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Effect of base isolation systems on increasing the resistance of structures subjected to progressive collapse

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Abstract. Seismic isolation devices are commonly used to mitigate damages caused by seismic responses of structures. More damages are created due to progressive collapse in structures. Therefore, evaluating the impact of the isolation systems to enhance progressive collapse-resisting capacity is very important. In this study, the effect of lead rubber bearing isolation system to increase the resistance of structures against progressive collapse was evaluated. Concrete moment resisting frames were used in both the fixed and base-isolated model structures. Then, progressive collapse-resisting capacity of frames was investigated using the push down nonlinear static analysis under gravity loads that specified in GSA guideline. Nonlinear dynamic analysis was performed to consider dynamic effects column removal under earthquake. The results of the push down analysis are highly dependent on location of removal column and floor number of buildings. Also, seismic isolation system does not play an effective role in increasing the progressive collapse-resisting capacities of structures under gravity loads. Base isolation helps to localize failures and prevented from spreading it to intact span under seismic loads.

Keywords: base isolation; seismic loads; progressive collapse; column removal; push down analysis; nonlinear time history analysis

1. Introduction

One of the most effective techniques to protect structures against seismic effects is to use of the seismic isolation systems (Kang *et al*. 2009). Energy caused by seismic excitations dissipates installing the seismic isolation in foundation of the building structure and reduces transmitted acceleration in to the superstructures (Jangid 2007). Seismic isolation provides safety of structure against structural damage caused by strong and moderate earthquakes. In structures with seismic isolation, due to creat flexibility between foundation and superstructure, Period of the isolation system becomes more than dominant period of earthquake. Thus, the first mode of vibration provides most of deformation in base isolation system so that superstructure stays rigid (Matsagar and Jangid 2003).

Lead rubber bearing (LRB) is considered as one of the most conventional isolation systems that

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have been examined in previous studies. Existence the layers of rubber and steel plates in bearing make horizontal flexibility and the vertical stiffness, respectively. Yieling the lead core in bearing leads to make hysteretic damping. The methodology of simulating LRB bilinear hysteretic behavior was studied (Providakis 2008). Jangid (2007) performed an analytical study on seismic response of the multi-story buildings with LRB under near fault excitations. Komodromos (2008) performed numerical simulations for evaluation of isolated buildings subjected to strong earthquakes. He stated that the performance of the base isolation may be considerably influenced by factors that increase floor accelerations and inter-story deflections so that the efficiency of the base isolation decreases. Nath *et al*. (2013) evaluated performance of the base-isolated buildings using the numerical models. They stated that calibrating the numerical models with low level of earthquake can be developed for modeling of nonlinear responses of building subjected to the intense earthquake.

The phenomenon of progressive collapse can be defined on the basis of an initial failure and spread of member to other members that eventually leads to total structure or part of it. To reduce the risks caused by progressive collapse and increase the resistance of new and existing buildings, strategies were proposed by NIST (Ellingwood *et al*. 2007). Factors that may cause failure are included construction/design errors, abnormal loads (gas, explosions, vehicular collisions, and sabotage) and fires are not regarded in conventional design of the buildings. Recently, many building codes offered proposals based on increasing strength, ductility and continuity for building that subjected to progressive collapse (Kim *et al*. 2009). Different analysis methods have been presented by General Service Administration (GSA) (2003) and Department of Defense (DoD) (2005) for assessment progressive collapse in buildings.

Using systems that prevent progressive collapse and analysis methods can be considered as important approaches in assessing progressive collapse (Kim *et al*. 2011b). According to the results of some previous studies can be found that providing resistant structures against earthquake may also be an increase in resistance of progressive collapse (Corley 2002, Corley *et al*. 1998, Hayes Jr *et al*. 2005). Crawford (2002) suggested utilizing devices such as Side Plate, mega-trusses in high-rise buildings, cables embedded in reinforced concrete beams that applied in earthquake to enable the catenary operation and withstand against progressive collapse. Kaewkulchai and Williamson (2003) noted that the use of linear static analysis can not be simulated dynamic effects caused by the sudden column removal. Hayes Jr. *et al*. (2005) investigated relationship between seismic design and progressive collapse-resisting capacity. Tavakoli and Kiakojouri (2013) studied framed structures under pogressive collapse using a new method for removing column in different scenarios and found suitable agreement between results of new method with those obtained from traditional method. Tavakoli nad Alashti (2013) carried out push-over analysis on 2-D and 3-D models to assess progressive collapse in 5- and 15-story buildings with 4- and 6 bays using alternate load path method as proposal of UFC guideline. They demonstrated that 2-D models are more sensitive to the removal of column than 3-D models. Karimiyan *et al.* (2013a) examined collapse distribution of 6-story reinforced concrete ordinary moment resisting frame buildings under seismic loads. They concluded that almost pattern of collapse distribution was not related to the earthquake record. Karimiyan *et al*. (2013b) studied on seismic progressive collapse of 3-story moment resisting buildings with different levels of eccentricity in plan. They presented that drift response can be extracted easily to assess progressive collapse in structures as compared to number of plastic hinges. In an investigation on Progressive collapse of symmetric and asymmetric building models that conducted by Karimiyan *et al*. (2014), they found that potential of progressive collapse is more in asymmetric models than symmetric models due to local damage concentration in asymmetric models . Tavakoli and Kiakojouri (2014) proposed approaches for evaluating robustness of steel moment-resisting frames subjected to progressive collapse and carried out linear and nonlinear analysis methods.

In this study aimed to investigate the effects of base isolation systems on increasing the strength of the progressive collapse of structures caused by seismic loads. The performance of the LRB isolation system installed in the below of RC building structures to increase progressive collapse resistance was investigated by GSA 2003 gravity loading. 4-, 8- and 12-story reinforced concrete moment frames were selected to perform analysis in both the fixed and isolated bases by removing a column under gravity and seismic loads in various positions and locations. Evaluation of the structural resistance against progressive collapse under gravity loads was conducted by push down static nonlinear analysis. Time history nonlinear dynamic analysis was carried out under applying seismic loads due to consider the dynamic effect caused by an earthquake during column removal. In this case, the column was removed under gravity and seismic loads at the same time.

