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# Vulnerability Assessment of Progressive Collapse of Steel Moment Resistant Frames

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## ABSTRACT

A structure collapse progressively when its vital vertical member fails. This cause the failure of adjoining members that finally leads to partial or total collapse of a structure. Many regulations such as the Unified Facilities Criteria (UFC) and General Services Administration (GSA) addressed the structural progressive collapse phenomenon. In the current study, the progressive collapse potential and vulnerability of steel moment resistant frame building structures, which are seismically designed based on Iranian National Building Codes (INBC) and Iranian code of practice for seismic resistant design of buildings (Standard No. 2800), was investigated. It was achieved by performing nonlinear dynamic analysis of steel moment frames, designed for very high seismic zones. The exterior frames of such structures were investigated for collapse progression while losing one primary column on height of the structures. The analysis results revealed that the columns of designed structures had not adequate strength to survive one corner column loss of the structures before the occurrence of next failure and so the structures have high potential for progressive collapse. Finally, the building structures do not meet the UFC acceptance criteria. Hence, some modifications have been proposed on the INBC codes to meet the UFC limits.

**Key words:** Steel structure, alternate path method, progressive collapse

## INTRODUCTION

Progressive collapse is one of the disastrous phenomena in which failure of one vital structural member leads to failure of other analogous members, as a result, the entire structure may collapse in some situations (ASCE, 2005). Plane impact, car collision and gas explosions usually are a few examples of the probable risks in addition to unusual loads which could produce this event (NIST, 2007). Structures are not generally designed for unusual events which usually can result in element removal and ultimately leads to devastating failure. The majority of building codes have got only general strategies for mitigating the effect of progressive collapse in structures that were overloaded further than their particular design loads. The American Society of Civil Engineering (ASCE) 7-05 is the solely well-known regular society which deals with the problem of progressive collapse in certain detail. The remarkable design guidelines for structures to resist against progressive collapse are presented in US Government documents, e.g., General Service Administration (GSA, 2003) and Unified Facility Criteria (UFC, 2009).

The particular methodology has been provided in the GSA guidelines in order to reduce the catastrophic effects of progressive collapse in structures according to the Alternate Path Method (APM). It describes cases in which one of the building columns is removed and the damaged structure is examined to check the system responses. The UFC methodology, in contrast, is a performance-based design strategy and is partially in line with the GSA procedures. Progressive collapse analysis of steel frames has been recently the main topic of a number of scientific studies.

Liu (2010) analyzed catenary action and demonstrated that it can reduce the bending moment considerably through axial restraining the beam. Also, two schemes were proposed for retrofitting the fin plate beam-to-column connection of tall steel framed structures subjected to a terrorist blast. Park and Kim (2010) investigated the progressive collapse potential of steel structures with three various seismic connections: WUFB, RBS and WCPF. The analysis results revealed that the RBS connections demonstrated the highest load resisting capacity against collapse owing to their high ductile behavior and that the loss of an outer column is more vulnerable for progressive collapse than the loss of an interior column.

Kim and Kim (2009) studied the progressive collapse-resisting capacity of steel moment frames by employing alternate path methods proposed in the GSA and UFC recommendations and noticed the nonlinear dynamic analysis triggered more substantial structural responses. In addition, they noticed the possibility of progressive collapse had been greatest when a corner column was abruptly eliminated. In addition, that was figured the progressive collapse potential diminished since the quantity of stories elevated. Khandelwal *et al*. (2009) figured an eccentrically braced frame is much less prone to progressive collapse compared to a special concentrically braced frame. Kim *et al*. (2009) represented that the dynamic amplification may be larger than two that is suggested by the GSA and UFC recommendations.

Fu (2009) stated that under the identical normal circumstances, a column elimination at a higher-level will certainly generate greater vertical displacement compared to a column elimination at ground level. Kim *et al*. (2011) deduced that among the variety of braced frame structures, the inverted-V type braced frame demonstrates remarkable ductile behavior through progressive collapse. England *et al*. (2008) researched the significance of evaluating the vulnerability of a structure to unexpected events and reviewed the character of unpredicted events. Besides, a hypothesis of structural vulnerability which in turn investigates the form of structure to look for the most vulnerable series of failure events had been defined.

