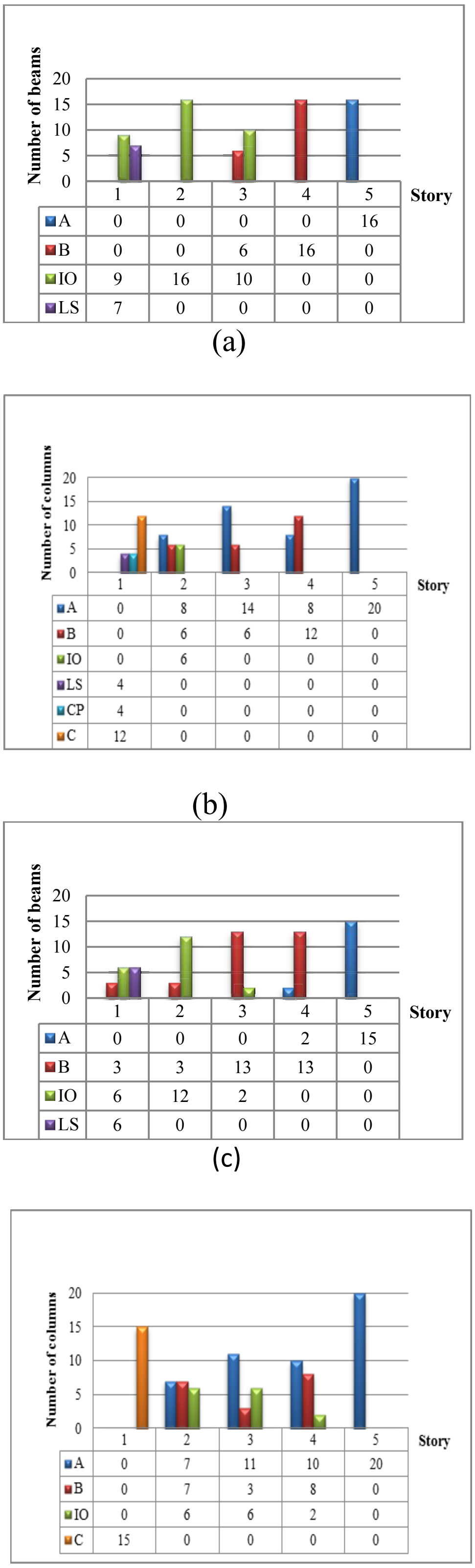

(a)

Story	1	2	3	4	5
A	0	0	0	0	16
B	0	0	6	16	0
IO	9	16	10	0	0
LS	7	0	0	0	0

(b)

Story	1	2	3	4	5
A	0	8	14	8	20
B	0	6	6	12	0
IO	0	6	0	0	0
LS	4	0	0	0	0
CP	4	0	0	0	0
C	12	0	0	0	0

(c)

Story	1	2	3	4	5
A	0	0	0	2	15
B	3	3	13	13	0
IO	6	12	2	0	0
LS	6	0	0	0	0

(d)

Story	1	2	3	4	5
A	0	7	11	10	20
B	0	7	3	8	0
IO	0	6	6	2	0
C	15	0	0	0	0

Fig.11. Beams and columns performance levels of (a-b) X direction (c-d) Y direction for collapse prevention earthquake by pushover analysis.

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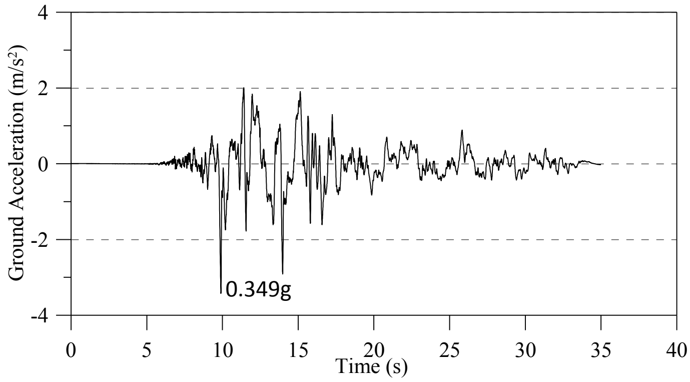

Fig. 12. Acceleration time history of Kocaeli earthquake (YPT330), 1999 (PEER, 2012).

