



Effect of Initial Local Failure Type on Steel Braced Frame Buildings against Progressive Collapse

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ABSTRACT

In many studies, the alternative load path method (APM) has been used for progressive collapse analysis. In this method, one or several columns of the building are removed and the building response is investigated. This method ignores the initial local failure cause of building and this can affect the structural response. Investigation of ignoring the initial local failure cause of steel braced frames is the main purpose of this research. The variables include the type of progressive collapse investigation (ignoring the initial cause of failure or APM, considering blast loading and the heat caused by the fire as the initial causes of failure) and the location of the initial local failure in plan (outer and inner frame) and in floors (1st, 2nd and 3rd). 4-story braced steel buildings were simulated using ABAQUS software and the responses were compared using different methods. The most important results showed that the axial forces are very noticeable in the columns around the damaged site if the initial local failure caused by explosive loading; while these forces are ignored when APM is used. Therefore, due to this significant difference, if the design of a steel building is to be considered against progressive collapse, it is recommended to consider the initial local failure in order to make the appropriate design in accordance with it. Therefore, the initial loading type has a very significant effect on the structure response, and ignoring the initial local failure can lead to incorrect predictions of the structure response.

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1. INTRODUCTION

According to the American Society of Civil Engineers (ASCE), progressive collapse occurs when an initial failure spreads in a structure from one element to another, eventually leading to the collapse of a total structure or a main part of it [1]. The reasons that can lead to progressive collapse can be vehicle impact, gas explosion, aircraft accidents, construction errors, fire load, blast events and etc. Most of these events have a short duration of impact which leads to structural dynamic response. In traditional structural design codes, the progressive collapse was considered indirectly with the definition of importance degree for structures; but recently, regulations have been developed about the progressive collapse of structures. The most acceptable of these regulations is Unified Facilities Criteria (UFC

[2] and General Services Administration (GSA) [3]. In progressive collapse, loads of damaged members transfer to adjacent members. Now if the adjacent members do not have the ability to withstand against this overload, they will fail and all or parts of the structure will destroy [4, 5]. In order to reduce these risks, it is necessary to design and analyze the structure against this destructive phenomenon. Alternative load path method (APM) is one of the methods for assessing the progressive collapse, which is done directly by removing one or more columns [1-3]. However, in this method, the damage that can be inflicted by the explosion and fire on adjacent columns is not considered, which could lead to a false prediction of progressive collapse. The mentioned reasons can damage beams and columns of structure and even cause them to collapse. Therefore, in this study, ignoring the initial local failure cause of steel braced frames are

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investigated. For this purpose, the finite element method (FEM) is used with ABAQUS [6] software.

In recent years, there have been many studies on progressive collapse and each of them has examined part of this event. Almusallam et al. [7] investigated the precast concrete beam-column joints against progressive collapse. The results showed that the joints have a considerable impact on the response of the structures against progressive collapse. Gao and Guo [8] investigated a 20-story building against progressive collapse and the effect of composite floor was evaluated. The results showed that the overall behavior of the structure depends on the position of the column removal. Gernay and Gamba [9] examined the progressive collapse caused by the fire. The results showed that fire-induced heat has a significant influence on the structural response against progressive collapse. Li et al. [10] studied the behavior of steel frames against progressive collapse. ABAQUS software was used for this purpose. The results showed that the type of steel frame (for example, weak beam strong-column or vice versa) and the location of column removal has important role on the strength of the steel frame against progressive collapse. Lou et al. [11] investigated the behavior of portal frames against progressive collapse caused by fire. The results showed that corner and outer columns have an important role in preventing the spread of collapse to adjacent buildings. Zhu et al. [12] investigated the progressive collapse of semi-rigid steel frames. Finally, an equation presented for the dynamic increase factor. This equation is a function of the energy demand and energy capacity ratio. Tavakoli and Kiakojoori [13] presented a new method for simulating the dynamic removal of columns in steel frames. They evaluated the structural response of a five-story steel building using this method in various column removing steps. The nonlinear effects of materials were also considered. The results indicated that the potential of progressive collapse is essentially dependent on the column removal position. Kaafi and Ghodrati Amiri [14] simulated a 5-story steel building with and without considering slab using ABAQUS software and the potential of progressive collapse was evaluated. The results showed that the lack of considering the slab during computation can lead to errors in evaluating the potential of progressive collapse in the structure. Elshaer et al. [15] evaluated the progressive collapse of multistory structures under the earthquake. The nonlinear dynamical analysis was performed and various parameters such as position of column removal were investigated. The results showed that removing the edge columns creates a more critical state than the corner columns. Hosseini and Ghodrati Amiri [16] investigated the potential of progressive collapse in steel frames with buckling restrained eccentric braces. For this purpose, APM was used and 7 buildings with five-