Gerasimidis (2013) assessed the progressive collapse vulnerability of steel frames to corner column loss. He proposed an analytical method to indicate the collapse mechanism of a steel frame for the case of a corner column loss through the development of critical ductility curves. Tavakoli and Alashti (2013) studied the potential of multi-story moment resisting steel frame buildings with damaged columns in different locations under seismic loading. The analysis results showed that in the case of middle column removal, the structure is more robust than in a corner column removal situation and also determined that, as the number of stories and bays increased, the capacity of the structure which resist progressive collapse under lateral loading is also increased, because additional elements participated in resisting progressive collapse. Hosseini *et al*. (2014) investigated the vulnerability of steel moment resistant frame structures and concluded that removing corner column in ground floor cause great value of axial forces in adjacent column which leads to failure of adjacent bay. Though progressive collapse of various kinds of structural frames is mainly deemed as vertical movement, analysts have proposed that seismic design of buildings causes mitigating this kind of event (Khandelwal *et al*., 2009).

Nevertheless, studying existing scientific resources, it can be mentioned that the quantitative impact regarding such application had not been completely concentrated. For this purpose, an official 10-story steel moment resistant frame with specific administrative application designed according to Iranian National Building Codes and with a focus on requirements of Iranian Seismic Code No. 2800 was designed. Their exterior frames were analyzed performing nonlinear dynamic analysis for collapse progression to determining their safety against progressive collapse by comparing the analysis results to criteria mentioned in UFC code. Finally, some modifications are proposed for such structures to resist progressive collapse.

## DESCRIPTION OF DESIGNED STEEL BUILDINGS

**Structural details:** In this study, an official 10-story steel moment resistant frame building which is designed based on Iranian code of practice for seismic resistant design of buildings (Standard No. 2800) (BHRC, 2004) and Iranian National Building Codes part 6 and 10 (INBCSD, 2006, 2008) considered to investigate the effect of column lose in structure. As shown in Fig. 1, the height of all stories is 3.2 m. Its first story is an open space for public the level of protection is medium. The structure has no irregularities in its elevation or plan and is square in plan. All floors are 5×5 panels, each of which 5×5 m and have 625 $m^2$ in sum with 25 $m^2$ areas at corner panels. This building has intermediate steel moment-resistant frame in both sides.

The column sections used for the stories 1-4, 5-7 and 8-10 are Box 340×340×30, Box 250×250×20 and Box 200×200×25, respectively (Table 1). The beam sections applied for the stories 1-4, 5-7 and 8-9 and roof story are IPE400, IPE360, IPE330 and IPE300, respectively (Table 1).

**Material properties and loading:** The materials used for the structure are based on Iranian National Building Codes part 10 (INBCSD, 2008). The compressive strength ($f_c$), used for concrete in all floors, is 25 MPa and showed in Table 2. The design yield strength and ultimate strength values are 235 and 400 MPa for beams and columns. The elasticity moduli of steel and concrete

