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RESEARCH PAPER

The Effect of Out of Plane Perpendicular Beams on the Ductility Demand of Steel Moment Framed Structures during Progressive Collapse

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ABSTRACT: Unexpected loading, induced by severe earthquake or blast, could cause local damage to a structure. In this case, the structure has the potential of progressive collapse phenomenon. Hence, further consideration is required to mitigate the consequences of such loading. This study is aimed to evaluate the progressive collapse capacity of steel moment frames with different heights under column removal conditions. Seven and twelve story buildings modeled in different conditions in order to view effects of various parameters like the out of plane frames, column removal location, and the height of buildings in the results. One of the middle column and/or the corner columns is removed in order to evaluate the effect of column removal location in response of structures. The General Services Administration and the Department of Defense guidelines are considered for defining load combination for the analysis of the collapse. Nonlinear dynamic analysis is conducted in order to obtain the ductility demand of structures when the out of plane effect is considered. The structures have welded cover plate connections, designed for high-seismic zone area. For evaluating the response of the structures, for each connection at the point of column removal, maximum vertical displacement is measured. For Finite Element analysis, a sub-assemblage of structures is modeled using ABAQUS software and the ability of beams deformation and it's out of plane effect is measured.

Keywords: Column Removal, Ductility, Finite Element Analysis, Nonlinear Dynamic Analysis, Progressive Collapse.

1. Introduction

Progressive collapse happens due to the failure of main structural components or local failure of structural elements. This complex phenomenon always accompanied with large deformations and nonlinear behavior of the structural elements. Subsequently, the damaged structure via the catenary action finds a new load path to transfer the forces from failed region to the stable one. Catenary action is an ability of

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beams to resist the vertical displacement and can further help a damaged structure to reach stability. The Catenary action can somehow control the structural damage as well as force redistribution from damaged portion of the structure to the robust parts of structure in order to avoid progressive collapse. This capability also decreases the bending force moment, rotation and vertical deformation in the exterior beams of structures.

US General Services Administration guideline (GSA, 2003) and Department of Defense guideline (DoD, 2016) are recommending different analysis methods for the structures in the progressive collapse analysis. Those guidelines have their specific load combination for nonlinear analysis. In the methodology of the progressive collapse, the impact of the collapse (abnormal load) is usually presented by column removal. Both guidelines offer different locations for column removal in external frame in order to simulate the abnormal load conditions. After column removal, the structure is reanalyzed to find out whether the initial damage could further extend to the stable part of structure or not. Three types of analysis procedure exist in the GSA and the DoD. Those are linear elastic static (called LSP analysis), nonlinear static (called NSP analysis), and nonlinear dynamic (called NDP analysis).

Marjanishvili and Agnew (2006) investigated the advantages and disadvantages of the above analytical procedure in details. It was shown that the most effective analysis procedure is nonlinear dynamic analysis. More research was conducted in order to prove that the results from nonlinear analysis are more realistic than results obtained from linear analysis. More importantly, the impact factor that exists in the GSA guideline in consideration of dynamic effects, could be decreased (Kim and Kim, 2009).

Several researchers investigated the progressive collapse response of steel moment frame and their connections. Khandelwal and El-Tawil (2007) focused on the ductility of special moment frame structures subjected to progressive collapse. They evaluated the ability of the deformed structure through the catenary action. Their results revealed that the ductility and strength were discordantly influenced by an increase in beam depth and also by growth of the yield to ultimate strength ratio.

Lee et al. (2009) recommended simplified nonlinear analysis methods for the progressive collapse response in the welded steel moment frame structures. The methods presented the relevance between the column load and the chord rotation of the beams. Kim and An (2009) investigated the influence of the catenary action on the progressive collapse response of the steel moment frame structures. Their results indicate that the maximum displacement caused by snap column removal would be less when the catenary action is considered. They also concluded that the influence of the catenary action would be superior when the constraint of lateral movement of the structure is increased. This would be achieved by using additional bays or braces.

Kim et al. (2009) used alternative load path method recommended in the aforementioned guidelines in order to examine progressive collapse resistance of the steel moment frame structures. Their results indicate that the nonlinear dynamic analysis has larger structural response. In addition, it was shown that progressive collapse potential increased when the corner column was eliminated and this potential was decreased as the story number increased.

Khandelwal and El-Tawil (2011) used the pushdown analysis in order to study the soundness of building systems by launching collapse conditions of a damaged system. It was concluded that with regard to the column removal, the dynamic impact factors associated with it were lower than the generally used. Li et al. (2012) assessed the progressive failure of steel structures numerically and they developed a procedure for multi-scale analyses on seismic damage and progressive failure and seismic damage in steel buildings with including meso-scale damage development.

Sadek et al. (2013) presented a computational method of the response of steel moment frame simulating a column removal situation. Using the Finite Element method of analysis and material nonlinearity with shell and solid elements and conducting fracture mechanics, they obtained results that matched well with the experimental results.

Li et al. (2013) carried out two full scale experiments on connections with steel I-beam to tubular column. They subjected the connections to a column removal and they observed two modes of failure; namely a continuous flexural and an interrupted flexural failure. Song et al. (2014) performed experimental and numerical study to study the progressive collapse of an existing steel frame structure.

Yang and Tan (2013) conducted several experiments of the behavior of the bolted connection under column removal situation. Their study presented the modes of the failure of different connection types and also the capacity of the connections to deform in catenary action.

Mashahdali and Kheyroddin (2014) numerically studied the progressive collapse of the new hexagrid structural. Their research focused on the collapse of 28 and 48 story buildings models. They demonstrated that the new hexagrid system has sufficient force redistribution during the progressive collapse mechanism. Also in a progressive collapse scenario, Kheyroddin et al. (2014) proposed an easy method in order to compute of the dynamic load amplification factor due to sudden column loss. Their concept was based on the kinetic energy transfer criteria.

Guo et al. (2015) studied the response of the flush endplate moment connection subjected to column removal in the composite frame. They also developed Finite Element model in order to simulate the experimental test. They found that the progressive collapse of their system is susceptible to the properties of bolts. Tavakoli and Kiakojouri (2015) investigated numerically the threat-independent progressive collapse in the steel moment frame structures. The influence of the pertinent parameters such as number of stories and location of primary failure were discussed.

Dinu et al. (2016) investigated the response of steel frame structures under the removal of a central numerically as well as experimentally. They found that the beam ultimate rotation of the system was larger than the deformation limit given in the codes. Abdollahzadeh et al. (2016) investigated the impacts of progressive collapse on four story steel building with a special moment frame system. They assessed the rate of the collapse risk and the reliability of the structure. Li et al. (2017) conducted a small scale analysis with the static push down. They revealed that the catenary and the flexural actions are the principal and primary source of the collapse resisting.

Zhong et al. (2017) presented the anti-progressive collapse performance of steel frames with different connections subjected to internal column removal. Different modes of failure were observed for different connections. Three dimensional Finite Element models developed by Rahnavard et al. (2018) in order to study the progressive collapse of high rise steel frame structures with different types of lateral systems including regular and/or irregular plans. From the obtained results, they made certain recommendations in order to prevent progressive collapse in future designs.