story were evaluated in different columns removal scenarios. The results showed that removing the corners columns leads to more critical responses in structures with ordinary and eccentric buckling restrained braces. Faghihmaleki et al. [17] evaluated progressive collapse in steel frames with different braces. To study the progressive collapse of structures against earthquake, three steel moment frame buildings with 8-story and concentric, eccentric and buckling restrained braces were selected. Several structural members removed and the effect of these scenarios on the dynamic behavior of structures during the earthquake was investigated. The potential and capacity of buildings for progressive seismic collapse and the failure modes were determined using incremental dynamic analysis. Pourasil et al. [18] proposed a procedure to evaluate the progressive collapse of blast loading in steel structures with moment frames. For this purpose, a 7-story steel building was simulated under blast loading, and the pressures equivalent to blast load were applied in four different states to structural members near the blast. The results showed that the potential of progressive collapse during considering blast loading as the cause of failure will be different compared to common methods used for investigating the progressive collapse.

In most of these studies, a number of structural columns were removed under different conditions and the structural response was evaluated under new conditions (the lack of a number of columns). The APM without considering the load type is relatively simple but it has less accurate and it doesn't seem to have reliable predictions. In general, the defect of APM is regardless of initial failure or damage of adjacent structural members due to the blast or fire load. According to the defect of APM, the used method in this study may predict more reliable predictions of progressive collapse caused by blast and fire loads. So that, more realistic responses will be investigated by considering the initial failure. In other words, the most important purpose of the present study is to compare three different methods for the analysis of braced steel buildings against progressive collapse. In the first method, the blast load is applied directly to the structure. In the second method, the initial cause of fire collapse is heat caused by fire and in the third method, the direct column removal method (APM) is used.

2. STUDY PROCEDURE

At first, a four-story braced steel building is simulated, analyzed and designed using ETABS software [19] based on sixth and tenth issues of national building regulations of Iran. The studied models have the same plan in floors and their height is 3.20 meters. The dimensions of all spans and staircase are 6 and 4 meters,

respectively. The lateral force resisting system in both directions is a simple frame with concentric diagonal braces. In the next step, ABAQUS software [6] is used to analyze four-story steel frames against the progressive collapse potential due to explosive loading and the load caused by fire. 3D finite element models of frames are simulated and the structure response is compared with each other in three types of loading conditions (Blast, Fire, and APM); so that the impact of loading type on the evaluation of progressive collapse can be examined in final behavior of frames. The studied modes are presented in Table 1. The variables in the present study include the type of initial loading (regardless of the initial cause of failure, blast loading and the heat caused by the fire), the location of the initial local failure in plan (outer and inner frame) and floors (1st, 2nd, and 3rd).

TABLE 1. Variables

Case	Initial local failure location of columns		Initial local failure type (Method of investigation)	Contraction
	Frame location	Story		
1	----	----		Without Removal
2	Inner	First		St1-I-B
3	Inner	Second	Blast	St2-I-B
4	Inner	Third		St3-I-B
5	Outer	First		St1-O-B
6	Inner	First		St1-I-F
7	Inner	Second		St2-I-F
8	Inner	Third		St3-I-F
9	Outer	First	Fire	St1-O-F
10	Outer	Second		St2-O-F
11	Outer	Third		St3-O-F
12	Inner	First		St1-I-APM
13	Inner	Second	Column removal (APM)	St2-I-APM
14	Inner	Third		St3-I-APM
15	Outer	First		St1-O-APM