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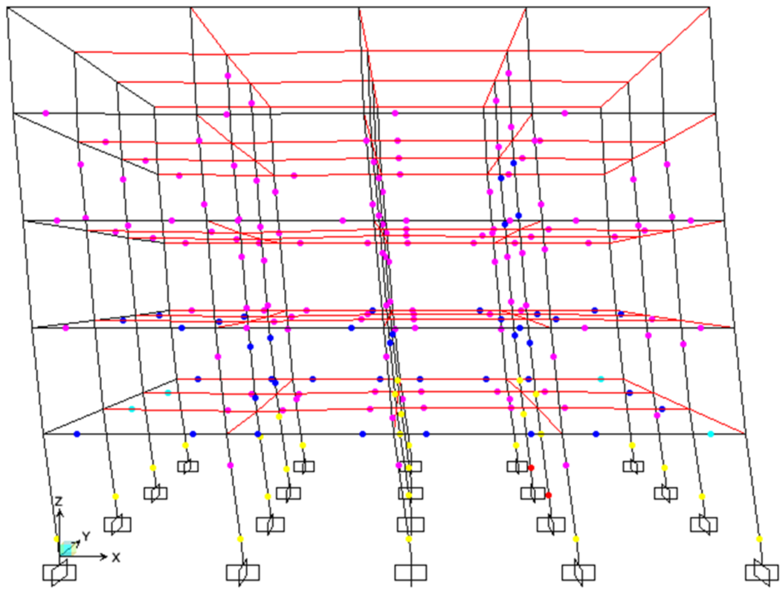

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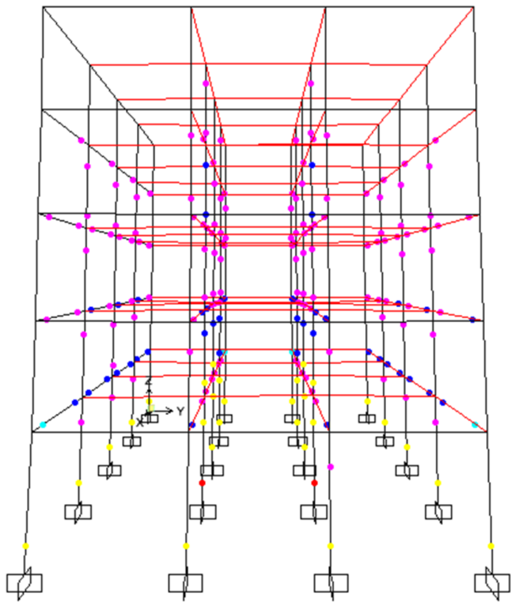

(b)

Fig.13.The plastic hinges occurred through the X and Y directions of the building after nonlinear dynamic analysis.

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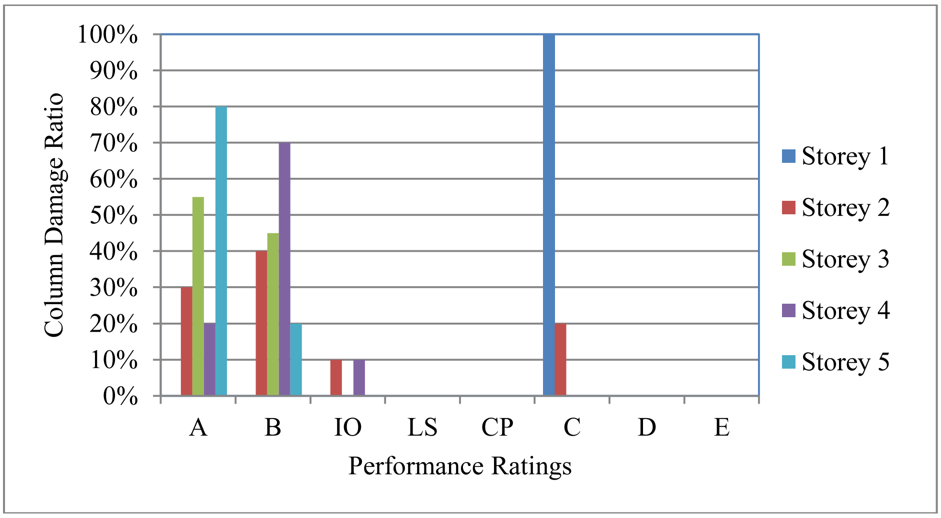

