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# Span Length Effects on the Progressive Collapse Behaviour in Concrete Structures

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## Article info

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**Abstract**

Progressive and general collapse of structures are extremely able to cause great casualties and financial losses. Accordingly, evaluation the performance of structures after losing primary elements is of great importance to prevent the structural collapse. This paper presents the progressive collapse assessment of RC moment frames with concrete shear wall system according to different span lengths. First, story column was destroyed by removing its reaction in different scenarios. Then, performance of the structure was evaluated under this circumstance to assess the different span length effects in RC moment frames with concrete shear wall. The results indicated that the shear wall has a positive effect on preventing progressive collapse and reduces the maximum vertical displacement by 30%. It was also observed that maximum Dynamic Amplification Factor (DAF) in the building occurs with the shear wall and the minimum length of the span. However, the maximum Demand Capacity Ratio (DCR) in a critical element for a building was obtained with the longest span and without the shear wall. Therefore, it was concluded that the DCR ratio is more suitable for evaluation of the progressive collapse severity than the DAF parameter.

## 1. Introduction

In recent years, with the increase of terrorist activities, safety and protection of the lives of the inhabitants subjected to such events are of particular importance. Progressive collapse can occur accordingly to natural or human factors. Possible hazards and abnormal loads that can cause progressive collapse includes aircraft crashes, design or construction errors, fires, gas explosions, accidental overloads, vehicle crashes, bombs etc. [1]. Progressive collapse may be investigated through a variety of analytical methods that range from very simple analyses to highly complex ones, which are generally based on the use of finite element software which is able to take into account dynamic and nonlinear properties. It is clear that the progressive collapse is a nonlinear dynamic phenomenon according to its occurrence in a very short timeframe and the imposition of nonlinear deformations on the elements before collapse [2]. The U.S. General Services Administration (GSA) [3] has made a great deal of researches in this field. The American Society of Civil Engineers (ASCE) [4] treats with progressive collapse in general and detailed issues. Furthermore, National Institute of Standard and Technology (NIST) [5] and Department of Defence (DOD) [6] provided extensive information and guidelines, including strategies to strengthen structures against progressive collapse. Among the various construction design methods for progressive collapse, Structural Standards generally choose alternate path methodology; in which, the structure is designed so that if one of the elements is destroyed, alternate paths are available for the distribution of load, and the total collapse does not

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occur [2]. The progressive collapse first caught the attention of the researchers in the 1970s, after the partial collapse of a Ronan Point tower in the United Kingdom [7]. After the collapse of the Ronan Point Building, Leyendecker et al. began studying and documenting information about the phenomenon of progressive collapse in the United States in 1976. As a distinguished main reference, these researches covered more than 375 references related to the abnormal loading conditions of structures and progressive collapse from the year 1948 to 1973. Here, the researchers provided an engineering compilation with vast references for future activities [8].

Girhammar in companion with the Swedish Association of Building Researches carried out a series of research activities related to the behaviour of steel structures exposed to abnormal events (removal of restrains in continuous beams) in 1980. Continuous double-span beams were investigated in this study in which the middle support was suddenly removed. The analytic models of rigid bodies were carried out and the axial partial restraint and the role of the chain function in the structure response were considered. This study showed that the chain reaction function plays an important role in the response of the structure during the sudden removal of the middle support of the double span system. For nearly two decades since 1980, no significant studies have been done on concrete structures and progressive collapse. However, a brief review was made by Mc Namara and GSA standard in 2003 and the DOD Guidelines for buildings resistant against progressive collapse in 2002. Despite its conciseness, general information was obtained and it was found that the elements did not yield until the initial design loading level was exceeded. It should be noted that in this research the membrane performance of the roof system was not considered in the analysis. Moreover, highly significant information was obtained from a 39-story steel structure analysis which was carried out using nonlinear static analysis. The analysis contained the removal of columns from the outside of the building and near the ground surface. Nonlinear static analysis was performed and the failure mechanism was located in the beams which were directly above the removed column [9]. Ronald Hamburger interpreted a variety of analysis methods based on the alternate load transfer path In 2007. based on his studies, analytical methods of progressive collapse evaluation were divided into four types: 1. Linear static method, 2. Nonlinear static method, 3. Linear dynamical method, 4. Nonlinear dynamical method; which are used by the researchers nowadays [10].

Hyun-Su Kim and Jinkoo Kim evaluated the dynamic amplification in 2009. It was shown that the dynamic amplification factor in the GSA and DOD standards could be more than 2 for static analysis. So, it was stated that it is inevitable to perform dynamic analysis to ensure safety against progressive collapse to consider the sudden removal of the column [2]. J. Kim and T. Kim evaluated the number of story effects in 2009. It was stated that the potential for progressive collapse decreased by increasing the number of stories [10]. In another research at the same year, Kim et al. showed that the dynamic amplification factor must be greater than two, which is also recommended by GSA and Unified Facilities Criteria (UFC) [11]. Liu acknowledged that chain operation can significantly reduce the moment by axial control [12]. Park and Kim concluded that the lack of an outer column causes more vulnerability of the steel structures when encountering the progressive collapse as compared to the removal of an internal column [16]. Murray and Sasani studied the shear failure of reinforced concrete column in a 10-story concrete block building and its impact on progressive collapse with vulnerable frame under earthquake load. For the shear failure of the column, they used opensees software and nonlinear static and dynamic analyses. It was found in their studies that how the load is distributed in the shear failure. Additionally, it was stated that the concrete structure has a suitable resistance to progressive collapse [14]. Elkoly and El-Ariss examined the positive effect of external cables in rectangular and T-shaped beams for the concrete structures, and found that the existence of a cable during the removal of the column had a significant positive effect on stability [15]. Keyvani et al. studied the effect of lateral bracing on concrete structure punching and used ABAQUS software for modelling, and found that punch strength significantly increased with the slab lateral bracing, which resulted from the formation of membrane forces in the slab [16]. Farshad Hashemi Rezvani et al. studied the effect of the span length on the steel frame structure, and concluded that by doubling the length of the span, the progressive collapse vulnerability would increase approximately 1.91 times [17]. Ghahremannejad and Park analyzed the effect of the number of floors in the building under the column removal scenario, and observed that the more the number of floors, the less the number of plastic joints in the beams and, practically, the progressive collapse decreases [24].