TABLE 3. Geometric properties of the sections

	Designation	Mass/meter (kg/m)	Depth (mm)	Width (mm)	Web thickness (mm)	Flange thickness (mm)
Beam section	IPE300	42.2	300	150	7.1	10.7
	IPE270	36.1	270	135	6.6	10.2
	Designation	Mass/meter (kg/m)		Width (mm)		Thickness (mm)
Column section	Box350×350×16	166		350		16
	Box300×300×16	141		300		16
	Box 200x200x20	1.18		200		20

16	Outer	Second	St2-O-APM
17	Outer	Third	St3-O-APM
Without Remove: Without column removal			I: Inner frame
St1: Column removal at first story			O: Outer frame
St2: Column removal at second story			B: Blast load
St3: Column removal at third story			F: Fire load
APM: Alternative load path method			

The sections are designed in a way that the stress ratios are as close as possible to each other so that different modes can be compared correctly. The studied building is residential and the earthquake loads were obtained with the assumption that the building is located in seismic zone 4 of Iran. The structural design results are presented in Table 2. Details of the used sections are also shown in Table 3. It should be noted that the design is done in several steps, on one hand, the selection of sections is close to optimal mode and on other hand, the design of components is simple and uniform. The general process of the study is shown in Figure 1.

3. FINITE ELEMENT SIMULATION

Elements of the structures in this study include beams, columns, and braces. These elements are 3D and deformable. The characteristics of the used materials in this study are steel and concrete. Beam elements were used to model the beams, columns, and braces. These elements are 3D and deformable. Concrete damage plasticity model was used to define concrete. This

TABLE 2. Design results

Story	Beam	Column	Brace
First	Box 350×350×20	2IPE 300	Box 200x200x20
Second	Box 350×350×20	2IPE 300	Box 200x200x20
Third	Box 300×300×20	2IPE 270	Box 200x200x20
Fourth	Box 300×300×20	2IPE 270	Box 200x200x20

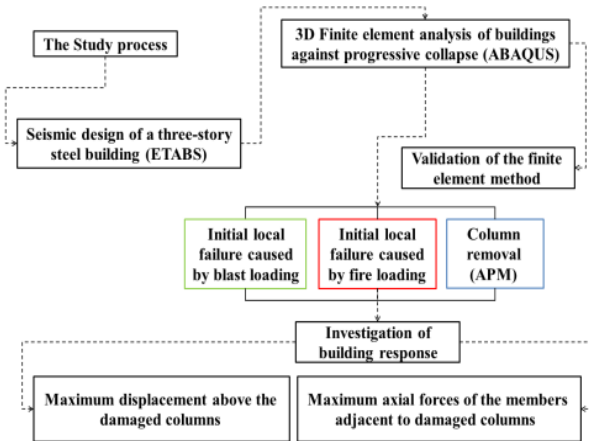


Figure 1. Study process flowchart

model considers two main assumptions in the failure mechanism, which includes tensile cracking and compression crushing. Dynamic explicit analysis was used to analyze the models. This analysis is explicitly used to design large scale models with short dynamic response durations and for processes with high discontinuities. This type of analysis allows us to use large deformations theory. Tie constraint has been used to define the interaction between members. This allows the user to combine two surfaces that their meshing is different from each other. In order to determine the optimal meshing in modeling, examine the convergence response method was used. The proposed meshing is reliable enough to ensure that the applied forces were accurately calculated. The imposed loads on building include the weight of structural elements (beam, column, floor, and brace), dead and live loads on the floor of structure. Three different methods were used to investigate the progressive collapse:

3. 1. Alternative Load Path Method

APM

analysis is a sophisticated analysis method that can be used to determine the potential for progressive collapse in a given facility. The applied loads to studied models are included [2, 3]:

1. Increased gravity loads for the floor above removed column: This load combination should be affected by gravity loads on the adjacent spans of removed elements at all top floors of these elements.

$$G_{LD} = \Omega_{LD} [(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S)] \quad (1)$$

where, G_{LD} is increased gravity loads for deformation controlled actions in linear static method, D is dead load including facade load and L is live load including live load reduction factor [1] and S is snow load and Ω_{LD} is load increased factor for deformation controlled actions in linear static method.