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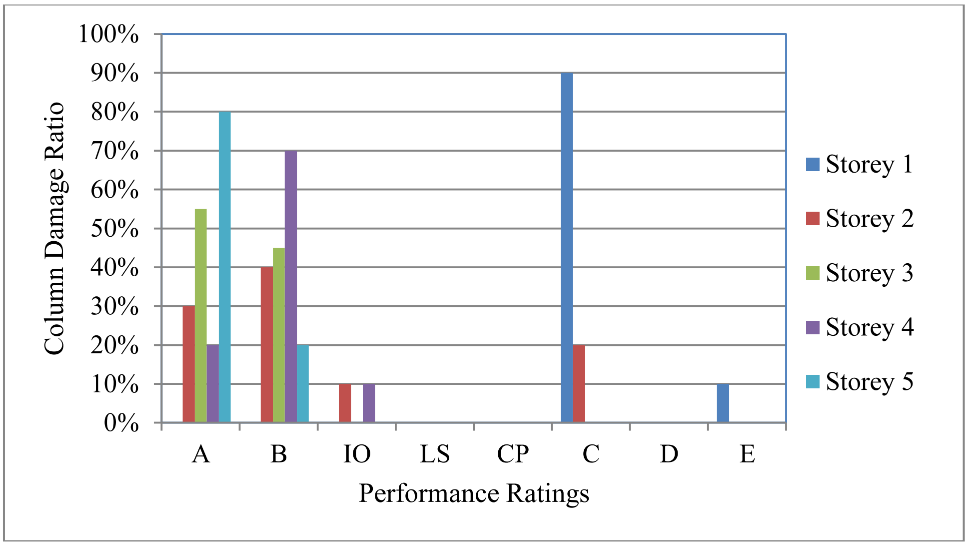

(b)

Fig.14. Columns performance levels of (a) X direction (b) Y direction of RC building obtained by nonlinear dynamic analysis.

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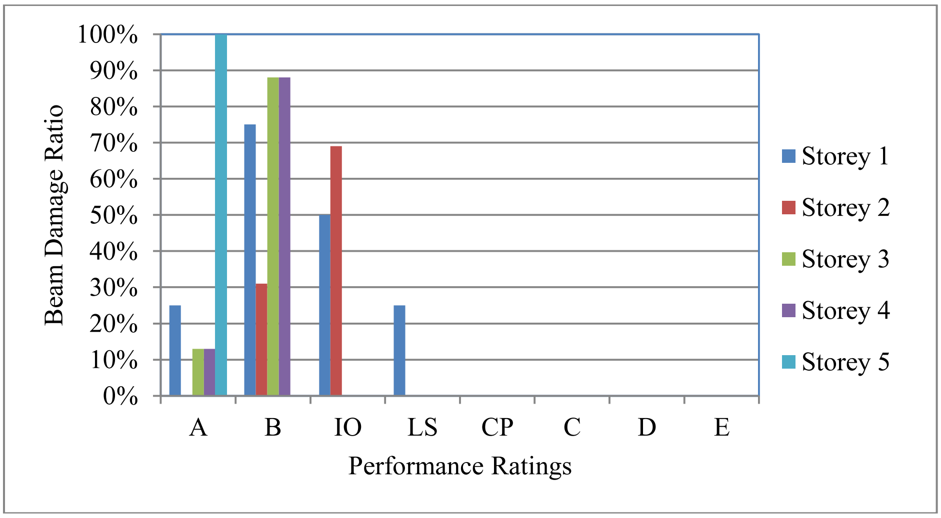

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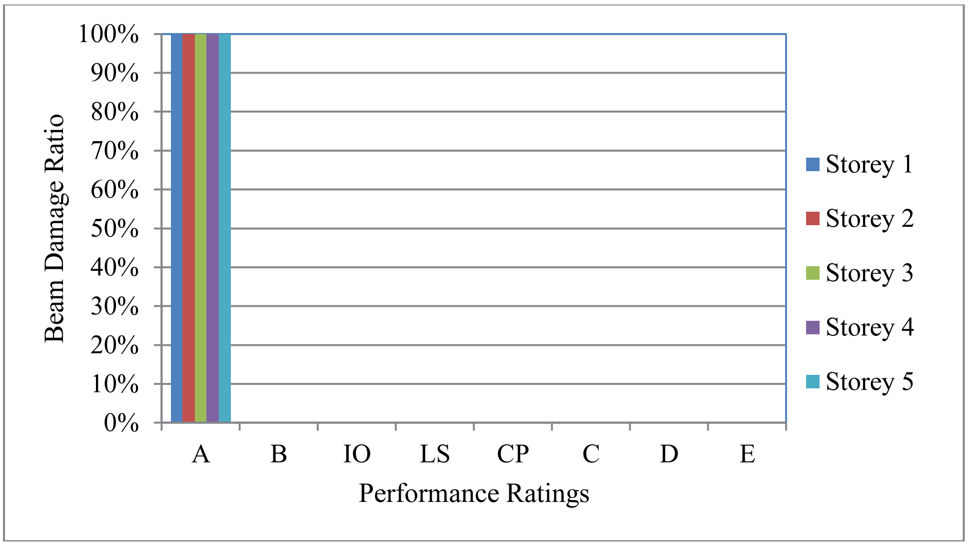

(b)

Fig.15. Beams performance levels of (a) x dimension (b) y dimension of health RC building obtained by nonlinear dynamic analysis.