Meng et al. (2018) studied the progressive collapse behavior of steel structures with different connections and dissimilar spans. Their research contained experimental as well as numerical study. They concluded that, when the stiffness of the beam and column are matching, then performance of the anti-progressive collapse mechanism can be improved. Lee et al. (2018), using the energy based approach, evaluated the progressive collapse behavior of the steel frame structures with several connection types. They demonstrated that the steel frame with RBS connections were least sentient to progressive collapse in comparison with the other type of connections. Sensitivity analysis was conducted by Rezaie et al. (2018) and Kheyroddin et al. (2019); and by following the GSA and DoD guidelines the key elements of the progressive collapse such as plan and height of the structure were identified. Yavari et al. (2019) investigated the effects of torsional irregularity together with seismic evaluation on the progressive collapse behavior of special steel moment frames. They demonstrated that buildings designed with greater torsional irregularities have better resistance to the progressive collapse.

As reported, most researchers only used two dimensional analyses in order to determine the progressive collapse behavior of steel structures and therefore the participation and influence of out of plane frames are neglected in the analysis. The main contribution of this research is to investigate the out of plane effects on the collapsing frames. It is included in the nonlinear dynamic analysis to evaluate the progressive collapse of special steel moment resisting frames more realistically. The results of nonlinear analysis obtained from the GSA and the DoD guidelines are compared in order to illustrate the role of out of plane frame stiffness in the progressive collapse potential. The influence of different parameters such as the position of column removal and the number of stories that could influence the behavior of steel moment structures during progressive collapse are examined. Furthermore, Finite Element analysis is carried out to evaluate the performance of flange plate connections. Numerical results are compared with both guidelines prediction of progressive collapse potential.

2. Analytical Procedure

2.1. Acceptance Criteria for Nonlinear Analysis

In regards to the nonlinear dynamic analysis, both GSA and DoD guidelines consider ductility and maximum plastic hinge for the criteria of progressive collapse potential. Tables 1 and 2 depict the complying criteria of steel beams and columns for progressive collapse in accordance with the GSA and the DoD guidelines respectively. It must be pointed out that here the definition of ductility is the ratio of maximum displacement at the column removal point to the yield displacement and the rotation criterion is calculated by proportioning the maximum vertical displacement to the beam's length.

2.2. Dynamic Analysis

The GSA recommends the following load combination for the dynamic analysis in every bay:

$$DL + 0.25LL \tag{1}$$

where *DL*: is the dead load and *LL*: is the live load.

In comparison with the GSA, the load combination for the DoD guideline is somehow different and it considers greater load factor for *DL* and *LL*. For the DoD, the wind load is also initiated in the load combination; as follows:

Table 1. Acceptance criteria for nonlinear analysis in the GSA guideline

Component	Ductility	Rotation (rad.)
Steel beams	20	0.21
Steel columns (Tension controls)	20	0.21
Steel columns (Compression controls)	1	-

Table 2. Acceptance criteria for nonlinear analysis in the DoD guideline

Component	Criteria
Steel beams	Collapse prevention
Steel columns	Life safety

$$(1.2DL + 0.5LL) + 0.2WL \qquad (2)$$

where WL: corresponds to wind load.

The following load criterion, as recommended by Kim and Kim (2009) is used for the dynamic analysis. First, column axial force (P), bending moment (M) and shear force (V) are calculated prior to the column removal. Second, as it is shown in Figure 1, the column is substituted by balanced and equal loads in order to elude from the unpleasant vertical displacement at the point where the column is removed.

For this purpose, the gravity load and the wind load as well as computed column loads are increased linearly and simultaneously until reaching their full intensity in the fifth second. Column loads remain unchanged for two seconds, while the gravity and wind loads will stay the same until the end of analysis. This step allows the system to reach a stable condition. In the final step, column forces are suddenly removed in the seventh seconds of the collapse time operation in order to resemble the progressive collapse event and perceive the dynamic influence of the column removal in structure.

2.3. Structural Characteristics

In this study, seven and twelve stories buildings are chosen to assess their progressive collapse potential. Figure 2a presents the structural plan for both structures, while Figure 2b and 2c show the elevation view of the analytical models. Story height and bay size are set as 3 m and 5 m; respectively. Each structure contains five bays in X direction and three bays in Y direction. These buildings are designed according to the AISC (2003) specification.

The location of the structures is assumed to be in Tehran, Iran, which is regarded as a highly seismic region. Special moment resisting frame system is used for the lateral system as well as gravity load resisting system. The structural elements such as columns and beams are designed with ST37 steel and the basic wind speed is considered to be 100 km/h. The dead loads of 56.2 kPa and 45 kPa are used for the floors and the top roof; respectively. The wall loading of 32 kPa and 15 kPa are used for the external and internal walls; respectively. The live loading of 20 kPa, 30 kPa and 15 Kpa are used for the floors, stairs and top roof; respectively.

The structural sections of the building models are presented in Tables 3 and 4.

2.4. Analytical Modeling

For dynamic analysis, the OpenSees program is used for two-dimensional modeling of the structure. The exterior frame, as illustrated in Figure 2a, is considered for the analysis purposes. For modeling the structural elements, the beam with hinges element of the program (called BeamWithHinge Element) is employed (Figure 3a). With this kind of element, the plastic hinge is defined at the starting and the finishing part of each column and beam elements. In beam with hinges elements, the nonlinear behavior of members must be specified and determined at plastic hinge. Therefore, the moment-rotation curvature of beam and column sections with considering the elastic and plastic effects are defined using steel 01 material, which is a bilinear material model and is provided in the OpenSees (Figure 3b).

Table 3. Main structural sections for the seven story building

Story number	Beam section	Column section
1	I330×160	Box 320×20
2	I330×160	Box 320×20
3	I330×160	Box 320×20
4	I330×160	Box 320×20
5	I330×120	Box 270×20
6	I330×120	Box 270×20
7	I320×100	Box 270×20

Table 4. Main structural sections for the twelve story building

Story number	Beam section	Column section
1	I380×240	Box 350×25
2	I380×240	Box 350×25
3	I380×240	Box 350×25
4	I380×240	Box 350×25
5	I330×160	Box 320×20
6	I330×160	Box 320×20
7	I330×160	Box 320×20
8	I330×160	Box 320×20
9	I330×120	Box 270×20
10	I330×120	Box 270×20
11	I330×120	Box 270×20
12	I330×120	Box 270×20

