Analytical Study of Seismic Progressive Collapse in one-Story Steel Building

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SUMMARY:
Progressive collapse is defined as total or remarkable partial collapse of structure following local damage at a small portion of the building. In most cases, the investigations are focused on the progressive collapse of structures due to explosion, vehicle impact, fire, or other man-made hazards, with a little attention paid to progressive collapse mechanism of structure due to earthquakes. In this study, to navigate the initial damage toward a specific part of the structure a corner-column was intentionally weakened. Then, push over analysis is carried out on the three dimensional model of the building and the behavior of structure, such as deformations are studied and the energy absorption of the frames are investigated and finally the collapse pattern of the building is obtained.

Keywords: progressive collapse, nonlinear analysis, collapse pattern

1. INTRODUCTION

Progressive collapse can occur as the result of natural or man-made hazard. If a structure has good alternative loading path, the initial failure will not expand to the other parts of the structure and the local damage will be restricted. Studies and researches about progressive collapse mostly evaluate the collapse mechanism of structures caused by man-made hazards. Current progressive collapse analysis procedures, which take into account only the gravity loads, may not have the capabilities to simulate the progressive collapse of structures due to earthquakes. On the other hand, natural hazards such as earthquakes can generate significant lateral loads and stress reversals which can overload structural elements and result in the loss of load carrying elements and trigger disproportionate collapse of the structures. So analysis of structures with removed columns during the earthquakes must be regarded. The effects of lateral loads should be considered in conjunction with those from gravity loads for seismic progressive collapse analysis. Progressive collapse of the structures has attracted much attention in the past few decades. M. Sasani, and S. Sagiroglu. (2008) evaluated progressive collapse resistance of an actual six-story reinforced concrete frame structure following simultaneous removal of two adjacent exterior columns and then changes in columns axial forces and load redistribution were discussed.

O. A. Pekau, Yuzhu Cui. (2006) modified a distinct element method to model the precast panel shear walls and investigated the progressive collapse processes of panel wall under gravity and lateral loads focusing on the shear ductility demands of mechanical connections. The differences between the behavior of model under gravity and earthquake loads were discussed and results indicated that if design of panel shear wall satisfies the seismic design requirements, it will meet the ductility demand with respect to progressive collapse without earthquake. S. Marjanishvili and E. Agnew. (2006) analyzed a nine-story steel moment-resistant frame building employing various analytical procedures: linear-elastic static, nonlinear static, linear-elastic dynamic and nonlinear dynamic procedures and compared the results obtained from analysis. They concluded dynamic analysis method acquires more accurate results and is also easy to carry out to evaluate the progressive collapse potential of structures.

Min Liu (2010) evaluated the progressive collapse potential of regular steel immediate moment frames with the minimized total steel weight. The design of frames satisfied both AISC seismic provisions and UFC progressive collapse requirements. The analysis were carried out using three analysis procedures (linear static, nonlinear static and nonlinear dynamic). The results showed that the seismic design which does not consider progressive collapse requirement, fails to meet the UFC alternate path criteria associated with any analysis procedure. Xinzheng Lu et al (2008) simulated the extreme nonlinear behavior of reinforced concrete structural elements with the fiber-beam-element model and multi-layer-shell-element model. They used simple reinforced concrete frames and reinforced concrete frame-shear wall structures to benchmark the capacity of the numerical model and then investigated the failure mechanism of the actual buildings.

Xinzheng Lu et al. also proposed a fiber beam model to simulate the collapse behavior of reinforced concrete frames. They simulated the collapse mechanism of two typical Chinese eight-story reinforced concrete frames with and without slabs and then compared the results acquired. Finally the positive influence of slabs on the progressive collapse behavior of structures was discussed. J. Kim and J., Park. (2008) evaluated the progressive collapse potential of three and nine-story special moment resisting frames by nonlinear static and dynamic analysis. They demonstrated that the model structures which are designed only for normal loads have high progressive collapse potential whereas the structures designed by plastic designed concept satisfies the GSA (2003), guideline failure criterion.