2. Gravity loads for floor areas away from the removed column:

This load combination should be affected on spans that are not loaded with G_{LD} .

$$G_{LD} = [(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S)] \quad (2)$$

where, G is gravity load. Loads and load locations for external and internal column removal according to UFC is shown in Figure 2. Also Locations of removed columns in this study for APM is shown in Figure 3.

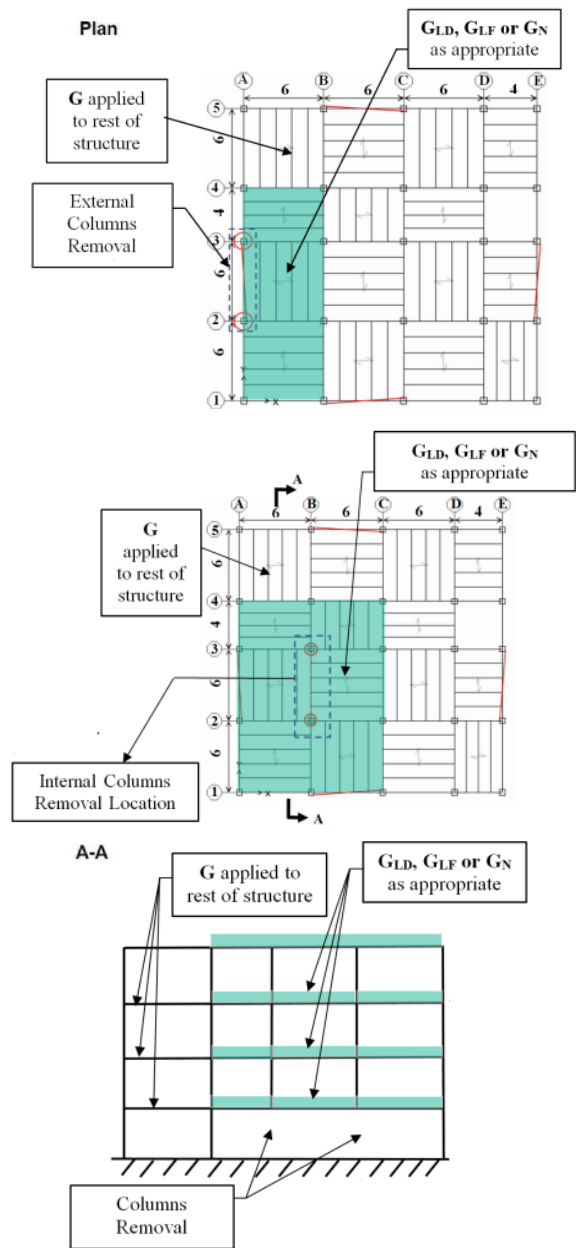


Figure 2. The applied loads for external and internal columns removal (APM) [2]

Column removal modes of APM were investigated in six modes in the first, second and third story of inner and outer frames, respectively. In the dynamic analysis of progressive collapse, both the UFC [2] and GSA [3] regulations do not recommend the use of dynamic load increase factor. For nonlinear dynamic analysis, the axial reaction of the column is calculated before the removal and then, during the process of removing the columns, these concentrated axial forces must be replaced with removed columns. Figure 4 shows the gravity loads and the supported reaction of removed columns. This loading pattern is in accordance with the UFC Code [2].