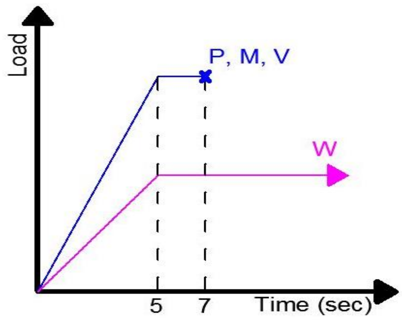
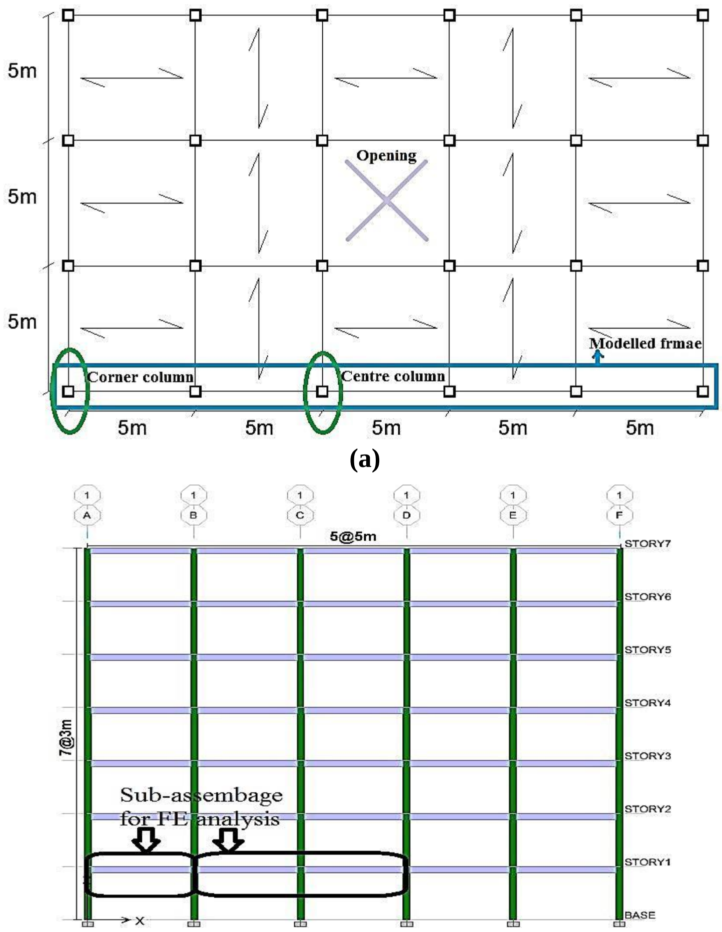
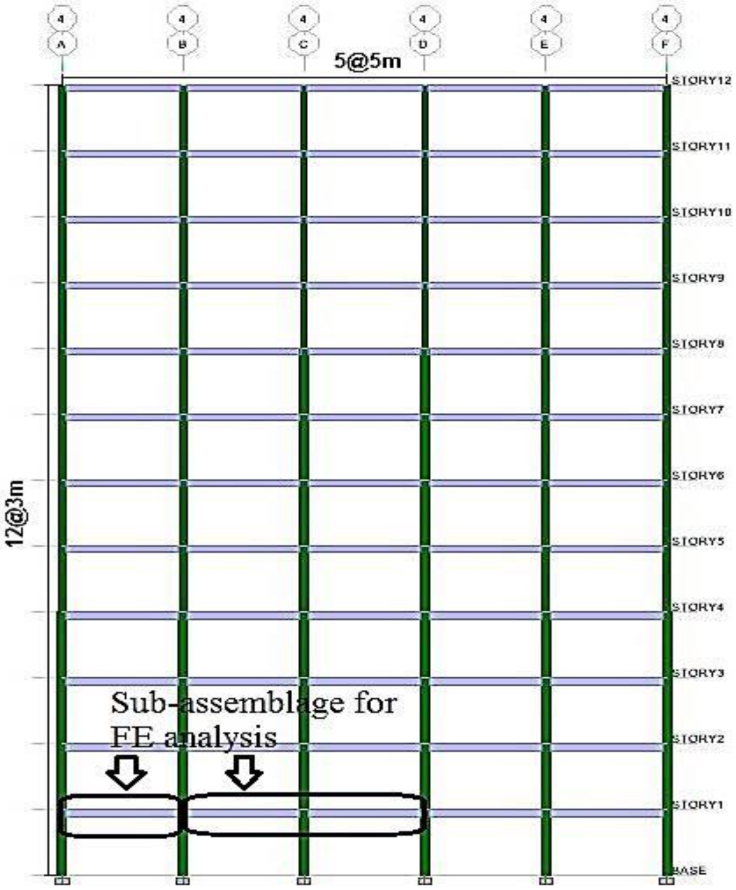


Fig. 1. The method of inserting loads

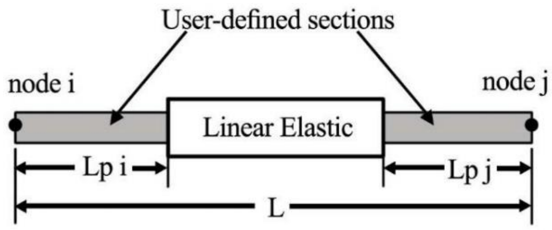


(b)

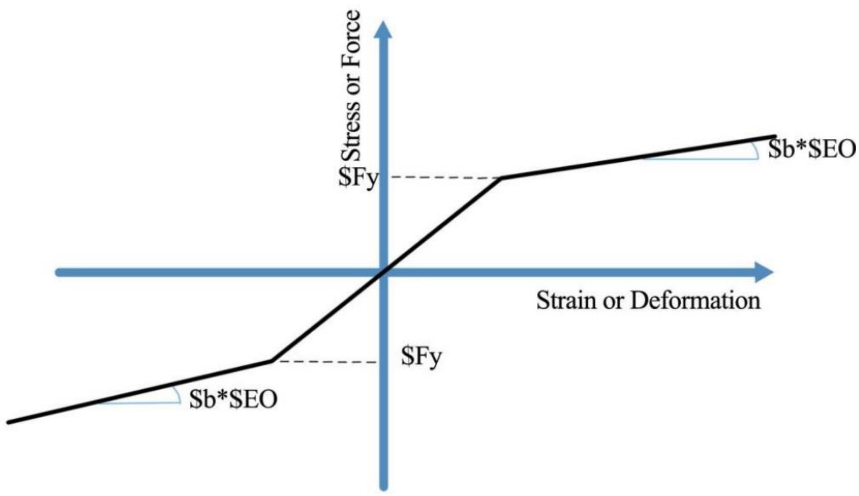


(c)

Fig. 2. Structural configuration of the models chosen: a) Structures plan; b) Seven story frame and; c) Twelve story frame



(a)



(b)

Fig. 3. Opensees element and material: a) Beam with hinge element and; b) Steel01 material model

The defined sections are assigned to the hinges with section aggregator command. As a definition of beam with hinges element, the middle section of elements will have an elastic behavior during analysis. Two different scenarios are considered for each frame. For the first case, the frame is perceived without considering out of plane stiffness, and in the other case, the frame is modeled with considering out of plane effect of vertical frames in 2D model to evaluate such effect. The influence of out of plane frames is considered by employing equivalent springs. The springs are regarded as the vertical stiffness of out of plane frames. They are attached to the intersection nodes of beams and columns in the model. The post yield stiffness of the structural elements is considered to be 2% of the initial stiffness and for large deformation analysis; the damping ratio is assumed to be of 5% of the critical damping.