F. Fu. (2009) evaluated progressive collapse potential of a twenty-story building using 3-D finite element model. Shell elements and beam elements were used to simulate the elements of the building. Analytical results were then compared with those obtained from experiments and good accordance between the results was obtained. The analytical model accurately represented the behavior of the twenty-story building under column removal. Jinkoo Kim and Dawoon An. (2009) evaluated the catenary action influence on the collapse potential of steel moment framed structures. They conducted Non-linear static and dynamic analyses on three- and six-story model structures with and without bracing according to the GSA provisions. Based on the results, the effect of the catenary action increased as the number of story and the number of bay increased. Also the results of non-linear dynamic analysis indicated that the maximum deflection of the structure during progressive collapse analysis decreases when the catenary action was taken into account.

J. Kim, J. Park, T. Lee. (2011), investigated the sensitivity of design variables of steel buildings subjected to progressive collapse. The results showed beam yield strength is the most significant design parameter in the dual system buildings. L. Kwasniewski. (2010) evaluated the progressive collapse potential of an eight-story steel frame structure under column removal using nonlinear dynamic finite element model based on the GSA guidelines. Finally the modeling parameters which affect the results were identified. A.G. Vlassis et al. (2008) presented a new design-oriented method for progressive collapse evaluation of multi-story buildings. The proposed method identifies the ductility demand and supply to determine progressive collapse potential under column removal. They concluded that besides the tying force requirements, ductility demand and supply must be considered in the support joints of the failed members to provide structural strength. W. J, Yi et al (2008) studied the progressive collapse failure of a reinforced concrete frame due to the loss of a column by static experiment and they investigated the redistribution and transition of the load resisting mechanism based on the experimental results.

In this article, a corner-column of a one-story steel frame building was weakened to navigate the initial damage toward a certain part of the structure and the nonlinear static analysis was carried out on the three dimensional model. Two one-story buildings were modeled which had various numbers of spans in two directions. The height of the structures was three meters and the length of the bays in both directions was five meters. The building was designed based on current design codes and special moment resisting frame was its lateral load resisting system which had rigid connections in both directions. Box profiles were the section profile for columns and beams, and the floor system was constructed by composite slab with the thickness of 12 centimeters. Table 1.1. shows the material used in the model.

Table 1.1. Materials′ properties used in the buildings

Steel	Modules of Elasticity =200 [Gpa]	F_y =250 [Mpa]	F_u =407.7 [Mpa]
Concrete	Modules of Elasticity =26.5 [Gpa]	f'_c= 28.1[Mpa]	

2. FINITE ELEMENT MODEL

A finite element software was used to model the building analytically and Rigid diaphragm assumption was included in the model. Finite element model of building is shown in Fig. 2.1. Nonlinear shell elements were used to model the beams and columns and after modeling the structure, a corner-column was intentionally weakened by reducing its yield and ultimate strength to navigate the initial damage toward a specific part of the structure. GSA progressive collapse guidelines apply the following load combination while analyzing for progressive collapse:

$$\text{Load} = 2 \times (DL + 0.25LL) \tag{2.1}$$

Where:
DL: Dead load and LL: Live load.

The live load and dead load (including the weight of elements) were assumed to be 150 kgf/m^2 and 600 kgf/m^2 respectively distributed uniformly on the beams. Then the static load combination, which was recommended by GSA, was applied during 60 seconds and then the pushover analysis was performed to simulate the behavior of the structure under lateral loads.

The results of the damaged model were compared with the results of the same analysis on the primary model (The primary model has not any weakened column) to have better perception of the behavior and the collapse mechanism of the model.

