Investigation of the Progressive Collapse Potential in Steel Buildings with Composite Floor System

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Abstract

Abnormal loads due to natural events, implementation errors and some other issues can lead to occurrence of progressive collapse in structures. Most of the past researches consist of 2- Dimensional (2D) models of steel frames without consideration of the floor system effects, which reduces the accuracy of the modeling. While employing a 3-Dimensional (3D) model and modeling the concrete slab system for the floors have a crucial role in the progressive collapse evaluation. In this research, a 3D finite element model of a 5-story steel building is modeled by the ABAQUS software once with modeling the slabs, and the next time without considering them. Then, the progressive collapse potential is evaluated. The results of the analyses indicate that the lack of the consideration of the slabs during the analyses, can lead to inaccuracy in assessing the progressive failure potential of the structure. The results show that a structure is subjected to unusual external loads such as a motor vehicle collision, explosion of a bomb in a vehicle, etc., the most critical columns are located in the nearest frame to the outer frame in the structure.

Keywords: Abnormal loads, Composite floor system, Intermediate steel moment resisting frame system, Progressive collapse.

1.Introduction

Following the devastation of the World Trade Center and other similar events worldwide, evaluation of progressive collapse potential in existing important structures, and also considering progressive collapse issue in the design phase have been studied by researchers around the world. The Progressive Collapse is a situation in which the incidence of a local damage in a structural element, leads to failure in the adjacent members and following it overall collapse in building happens [1]. Several factors can cause local damage in structural elements and eventually lead to progressive collapse in structures. One of these important factors is the occurrence of explosion in a building and clash to surrounding columns of the structure that may lead to local damage of one or more key structural elements and occurrence of progressive collapse in the structure. Often the progressive collapse is not proportional to the cause of damage creation, and due to a small incident the structures may be exposed to the progressive collapse. In other words, during the progressive collapse, mechanism destruction is much greater than the creator factor [2]. Existing standards for design of structures under common loads generally use a degree of strength and ductility in a structural system for preventing the progressive collapse. Old buildings generally consist of frames with small spans and have inherent strength and resilience against the progressive collapse. But changes in architectural styles in combination with the evolutionary design by computers and use of high performance materials lead to advanced building systems which have long spans and are relatively light and ductile [3]. Currently, there are some design procedures to mitigate the potential of progressive collapse in both UK and US. The UK Building Regulations [4] and BS5950 [5] are state requirements for the avoidance of disproportionate collapse. In the United States, the Department of Defense (DoD) [6] and the General Services Administration (GSA) [7] provide detailed guidelines regarding the methodologies for building structures to resist against the progressive collapse. Both of them use the alternate path method

(APM).The methodology is generally applied in the context of a 'removal column' scenario to assess the progressive collapse potential and checks this issue that if a building can successfully absorb loss of a critical member or not. FEMA 403 [8] and NIST 2005 [9] also provide some general design recommendations that according them, when the structural damage occurs, if steel-framed structural systems have enough redundancy and resilience, to provide alternative load paths and additional capacity for redistribution of gravity loads. There are four procedures for alternate path method: linear static (LS), linear dynamic (LD), nonlinear static (NS), and nonlinear dynamic (ND) methods.

So far, some analytical studies have been performed on the behavior of buildings against the progressive collapse. Kaewkulchai and Williamson [11] proposed a beam element formulation and solution procedure for dynamic analysis of progressive collapse, which provide guidance for further study on simulation of progressive collapse. Powell [12] reviewed the principles of progressive collapse analysis for the alternate path method. Khandelwal et al. [13] studied the progressive collapse resistance of steel braced frames, designed under seismic loads, and performed a validation for 2D models. The simulation results show that the eccentrically braced frame is less vulnerable against the progressive collapse than the special concentrically braced frame. Kim et al. [14] studied the progressive collapse-resisting capacity of steel moment frames using the alternate path methods recommended in the GSA and DOD guidelines. It was observed that the nonlinear dynamic analysis provides larger structural responses and more variable results. However the linear procedure provides a more conservative decision for progressive collapse potential of structural models. Using the commercial program SAP2000, Tsai et al. [15] conducted the progressive collapse analysis by using the linear static analysis procedure recommended by the US General Service Administration, GSA. Liu [16] investigated the methods of progressive collapse prevention by strengthening the beam-to-column connections. Shi et al. [17] proposed a new method for progressive collapse analysis of RC frames under blast loading. Rather than using sudden column removal methods, they directly applied the blast load on the structure. Mohamed et al. [18] used the direct element removal method to model the progressive