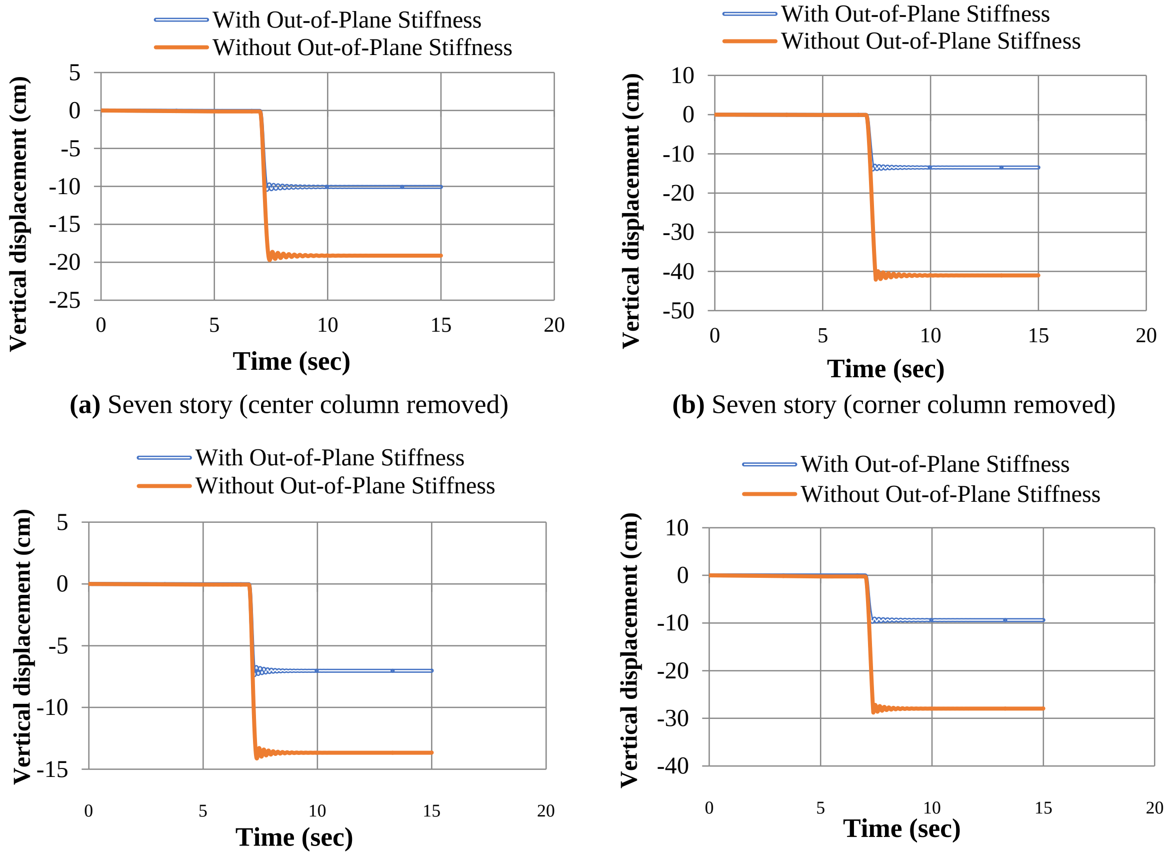
3. Nonlinear Dynamic Analyses of Structures

Nonlinear dynamic analyses are conducted subjected to different states by removing the corner column and the center column of the structure in order to resemble the progressive collapse. The vertical displacement is evaluated at the position of column removal; mainly because this parameter plays an important role to evaluate the collapse behavior as well as ductility demand of structures. Plastic hinge rotation of beams is also assessed and the results are used to provide useful comparison between guidelines criteria and analyzed structural response. Each structure is modelled with two different conditions; with and without considering stiffness of out of plane frames.

In Figures 4a and 4b, the vertical displacement due to the time history for the seven story structure with and without the inclusion of out of plane frame stiffness are depicted based on the GSA loading recommendation. In Figures 4c and 4d, the same graphs are also presented for the twelve story structure. Figure 5 shows also the same parameters based on the DoD instructions for both the seven story and the twelve story structure. The red graph shows the state of the model that the out of plane effect is not included and the blue graph shows the frame which considers the out of plane effect of frames.

It can be concluded from Figures 4 and 5 that the maximum vertical displacement is decreased when the number of floors rises. Thus, the results are largely dependent upon the number of stories. The vertical displacement of structures in case of corner column removal is greater than the models where their center column is removed. This conclusion is valid to both circumstances (with and without considering out of plane effect). The results show that considering out of plane effect has significant effects on the response of the structures; and structural demand is decreased if their effects are taken into the account.

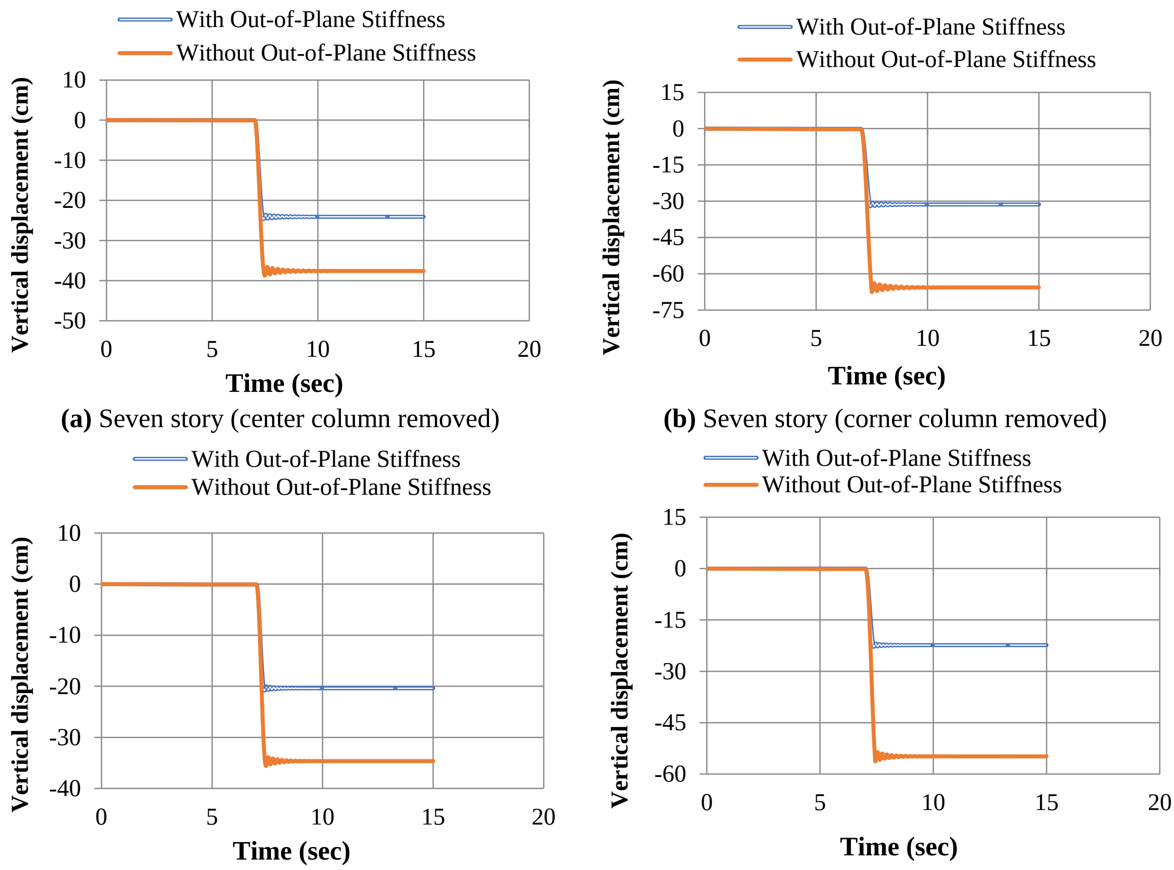
Beams rotation (in radian) for seven story models with and without considering out of plane effect due to the GSA (2003) guideline is presented in Figure 6. The numbers in purple color demonstrate the rotations in which out of plane effect excluded and the numbers in black color present the rotations in which out of plane effect included. As it is shown, the beams rotation at the bay where the center column is removed, does not exceeded the maximum specified rotation given by the GSA (2003) guideline (0.21 rad). For the corner column removal scenario, the plastic hinge rotation increased; although the acceptance criteria still satisfied. When the structure is analyzed subjected to the DoD load combination, the end beams rotation increased compared with the GSA (2003) results (Figure 7). The maximum beam rotation in Figure 7 is 15% whereas this number is 10% in Figure 6. Both beam rotations are less than the maximum acceptance criterion of the GSA which is 21%. In summary, maximum beam rotations of each model is presented in Table 5.