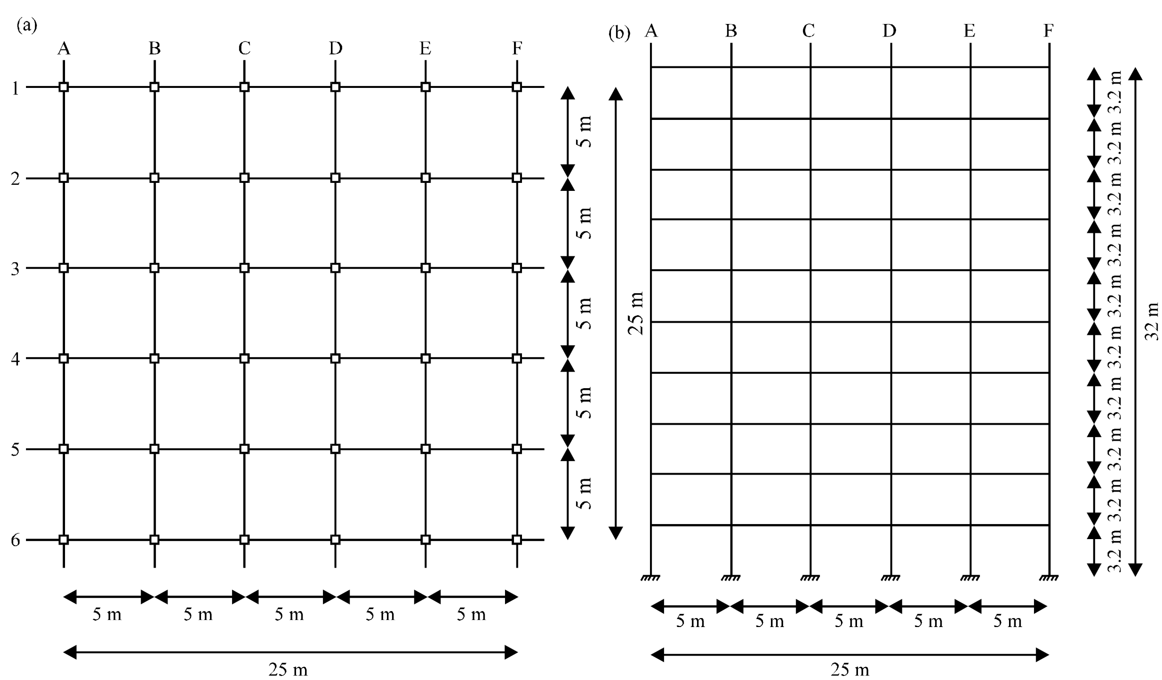


Fig. 1(a-b): Elevation and plane of the designed structure, (a) Plane and (b) Elevation view

Table 1: Dimensions of structural members (mm)

| Beam members | | Column members | |
|---|---|---|---|
| Story | 5 m | Story | 3.2 m |
| 1- 4 | IPE400 | 1-4 | Box 340×340×30 |
| 5-7 | IPE360 | 5-7 | Box 250×250×20 |
| 8-9 | IPE330 | 8-10 | Box 200×200×25 |
| 10 | IPE300 | --- | --- |

Table 2: Material properties

| Material | Poisson's ratio | Young's modulus (MPa) | Compressive strength (MPa) | Yield stress (MPa) | Ultimate strength (MPa) |
|---|---|---|---|---|---|
| Steel | 0.30 | 202,000 | --- | 235 | 400 |
| Concrete | 0.15 | 20,000 | 25 | --- | --- |

are 202000 and 20000 MPa, respectively. Based on INBC part 6 (INBCSD, 2006), a Live Load (LL) of 1.47 kN $m^{-2}$ is applied to the roof and 2.45 kN $m^{-2}$ to other floors. In addition to the self-weight of structural elements, the Dead Loads (DL) of 4.41 and 5.4 kN $m^{-2}$ are considered for the roof and all other floors, respectively. Furthermore, the perimeter wall weights of 3.04 and 5.88 kN $m^{-1}$ are applied to the roof and other floors, respectively.

## ANALYZING AND MODELING THE PROGRESSIVE COLLAPSE

Three procedures exist in the UFC regulations for analyzing the structures subjected to progressive collapse: (1) Linear Static Procedure (LSP), the simplest one, is common practice utilized in the structural analysis and design. The material is assumed linear elastic no geometric nonlinearity exists and the structure experiences small deformations. (2) Nonlinear Static Procedure (NSP), in which not only geometric but also material nonlinearities are considered. (3) Nonlinear Dynamic Procedure (NDP) which because of its accuracy and realistic results comparing to the other procedures has been more used by the researchers. In addition, it considers both geometric and material nonlinear behaviors. The structure is prone to dynamic loads and can be more susceptible to experience large deformation without any restriction for irregularities in elevation or plan. Nonlinear dynamic analysis seems to be more suitable for analyzing progressive collapse phenomenon since it possess the nonlinearity and dynamic phases. Hence, the mentioned procedure has been utilized for pre-analyzing the model structures.

**Load combination:** In order to study the structures after column removal scenarios, the following gravity load combination ($G_{ND}$) has been applied to the entire structure based on UFC guidelines:

$$G_{ND} = 1.2DL+0.5LL \tag{1}$$

where, LL and DL are live load and dead load, respectively.