collapse in reinforced concrete buildings. They presented a new analytical formulation of an element removal algorithm based on dynamic equilibrium and the resulting transient change in kinematics of the system.

As mentioned above, most of the past researches consist of 2D models of steel frame structures without considering the floor system effects, which leads to the inaccuracy of the model. Recent studies by the author and other researchers found the importance of considering 3D model effects and showed that the concrete slab system for floors has a crucial role in the progressive collapse evaluation. To solve the problem of neglecting the floor system in 2D models, Fu [20] proposed a 3D finite element model created using ABAQUS [19] to investigate the progressive collapse of high-rise buildings in different column removal scenarios. Fu then extended his study of progressive collapse to the multi-story buildings with concentric bracing and found that, with normal column spacing, the beams may still be in the elastic stage after the removal of one column if they are designed with the current design codes [21]. He showed that plasticity is normally observed in more than two column removal scenarios. As the plasticity is very important in absorbing the energy caused by the columns removal, so, in this paper, two column removal scenarios are studied in detail and the plasticity developed in the steel member and the response of the slabs are studied in detail.

Therefore, in this study, 3D finite element models of a 5-story steel building According to Figure 1, with consideration of the slab and without considering it, were simulated using the ABAQUS software [19] and their progressive collapse potential was evaluated.

2.Three-dimensional Finite Element Model

2.1.Description of building specifications

The considered model is a 5-stroy steel building with the typical story plan shown in Figure 1.The story heights are equal to 3.20 m. The floor system is a full shear interaction metal deck with a slab thickness of 150 mm; the shear studs are evenly distributed along the steel beams. The steel rebar used in the rebar mesh for the slabs is A252. Lateral force resisting system of the building in both x and y directions is the intermediate steel moment resisting frame. The connections between beams and columns

are rigid and are made of the ST37 steel, which its yield stress is 2400 $\frac{kg}{cm^2}$ and its ultimate stress is 3700 $\frac{kg}{cm^2}$. The conventional design of the structure was carried out according to the tenth topic of the Iranian Building National Regulations [22] by using the ETABS software [23]. Dead, live and earthquake loads were calculated based on the Sixth topic of the Iranian Building National Regulations [24]. The structural design of the building was done in a few steps, on one side, the selection of near-optimal levels (in terms of stresses and lateral displacements of the structure), and on the other hand design of components to a have simple and uniform arrangement. In the future in order to study the progressive collapse of structures, the effect of each of the various members on the general behavior of the structure can be analyzed in an appropriate and comprehensible manner. The results of structural design are presented in Table 1.
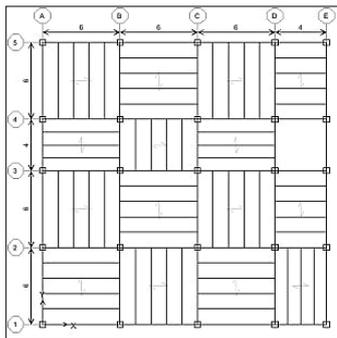

Fig. 1. The plan of the case study building

Table1
The Results Of Five-Story Steel Building Design

Story	Column	Main Beam	Composite Beam
base	Box 40×40×1.6	2IPE 300	IPE 300
1	Box 40×40×1.6	2IPE 300	IPE 300
2	Box 30×30×1.6	2IPE 270	2 IPE 180
3	Box 30×30×1.6	2IPE 270	2 IPE 180
4	Box 30×30×1.6	2IPE 270	2 IPE 180
5	Box 30×30×1.6	2IPE 270	2 IPE 180

2.2.Finite element modeling- material properties

In this study two types of materials (i.e., steel and concrete) were employed for finite element modeling. The properties of the materials and elements used for modeling are described in this section.