Kordbagh and Mohammadi investigated the impact of seismicity and the height of steel buildings in a progressive collapse and found that taller buildings are safer with respect to the progressive collapses [25].

By studying the literature in this area, it is obvious that the impact of the span length on the progressive collapse behaviour of concrete structures has not been considered in previous studies. So, this study focuses on "investigating the effects of the span length on the progressive collapse behaviour of the concrete structures with earthquake resistant design". For this purpose four concrete buildings with different span lengths were designed. Then, external frames were studied in the ground floor column removal scenario based on UFC2009. Furthermore, the Dynamic Amplification Factor (DAF) and Demand Capacity Ratio (DCR) factor were calculated to better understand the structural behaviour by removing a column from the first story [6].

## 2. Analysed Structures

In this study, four 5-story concrete buildings were designed with a moderate ductility system for an area with a high seismic hazard level placed on the soil type II. Three moment resistant buildings were designed with a shear wall in X direction. One building had moment resistant frame in both directions. As shown in Figs. 1 to 5, the plan shape of the buildings have equal dimension in both directions. Buildings were with 6 spans of 4m, 4 spans of 6m, and 3 spans of 8m. Total length of 24m in each direction was considered. Moreover, the story height in all models were 3.2m. Seismic design of structures were based on the Iranian code for seismic resistant design of buildings (2800 standard) with considering Tabas earthquake as the lateral design load. It should be noted that the seismic behaviour coefficient of structures considered as 6, according to 2800 standard for moderate ductility reinforced concrete moment frame with shear wall [18].

The initial design of the structures was performed by the ETABS 2013 software and is in accordance with the American Concrete Institute (ACI) standard for the concrete structure design [23]. Additionally, Openness software (version 2.5) used in order to implement progressive collapse modelling and column removal procedure [22].

In this study, the effect of the span length in concrete structures was studied under the conditions of sudden collapse of a column in a residential 5 story building. Accordingly, for concrete residential buildings, four different configurations are selected as follows:

1. Without a shear wall with three 4m spans (building A)

2. Having a shear wall with six 8m spans (building B)

3. Having a shear wall with three 8m spans (building C)

4. Having a shear wall with four 6m spans (building D).

The entire structure was modelled in three dimensions with six degrees of freedom, and since the area of each story and the gravity loads are the same, the lateral design force was calculated with the same value. The selected cross-sections for the frame members are shown in Table 1.

## 3. Progressive Collapse Analysis Method

According to UFC, there are three methods for the analysis of collapsed structures:

1. Linear static analysis, which is the simplest method and is a common process in structural analysis and design. In this analysis, it is assumed that the aggregates are of linear elasticity and no geometry nonlinearities are considered.

2. Nonlinear static analysis in which both geometry nonlinearities and aggregates nonlinearity are considered.

3. Nonlinear dynamic analysis that includes inertia and damping effects and is the most accurate and complex method [6].

In the present study, this method was used to consider the precision of nonlinear dynamic analysis.

Twelve column removal scenarios were considered in order to investigate the effect of column removal on the behaviour of concrete structures. These scenarios contained the sudden removal of a corner and interior column on the ground floor, for which, all column removal modes are shown in Figs. 1 to 5.

## 4. Numerical Modeling

External frames of the buildings were modeled using the opensees software (version 2.5) in two-dimensional method. Several nonlinear dynamic analyses were conducted to investigate the structural response in each removing scenario. As shown in Fig. 6, concrete behaviour modelling has two important parts. Internal mass of the concrete in member was confined by the rebar and had greater resistance compared to the unconfined one. So, two types of concretes were used. Unconfined concrete defined with maximum compressive strength equal to 25MPa and linear tension softening behaviour with cracking tension stress equal to 5MPa. Confined concrete defined with maximum compressive strength equal to 28MPa and linear tension softening behaviour with cracking tension stress equal to 5.6MPa. Respectively, Confined and unconfined concrete were modeled in opensees with Concrete02 and Confined Concrete01 uniaxial material property based on mander stress-strain model [25]. For the steel rebar shown in Fig. 7, the Steel02 was used in opensees, based on the Iranian S340 rebar with Fy=340MPa and E=202GPa. In this model, the nonlinear elements of the column and beam were used for precise modeling.

Plasticity was also considered along and across the element, through using nonlinear force Beam-Column element. Moreover, large displacement effects and rotational aspects of the geometric hardness matrix were used to consider nonlinear effects. In addition, the connections of the beam to the column and column to the foundation were assumed to be rigid.

Dead and live loads were 6.54 (kN/m$^2$) and 1.25 (kN/m$^2$) for the roof and 2 (kN/m$^2$) and 7.31 (kN/m$^2$) for the floors. P-$\Delta$ effects were considered with correlational geometry method. To calculate the total rotation, the plastic rotation method was used, and for the shear modeling, the element MVLEM was used in the opensees software.