3. 2. Blast Loading Numerical simulation of buildings against the explosion is done using ABAQUS Software. Simulation of explosion was carried out on models using the CONWEP option. This is an experimental model of Kingery and Bulmash [20], and is the result of many explosive tests on a wide range of explosive charges. The considered blast was an airblast type. The explosive loading intensity applied to the building is equivalent to 500 kilograms of TNT and is 6 meters from the explosion center. For this purpose, according to Hopkinson-Cranz law [21, 22], by determining the scaled distance (Z) from Equations (3) to (5), the overpressure generated by the explosion can be calculated.

$$Z = \frac{R}{W^{\frac{1}{3}}} \tag{3}$$



Figure 3. Locations of removed columns in this study (APM) a: Outer frame b: Inner frame

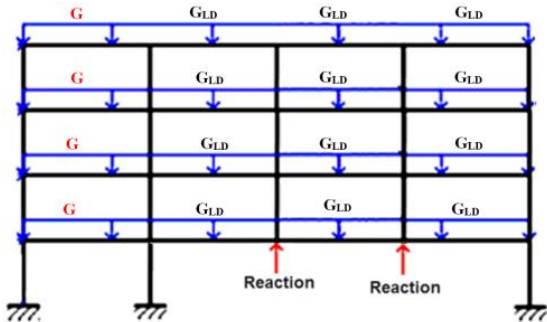


Figure 4. Column reaction and applied loads according to UFC [2]

$$X = \log_{10}(Z) \tag{4}$$

$$\log_{10}(Z)[\log_{10} P_s] = -1319X^2 - 0.2331X + 0.4644 \tag{5}$$

where, R is the distance from the detonation source to the point of interest (m) and W is the weight (more precisely: the mass) of the explosive (kg). The Friedlander explosive load equation is used in this study. This equation is one of the most accurate and complete numerical solutions for the explosion wave (Equations (6) and (7)). Where, $p_{r,max}$ is the maximum overpressure, t_d is the positive phase duration and b is a decay coefficient of the waveform [23]. These parameters are shown in Figure 5. The position of explosive charge of the outer and inner frame is shown in Figure 6.

$$p_r(r) = p_{r,max} \left(1 - \frac{t}{t_d}\right) e^{-bt/t_d} \tag{6}$$

$$p_{r,max} = p_0 \frac{808 \left[1 + \left(\frac{Z}{4.5}\right)^2\right]}{\left\{ \left[1 + \left(\frac{Z}{0.048}\right)^2\right] \left[1 + \left(\frac{Z}{0.32}\right)^2\right] \left[1 + \left(\frac{Z}{1.35}\right)^2\right] \right\}^{0.5}} \tag{7}$$

3. 3. Fire Loading Fire is applied to the column after applying axial loads. It is necessary to select and apply a fire curve to simulate the effect of fire on the

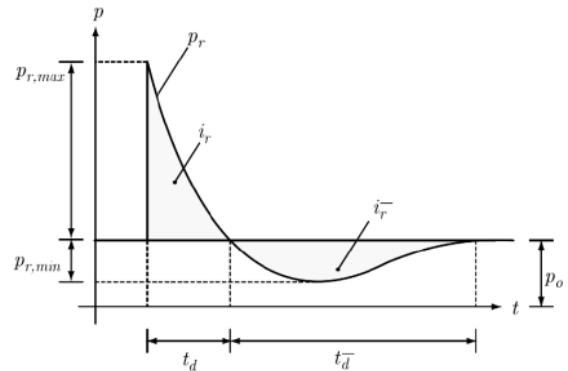


Figure 5. The ideal blast wave's pressure-time history [18, 22]



Figure 6. The position of explosive charge a: Outer frame b: Inner frame

structure. The fire curve is a temperature-time curve that shows the gradual change of temperature over time to indicate the environment in which the structure is located.

The ISO 834 fire curve [24] was used to apply fire to the members (Equation (8)). Fire curve refers to changes in temperature and heat caused by local fire to adjacent members of the column removal location in accordance with ISO834 is shown in Figure 7. The position of the damaged columns in the fire is shown in Figure 8.