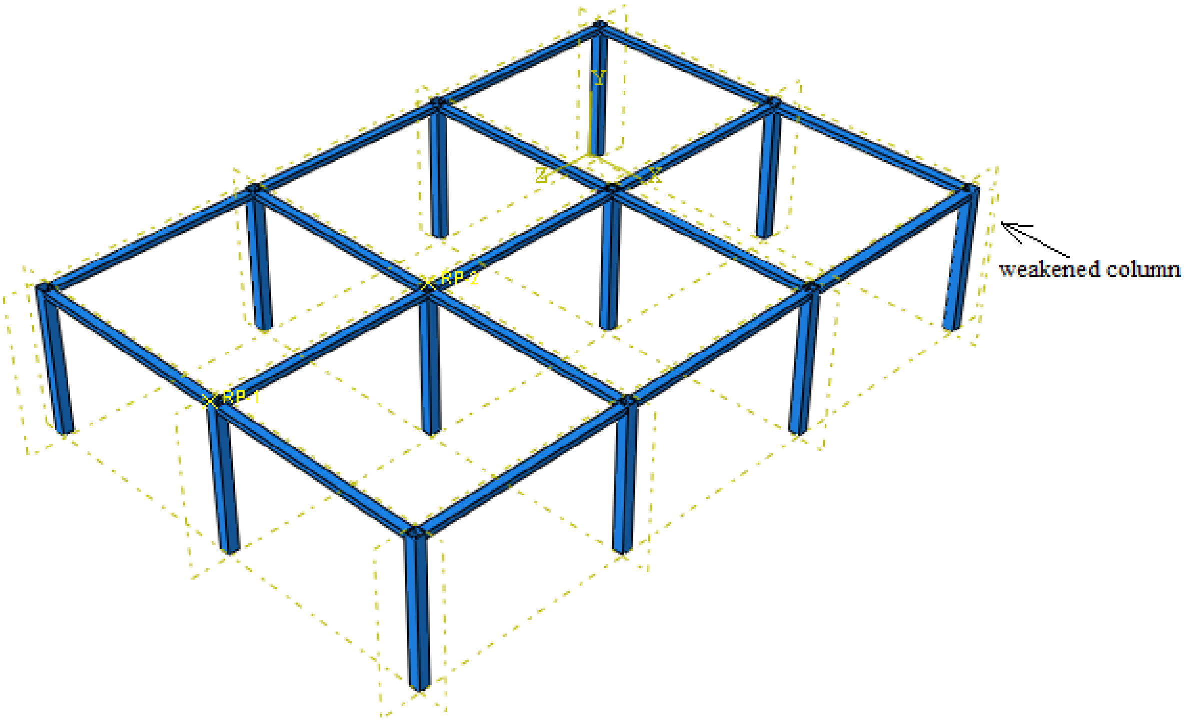


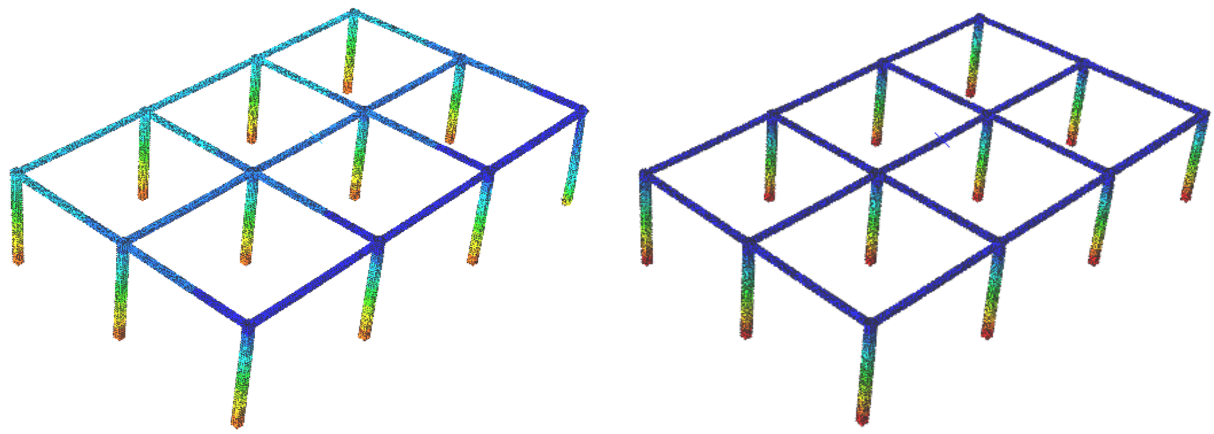
Figure2.1. Three-dimensional finite element model of the first building

3. NUMERICAL ANALYSIS

Results of the analysis indicated the different lateral displacements at the frames of the damaged structure and this behavior can be interpreted as follows:

In the first few steps of the analysis, the damaged frame (the frame in which the damaged column located) supports much deformation, so part of lateral loads supported by damaged frame is navigated toward nearby one because of stiffness reduction caused by weakening the corner-column. At the next steps, damaged frame and the nearby one support much deformation in comparison with the other ones, which can be due to torsion in the structure as the effect of shifting the stiffness center to another point far from the damaged column. Figs. 3.1.(a) and 3.1.(b) show the lateral displacement of both damaged and primary structure models respectively during push-over analysis.