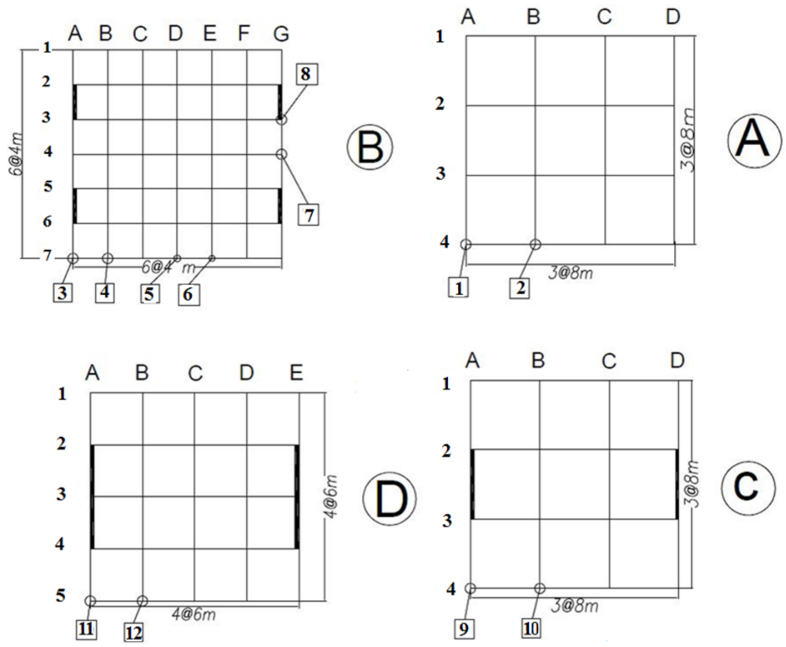


**Fig. 1.** Twelve scenarios of the column removal examined in four concrete buildings.

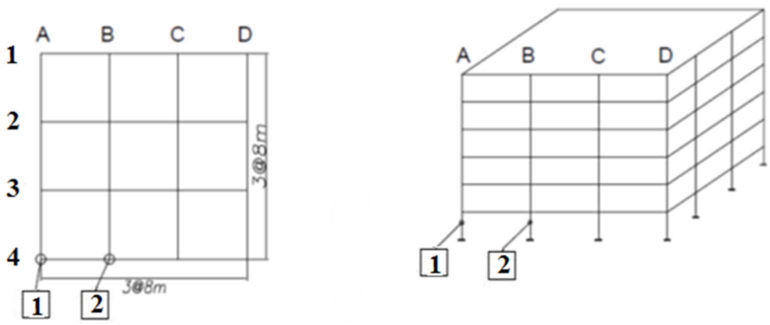


**Fig. 2.** Building without shear walls with a length of 8m, with columns A/4 and B/4 removed.

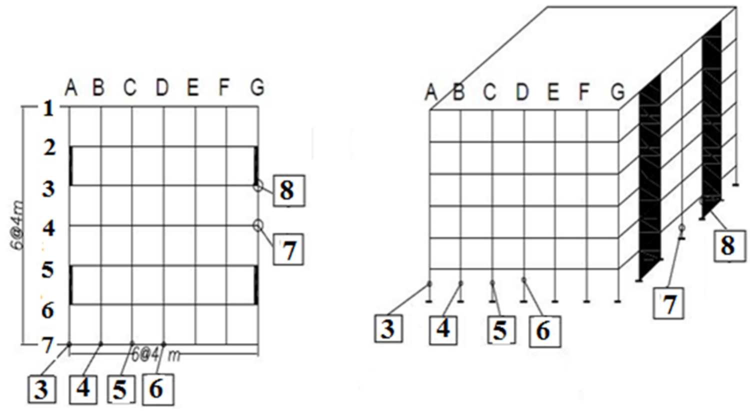


**Fig. 3.** Building with a shear wall with a length of 4m, with the columns of the axis 7 and the G axis removed.

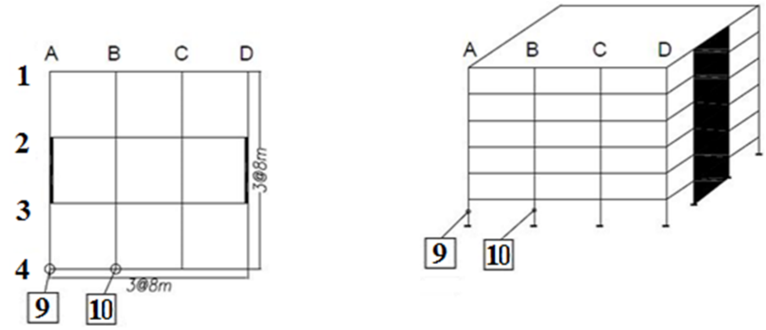


**Fig. 4.** Building with a shear wall with a length of 8m, with columns A/4 and B/4 removed.

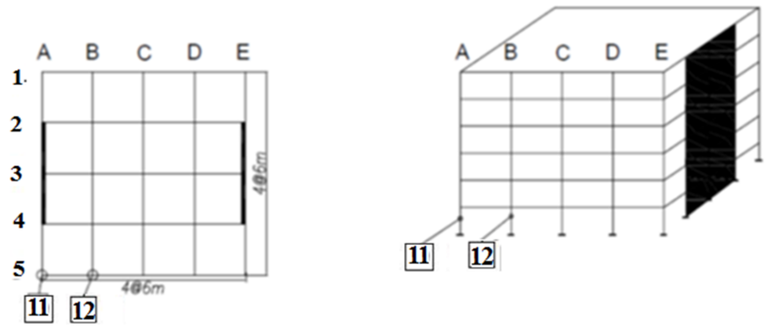


**Fig. 5.** Exhibits of a building with a shear wall with a span length of 6m, with columns A/5 and B/5 removed.

**Table 1**
The sections used in concrete buildings studied.

| Building | Section | Ground to 2nd floors | $3^{rd}$ to $5^{th}$ floors |
|---|---|---|---|
| A | Column sections | Section of 65×65cm with 24 longitudinal rebar No. 25 | Section of 60×60cm with 20 longitudinal rebar No. 25 |
| | Beam sections | Section of 45×45cm with 8 longitudinal rebar No. 16 | Section of 45×45cm with 8 longitudinal rebar No. 16 |
| | Shear wall sections | None | None |
| B | Column sections | Section of 55×55cm with 20 longitudinal rebar No. 25 | Section of 50×50cm with 16 longitudinal rebar No. 16 |
| | Beam sections | Section of 45×45cm with 8 longitudinal rebar No. 16 | Section of 45×45cm with 8 longitudinal rebar No. 16 |
| | Shear wall sections | Section with a width of 30cm and longitudinal/traverse rebar No. 16 in 15cm intervals | Section with a width of 25cm and longitudinal/traverse rebar No. 16 in 15cm intervals |
| C | Column sections | Section of 65×65cm with 20 rebar No. 25 | Section of 55×55cm with 16 rebar No. 25 |
| | Beam sections | Section of 45×50cm with 8 rebar No. 16 | Section of 45×50cm with 8 rebar No. 16 |
| | Shear wall sections | Section with a width of 35cm and longitudinal/traverse rebar No. 16 in 15cm intervals | Section with a width of 30cm and longitudinal/traverse rebar No. 16 in 15cm intervals |
| D | Column sections | Section of 60×60cm with 20 rebar No. 25 | Section of 55×55cm with 16 rebar No. 25 |
| | Beam sections | Section of 45×45cm with 8 rebar No. 16 | Section of 45×45cm with 8 rebar No. 16 |
| | Shear wall sections | Section with a width of 30cm and longitudinal/traverse rebar No. 16 in 15cm intervals | Section with a width of 25cm and longitudinal/traverse rebar No. 16 in 15cm intervals |

## 5. Dynamic Analysis Process

Removing column and using the column-removal reactions was performed as follows:

1. The structure undergoes the gravitational static load analysis for the combined load of the following relation, including the dead load (DL) and the live load (LL).

$$W = 1.2DL + 0.5LL \quad (1)$$

2. The axial force, moment, and shear force are recorded in the column to be removed.

3. Take the structure out from the analysis and remove the column. Instead, the force mentioned in the previous step should be replaced as the reaction.

4. To model the column removal, concentrated force equal and opposite to the documented reaction from the previous step should be replaced to the desired column, and certain time history should be considered for this force.

5. The structure was analysed under nonlinear dynamic analysis with the static loads as initial conditions (note that the principle of superposition cannot be used because this is a nonlinear analysis; therefore, they should be stacked behind each other).