Steel:According to UFC4-023-03 regulations [6], because the yield strength of steel is approximately 25% greater than the characteristic strength, the Strength Increase Factor (SIF) was used. Also, in accordance with the regulations the coefficient of the Ultimate stress of steel is equal to 1.05 [25]. The stress-strain curve of the steel material considered for modeling is shown in Figure 2 schematically.
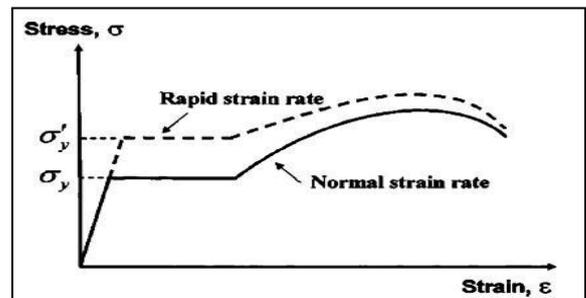

Fig. 2. The diagram of strength increase factor [26]

Reinforced Concrete:Reinforced concrete is one of the complex materials in the finite element modeling. The correct definition of reinforced concrete in finite element modeling for the elastic and plastic parts of the compressive and tensile behavior can have a decisive role in the responses and outputs. In the ABAQUS software [19], for considering the concrete failure, three models can be applied. In this study the plasticity damage model was used for modeling the inelastic behavior of concrete [19]. The mechanical properties of the materials used in the finite element modeling are presented in Tables 2- 4.

Table 2
Mechanical Properties Of The Rebar In The Elastic Range

Modulus of Elasticity (E)	2625×10⁶ (Pa)
Poisson's Ratio (μ)	0.3
Density (ϑ)	7850 kg/m³

Table3
Mechanical Properties Of Bar In Plastic Range

Yield Stress (MPa)	Plastic Strain
280	0
370	0.09

Table 4
Mechanical Properties Of Concrete In Elastic Range

Modulus of Elasticity(E)	24757×10⁶ (Pa)
Poisson's Ratio(μ)	0.2
Density(ϑ)	2400 kg/m³

All of the beams and columns are modeled by using the beam elements. The slabs are modeled by using

the four node shell element. The reinforcement was imbedded in each shell element by using the rebar element as smeared layers. The beam and shell elements were coupled together using the rigid beam constraint equations to give the composite action between the beam elements and the concrete slab. The model also incorporates the nonlinear characteristics for the materials. The behavior of steel beams and columns was modeled using an elastic–plastic material model in the ABAQUS [19]. All columns in the base are fixed; also the mesh size of the model was good enough to ensure that the applied forces are measured precisely. The beam-to-column connections were assumed to be fully fixed. The continuity across the connector by the presence of a composite slab in all above parts of the connection is established. The 3D finite element model of the building is presented in Figure 3.
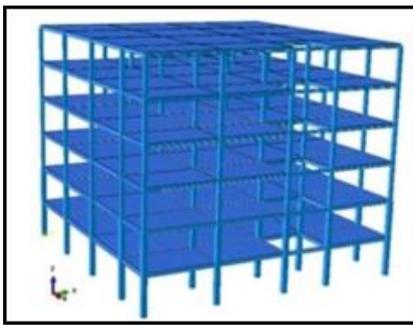

Fig. 3. 3D finite element model of the building

The assessment of progressive collapse potential was done by considering the alternative load path method. The general idea of this method is that the structure should be designed so that if the normal pathways of charge transfer are removed or damaged the other charge transfer alternative routes exist. Thus, the structures are designed for removing of columns or walls. Therefore, in this study the progressive collapse potential of the structural model was evaluated during 3 cases of column removal. The column removal scenarios are presented in Table 5.