To have better perception about the behavior of one-story buildings, another structure with five frames at each direction was modeled. Linear elements were used to model the columns and beams and plastic hinges were used to define the non-linear behavior of the elements. We defined deformation controlled frame hinge properties which represents only plastic behavior of the elements. The elastic behavior of the frame element was determined by the material properties of the frame element sections assigned to them. The behavior of the hinges was defined based on FEMA 356 provisions. Fig. 3.2. displays the behavior of the plastic hinges of the beam elements. In this analysis, the same combination of gravity loads was applied to the structure and then the push-over analysis was carried out. Fig. 3.3. shows the finite element model of the structure.



(a) **(b)**

Figure 3.1. Deformation of the (a): damaged and (b): primary structures subjected to lateral loading

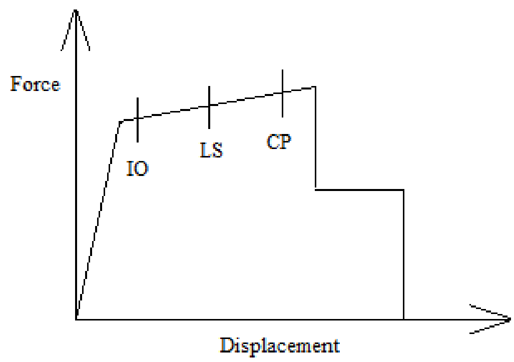


Figure 3.2. Behavior of the plastic hinges of the beams

Based on the results, at the damaged model first of all the rotation of plastic hinges formed at the frames near the damaged one, exceeded the rotation of the plastic hinges at the other frames which were far from the damaged frame. On the other hand at the primary structure hinges rotation of the corresponding frames has approximately the same values. Figs. 3.4. and 3.5. display the plastic hinges formation at the both damaged and primary models. As sum of the plastic hinges rotation represent the energy absorption of the structure, by summing the plastic hinges rotation of the elements at the various stages of the analysis, energy absorption versus the lateral displacement of the different frames of the structure is obtained. Comparing the graphs of the damaged model and the primary one, the amount of energy absorbed by different frames are discussed.

Based on the obtained graphs in the Fig. 3.6., the energy absorption of the damaged frame has much value in comparison with the primary model and far away from it sum of plastic hinges rotations of the frames decreases, and at the 4th and 5th frames, have larger values at primary structure. So it is noticeable that far away from the damaged frame, energy absorption of the frames decreases, so the frames near the damaged one have most participation in supporting lateral deformations. The energy absorption of various frames of damaged structure is shown in the fig. 3.7.