The loads from step 4 should be applied to the structure with a sudden manner to model the column removal from the structure. To carry out nonlinear dynamic load analysis with predetermined load, the load increases linearly for 5 seconds until the final value is obtained and then remains for 2 seconds to avoid any dynamic stimuli. Subsequently, a column for each of the above scenarios is suddenly removed and the reaction of the structure investigated [6]. Dynamic analysis was performed with a 5% damping coefficient. Furthermore, for greater accuracy and convergence of nonlinear analysis in Newton- Raphson analysis, each second was divided into 100 time intervals.

Admission criteria for performance-based analysis Building structures are typically designed to meet a certain level of performance such as the limit states, immediate occupancy (IO) and Life Safety (LS), and their structural elements generally collapse when exposed to excessive loads. In this study, based on the acceptance issues, structural performance of all elements was controlled at all stages of analysis [6]. These issues contain the deformation and force control in accordance with reference [6]. Rotation ($\theta$) is described as the deflection ratio ($\Delta$) of the elements of the structure to the length (L) of the elements. In this study, the rotation of beams and columns was obtained using nonlinear dynamic analysis and compared with the acceptable values in the ASCE41-13 standard [4].

The control measures of the columns depend on the level of the load bearing on them, and can be of the deformation control type of the force deformation type. In this case, when the axial compressive loads are less than half the axial pressure capacity, the column is considered to be a deformation-controlled column. Otherwise, the column will be considered as a force-controlled one. The relationship of the axial load capacity of the column is determined using Eq. (2), and accordingly, it is assumed that the column loses its efficiency in case the DCR ratio increases from the unit value based on the following equations. For the beams, on the other hand, only the deformation control is considered. This will determine the beam rotation at every stage of the analysis and specifies its performance. In this case, a beam is considered defected or collapsed when its performance exceeds LS (life safety) [6].

$$DCR = \frac{Q_{UD}}{Q_{UE}} \quad (2)$$

Where: $Q_{UD}$ is the acting demand determined in component or connection (moment, axial force, shear and possible combined forces) and $Q_{UE}$ is the expected ultimate un-factored capacity of the component or connection (moment, axial force, shear and possible combined forces), which results from dynamic analysis.

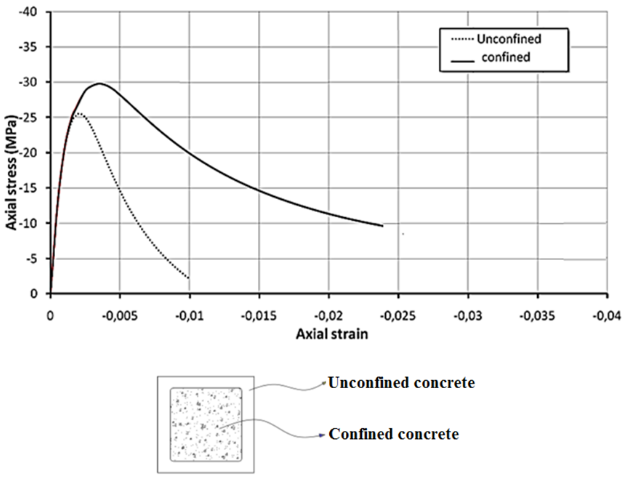


**Fig. 6.** Behaviour of the concrete modelled [23].

In this study, progressive collapse analysis is a strength assessment, which determines whether the sudden removal of a column leads to a collapse progression. In this assessment, for each scenario, the alternate path method is studied by performing a nonlinear structural analysis of the reaction. Then, the results of the analysis are compared with the accepted values in order to check the probability of their progressive collapse. If one element is destroyed, it can be said that the building is susceptible to progressive collapse and that analysis of that scenario is stopped. Otherwise, it will be obvious that under the load of the structure, the structure is able to bridge other elements after the removal of the element.