2. Descriptions of modeling

2.1 Building modeling

To carry out analysis, 4-, 8- and 12-story reinforced concrete model structures were considered as intermediate moment-resisting frames in longitudinal and transverse directions. Structural plan is identical in all models. The plan has five spans with 5 m in two directions. One of the internal frames was selected to perform dynamic analysis as shown Fig. 1 by a dotted rectangle. All floors have the same height and to be equal 3.2 m. The dead and live loads of the top story are 5.9 KN/m^2 and 1.47 KN/m^2, respectively. Other floors have a dead load of 6.4 KN/m^2 and the live load of 1.96 KN/m^2. The models were designed to withstand both gravity and lateral loads. The building structures used are intermediate moment frames. Beam and column sections were designed in such a way that stress ratios are approximately equal to 1. Geometric properties designed for structural elements have been shown on the elevation of the frames in Figs. 2(a)-(c). Based on ACI 318 (2008), concrete with compressive strength equal to 24.5 MPa, poison ratio 0.2 and Young's modulus 23 MPa was considered for material properties. Longitudinal reinforcement of the sections is 3% with yield stress 338 MPa, poison ratio 0.3 and Young's modulus 2E5 MPa for both

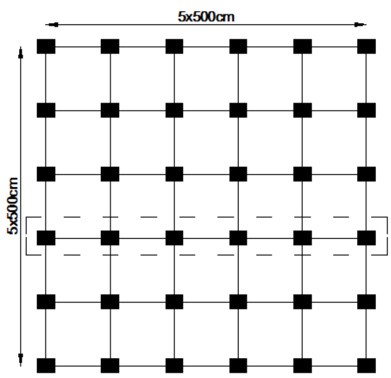
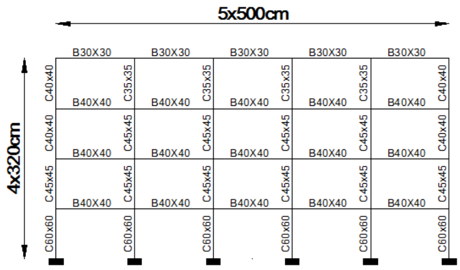
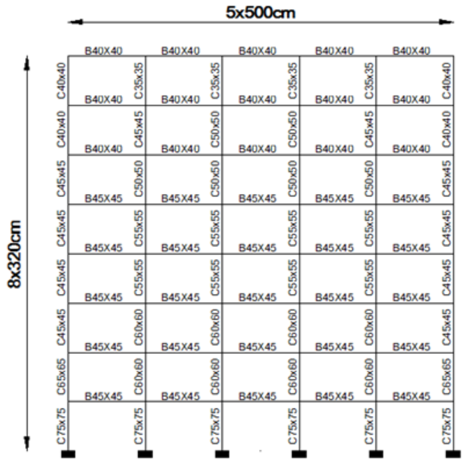


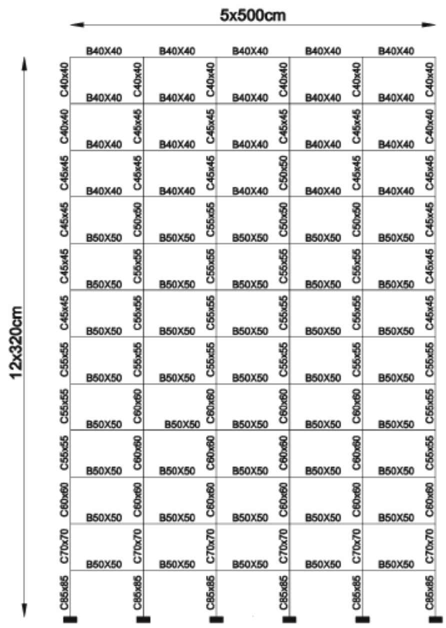
Fig. 1 Typical plan



(a) Elevation of 4-story



(b) Elevation of 8-story



(c) Elevation of 12-story

Fig. 2 Elevation of model structures

Table 1 Longitudinal Reinforcement of beams and columns

Model	Story Number	End of beam		Mid-span of beam		Interior Columns	Exterior Columns
		Top	Bottom	Top	Bottom		
4-story	1	6-D22	3-D18	2-D18	3-D18	32-D18	32-D18
	2	6-D25	6-D16	3-D16	4-D18	20-D18	20-D18
	3	7-D22	3-D20	3-D16	3-D20	20-D18	16-D18
	4	6-D22	3-D16	2-D14	3-D22	12-D18	16-D18
8-story	1	7-D22	7-D16	4-D14	6-D14	44-D22	44-D22
	2	7-D28	9-D20	6-D14	6-D18	32-D18	44-D18
	3	8-D28	7-D25	5-D18	7-D18	32-D18	20-D18
	4	8-D28	7-D25	5-D18	7-D18	28-D18	20-D18
	5	9-D25	5-D25	5-D16	8-D16	28-D18	20-D18
	6	8-D25	4-D25	5-D14	8-D14	24-D18	20-D18
	7	8-D22	4-D20	4-D14	5-D16	20-D18	16-D18
	8	5-D22	5-D14	3-D14	5-D14	16-D18	16-D18
12-story	1	7-D22	7-D18	3-D16	4-D18	56-D22	56-D22
	2	8-D28	7-D25	7-D14	6-D18	36-D22	36-D22
	3	10-D28	7-D28	5-D18	7-D18	32-D18	32-D18
	4	10-D28	7-D28	5-D18	7-D18	32-D18	32-D18
	5	10-D28	7-D28	5-D18	7-D18	32-D18	32-D18
	6	9-D28	8-D25	5-D18	6-D18	28-D18	28-D18
	7	9-D28	8-D25	5-D18	6-D18	28-D18	20-D18
	8	7-D28	7-D25	5-D18	6-D18	28-D18	20-D18
	9	7-D28	6-D22	4-D18	7-D16	24-D18	20-D18
	10	8-D25	4-D25	4-D18	7-D16	20-D18	20-D18
	11	8-D22	6-D18	3-D18	5-D18	20-D18	16-D18
	12	7-D18	4-D16	3-D14	4-D16	16-D18	116-D18

columns and beams. Longitudinal reinforcement of beams and columns have been shown in Table 1. Models were built in case of both the fixed and isolated bases with LRB isolator that LRB Isolators were placed under each column.

2.2 Base isolation modeling

In this study, the effect of the progressive collapse of structures with LRB isolation systems has been investigated. LRB is comprised of layers of rubber and steel plates. Layers of rubber cause that the structure to be flexible in the horizontal direction. Steel plates provide vertical stiffness of the structure. LRB is also composed of lead-plug that yielding of the lead-plug dissipates seismic energy and mitigates isolator displacement. Fig. 3 shows the configuration of LRB isolator (Providakis 2008). Behavior of the LRB isolation system were modeled by the bilinear force-deformation curve as a suggestion of win (2008) that has been shown in Fig. 4.