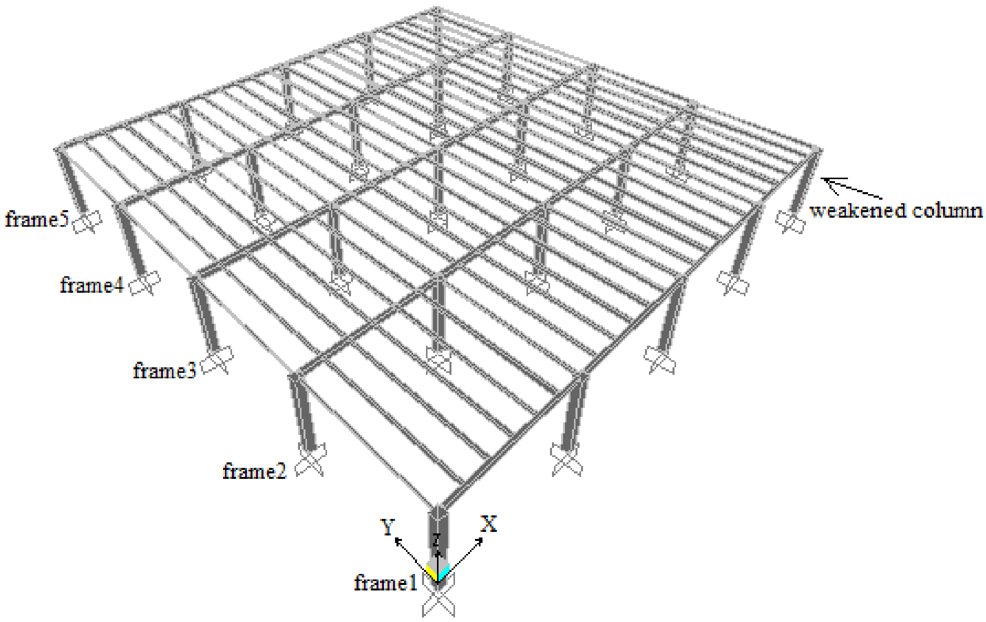
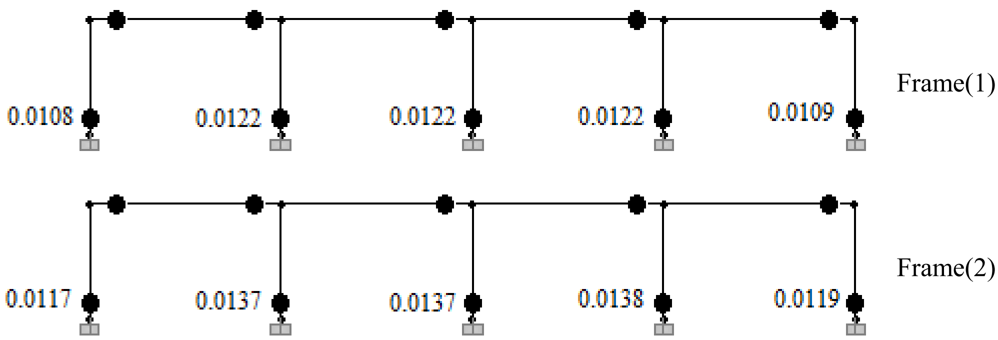


Figure 3.3. Three-dimensional finite element model of the second building



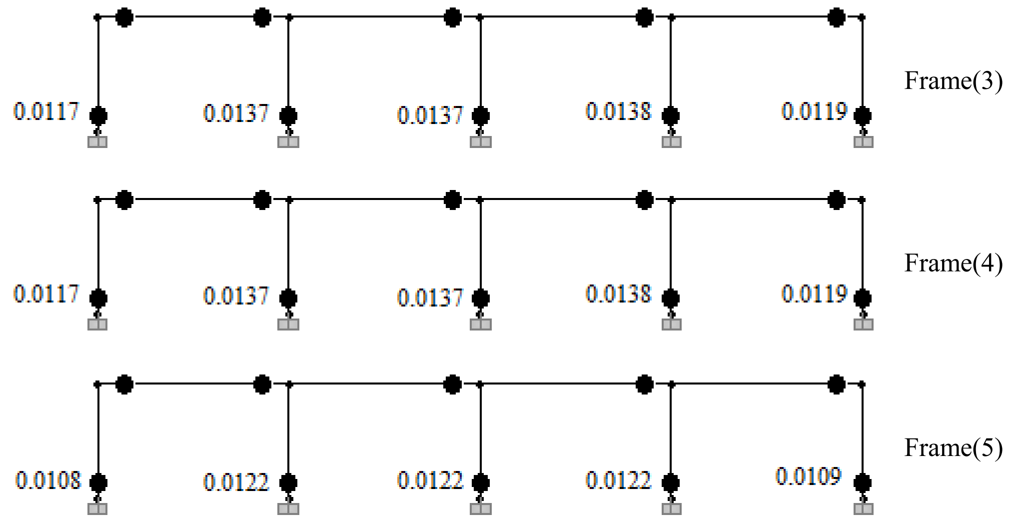


Figure 3.4. Rotation of the plastic hinges of the primary building at the displacement of 6.4cm