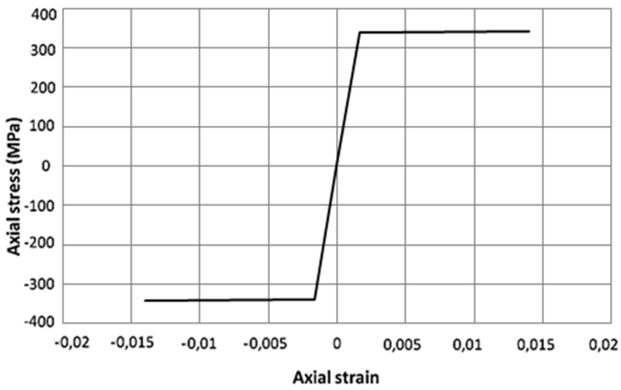


**Fig. 7.** Behaviour of steel rebar's modelled [23].

Another benefit of a strength analysis is the possibility of calculation of the DAF and DCR. DAF considers the dynamic nature of the sudden removal of the column and the DCR implies the most critical location of the structure for such collapses and possible collapse modes. Fig. 8 and Eq. (3) show how to calculate DAF.

$$\mathrm{DAF} = \frac{\text{Maximum load input}}{\text{Constant input force before column removal}} \quad (3)$$

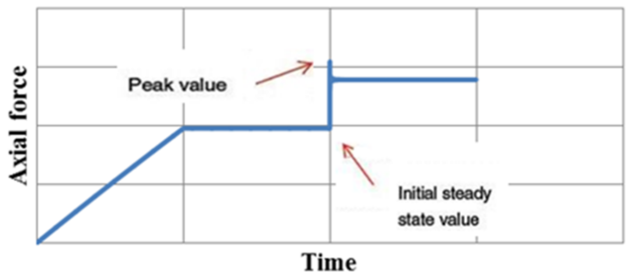


**Fig. 8.** Parameters required to calculate the dynamic amplification factor (DAF).

## 6. Results and Discussion

Figs. 9 to 12 show the vertical movements of the upper nodes of the removed columns. In these figures, reduction of the length of the span and the presence of the shear wall reduce the vertical displacement. As shown in Fig. 9, the existence of a shear wall orthogonal to the frame caused the vibration and displacement to be reduced by approximately 15% after the removal of the column than when there is no shear wall.

According to Figs. 10 and 11, when the span length doubles (for buildings with shear walls), the maximum vertical displacement increases by approximately 22%. It is also observed that the vertical displacement for the corner columns is approximately 2 times the internal columns, which does not necessarily mean that the structure is more prone to progressive collapse when one of the corner columns is removed.

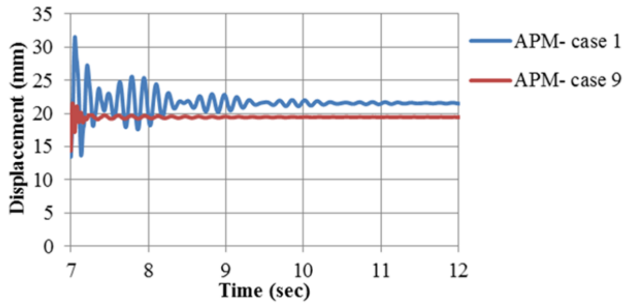


**Fig. 9.** Vertical displacement of column removal points in nonlinear dynamics analysis in the scenarios 1 and 9.

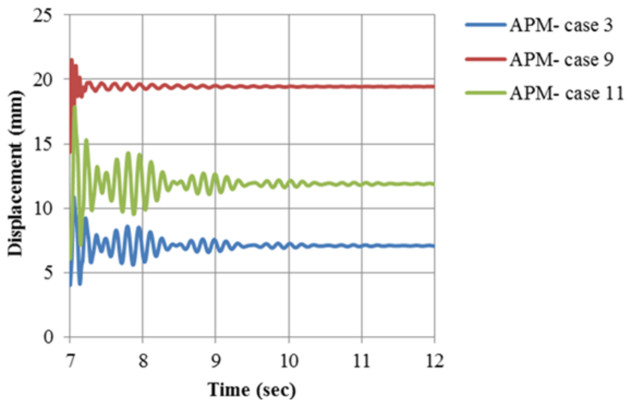


**Fig. 10.** Vertical displacement of column removal points in nonlinear dynamic analysis in Scenarios 3, 9 and 11.

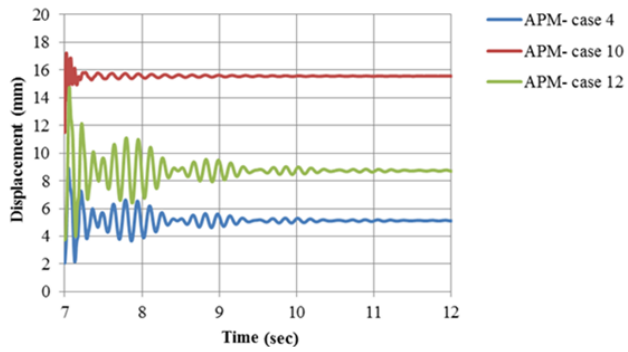


**Fig. 11.** Vertical displacement of column removal points in nonlinear dynamic analysis in Scenarios 4, 10 and 12.

The graphs shown in Fig. 12 are related to structure B. as shown in the figure, the existence of a shear wall inside the frame has caused a 30% decrease in the maximum displacement in the direction in which the shear wall exists than in the direction in which the shear wall doesn't exist. In addition, it can be concluded that when the length and the number of spans are large, in the same conditions, the maximum displacement is equal for each of the middle columns in a frame. It is also observed that when the column, which is attached to the shear wall, is removed, a slight vertical displacement occurs, but the amount of this displacement is directly related to the resistance and the dimensions of the shear wall.