(a) Seven story (center column removed) **(b)** Seven story (corner column removed)

(c) Twelve story (center column removed) **(d)** Twelve story (corner column removed)

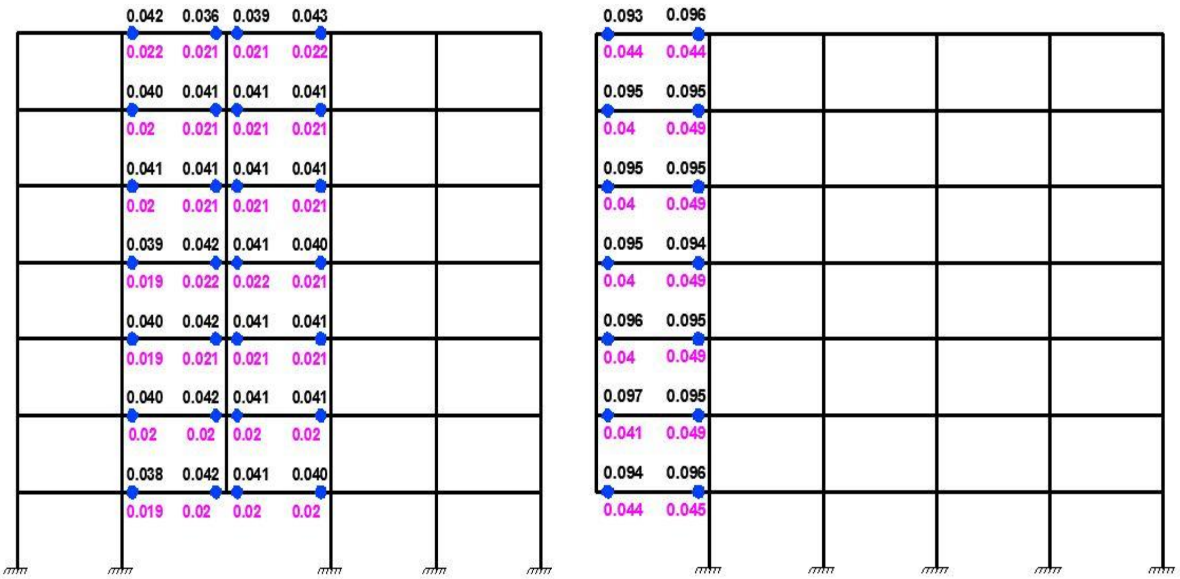
Fig. 4. Vertical displacement for seven and twelve story models with GSA (2003)



(a) Seven story (center column removed) **(b)** Seven story (corner column removed)

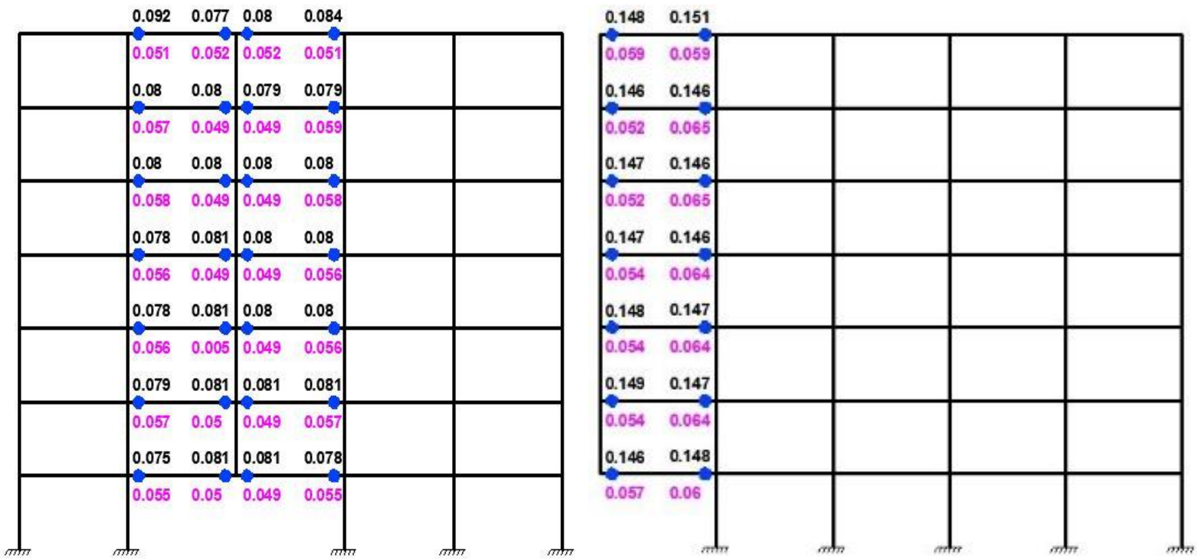
(c) Twelve story (center column removed) **(d)** Twelve story (corner column removed)

Fig. 5. Vertical displacement for seven and twelve story models due to the DoD loading



(a) Center column removal **(b)** Corner column removal

Fig. 6. Rotation of beams (in radian) for seven story building with and without considering out of plane effect with GSA loading



(a) Center column removal **(b)** Corner column removal

Fig. 7. Rotation of beams (in radian) for seven story building with and without considering out of plane effect with DoD loading

Also from the above results, the ductility demand can be determined. Ductility demand is limited to 20 for steel beams in accordance with the GSA guidelines. Without considering the out of plane effect, the ductility demand in the beams where it is located in the bay in which the column is removed is presented in Table 6. In addition, the ductility demands of structures with plane support included are displayed in Table 7.

Table 5. Maximum beam rotation

Story structure	Guidelines / Column removal place	GSA (2003) rotation (rad.) Corner	GSA (2003) rotation (rad.) Center	DoD rotation (rad.) Corner	DoD rotation (rad.) Center
7	Out of plane effect included	0.049	0.022	0.065	0.059
	Out of plane effect excluded	0.097	0.043	0.151	0.084
12	Out of plane effect included	0.019	0.014	0.043	0.042
	Out of plane effect excluded	0.055	0.028	0.11	0.07

Table 6. Ductility demand without considering out of plane effect

Story structure	Corner column removal Yield disp. (mm)	Max. disp. (mm)	Ductility	Center column removal Yield disp. (mm)	Max. disp. (mm)	Ductility
GSA						
7	72	410	5.69	55	190	3.45
12	60	280	4.67	51	135	2.65
DoD						
7	72	665	9.24	55	375	6.81
12	60	510	8.5	51	345	6.76

Table 7. Ductility demand with considering out of plane effect

Story structure	Corner column removal Yield disp. (mm)	Max. disp. (mm)	Ductility	Center column removal Yield disp. (mm)	Max. disp. (mm)	Ductility
GSA (2003)						
7	62	135	2.18	50	100	2
12	52	94	1.81	45	70	1.55
DoD						
7	62	315	5.08	50	240	4.8
12	52	225	4.33	45	205	4.55

As presented in the above tables, the ductility demand values are not over the specified value set by the GSA (2003). However, the ductility demand values for the DoD loading analyses illustrates a large discrepancy with respect to the GSA (2003) results. As expected, the DoD guideline load pattern requests larger ductility demand in comparison with the GSA (2003) guideline. It is concluded from Tables 6 and 7 that a ductility ratio obtained from corner column removal is larger than the state of center column removal.