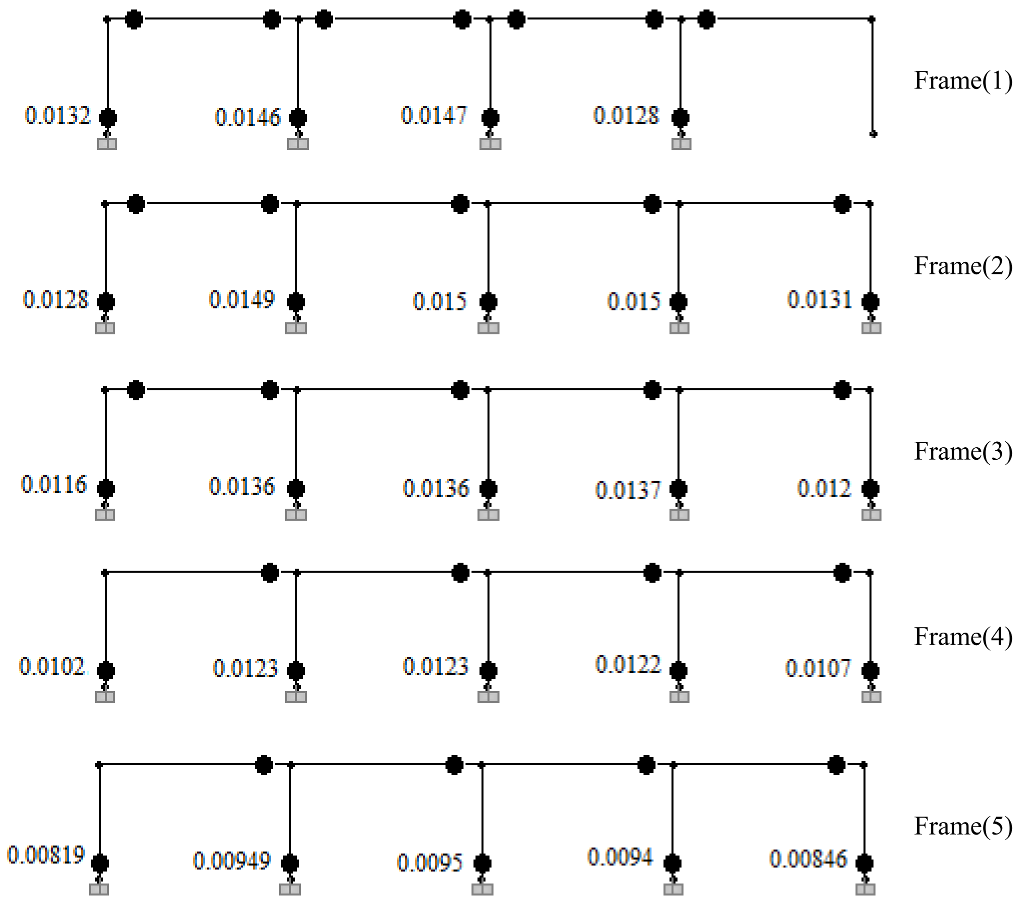


Figure 3.5. Rotation of the plastic hinges of the damaged building at the displacement of 6.4cm