In addition to gravity loads, the lateral loads ($L_{LAT}$) should be applied to the structure sides as follows:

$$L_{LAT} = 0.002\sum P \tag{2}$$

where, $\Sigma P$ is sum of the gravity loads applied on each floor and substituted for wind load.

It should be noted that four separate analyses should be conducted on the structure. In the other words, lateral loads should be applied to each sides of the building, south to north, north to south, east to west and west to east. Helmy *et al.* (2012) investigated the influences of lateral load directions. Apart from these effects, they considered the same direction that has been used in this study for lateral load. This direction leads to the largest deflection in the structure.

**Applied loads and loading procedure:** The internal forces acting on the corner column is computed in order to perform nonlinear dynamic analysis. The corner column is removed and its equivalent forces are applied to the connection of the removed column. The equivalent internal forces are M, P and V, defined as bending moment, axial force and shear force. The variables $G_{ND}$ and $L_{LAT}$ denote gravity loads and lateral loads. According to Fig. 2, the equivalent internal forces started from 0 sec and increased linearly upto 5 sec for simulating the dynamic effect of column removal phenomenon. The internal forces are kept constant until the 7th second after they met their full capacity and met their full capacities and the system reaches its stable condition. Then, arriving to 7th second, the internal forces are removed suddenly (Kim and Kim, 2009).

**Column removal scenarios:** To investigate the effects of losing columns on the structural members and their behavior, corner column of the model have been chose to be removed and analyzed in four cases based on the UFC regulations (Table 3).

The locations of removed corner columns in the studied ten-story building are as follow:

- First story, above the ground
- Fifth story, at mid-height of the building where the column sections is changed
- Eighth floor, where the column sections is changed
- Ninth story, the story below the roof

Table 3: APM analysis cases (scenarios)

| APM case/scenario | Element removed | Frame |
|---|---|---|
| 1 | Col-A-1 | 1 (5.0 m-Bay frame) |
| 2 | Col-A-1 | 1 (5.0 m-Bay frame) |
| 3 | Col-A-1 | 1 (5.0 m-Bay frame) |
| 4 | Col-A-1 | 1 (5.0 m-Bay frame) |

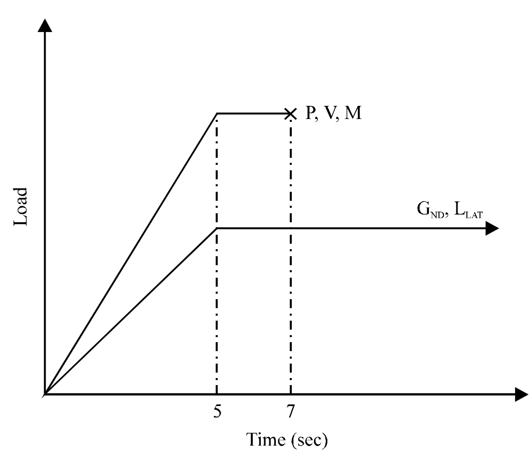


Fig. 2: Applying dynamic loads for nonlinear dynamic procedure (Yousefi, 2014)

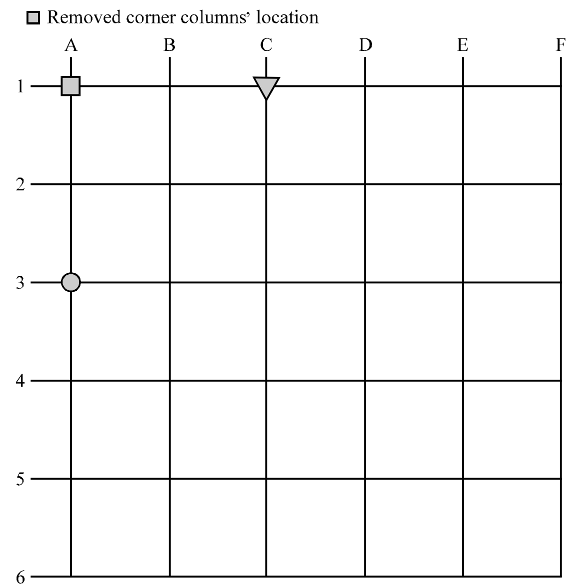


Fig. 3: Notionally removed corner column for AP analysis

Table 4: Ultimate capacity of the column

| Story | Column members | Ultimate capacity (kN) |
|---|---|---|
| 1-4 | Box 340×340×30 | 8887.8 |
| 5-7 | Box 250×250×20 | 4333.4 |
| 8-10 | Box 200×200×25 | 4068.9 |

According to Table 4, a corner column and an internal column in each model was removed and the results were analyzed.