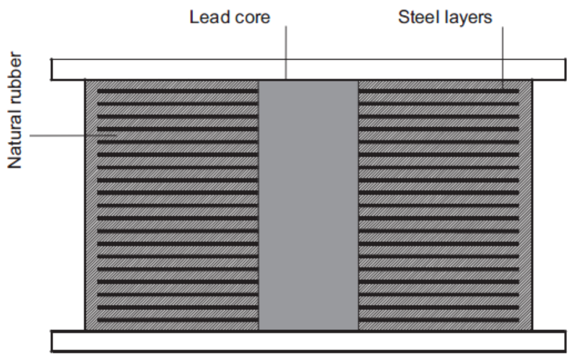


Fig. 3 Configuration of LRB isolation system

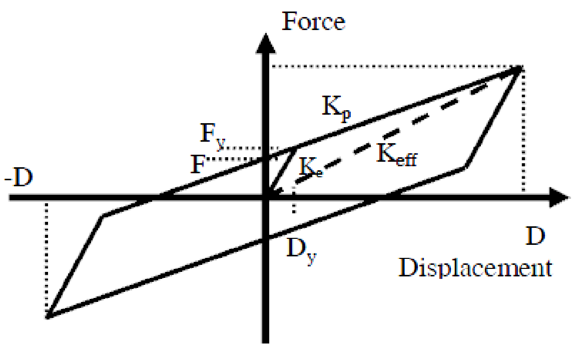


Fig. 4 Bilinear force-displacement behavior of the LRB

Table 2 Mechanical properties of the LRB isolation system

Model	K_{eff} (KN/m)	K_e (KN/m)	K_p (KN/m)	Q_D (KN)	F_y (KN)	α	T_D (s)	D_D (m)
4-story	437.4	3083.22	265.6	54.55	64.75	0.086	2.5	0.295
8-story	797.93	4504.59	484.59	92.46	103.6	0.1	2.7	0.32
12-story	923.32	5886.4	560.75	133.14	147.16	0.095	3	0.354

LRB isolates were designed considering total weight value obtained from the static analysis procedure. In order to obtain the mechanical properties of the LRB as given in Table 2, the fundamental isolation period was assumed in the range of 2.5-3 s (Tavakoli *et al*. 2014). Isolators design calculations were conducted according as works of Kelly (1997) taking into account the additional recommendations and practical limitations in flexural and shear deformation.

Where K_{eff}, K_e and K_p are effective stiffness, post-yield stiffness and elastic stiffness, respectively, yielding force F_y, design displacement D_D, fundamental isolation period T_D, characteristic strength Q_D, and α is post- to pre-yielding stiffness ratio. Table 3 presents the results of calculations of the dimensions of LRB. Where h is isolator height, the number of the rubber layers N, the layer thickness t_r, the number of the steel plates Ns, bottom and up steel plates of isolator t_{tp}, lead-plug diameter *dp*, and isolator diameter *d*.

Table 3 Dimensions obtained from the design LRB isolation system

Model	*h* (cm)	*N*	t_r (cm)	N_s	t_s (cm)	t_{tp} (cm)	d_p (cm)	*d* (cm)
4-story	29.4	9	1	18	0.3	2.5	11	50
8-story	32.5	4	1.5	13	0.5	2.5	13	60
12-story	36.5	6	1.5	15	0.5	2.5	15	70

3. Methodology of analysis

To evaluate the effect of isolation systems for progressive failure that made due to weak design/construction is required first to investigate the resistance of structures against progressive collapse under gravity loads. For this purpose, one of the columns was removed. Then, effect of column removal was evaluated in first floor of the models by nonlinear static push down analysis method. The analysis was done using SAP 2000 (2006) software. To carry out nonlinear analysis need to consider nonlinear frame members by assigning plastic hinges at the end of beams and columns as FEMA-273 (1997) or ATC-40 (1996) that shown in Fig. 5. Three points IO, LS and CP represent performance levels that they are Immediate Occupancy, Life Safety and Collapse Prevention, respectively. Gravity loads were applied on models as a recommendation of the GSA 2003 guideline. At the suggestion of GSA 2003 guideline, amplification factor of 2 was considered for static analysis. Load combination used is including of 2 (Dead Load+0.25×Live Load) that applied only on the spans that a column was removed and without amplification factor of 2 on intact spans. Strength capacity against progressive collapse can be mentioned based on load factor obtained for specific vertical deformation under nonlinear static push down analysis. Considering the a maximum deflection of 20 cm as a work of Tsai and Lin (2008), gravity loads were gradually increased. Then, load factor was calculated on the basis of the ratio of the equivalent load corresponding to the deflection and load specified in GSA 2003 in each step of the analysis. To consider the effect of seismic loads on frames with removal of column need to perform dynamic analysis. As a recommendation of GSA 2003, gravity loads applied on all spans are without dynamic amplification factor in dynamic analysis. Figs. 6(a) and 6(b) show samples of gravity loading in frames with a removal column under nonlinear static and dynamic analysis, respectively. Simulating removal of column was carried out by replacing point load of the column that calculated before removing it as Fig. 7, where P, V, M and W represent axial force, shear force, bending moment and vertical load, respectively. It is seen that forces were enhanced for five

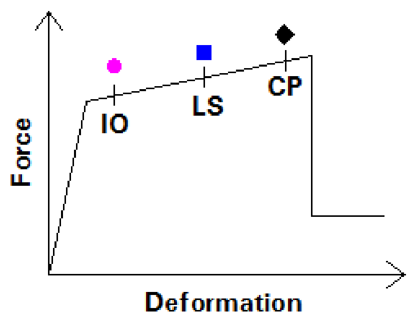
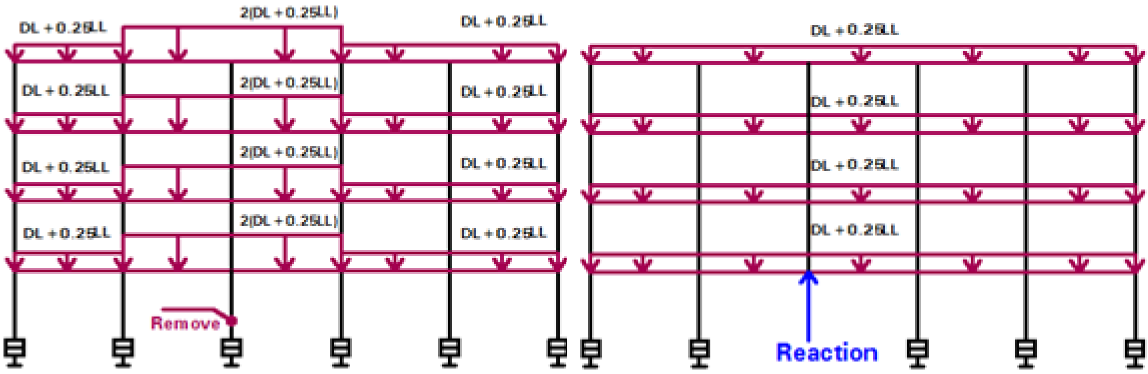


Fig. 5 Plastic hinge model



(a) Nonlinear static method (GSA 2003) (b) Nonlinear dynamic method (GSA 2003)