Table 5
Column Removal Scenarios

Case	Column Location
1	Without Removal
2	E1 (Ground Floor)
3	D1 & E1 (Ground Floor)

-Loading
The definition of loads and boundary conditions, and applying them in finite element method is very important because the structural behavior can vary with the variation of the loading and boundary conditions. The applied loads on the structure consisted of the weight of structural components (i.e., beams, columns), and dead and live loads exerted on the floors. These loads are presented in Table 6.

Table 6
Values Of The Applied Dead And Live Loads

Location	Dead Load (Kg/m^2)	Live Load (Kg/m^2)
story	335	200
Roof	310	150

The load combination used in the nonlinear dynamic analysis for all of the cases considered in accordance with UFC guideline [6] is:

$$1.2D + 0.5L + 0.2W \qquad (0)$$

Guidelines do not recommend using the dynamic amplification factor for dynamic analysis. To carry out the dynamic analysis, the axial force acting on a column is computed before it is removed. Then, the column is replaced by point loads equivalent of its member forces as shown in Figure 4 and damping ratio was assumed to be 5% of the critical damping, which is usually adopted for analysis of structures undergoing large deformations. The progressive collapse analyses were carried out by removing a column in various locations in accordance with the UFC guideline [27].
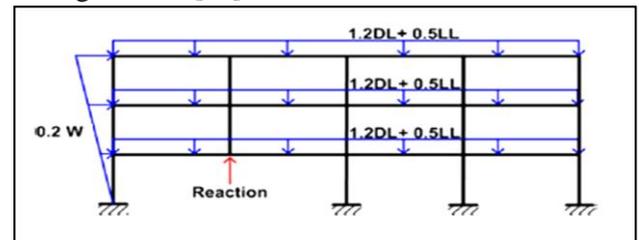

Fig. 4. Dynamic procedure [27]

3. Evaluation of Structural Response

The building response to the sudden removal of column was evaluated using the nonlinear dynamic analysis of a 3D finite element models. After modeling and analysis, the results in the form of axial force, bending moment diagrams and displacement values for each of the models were presented separately.

4

3.1. Results of the first case

The displacement of the structure in the vertical direction (U3), axial force, and bending moment diagrams in the first case of column removal scenarios are presented in Figures 5– 7, respectively. As shown in Figure 5, in case 1, the building was analyzed before removing the columns. This figure shows that the maximum displacement of the structure is equal to 10.85 centimeters. Figure 6 shows that column C2.

undergoes the maximum axial force, and Figure 7 shows that the maximum moment occurs in column D1.
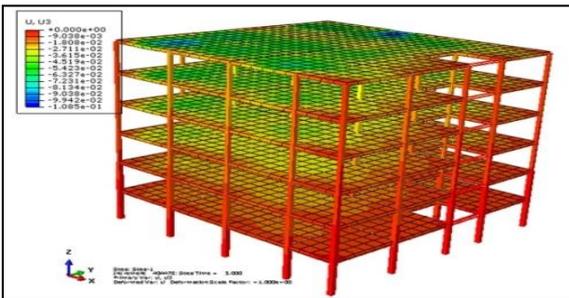
Fig. 5. Displacement of structure in the vertical direction (U3) in the first case
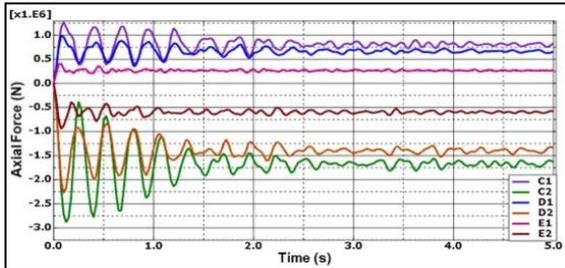
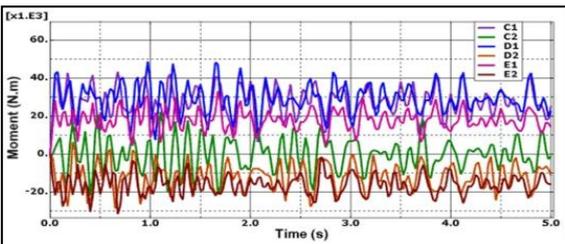Fig. 6. Axial force of the column in the first case