Figure 3 shows the locations of removed columns in the elevation and plan. A-1 (intersection of axes A and 1) columns as representative of corner columns are notionally removed in the height of the structure one at a time in stories in distinct analyses.

**Analytical modeling:** OpenSees (Mazzoni *et al.*, 2007) finite element program code was used for modeling and numerical analysis of the exterior frames subjected to column removal scenarios. It is a powerful nonlinear software for simulating the applications in the earthquakes as well as structural engineering using finite element methods. A series of nonlinear dynamic analyses have been performed for each analysis in order to analyze the external steel frames. To model structural members, beams and columns have been presented by nonlinear beam-column element in combination with fiber cross sections having fatigue material and Steel02 hysteretic material model with isotropic strain hardening and bilinear kinematic stress-strain curve. The post-yield stiffness of members is considered to be 2% of the initial stiffness. The values of controlling the transition from elastic to plastic branches $R_0$, $R_1$ and $R_2$ are assumed as 10, 0.925 and 0.15, respectively. The steel behavior of Steel02 material is shown in Fig. 4. Due to the possibility of large deformations, corotational coordinate transformation has been used to perform a precise geometric transformation of members' stiffness and resisting forces and the common damping ratio which usually applies to

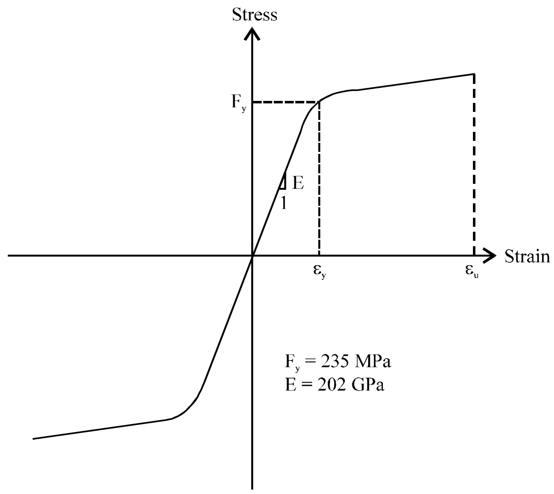


Fig. 4: Constructive material of the models

the structures is considered as 5% of the critical damping. The progressive collapse analyses are conducted by removing a column in ground level of the frame buildings according to UFC (2009) guidelines.

## RESULTS AND DISCUSSION

To assess whether or not the building structure can bridge over the notionally removed corner column under extra forces imposed to the adjacent columns, Four APM scenarios have been analyzed. In Fig. 5-9, time history responses columns' axial forces for APM cases are shown. The analyses interpret according to described scenarios in Table 4. The simulation results revealed that the system was not able to successfully absorb the loss of structural members predefined in Table 4 and a large distribution of forces were observed to happen. In all cases, column of B-1 was critical. For instance, when the corner columns removed in the first case, the axial force spiked from 7816.14 kN to a peak value of 12324.31 kN. For the same element, in the second case, the axial force spiked from 4219.53 kN to a peak value of 7189.19 kN. Accordingly, in the third case, the axial force spiked from 2025.18 N to a peak value of 3486.85 kN and in the fourth scenario, the axial force in this element spiked from 1634.28 kN to a peak value of 2278.36 kN.