Where the influence of out of plane frames considered in the models, the ductility demand decreases significantly in comparison with the situation in which the out of plane frames are ignored. It is also observed that the story number has the reverse relation with the ductility demand and with increasing the story number, the ductility demand is reduced. It could be mentioned that none of the ductility demand of structures exceeds the acceptance criterion of 20. It can be deduced that structures with moment resisting frame system have a reasonable reserve functional abilities when as an initiation of progressive collapse is subjected to column removal due to the investigated guidelines.

4. Finite Element Modeling of Beam-Column Connections

4.1. Characteristics of Models

The welded flange plate connection (Ghobadi et al., 2009), as shown in Figure 8 is selected in order to study the progressive collapse potential.

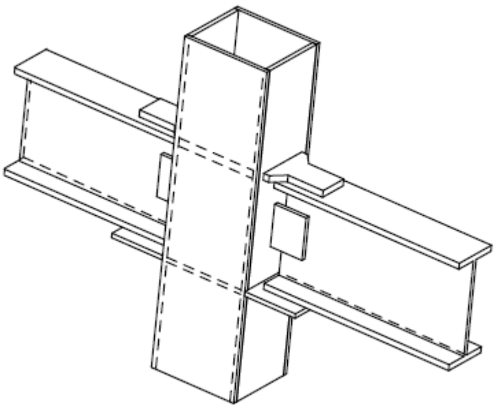


Fig. 8. Welded flange plate connection

For increasing the capacity of beam section in the beam-column connection area cover plates are added instead of reducing beam section. As a result, a plastic hinge will not be formed at the connection zone and brittle fracture of connection will be prevented. Beam and column sections of models are shown in Table 8. In Finite Element modeling, beams are modeled with their real length and columns are modeled with 1 m height. Therefore, the boundary condition can be applied at the ends of the column and then the catenary action can be started. Cover plate dimensions for both models are shown in Figure 9. For seven story and twelve story models, the top plate has thickness of 20 mm and the bottom plate thickness is 15 mm.

4.2. Material Properties

In the Finite Element model, in order to capture the realistic connection behavior, it is most proper to describe the material in the form of true stress and strain correlation. Since mostly the material data is available in the form of engineering stress and strain relationship, hence it is essential to change material data from engineering to true relationship. The following equations can be utilized in order to do the conversion.

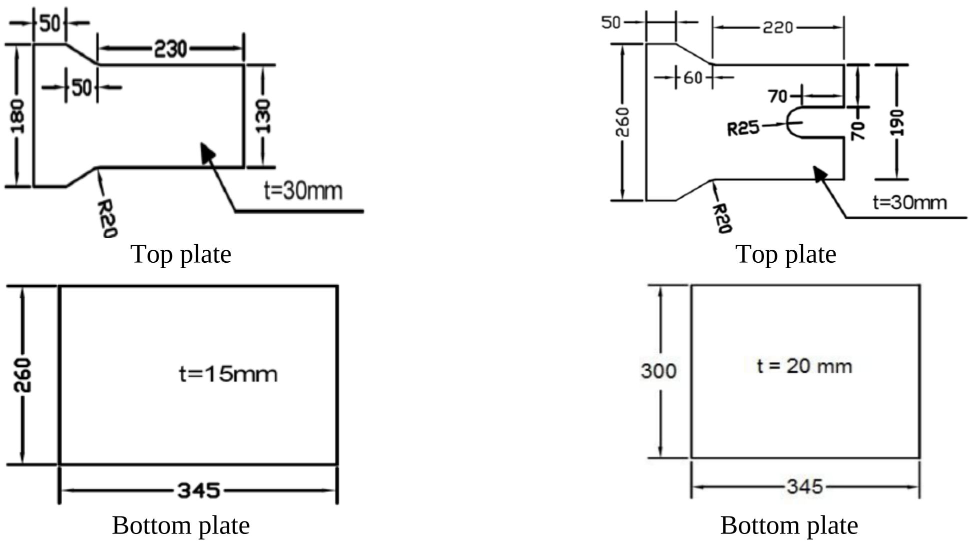
$$\sigma = \sigma_{nom}\,(1 + \varepsilon_{nom}) \quad (3)$$
$$\varepsilon = \ln(1 + \varepsilon_{nom}) \quad (4)$$

in which σ: is the true stress, ε: is the true strain, σ_{nom}: is the nominal engineering stress and ε_{nom}: is the nominal engineering strain.

In ductile materials such as mild steel, fracture initiation could start by plasticity. Ductile fracture will arise as a consequence of vast necking and plastic deformation in the necked vicinity. When the specified equivalent plastic strain is reached, the model presumes the damage begins. Since the triaxial test data for the mild steel materials used in this study is not available, hence the triaxial effect could not be documented. Thus, the fracture strain defined in the ductile criterion is independent of hydrostatic stress; however, previous researchers (Ghobadi et al., 2009; Mehr and Gobadi, 2017; Jazany and Ghobadi, 2018) have demonstrated that the ductility of steel rests considerably on the stress triaxiality.

Table 8. Geometrical properties

Story structure	Member	Section (mm)	Member length (m)
7	Beam Section	I 330×160	5
	Column Section	Box 320×20	1
12	Beam Section	I 380×240	5
	Column Section	Box 400×20	1