Fig.7. Moment of column D1 in the first case

3.2.Results of the second case

The displacement of the structure in the vertical direction (U3), axial force and bending moment diagrams in the second case of column removal scenarios are presented in Figures 8– 10, respectively. Figure 8 shows the case 2, which column E1 at ground floor was removed. It can be seen that, the maximum displacement of the structure is equal to 10.89 centimeters. Figure 9 shows that node C2 reached a peak axial force of

2800 KN, and then continued to vibrate . When the first column was removed, the redistribution of major moments in the adjacent columns was observed. In Figure 10, it can be seen that, the moment at column C1 reached a peak value after the removal of column E1.
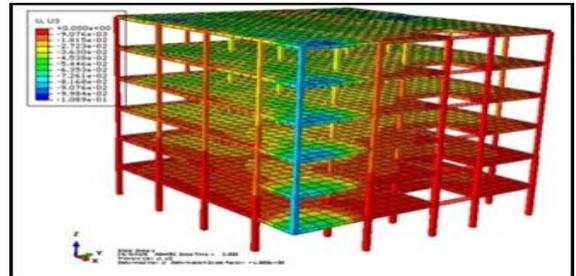

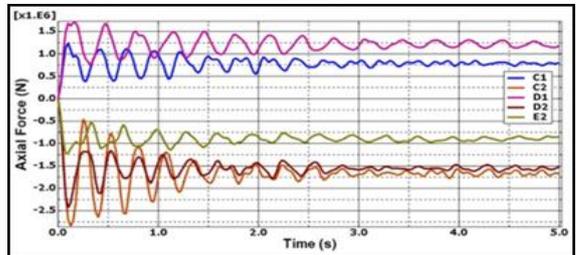Fig. 8. Displacement of structure in the vertical direction (U3) in the second case

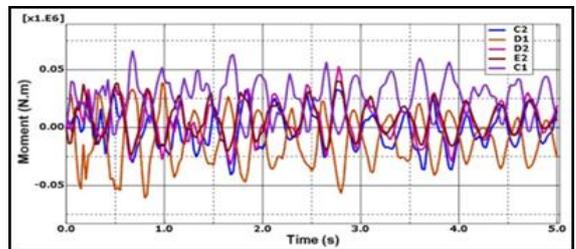Fig. 9. Axial force of the column in the second case

Fig. 10. Moment of the column in the second case

3.3. Results of the third case

The displacement of the structure in the vertical direction (U3), axial force and bending moment diagrams in the third case of column removal scenarios are presented in Figures 11– 13, respectively. As shown in Figure 11, in case 3, columns E1 and D1 at ground floor were removed, and it is shown that the maximum displacement of the structure is equal to 64.52 centimeters.

Figure 12 shows that node D2 reached a peak axial force of 3300 KN, and then continued to vibrate. When the first column was removed, a redistribution of major moments in the adjacent columns was observed. As shown in Figure 13, it can be seen that, the moment at column D2 reached a peak value after the removal of columns E1 and D1.
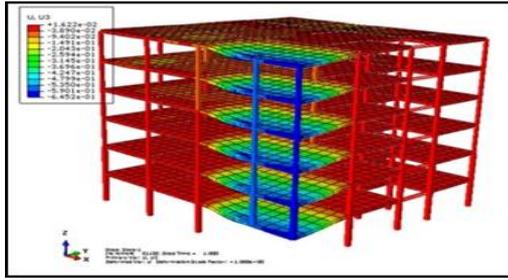
Fig. 11. Displacement of the structure in the vertical direction (U3) in the third case
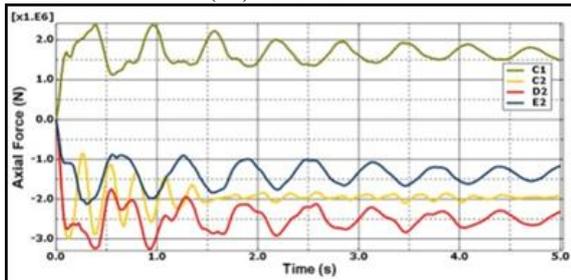