After removing corner column, in first scenario, the maximum axial force in the adjacent column of C-1 was 7218.41 kN. For the same element, in the second case, the maximum axial force was 4198.24 kN. Accordingly, in the third case, the maximum axial force was 2004.83 kN and in the fourth scenario, the axial force in this element became at highest level at 1219.34 kN. According to the results, in first scenario, the maximum axial force in the adjacent column of D-1 was 7214.63 kN. For the same element, in the second case, the maximum axial force was 4211.53 kN. Accordingly, in the third case, the maximum axial force was 2001.23 kN and in the fourth scenario, the axial force in this element became at highest level at 1224.82 kN.

According to the gained results, in first scenario, the maximum axial force in the adjacent column of E-1 was 7186.14 kN. For the same element, in the second case, the maximum axial force was 4122.18 kN. Accordingly, in the third case, the maximum axial force was 2016.6 kN and in the fourth scenario, the axial force in this element became at highest level at 1276.36 kN. When the corner columns removed in the first case, the axial force of F-1 spiked from a peak value of

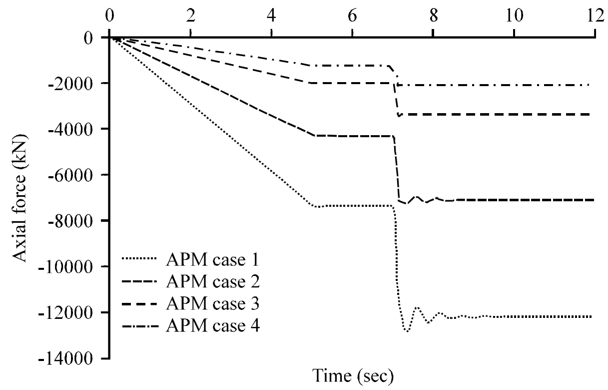


Fig. 5: Time history of axial forces at the column of B-1 in four cases

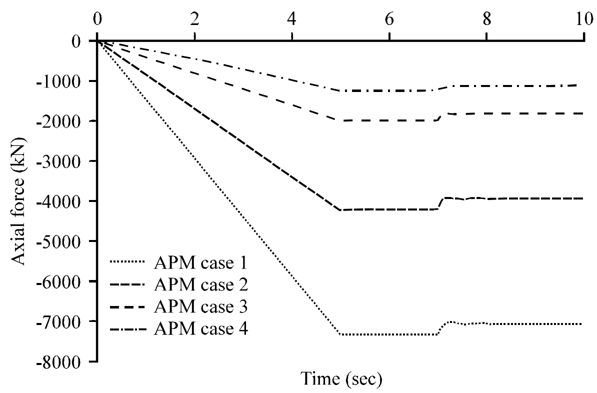


Fig. 6: Time history of axial forces at the column of C-1 in four cases

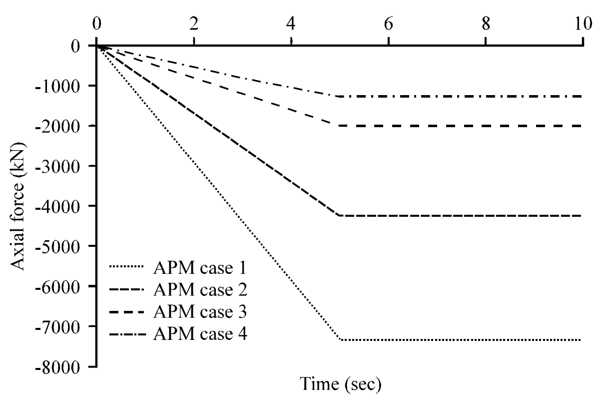


Fig. 7: Time history of axial forces at the column of D-1 in four cases

3761.42 kN to a steady state value of 2643.19 kN. For the same element, in the second case, the axial force spiked from a peak value of 2186.19 kN to a steady state value of 1525.32 kN. Accordingly, in the third case, the axial force spiked from a peak value of 1006.19 kN to a steady state value of 892.55 kN and in the fourth scenario, the axial force in this element spiked from 634.25 kN to a value of 503.11 kN.