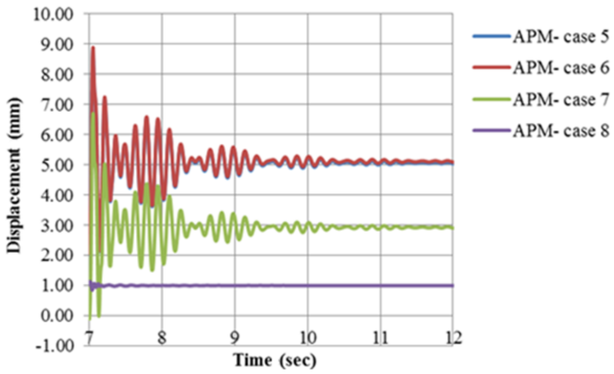


**Fig. 12.** Vertical displacement of column removal points in nonlinear dynamic analysis in Scenarios 5, 6, 7, and 8.

Figs. 13 and 14 show the coefficients of the total rotation relative to the yield rotation in 1$^{th}$ and 9$^{th}$ Scenarios. Data is shown considering some critical beams and columns. It can be seen that the beam AB on the fourth axis has the most critical state. Table 2 shows these coefficients along with the most critical structural elements, all of which are less than the values permitted in the ASCE41-13 standard [4].

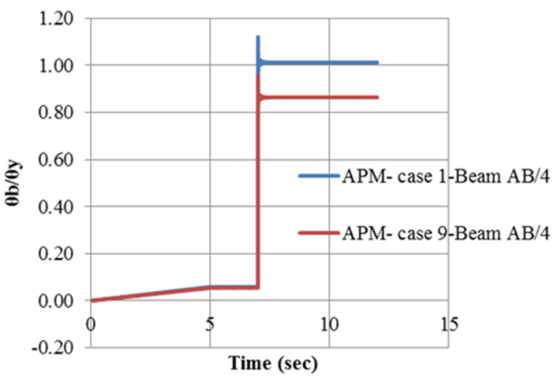


**Fig. 13.** Total rotation to yield rotation of the beams in Scenarios 1 and 9.

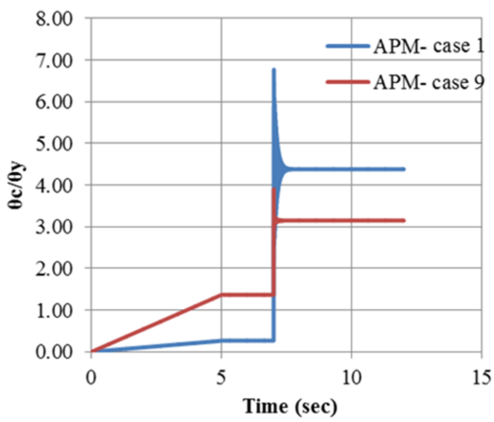


**Fig. 14.** Total rotation to yield rotation of the columns in Scenarios 1 and 9.

**Table 2**
Analysis results (most critical elements to be removed).

| Rotation ratio | Critical beam rotation ratio | | Scenario |
|---|---|---|---|
| 1 | 4.5 | AB/4 | Scenario 1 |
| 0.8 | 3.9 | AB/4 | Scenario 2 |
| 0.9 | 1.6 | AB/7 | Scenario 3 |
| 0.85 | 1.5 | AB/7 | Scenario 4 |
| 0.83 | 1.4 | AB/7 | Scenario 5 |
| 0.8 | 1.2 | AB/7 | Scenario 6 |
| 0.1 | 0.35 | G/3-4 | Scenario 7 |
| 0.03 | 0.06 | G/3-4 | Scenario 8 |
| 0.95 | 4.8 | AB/4 | Scenario 9 |
| 1.03 | 5 | AB/4 | Scenario 10 |
| 1.13 | 1.9 | AB/5 | Scenario 11 |
| 1.10 | 1.7 | AB/5 | Scenario 12 |

As it can be seen, the ratio of the critical beam rotations is higher than that of the columns. It occurs because of the higher rotation of the beams according to the column removal. Additionally, it is observed that the column rotation ratio for the structure D is higher than the others, and the beam rotation ratio for the structure C is higher than others. This information indicates that the ratio of the angles of the beam is directly related to the length of the span. Moreover, with an increase in the span length, the beam rotation ratio increases. Although, the column rotation ratio does not have a direct relation with the span length.

Fig. 15-19 show the axial forces and moment of the critical columns. The simulation results show that the maximum moment and axial force are obtained after the removal of the column in building A. this building did not have a shear wall and its span length was larger than other buildings. In buildings with a shear wall, longer span causes the higher moment. So, when the span length doubled, the maximum moment increased by 54.2% after the removal of the column and the maximum axial force of the column increased by 257.9%.

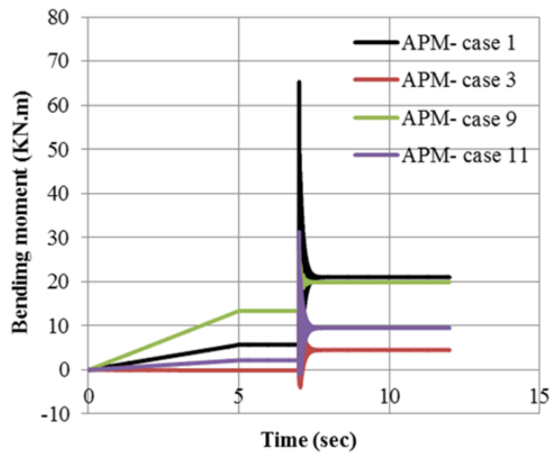


**Fig. 15.** Bending moments of columns in Scenarios 1, 3, 9 and 11.

In this case, the forces are redistributed and the forces applied to the removed column are moved to other columns by the beams. For example, in Building C, when the corner column was suddenly removed in $9^{th}$ Scenario, the axial force of the column Col-B-4 reached to a maximum of 2120kN from 1261kN, then decreased to a uniform and stable value of 1402kN which is lower than nominal capacity (4330kN). When this is combined with the maximum moment of 23.74kN.m which was produced on the column, it shows that the column does not undergo an additional load and overall collapse does not occur in the structure.