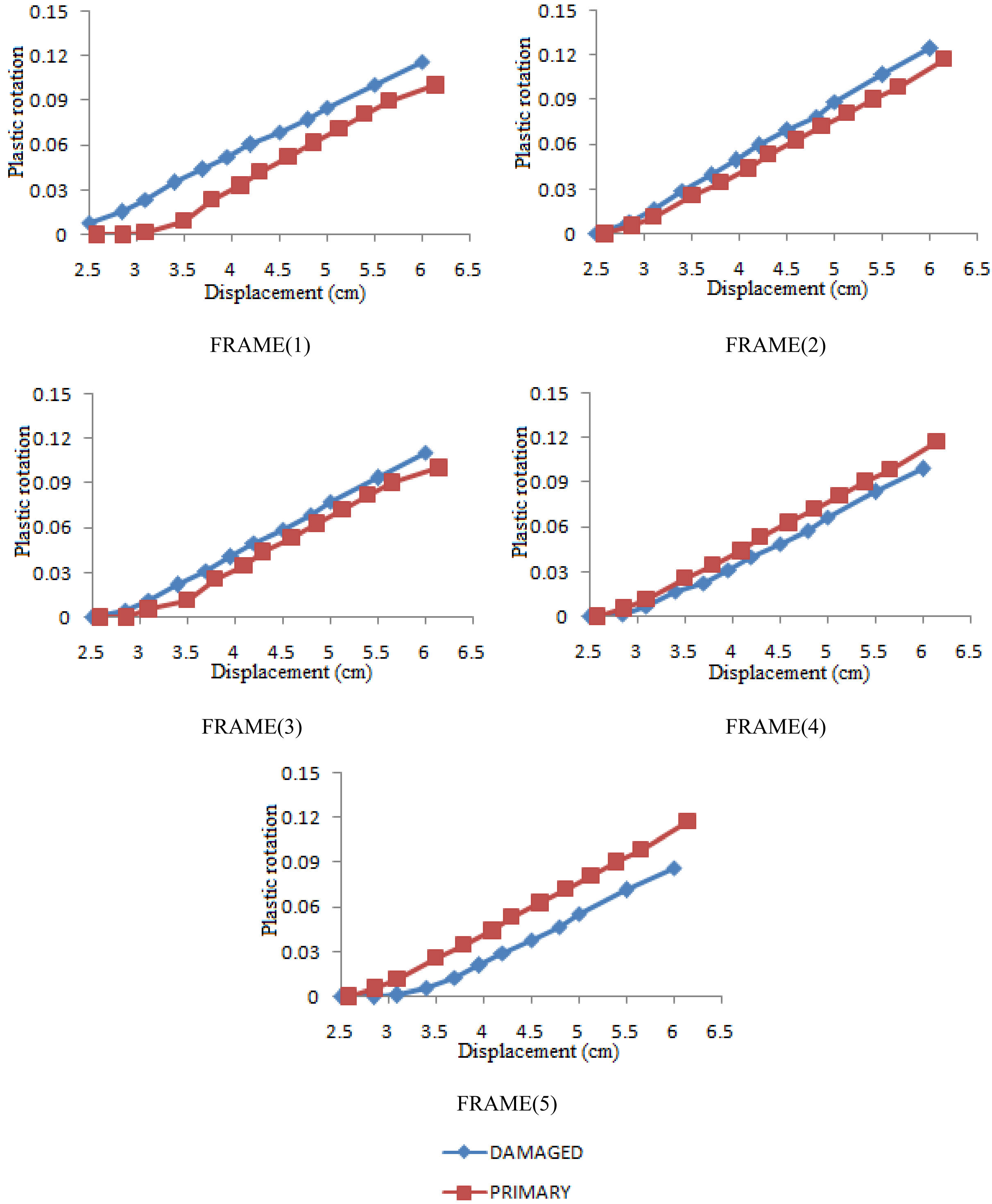


Figure 3.6. Energy absorption of the different frames of the damaged structure in comparison with the primary structure

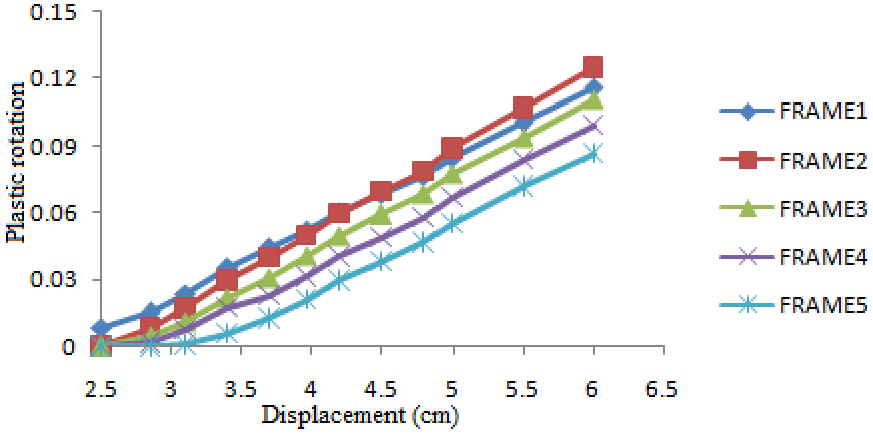


Figure 3.7. Different levels of energy absorption of the frames of the damaged structure