(a) Seven story building **(b)** Twelve story building

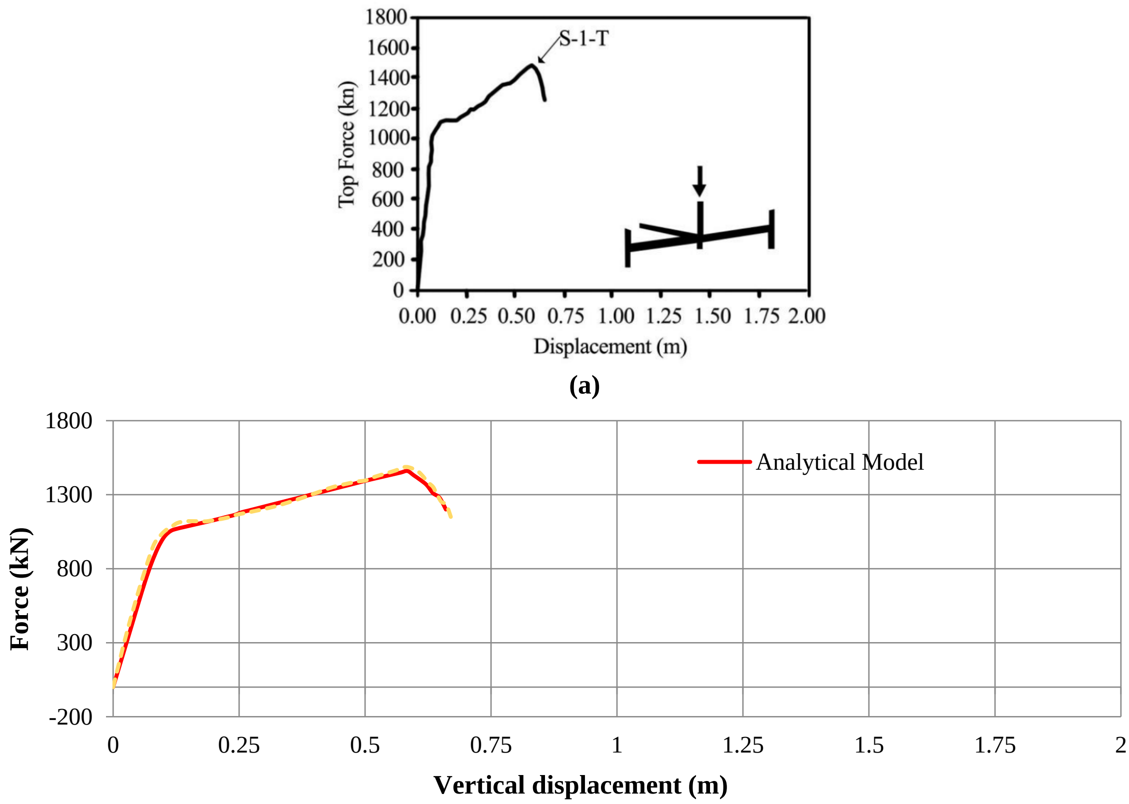
Fig. 9. Cover plate dimensions

Therefore, model parameters needed for ductile criterion that have been calibrated to a specific experiment cannot be considered as material properties and may not be able to correctly simulate ductile fracture in circumstances which vary mainly from the prototype experiment used for model calibration. It should be specified that this fracture criterion is also dependent on mesh size. The steel yield stress was taken as 244.8 Mpa and the ultimate stress was taken as 444 Mpa. The ultimate equivalent plastic strain value is taken as 0.16.

4.3. Finite Element Modeling

ABAQUS Finite Element software is used in order to perform numerical modeling of the connections. Two exterior bays of a perimeter moment frame and corner bay of exterior frame are considered for analysis (Figure 2). For all models considered, both material as well as geometric nonlinearity are considered. Material nonlinearity is being considered in the ABAQUS software using standard metal plasticity material model; which is based on an incremental plasticity formulation. Three dimensional solid brick elements with twenty nodes (three degrees of freedom for each node which includes the secondary effects) were used for the modeling of the connection area and three dimensional solid brick elements with eight nodes were used for the beam and column elements. For verifying an approach of the present study, comparison of results between Khandelwal and El-Tawil (2007) study and this study is undertaken, as shown in Figure 10.

Based on the results from Figure 10, it can be concluded that the results obtained from present study is quite close to the results of Khandelwal and El-Tawil (2007); and the differences is quite insignificant. Four different Finite Element models are generated in order to investigate four different conditions. They are middle beam-column connection with and without transverse beam and corner beam-column connection with and without transvers beam. Sub-assemblages are assumed located at the first story.



(b)

Fig. 10. First story Force-Displacement relation for non RBS connections without transverse beam: a) Khandelwal and El-Tawil (2007) and; b) Present analysis

The top and the end of the middle column are supported with the exception of the vertical displacement which set free and is prescribed. Since the transverse beam might impose the out of plane effect, then the transverse beam is modeled separately and its ends are fixed.

For evaluating the influence of out of plane effect, other configurations in which the transverse beam is not modeled are also taken into account. Prescribed displacements obtained from time history analyses, is applied to the bottom of the center and the corner column with the rate of 127 cm/sec. This loading rate represents the phenomenon that the column loses its load carrying capacity in an extreme loading condition. The final Finite Element converged mesh models are illustrated in Figure 11.

The initial increment for analysis is 0.001, the minimum increment is 1×10^{-10} and the maximum increment number in each step is 10000. For material properties St 37 steel is used. The elastic modulus and Poisson ratio for St 37 steel is $2.0\times10^5\ Mpa$ and 0.3 respectively.

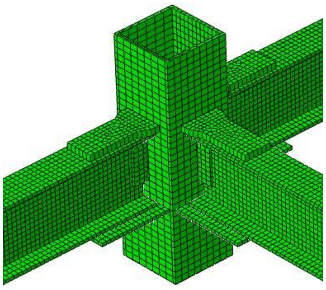
As for elements, solid element type C3D8R was employed. This element has the ability to present large deformations and both geometric and material nonlinearities.

For applying boundary condition, Discrete Rigid plate is placed at the ends of all columns. The maximum vertical displacement, which is obtained from time history analysis, is used as a displacement load pattern. Displacements are applied to the bottom of removed column.

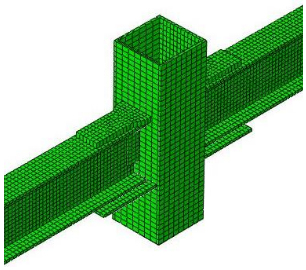
4.4. Results

Force-vertical displacement charts for four discussed conditions are shown in the Figures 12 and 13. The Finite Element analysis is conducted based on the maximum displacement demand of both guidelines. Stress distribution of seven story sub-assemblages are shown in Figure 14 for middle and corner column models.

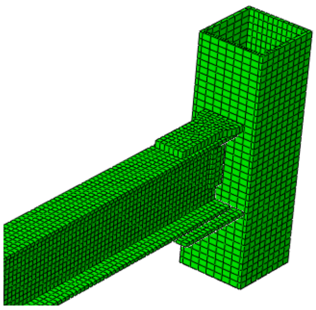
From the Finite Element analyses, in summary, Table 9 presents the results of progressive collapse potential of different story structures based on the GSA and DoD guidelines. As shown, only seven story sub-assemblage with corner column and excluding the transverse beam effect is unable to redistribute the load under required vertical displacement; while other cases managed to redistribute the loads. It can be concluded the other structures managed to have the acceptable progressive collapse performance.



(a) Middle column with transverse beam



(b) Middle column without transverse beam



(c)

Fig. 11. Finite Element model of seven story building in two different conditions