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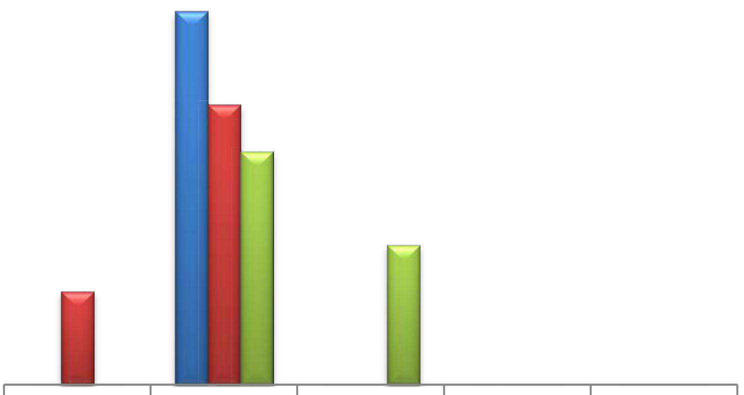
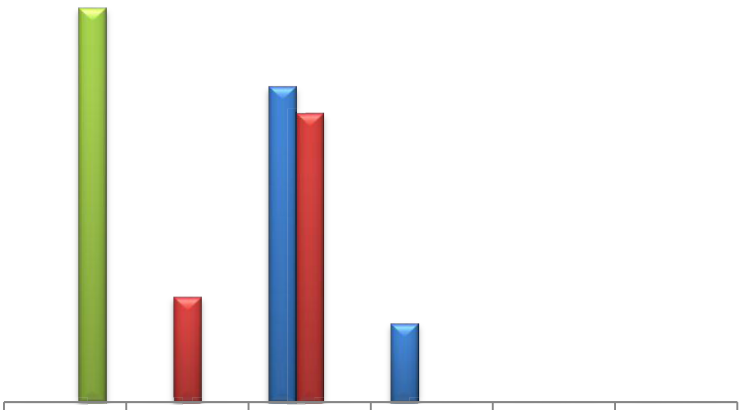


	B	IO	LS	CP	CO
LA	0	16	0	0	0
PUSHOVER	4	12	0	0	0
NDA	0	10	6	0	0

(a)



	A	B	IO	LS	CP	CO
LA	0	0	12	3	0	0
PUSHOVER	0	4	11	0	0	0
NDA	15	0	0	0	0	0

(b)

Fig.16. Comparison chart of the methods (a) X direction and (b) Y direction for the first story beams obtained by the design earthquake.

Fig.17.pdf

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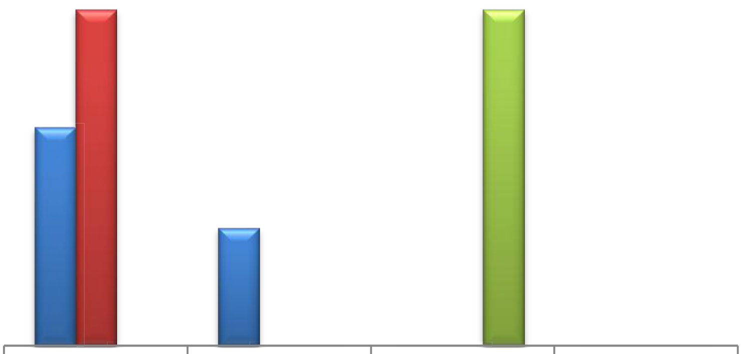
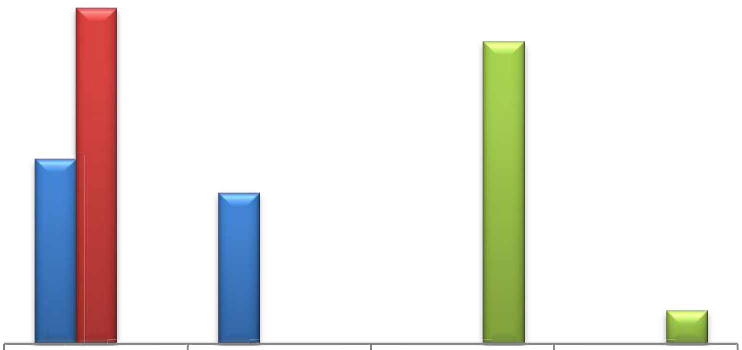


	IO	LS	CP	CO
LA	13	7	0	0
PUSHOVER	20	0	0	0
NDA	0	0	20	0

(a)



	IO	LS	CP	CO
LA	11	9	0	0
PUSHOVER	20	0	0	0
NDA	0	0	18	2

(b)

Fig.17. Comparison chart of the methods (a) X direction and (b) Y direction for the first story columns obtained by the design earthquake.