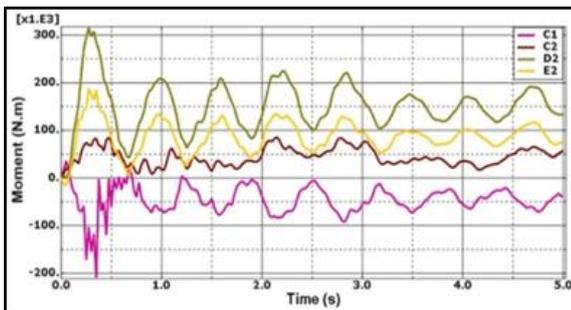Fig. 12. Axial force of the column in the third case

Fig. 13. Moment of the column in the third case

4.The Evaluation Results

4.1. The axial force criteria

According to Figure 14, in the 5-story building it can be seen that in the second case of column removal scenarios, when column E1 was removed, the greatest force was created in column C2, and when columns E1 and D1 were removed, more axial force was applied to column D2. For this reason, it can be concluded that the critical columns are those located in the nearest axis to the perimeter axis.
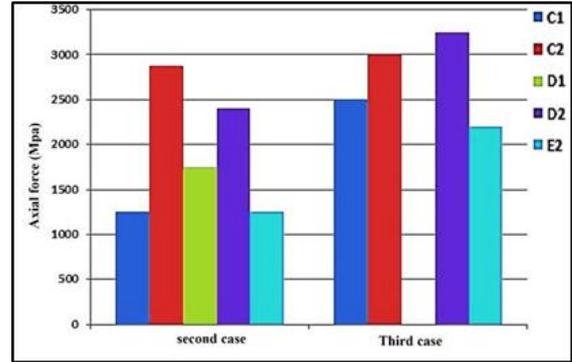

Fig. 14. The percent of change in axial force

4.2. The resistance criteria

One of the admission criteria in the alternative path method is the DCR criteria that is the ratio of demand to capacity. According to UFC 4-023-03 guideline, if the DCR ratio is larger than the 2, the member is damaged severely and will collapse, so by removing it from the model, the level of damage should be compared with the permissible values. The DCR values for the critical beams of the studied models were calculated and are shown in Figure 15. It can be seen that the DCR values for all of the beams are less than 2, so it can be inferred that these beams are in accordance with the specified criteria in the bylaws.
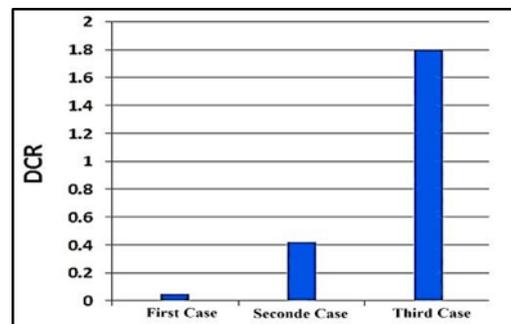

Fig. 15. DCR values of the model beams

5.Conclusion

The following results can be concluded from this study:

1) When a structure is subjected to unusual external loads such as a motor vehicle collision, explosion of a bomb in a vehicle, etc., the most critical columns are located in the nearest frame to the outer frame in the structure. So, the engineers should focus more on the resistant design against the progressive collapse, because it could be a key factor that has a significant role in reducing the progressive collapse

potential. In progressive collapse evaluation, when external columns are removed and the structure is damaged, the near columns to the external frame are critical

2) After removing the columns in different modes, they would split the loads to the adjacent members; hence, these members must have sufficient ability to withstand the additional forces. Therefore, the distribution of the forces in these members, before and after the column removal can be seen by monitoring the axial force values for adjacent members of the removed column. Because all of the members are designed to withstand the earthquake loads and non-interference of related loads (i.e., earthquake ground motion) with progressive collapse, even with the removal of the main load bearing members, other columns still have enough capacity to carry the existing loads.

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