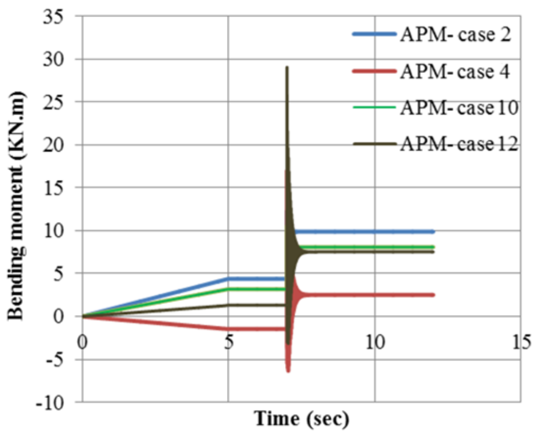


**Fig. 16.** Bending moments of columns in Scenarios 2, 4, 10 and 12.

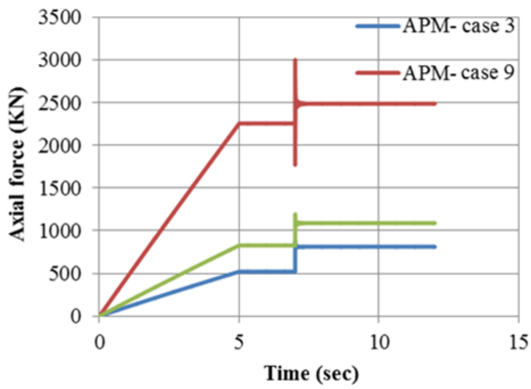


**Fig. 17.** Axial forces of the columns in Scenarios 3 and 9.

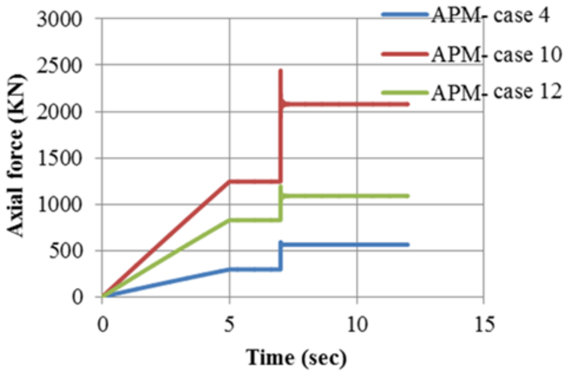


**Fig. 18.** Axial forces of the columns in Scenarios 4, 10 and 12.

As shown in Fig. 19, the removed column that is next to the shear wall has a lower axial force than others according to the absorption of forces by the shear wall.

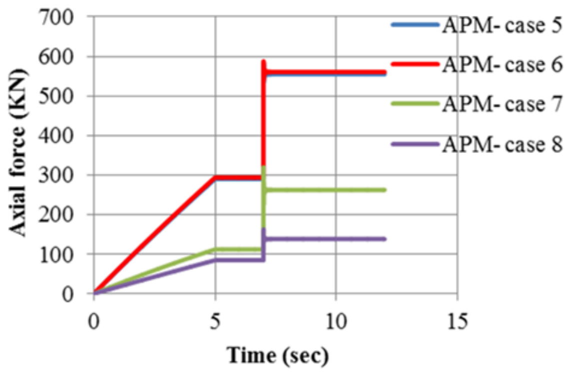


**Fig. 19.** Axial force of the columns in Scenarios 5, 6, 7 and 8.

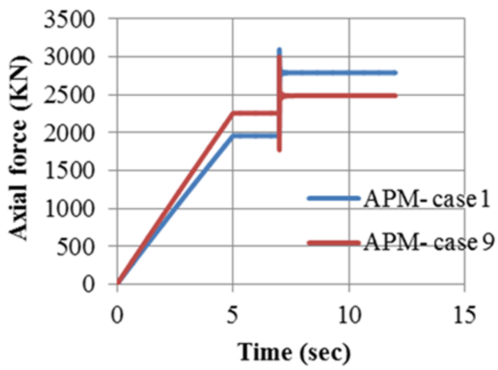


**Fig. 20.** Axial force of the columns in Scenarios 1 and 9.

The performance of the columns reveals the fact that apart from such increases in axial forces and moments, the structural system is able to with stand the removal of one of the predetermined columns and redistribution of the additional forces on the other columns. Therefore, no progressive collapse is expected. This situation is according to the fact that the frames are studied in terms of the forces induced by the earthquake. Therefore, large-scale columns can successfully handle all forces. In these frames, the spans affected by the removal of the elements obtained their stability from a healthy span and they did not collapse.

As can be seen in Table 3, with an increase in the span length the DCR parameter increases. This increase indicates potential for progressive collapse. Table 3 shows the coefficients of DAF and DCR based on the nonlinear dynamic analysis results that are lower than the permitted values in UFC2009 regulations [6]. According to the table, it is clear that the sudden removal of a corner column on the first story leads to the highest DAF and DCR. Additionally, it is observed that the maximum DAF is 2.7 and is related to the $7^{th}$ scenario, while the critical DCR is 0.65 and is related to the first scenario. The maximum amplification ratio in the building occurs with a shear wall and a minimum length of the span. However, the maximum demand capacity ratio in the critical element occurs for the building with the maximum length of the span and without the shear wall. Therefore, it is known that the DCR ratio is more suitable for evaluation of the progressive collapse severity than the DAF parameter.