Fig. 6 Applying gravity loads in progressive collapse analysis as GSA 2003

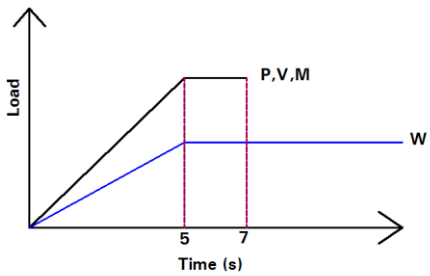


Fig. 7 Function of column removal in dynamic analysis

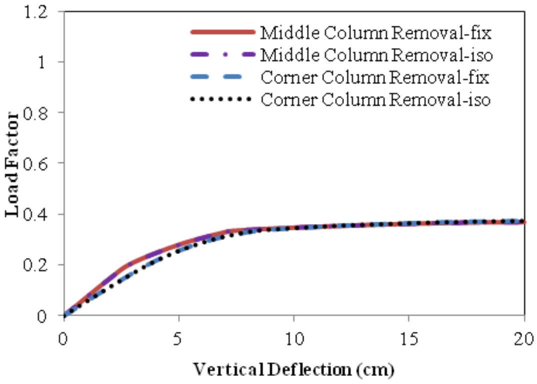
Table 4 The features of three records from ground motions

Record/Component	Station	Magnitude	Distance closest to the fault (km)	PGV (cm/s)
Imperial Valley	5052 Plaster City	6.5	31.70	5.4
Northridge	24576 Anaverde Valley-City R	6.7	38.40	5.5
Duzce, Turkey	Cekmece	7.1	188.40	2.5

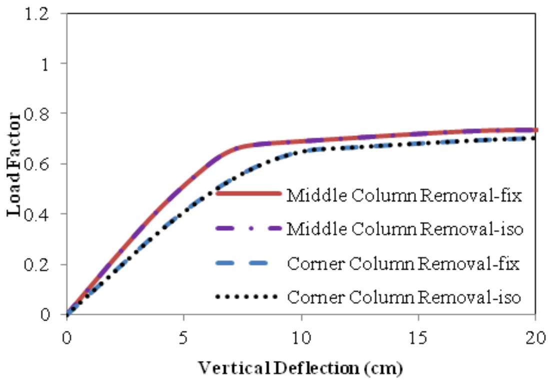
seconds until arriving at their maximum amount, member forces were fixed for two seconds to be stable system. Finally, upward force was abruptly removed in seventh second. To assess progressive collapse under seismic loads, three records were considered as Table 4. Peak ground accelerations in ground motions were scaled to 0.35 g and employed in the analysis.

4. Progressive collapse-resisting capacity

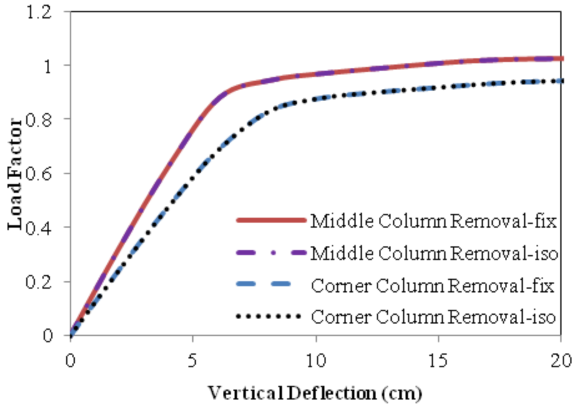
Push down analysis was performed by removing the column in the first floor of model structures in corner and middle situations. In this method, gravity loads were increased until the deflection at the point of removal of the column reached to 20 cm. Load factor in each step corresponding to vertical deflection was stated as shown in Fig. 8. As shown in the figures the



(a) 4-story



(b) 8-story



(c) 12-story

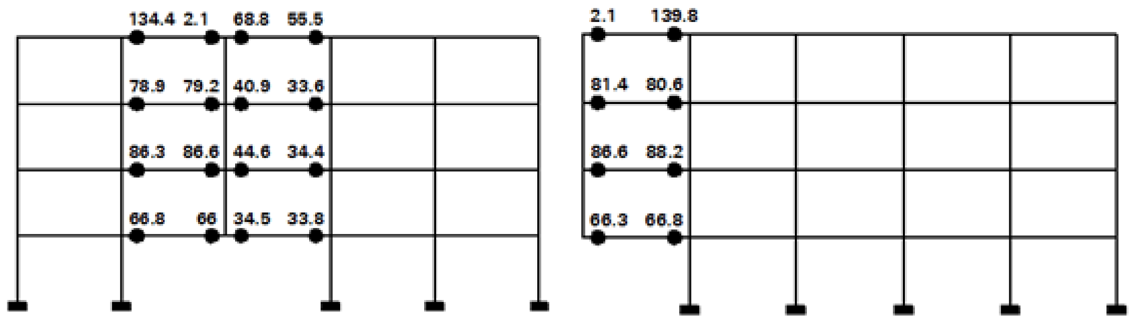
Fig. 8 Push down Load-deflection curves of the fixed and isolated models in the removal of the corner and middle columns

symbol fix and iso represent the fixed-base and base-isolated models. Load factor is the ratio of push down force in each step to total gravity loads applied to structure as GSA 2003 guideline. Results of load factor for both the fixed and base-isolated models are same together. Maximum load factors for models 4-, 8- and 12-story with removal of corner column are 0.37, 0.7 and 0.94, respectively. Also, removing middle column the maximum load factors are 0.37, 0.74 and 1. It can be expressed that the load factors of models with removal of middle column are more than when the corner column was removed. When a middle column was removed catenary action of beams is more due to existence two spans in above of removal of column than one span in loss of corner column.

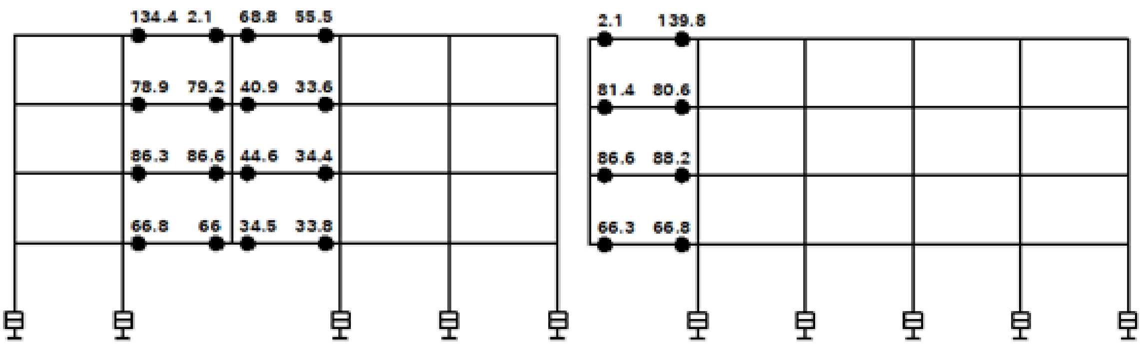
Increasing number story causes that maximum load factor increases in models with the isolator and without it. As statement of Kim *et al*. (2011a) structures with maximum load factor less than 1 have a probability of progressive collapse. Also, load factor less than 0.5 presents severe progressive. As it can be seen from figures, even base isolation system has no effect on increasing progressive resistance of the structures under gravity loads. Figs. 9-11 show rotation of plastic hinges in member of structures that obtained from push down analysis in both fixed and isolated models after the loss of the column subjected gravity loads of GSA 2003. Acceptance criteria

10.5% radian for rotation hinge was considered in GSA 2003 for concrete beam. Rotation hinges are similar in isolated and fixed structures as results of maximum load factor. Plastic hinges were only formed in spans damaged of the column removal. When corner and middle columns were removed in 4-story models, almost all the hinges formed in beams were exceeded from acceptance criteria. There is strong probability of the progressive collapse in 4-story models as results corresponding to the maximum load factor 0.37 that it is less than 0.5. Therefore, these results are compatible with the statements of Kim *et al*. (2011a).