Fig. 3.8. indicates two push-over curves obtained from nonlinear analysis. It could be observed from comparing two acquired graphs from initially damaged and primary models, that damaged model has less secondary stiffness rather than the primary one.

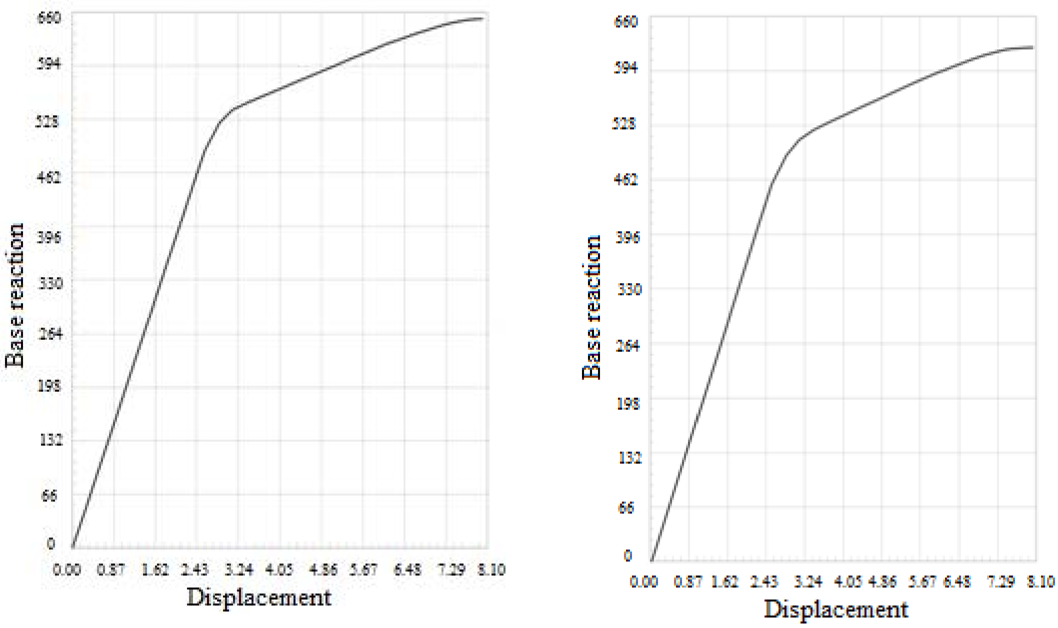


Figure 3.8. Push-over curve of (a): primary and (b): damaged one

4. CONCLUTION

In this paper progressive collapse potential of a special moment resisting steel building was investigated under earthquake action. A three-dimensional model of the structure with an initially damaged corner-column was analyzed by increasing lateral loads, through nonlinear static procedure. Based on the results, in the first few steps of the analysis, the damaged frame supports much deformation, so part of lateral loads supported by damaged frame is navigated toward nearby one because of stiffness reduction caused by weakening the corner-column. At the next steps, damaged frame and the nearby one support much deformation in comparison with the other ones, which can be due to torsion in the structure as the effect of shifting the stiffness center to another point far from the damaged column.

Another one-story building with five frames at both directions was modeled to have better perception about the behavior of one-story buildings. Linear elements were used to model the columns and beams and plastic hinges to define the non-linear behavior of the elements. As it is indicated, energy absorption of the damaged frame has much value in comparison with the primary model. By getting away from the damaged frame, sum of plastic hinges rotations of the frames decreases. As it seems, collapse pattern is in a way that the deformation of damaged frame increases and further away from it, deformation of the frames decreases. So during an earthquake progressive collapse gets started from damaged frames then passes through the others beside it.

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