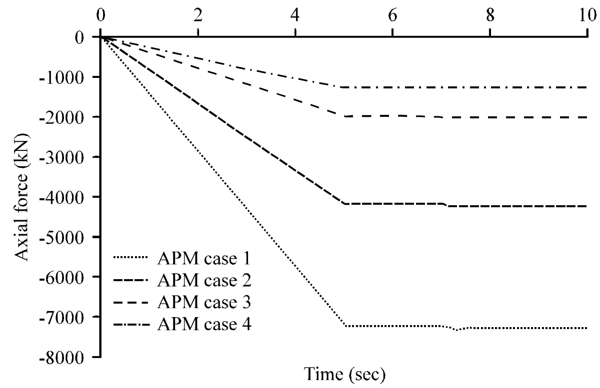


Fig. 8: Time history of axial forces at the column of E-1 in four cases

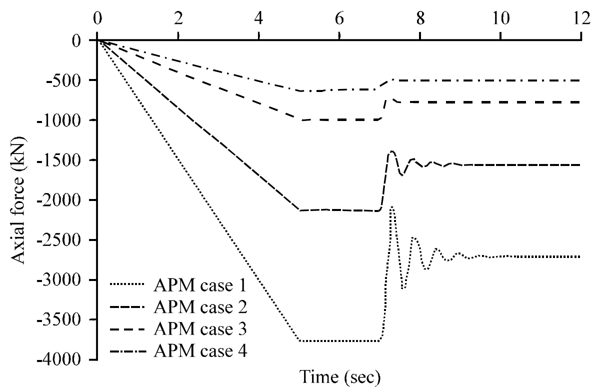


Fig. 9: Time history of axial forces at the column of F-1 in four cases

According to Iranian National Building Code part 10 (INBCSD, 2008) and ASCE (2006), the ultimate axial strength have been calculated for all columns in four scenarios and presented in Table 3.

To evaluate the vulnerability of the structure against progressive collapse phenomenon, the column buckling and local web buckling or local flange buckling have been investigated. The results obtained from analyses have been compared with those of the criteria mentioned in those codes. Regarding the analyzed scenarios, the adjacent $B_1$ columns in all four scenarios did not meet the limits. In case of $A_1$ corner column removal, the axial forces of B1 columns are 1.3-1.4 times greater than the ultimate strengths in the first and fifth stories, respectively. This is explained by the fact that, when the corner columns are removed, the weight of the structural elements tolerated by removed columns, are transferred to other columns particularly to the nearest ones. This leads to increase axial force and buckle the columns.

Based on the obtained analysis results, losing column in all scenarios had a significant effect to increase compressive axial forces on adjacent columns. Estimations show that these forces are about 2-3 times more than the situation in which the columns had not been removed, so the structure experience a large amount of extra imposed internal forces and consequently more susceptible to the progressive collapse. In the other words, by removing a column, the adjacent columns impose significantly the stresses and forces. This might be due to the removing of vertical load bearing member and transferring the weight of the elements to other adjacent columns.

However, despite of this increases in forces, the structural systems were not able to successfully absorb the loss of predefined members and redistribute extra forces among other columns. Hence, collapse prediction exists in the corner span of the designed structure. Despite the fact that model structure was designed to support the gravity as well as seismically induced forces so, the structure was not able to successfully carry all the gravity loads.

## CONCLUSION

The purpose of the current study was to investigate the vulnerability of steel moment resistant frames in building structures seismically designed according to the regulations of Iranian National Building Codes for design and construction of steel structures, with respecting the guidelines of UFC regulations for alternative path method. For this purpose, an official 10-story steel moment resistant frame building was designed for very high seismic zones (city of Tehran) according to Iranian code of practice for seismic resistant design of buildings (Standard No. 2800) and by considering regulations mentioned in INBC codes. Their exterior frames were studied under four scenarios for collapse progression while losing one primary column i.e., corner column in the ground floor, fifth floor, eighth floor and below the roof floor. Nonlinear dynamic analysis results revealed that structures designed for very high seismic zone collapsed progressively after such removal. In case of corner column removal in first and fifth story (middle height of the structure), the adjacent columns, especially the nearest to the removed member cannot tolerate the pressure created by the weight of the structural members. These extraforces are more than ultimate capacity calculated for these columns which lead to buckle the columns and finally progressive collapse the structure. These axial force values of adjoining columns are 1.3 and 1.4 times greater than their ultimate strengths in the corner column removal of the first and fifth stories, respectively. Thus, the steel moment resistant structure designed according to the Iranian National Building Codes and Iranian Seismic Code No. 2800 does not meet the UFC code limit. Therefore, some modifications are needed to increase the capacity of these columns to 1.4 times more than the initial capacity to progress the safety margin. This can be achieved by changing used column types and increasing column dimensions or using new materials and methods.