Hinge rotation of the 8-story models against loss of the column was significantly improved compared with 4-story as shown in Fig. 10. More number of hinges were in accepted range of rotation. Also, models with the middle column removal had hinge rotation relatively less than those with corner ones. In 12-story models as Fig. 11, no plastic hinges even formed in some members. In addition, entire hinges were satisfied the acceptance criterion, hinge rotations were much less than 10.5% radian. Then, 12-story models were not very susceptible against loss of the column. These Results are also compatible with the results mentioned of the maximum load factor almost 1 for 12-story that they are safe subjected to progressive collapse. Fig. 12 shows the displacement time history at the point of the column removal in time duration of 20 seconds under gravity load that specified in GSA 2003. By removing column in seventh second under gravity loads were attained same results for this displacement in both the fixed and isolated buildings. It can be observed that increasing story number increases resistance of the structures against progressive collapse. This displacement at the point of middle column removal is almost less than corner one.

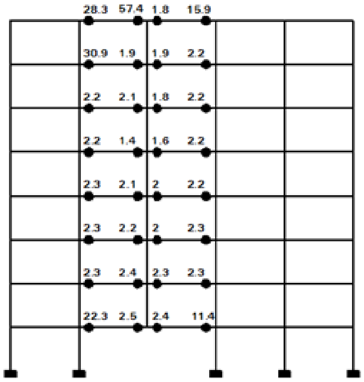


(a) Removal of middle column, fixed base (b) Removal of corner column, fixed base

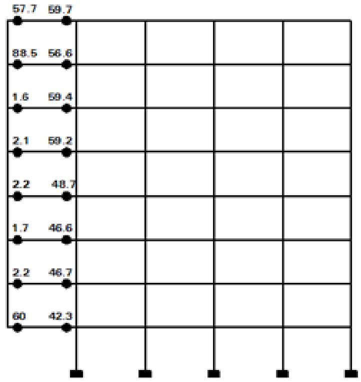


(c) Removal of middle column, isolated base (d) Removal of corner column, isolated base

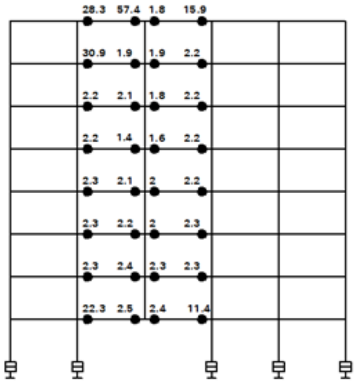
Fig. 9 Rotation of plastic hinges in 4-story models in radian (%) under gravity loads (GSA 2003)



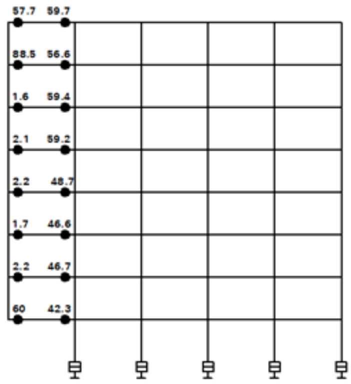
(a) Removal of middle column, fixed base



(b) Removal of corner column, fixed base

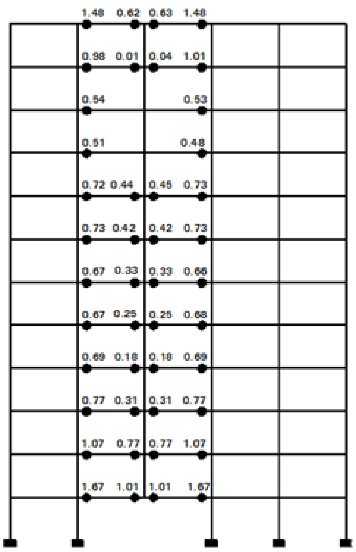


(c) Removal of middle column, isolated base

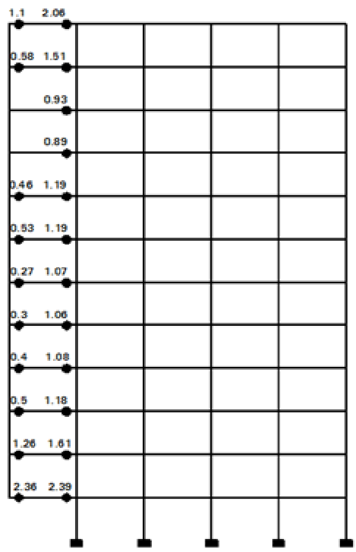


(d) Removal of corner column, isolated base

Fig. 10 Rotation of plastic hinges in 8-story models in radian (%) under gravity loads (GSA 2003)