**Table 3**
Critical DAF and DCR for the columns.

| DCR | DAF | Critical beam element | Scenario |
|---|---|---|---|
| 0.65 | 1.55 | AB/4 | Scenario 1 |
| 0.51 | 1.9 | AB/4 | Scenario 2 |
| 0.43 | 1.84 | AB/7 | Scenario 3 |
| 0.39 | 2.05 | AB/7 | Scenario 4 |
| 0.41 | 1.96 | AB/7 | Scenario 5 |
| 0.42 | 2 | AB/7 | Scenario 6 |
| 0.43 | 2.7 | G/3-4 | Scenario 7 |
| 0.44 | 1.68 | G/3-4 | Scenario 8 |
| 0.6 | 1.3 | AB/4 | Scenario 9 |
| 0.58 | 1.63 | AB/4 | Scenario 10 |
| 0.52 | 1.54 | AB/5 | Scenario 11 |
| 0.49 | 1.56 | AB/5 | Scenario 12 |

## 7. Conclusions

In this study, the effect of the span length was investigated on structural safety level against the progressive collapse of buildings with concrete moment frame in case of earthquakes. For this purpose, four concrete structures of different lengths and structural systems were designed for a constant frame length. Their exterior frames were investigated under 12 progressive collapse scenarios by removing the column from the first story, i.e. the corner column and the inner column. Alternate loading path and nonlinear dynamic analysis were used in this study and the following results were obtained:

1. A structure with a shorter span has a greater resistance to progressive collapse, such that it has the highest strength in a structure modeled with the span length of 4m, and the lowest strength in a structure with a span length of 8m and without a shear wall in both case of the external or internal column removal.

2. The maximum displacement is approximately doubled after removing the column by doubling the length of the span.

3. When the column is removed, the shear wall has a positive effect on preventing progressive collapse, and it reduces the axial force of the column by 12%. Furthermore, the vertical displacement decreased about 15% in comparison with the building without shear walls.

4. If the shear wall exists in the same frame that the column is removed, it has an increased effect so the maximum displacement decreases by 30%.

5. The maximum amplification ratio occurs in a building with a shear wall and the minimum span length. However, the maximum demand to capacity ratio occurs for a building with the maximum span length and without a shear wall. Therefore, it is concluded that the DCR ratio is more suitable for evaluation of the progressive collapse severity than the DAF parameter.

## References

[1] H. Wibowo, D.T. Lau, Seismic progressive collapse: qualitative point of view, Civ. Eng. Dimension., 11(1) (2009) 8-14.

[2] J. Kim, T. Kim, Assessment of progressive collapse-resisting capacity of steel moment frames, J. Constr. Steel. Res., 65(1) (2009) 169-179.

[3] U. GSA, Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects. Washington D.C., (2003).

[4] American Society of Civil Engineers, Structural Engineering Institute, Minimum design loads on buildings and other structures standards committee, Am. Soc. Civ. Eng., 4 (2013).

[5] B.R. Ellingwood, R. Smilowitz, D.O. Dusenberry, D. Duthinh, H.S. Lew, N.J. Carino, Best practices for reducing the potential for progressive collapse in buildings (No. NIST Interagency/Internal Report (NISTIR)-7396), (2007).

[6] R.J. Mcnamara, Conventionally designed buildings: blast and progressive collapse resistance, Blast and progressive collapse resistance, http://www.emergencymgt.net, (2003) 29-32.

[7] M. Kfir, Progressive collapse: comparison of main standards, formulation and validation of new computational procedures, PhD Thesis, (2009).

[8] E.V. Leyendecker, J.E. Breen, N.F. Somes, M. Swatta, Abnormal loading on buildings and progressive collapse: an annotated bibliography (No. 67-68). US Department of Commerce, National Bureau of Standards, (1976).

[9] R.J. Mcnamara, Conventionally designed buildings: blast and progressive collapse resistance, Blast and progressive collapse resistance, Steel Conf., AISC, San Antonio, http://www.emergencymgt.net, (2006).

[10] H.S. Kim, J. Kim, D.W. An, Development of integrated system for progressive collapse analysis of building structures considering dynamic effects, Adv. Eng. Softw., 40(1) (2009) 1-8.

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

[12] J. Park, J. Kim, Fragility analysis of steel moment frames with various seismic connections subjected to sudden loss of a column, Eng. Struct., 32(6) (2010) 1547-1555.

[13] J.A. Murray, M. Sasani, Seismic shear-axial failure of reinforced concrete columns vs, system level structural collapse, Eng. Fail. Anal., 32 (2013) 382-401.

[14] S. Elkoly, B. El-Ariss, Progressive collapse evaluation of externally mitigated reinforced concrete beams, Eng. Fail. Anal., 40 (2014) 33-47.

[15] L. Keyvani, M. Sasani, Y. Mirzaei, Compressive membrane action in progressive collapse resistance of RC flat plates, Eng. Struct., 59 (2014) 554-564.

[16] F.H. Rezvani, A.M. Yousefi, H.R. Ronagh, Effect of span length on progressive collapse behaviour of steel moment resisting frames, Structure., 3 (2015) 81-89.

[17] No. Standard 2800. Iranian Code of Practice for Seismic Resistant Design of Buildings and Housing Researchers Center, (2014).

[18] S. Tahooni, Design of Concrete Structures. 7th edition, Tehran; Elm and Adab Publications, (2001).

[19] B. Asgarian, A. Sadrinezhad, P. Alanjari, Seismic performance evaluation of steel moment resisting frames through incremental dynamic analysis, J. Constr. Steel. Res., 66(2) (2010) 178-190.

[20] F. Hashemi Rezvani, B. Asgarian, Effect of seismic design level on safety against progressive collapse of concentrically braced frames, Steel. Compos. Struct., 16(2) (2014) 135-156.

[21] S. Mazzoni, F. McKenna, M.H. Scott, G.L. Fenves, B. Jeremic, Opensees command language manual, http://opensees.berkeley.edu/OpenSees/manuals/usermanual/, (2007).

[22] ACI318, Building code requirements for structural concrete and commentary, ACI Commitee 318, Am. Concr. Inst., (2008).

[23] M. Ghahremannejad, Y. Park, Impact on the number of floors of a reinforced concrete building subjected to sudden column removal, Eng. Struct., 111 (2016) 11-23.

[24] B. Kordbagh, M. Mohammadi, Influence of seismicity level and height of the building on progressive collapse resistance of steel frames, Struct. Des. Tall. Spec. Build., 26 (2017) e1305.

[25] J.B. Mander, M.J.N. Priestley, R. Park, Theoretical stress-strain model for confined concrete, J. Struct. Eng., 114(8) (1988) 1804-1826.