$$T = \text{Log}_{10}(8t + 1) + T_0 \quad (8)$$

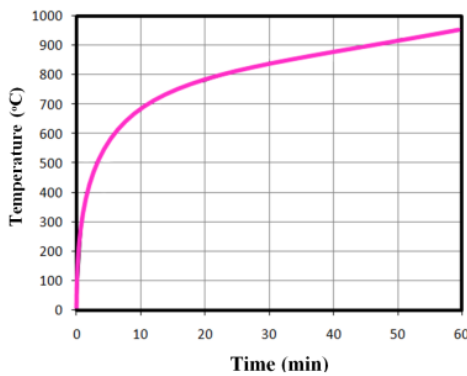


Figure 7. ISO834 Temperature-Time curve caused by local fire [23]

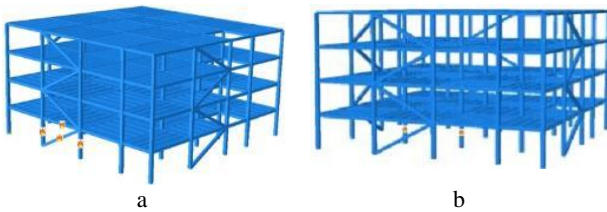


Figure 8. The position of initial failure caused by fire loading a:Outer frame b: Inner frame

4. VALIDATION

Three different experimental studies are utilized in order to validate the FEM used in the modeling of APM (loss of column), blast and fire loading. The accuracy of the used method to simulate the behavior of APM is investigated using Guo et al. [25]. The studied frame was a one story composite frame with four spans, which was built with a 1/3 scale in the laboratory (Figure 9). The length of the frame spans is 2 meters and its height is 1.20 meters. The beam section is H200×100×5.5×8 and the cross-section of the columns is H200×200×8×12. The depth and width of the slabs are 100 and 80 mm, respectively. The reinforcement mesh ratio for

RC slab was considered 0.85%. In order to simulate the column removal, the middle column was not supported. Material properties are shown in Table 4.

The behavior of the frame, as well as the concrete slab during the test, were evaluated. The static vertical load was applied at the top of the middle column. More information about the material and the details of the test can be found in Guo et al study [25]. The frame investigated in Guo et al. [25] test is simulated using the FEM used in the present study (Figure 9). Load-displacement values of the frame made in Guo et al. test and the simulated finite element model are compared in Figure 10. As it is seen, the force and deformation values corresponding to the experimental and numerical samples are in good agreement.

Figure 11 shows comparison of load-displacement curves of simulation and experimental results. The second validation of the FEM in modeling of blast loading on steel members is performed using Lawver et al. [26] study. In the mentioned study, seven steel columns were exposed to the different amount of

TABLE 4. Material properties [25]

Section		f_y (MPa)	F_u (MPa)	E_s (10 ⁵ MPa)
Beam	Flange	269	401	1.96
	Web	275	411	2.09
Column	Flange	247	396	2.00
	Web	276	415	1.98
Reinforcement	8T	325	487	-
	12T	331	464	1.95



Figure 9. Vertical displacement of the tested frame in the laboratory [27]

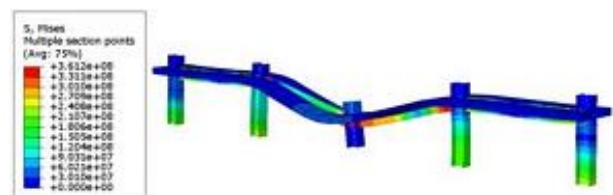


Figure 10. Vertical displacement of the simulated frame using FEM

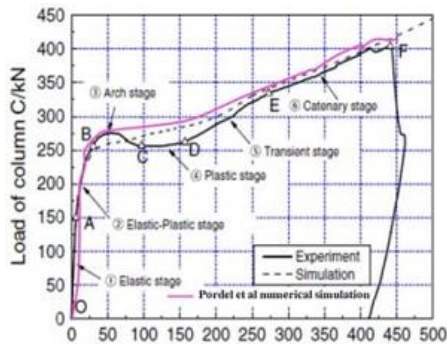


Figure 11. Comparison of load-displacement curves of simulation and experiment

explosive loads and only the details of the column with a cross-section of W360×122, which was exposed to a close explosion loading, were presented. The height of the column was 4.30. The mentioned column was loaded simultaneously with an axial load of 670 N and a blast load in the lateral direction. The column is connected to a concrete foundation through a baseplate and 8 bolts. The column was instrumented with accelerometers and pressure transducer. An overview of the experimental test and numerical model is presented in Figures 12 and 13. Also, the comparative graph of the lateral displacement of experimental results and numerical simulation is presented in Figure 14. The simulation results showed a good agreement with experimental results.