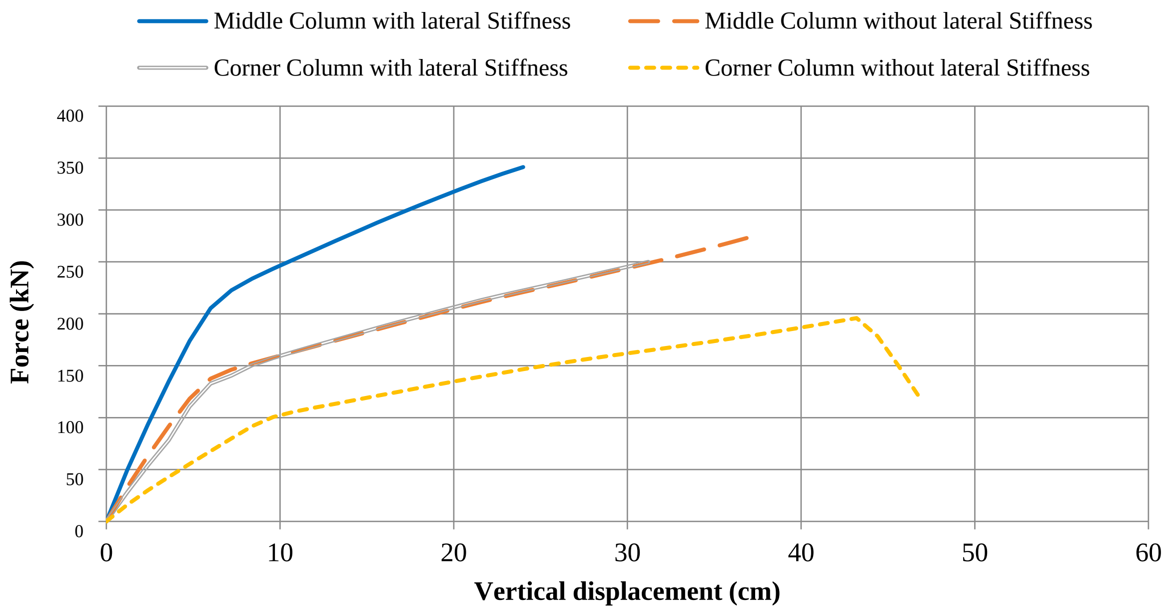


Fig. 12. Seven story building beam-column connection models

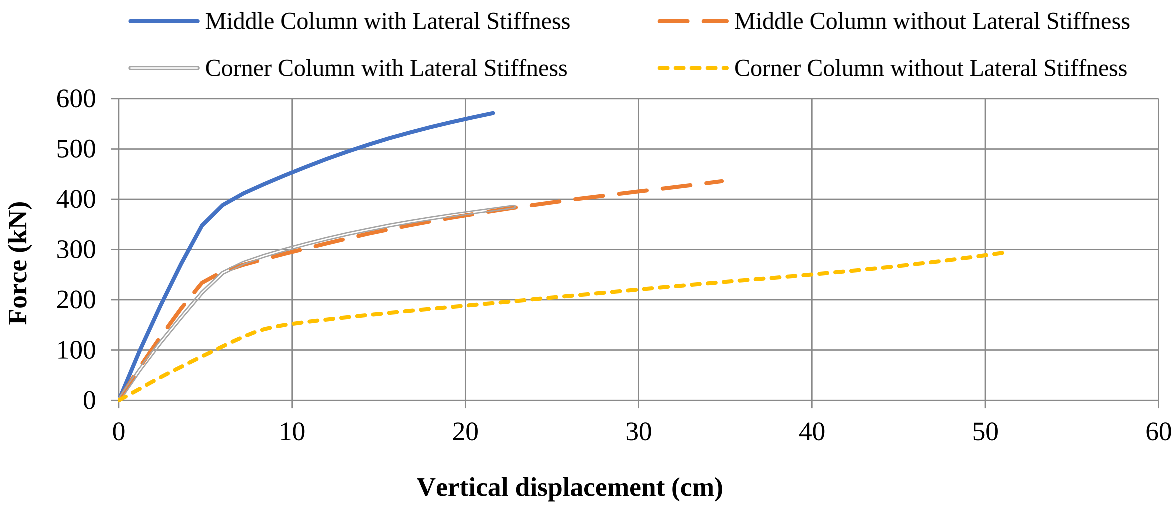
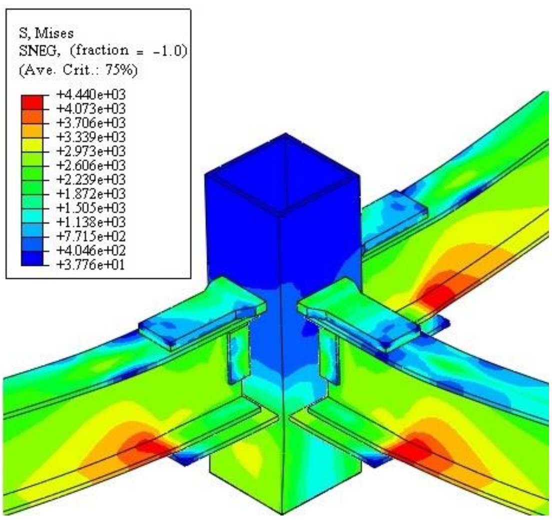
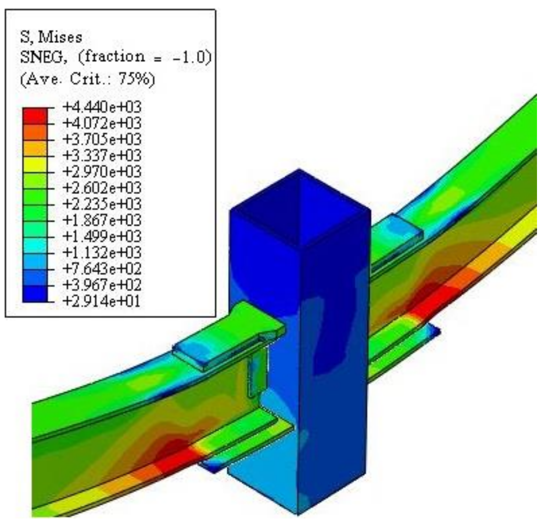


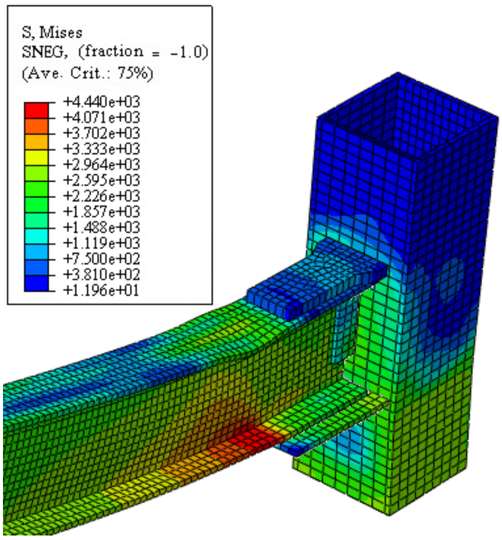
Fig. 13. Twelve story beam-column connection models



(a) Stress distribution for middle column with transverse beam at 61 cm vertical displacement



(b) Stress distribution for middle column without transverse beam 51 cm vertical displacement



(c) Stress distribution for corner column without transverse beam at 43 cm vertical displacement

Fig. 14. Stress contour for seven story beam-column connections

Table 9. Progressive collapse potential of structures

Story structure	Guideline / Column removal place	GSA (2003) Corner	GSA (2003) Center	DoD Corner	DoD Center
7	With transverse beam	No	No	No	No
	Without transverse beam	No	No	Yes	No
12	With transverse beam	No	No	No	No
	Without transverse beam	No	No	No	No

5. Conclusions

This study focused on the progressive collapse behavior in the special moment resisting framed structures. The influence of the pertinent parameters such as the story number, position of column removal, impact of out of plane frames on maximum vertical displacement and ductility demand of beams are investigated. The analyses carried out using the General Services Administration as well as the Department of Defense guidelines; and the differences between the results are assessed.

Results reveal that the special steel moment resisting frames designed for lateral loads have a reasonable performance during progressive collapse simulation. It is concluded that when the structure suffers from losing its corner column, it becomes more vulnerable in comparison with the structure losing one of its central columns. It is observed that there is a relation between the number of stories and the potential of progressive collapse. As the story number increased, the ductility demand as well as beams rotation reduce indicating the decrease of potential of progressive collapse. The structures are analyzed due to the GSA (20030 and the DoD guidelines and their progressive collapse potential are illustrated.

Moreover, the effect of out of plane frame is evaluated independently. As explained, out of plane have an effective influence over the results and decreased the potential of progressive collapse. In some cases, due to this affect, the ductility demand decreased by more than one-half. The importance of considering out of plane and its influence on the maximum vertical displacement is demonstrated.

This advantage could considerably reduce the ductility demand of structures. Based on the Finite Element analysis, the structures showed that they have more resistance to progressive collapse than it was expected regarding both guidelines. Only one sub-assemblage showed not having the ability of the demand required and it loses its capacity before reaching the required displacement. In some cases, the results are dissimilar from guidelines expectations and therefore further investigation is required.

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