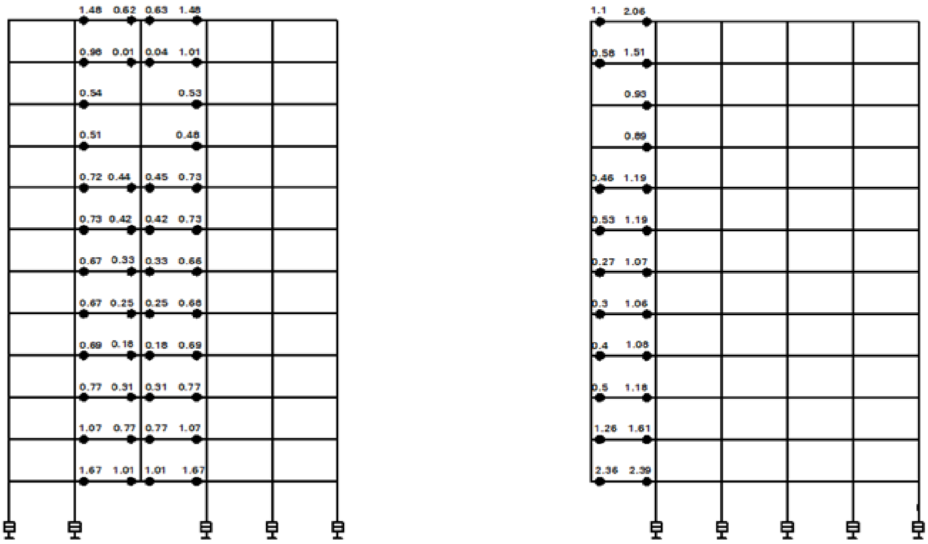


(a) Removal of middle column, fixed base



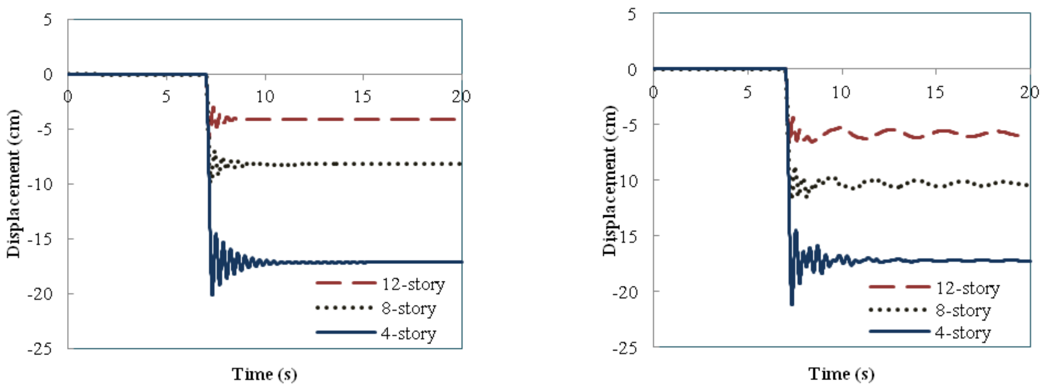
(b) Removal of corner column, fixed base

Fig. 11 Rotation of plastic hinges in 12-story models in radian (%) under gravity loads (GSA 2003)



(c) Removal of middle column, isolated base (d) Removal of corner column, isolated base

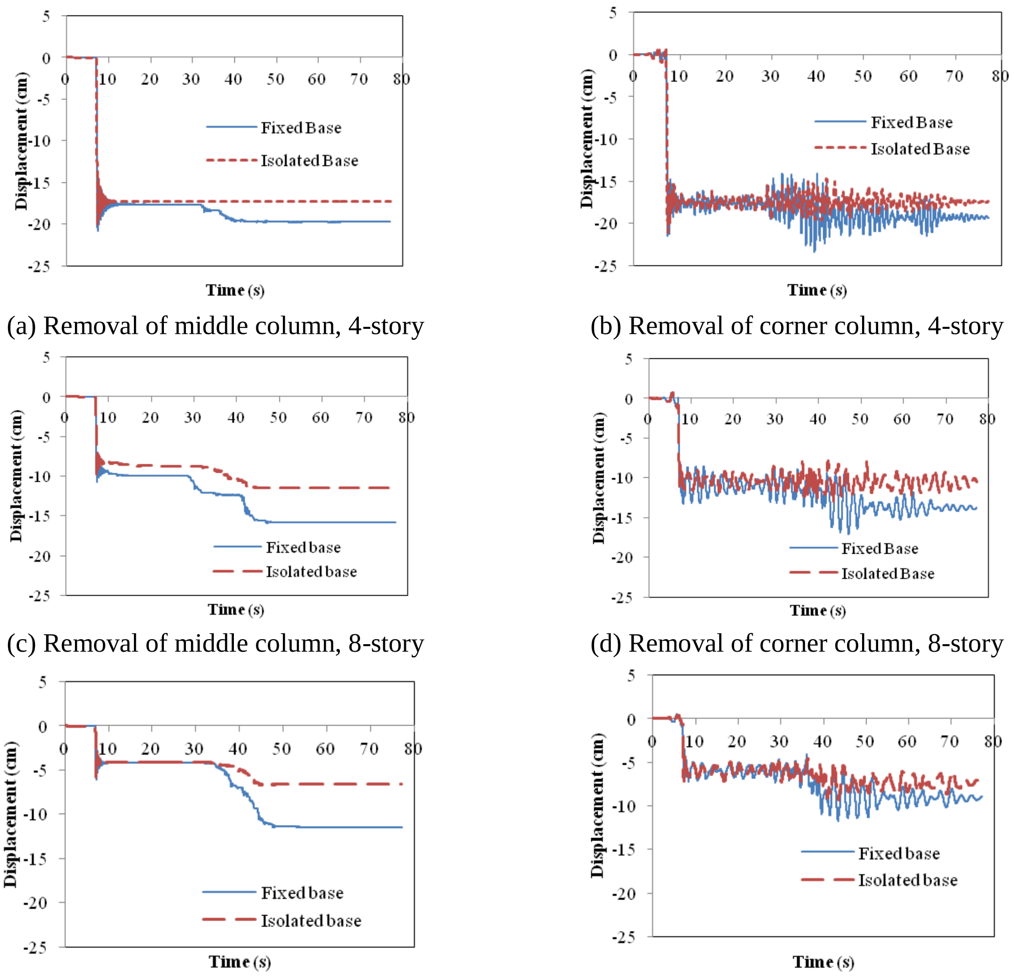
Fig. 11 Continued



(a) Removal of middle column (b) Removal of corner column

Fig. 12 Comparison of vertical displacement time history at the point of column removal under gravity loads specified in GSA 2003 in dynamic analysis

Fig. 13 shows the time history of vertical displacement at the point of removal of the column at the first story when that subjected to seismic loads in addition to gravity loads. The results obtained from nonlinear dynamic analysis that applied to structures with removal of a column in during three ground motions. These results are for maximum response of the three earthquakes that belongs to earthquake in Duzce, Turkey. Vertical displacement of structures with seismic isolation is less than base-fixed structures. Especially, after removing middle column vertical deflection was less subjected to the fluctuations in the rise and fall so that quickly reach equilibrium. During the earthquake, vertical displacement was intensified comparison to initial time of the column removal in base-fixed models, while the decline has even been in some base-isolated models. After removal of column in seven seconds in case of fixed-base frames as shown in the figures, structural system loses its stability under seismic loads. Due to the absence of control devices such as seismic isolation, seismic loadings increase maximum vertical displacement at the point of column removal in all cases of fixed base models, while this is generally not occur in isolated models. Base isolation system not only prevents the high increase in the maximum vertical displacement of the point of column removal in 8- and 12-story models, but also reduces the maximum vertical displacement for 4-story model in the time period of 30s-40s. Also, it can be observed as the Fig. 14 maximum response of this vertical displacement under seismic loads that compared with each other in all model structures. The results vary depending on the position of the removal of column and type of structure. In all models, when middle column was removed displacement response is small in comparison with the removal of corner column. Increasing story number causes decreasing displacement and also increasing resistance of progressive collapse in both the fixed and base-isolated structures under seismic loads.