Figure 12. Single column tested [26]

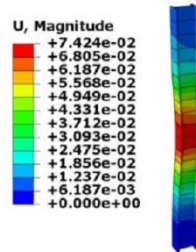


Figure 13. Finite element model of column

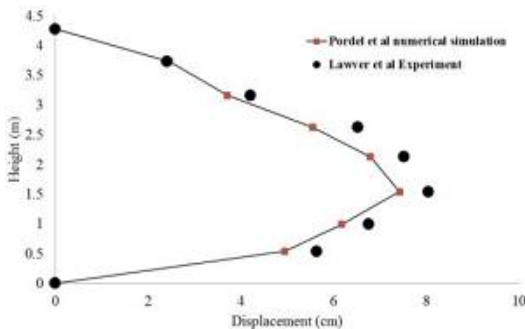


Figure 14. Comparison of experimental results and numerical simulation

Also, the effect of fire loading on the steel building response is one of the most important purposes of the present study; therefore, in order to validate the used method in simulating fire loading, an experimental study that was performed by Jiang et al. [27] is used. As it was shown in Figure 15, the tested frames had two stories and four spans. The lengths of central and side spans were 2.2m and 2m, respectively. The height of the first and second story was 1.3 and 1.2 meters, respectively. The dimensions of the steel members with tubular rectangular sections for columns and beams are 30×3×50 cm and 40×60×60 cm, respectively. The central column located on the first floor was heated by an electric furnace. The initial axial force of the heated column was between 9.8 kN, 14.0 kN and 20.9 kN for Frames 1, 2 and 3, respectively. Plastic strain due to fire in the center column of Frame 1 is shown in Figure 16. During the tests, parameters such as temperatures, displacements, and strains were measured. Before turning off the furnace and cooling down the column, the furnace gas temperature increased to 950, 829 and 735°C for Frames 1, 2 and 3, respectively. The temperatures measured in the section of heated columns were almost the same. Further information of the tests is presented in Re [28]. The model is established by the same modeling techniques mentioned in this paper. Comparison of measured and computed temperatures in the heated column is presented in Figure 17. The predicted temperature by the model is in good agreement with the experimental result.



Figure 15. Deformations of test frame after unloading [27]

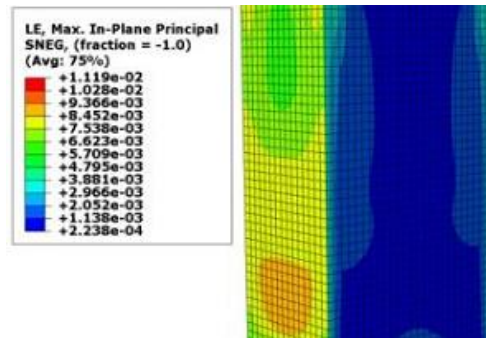


Figure 16. Plastic strain due to fire in the center column of Frame 1

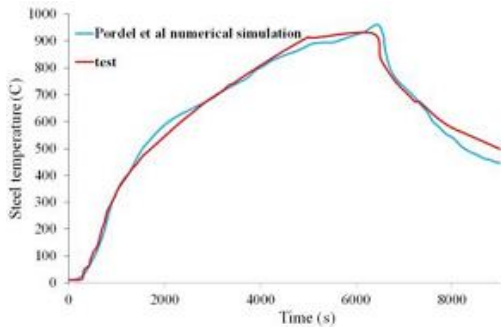
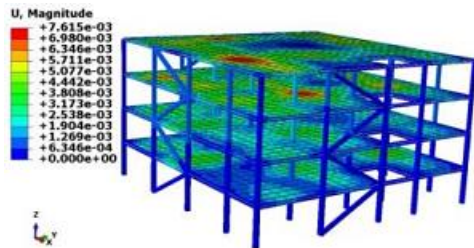