## REFERENCES

ASCE., 2005. Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-05. American Society of Civil Engineers, New York.

ASCE., 2006. Seismic Rehabilitation of Existing Buildings, ASCE 41-06. American Society of Civil Engineers, New York.

BHRC., 2004. Iranian code of practice for seismic resistant design of buildings, Standard No. 2800. 3rd Edition. Building and Housing Research Center, Tehran, Iran. http://www.bhrc.ac.ir/portal/Default.aspx?tabid=487.

England, J., J. Agarwal and D. Blockley, 2008. The vulnerability of structures to unforeseen events. Comput. Struct., 86: 1042-1051.

Fu, F., 2009. Progressive collapse analysis of high-rise building with 3-D finite element modeling method. J. Constr. Steel Res., 65: 1269-1278.

GSA, 2003. Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects. General Services Administration, June, 2003, Washington, USA.

Gerasimidis, S., 2013. Analytical assessment of steel frames progressive collapse vulnerability to corner column loss. J. Constr. Steel Res., 95: 1-9.

Helmy, H., H. Salem and S. Mourad, 2012. Progressive collapse assessment of framed reinforced concrete structures according to UFC guidelines for alternative path method. Eng. Struct., 42: 127-141.

Hosseini, M., N. Fanaie and A.M. Yousefi, 2014. Studying the vulnerability of steel moment resistant frames subjected to progressive collapse. Indian J. Sci. Technol., 7: 335-342.

INBCSD., 2006. Part 6: Minimum building loads. Iranian National Building Codes for Structural Design, Tehran, Iran. http://www.bhrc.ac.ir/portal/Default.aspx?tabid=485.

INBCSD., 2008. Part 10: Design of steel buildings. Iranian National Building Codes for Structural Design, Tehran, Iran. http://www.bhrc.ac.ir/portal/Default.aspx?tabid=485.

Khandelwal, K., S. El-Tawil and F. Sadek, 2009. Progressive collapse analysis of seismically designed steel braced frames. J. Constr. Steel Res., 65: 699-708.

Kim, H.S., J. Kim and D.W. An, 2009. Development of integrated system for progressive collapse analysis of building structures considering dynamic effects. Adv. Eng. Software, 40: 1-8.

Kim, J. and T. Kim, 2009. Assessment of progressive collapse-resisting capacity of steel moment frames. J. Constr. Steel Res., 65: 169-179.

Kim, J., Y. Lee and H. Choi, 2011. Progressive collapse resisting capacity of braced frames. Struct. Des. Tall Spec. Build., 20: 257-270.

Liu, J.L., 2010. Preventing progressive collapse through strengthening beam-to-column connection, Part 1: Theoretical analysis. J. Constr. Steel Res., 66: 229-237.

Mazzoni, S., F. McKenna, M.H. Scott, G.L. Fenves and B. Jeremic, 2007. OpenSees command language manual. Open System for Earthquake Engineering Simulation, July, 2007.

NIST., 2007. Best practices for reducing the potential for progressive collapse in buildings. NISTIR 7396, February 2007. National Institute of Standard and Technology.

Park, J. and J. Kim, 2010. Fragility analysis of steel moment frames with various seismic connections subjected to sudden loss of a column. Eng. Struct., 32: 1547-1555.

Tavakoli, H.R. and A.R. Alashti, 2013. Evaluation of progressive collapse potential of multi-story moment resisting steel frame buildings under lateral loading. Scientia Iranica, 20: 77-86.

UFC., 2009. Design of building to resist progressive collapse. UFC 4-023-03, US. Department of Defense, Washington, DC., USA.

Yousefi, A.M., 2014. Investigation on progressive collapse of steel moment resistant frame buildings. M.Sc. Thesis, Lorestan University, Iran.