(a) Removal of middle column, 4-story

(b) Removal of corner column, 4-story

(c) Removal of middle column, 8-story

(d) Removal of corner column, 8-story

(e) Removal of middle column, 12-story

(f) Removal of corner column, 12-story

Fig. 13 Comparison of vertical displacement time history at the point of column removal in fixed and isolated buildings under seismic loads

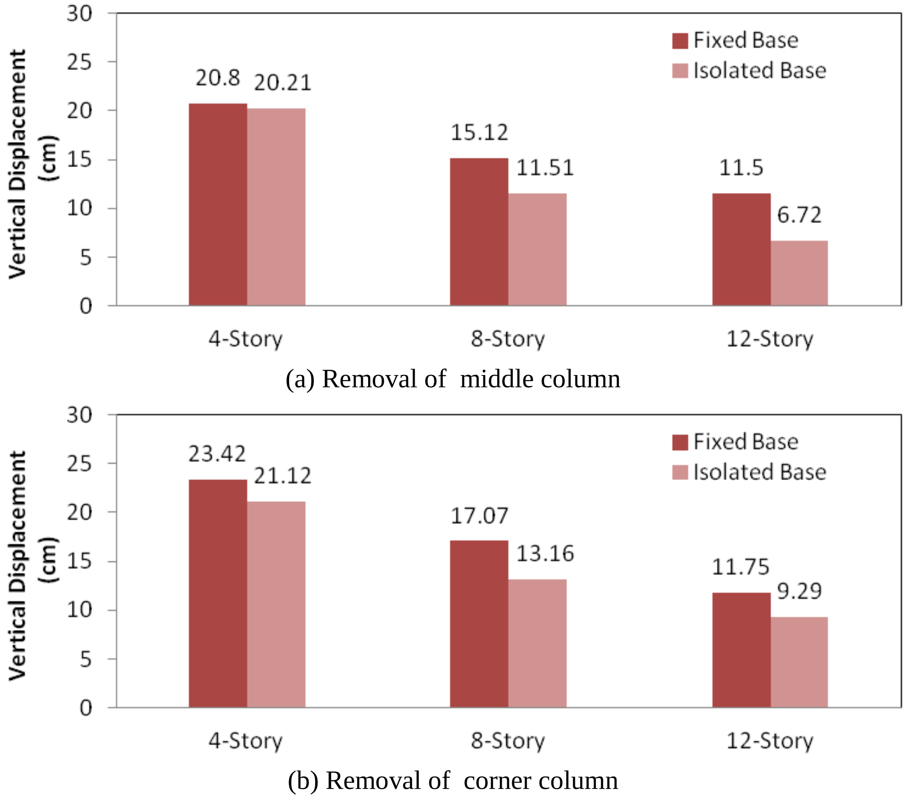
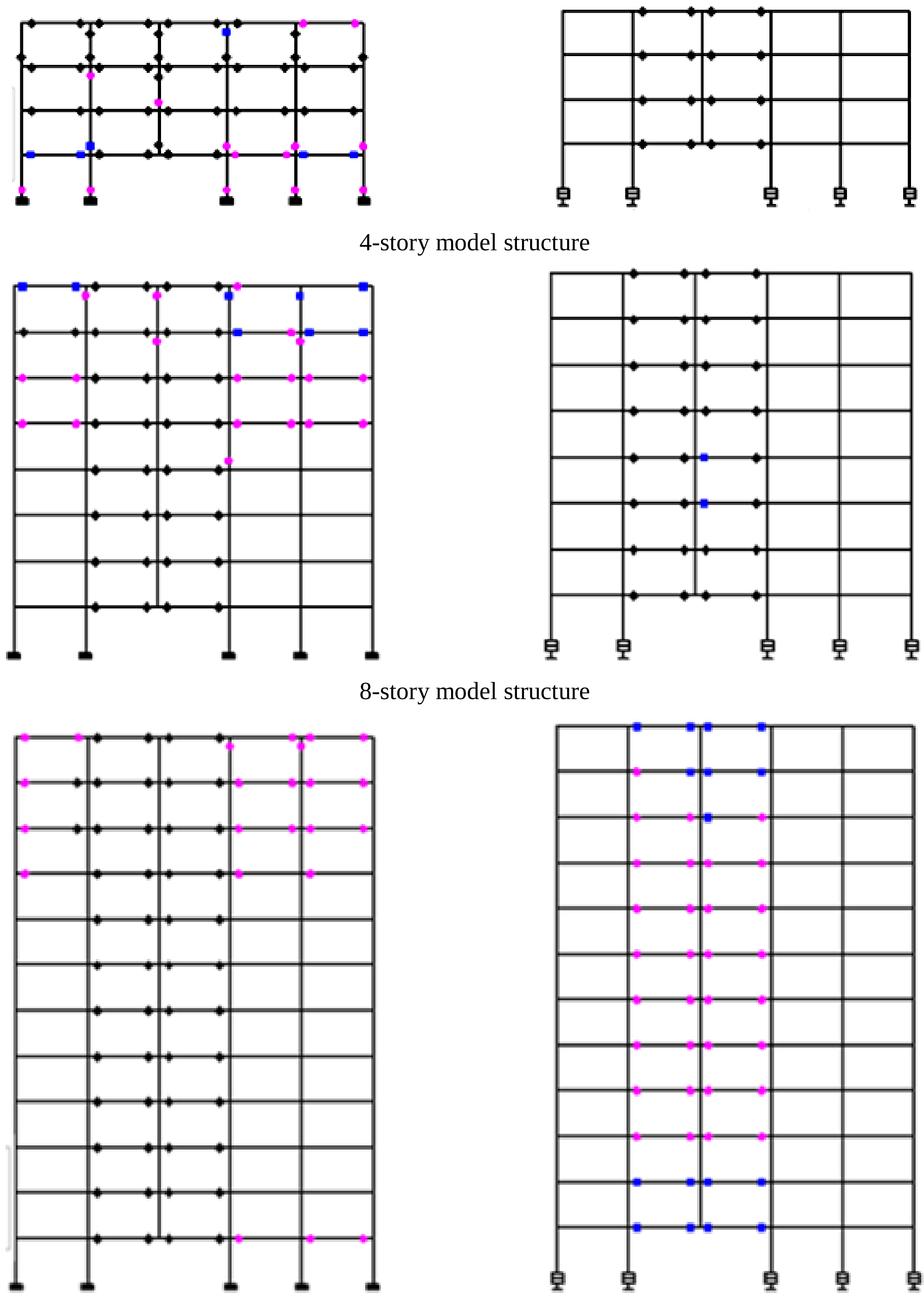


Fig. 14 Comparison of maximum vertical displacement at the point of column removal under seismic loads

Figs. 15 and 16 show performance levels of the plastic hinges based on symbols specified in Fig. 5 that obtained from the time history analysis. These hinges were formed in structures subjected to removal of the column at the first story under seismic loads. Performance levels were assessed at the ultimate displacement of model structures. As it can be observed applying seismic loads causes formation plastic hinges at both sections of beams and columns of the base-fixed structures while, no plastic hinge was formed in columns of the base-isolated models after removing a column. The number of plastic hinges formed in the base-isolated buildings is significantly less than fixed bases. When both the corner and middle column were removed at the first story of base-fixed buildings, especially 4-story, high number of plastic hinges were formed at the CP performance level and progressed to intact spans. Damage progression in intact spans of 8- and 12-story base-fixed models were considerably decreased compared to 4-story case due to increase redundancy and catenary action. It also can be seen that no plastic hinges formed in intact spans of isolated models when a middle column was removed. When a corner column was removed, hinges progressed to intact spans of isolated building were in IO performance level. Also, no hinges exceeded from LS performance level in 12-story isolated buildings even in damaged spans. Therefore, base isolation system plays an effective role in localization failures and not to spread to undamaged spans. Based on the nonlinear time history analysis, it can be concluded that isolated buildings were safe against progressive collapse caused by column removal under gravity loads also remained safe subjected to seismic loads.



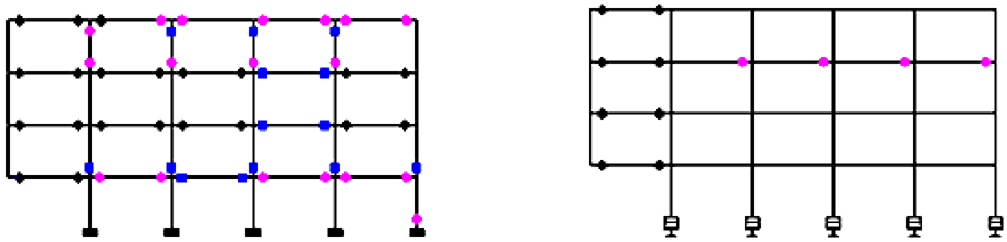
4-story model structure

8-story model structure

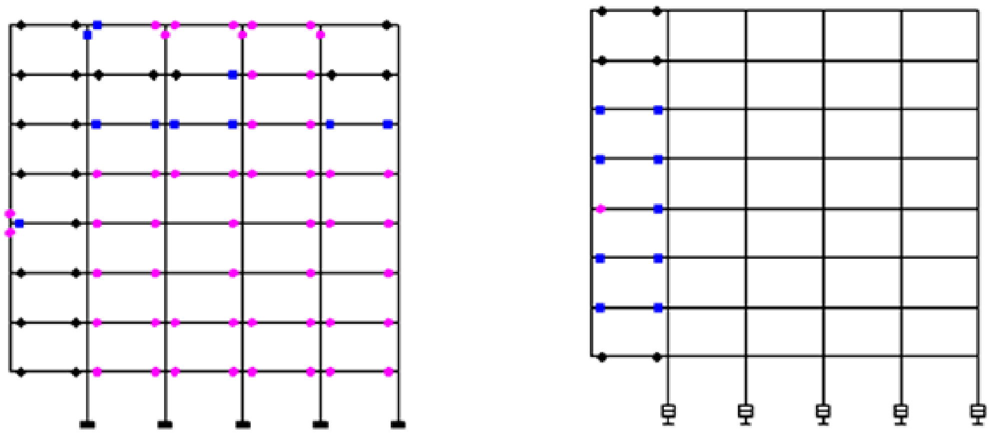
12-story model structure

(a) Fixed-base (b) Isolated-base

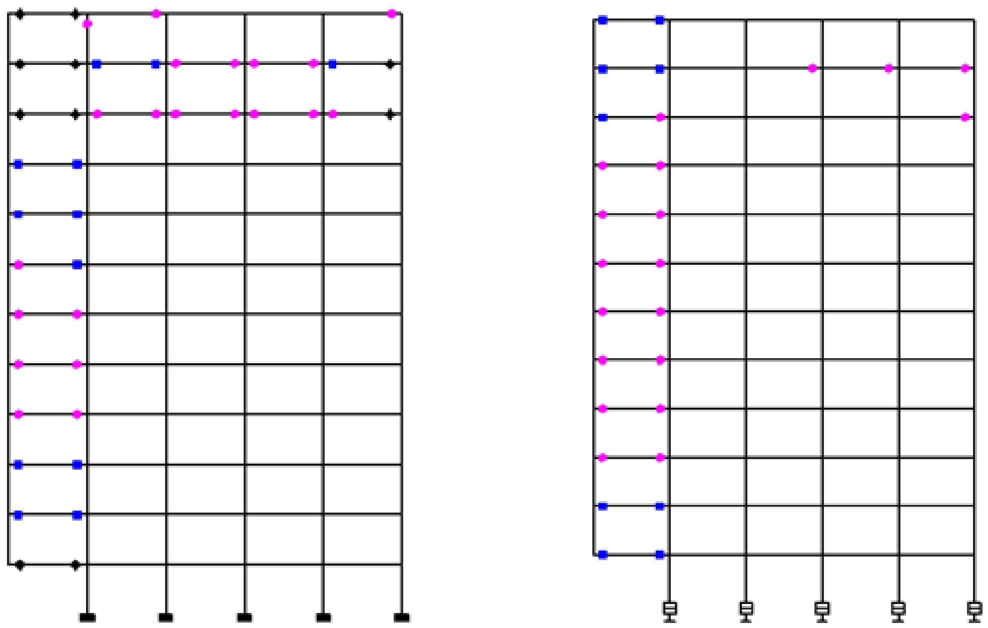
Fig. 15 Plastic hinge formation in fixed and base-isolated model structures when the first story middle column was removed under seismic loads



4-story model structure



8-story model structure



12-story model structure

(a) Fixed-base (b) Isolated-base

Fig. 16 Plastic hinge formation in fixed and base-isolated model structures when the first story corner column was removed under seismic loads

5. Conclusions

In the present study effectiveness of the LRB isolation system in increasing resistance of progressive collapse for reinforced concrete frames was investigated under both the gravity and seismic loads. Analysis was performed using the nonlinear static (push down) and dynamic time history methods. Gravity loads were applied on frames as specified in the GSA 2003 guideline. 4-, 8- and 12-story model structures were considered for analysis. The Imperial Valley, Northridge and Duzce, Turkey records have been selected as seismic excitations to carry out nonlinear dynamic time history analysis. Results are given in following:

• LRB isolation systems have no effect on increasing the resistance of the progressive collapse of structures that caused by gravity loads.

• The deflection at the point of first story column removal increases under seismic loads than when gravity loads were only applied. Also, deflection is less than in isolated buildings compared with the base-fixed buildings when subjected to seismic loads.

• Seismic isolation systems prevent the spread of failures to intact spans and maintain these spans in life safety performance level.

• Using the seismic isolators, model structures were highly resistant to progressive collapse under gravity loads as GSA 2003 can be kept absolutely safe against progressive collapse under seismic loads.

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