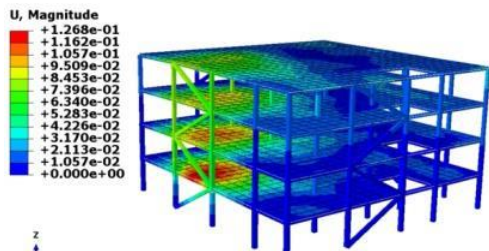
Figure 17. Comparison of measured and computed temperatures in the heated column

5. ANALYTICAL RESULTS

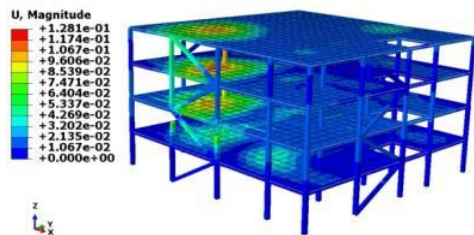
In this section, the results have been analyzed. For this purpose, the displacement of the column removal area, the axial force values of the columns adjacent to the column removal location, and the values of the angle of rotation of the beam to the column are compared in different states. For this purpose, the displacement of the columns that were subjected to initial failure and the axial forces of the columns adjacent to the location of the initial local failure are compared.



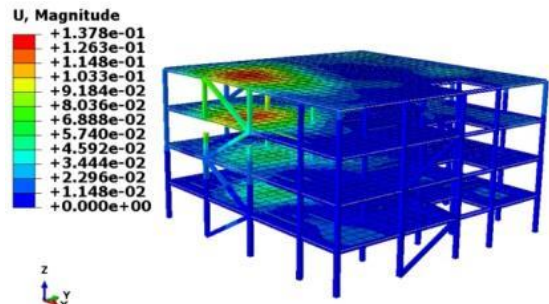
Without Removal



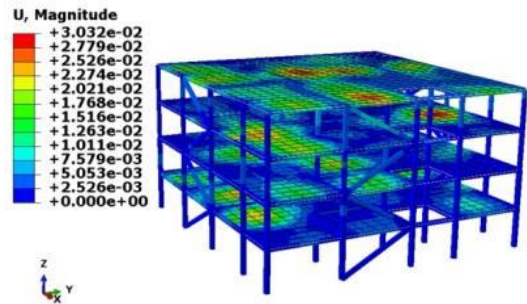
St1-I-B



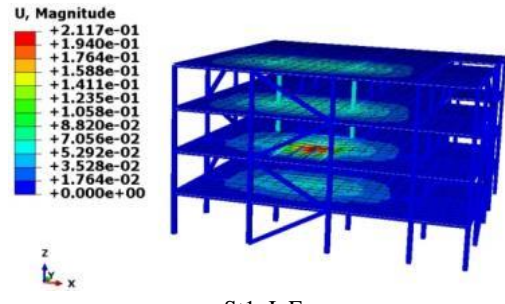
St2-I-B



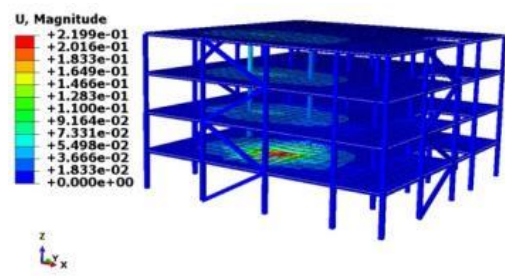
St3-I-B



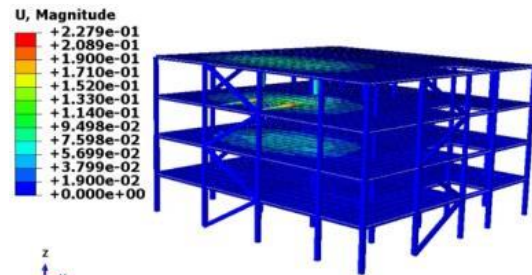
St1-O-B



St1-I-F



St2-I-F



St3-I-F

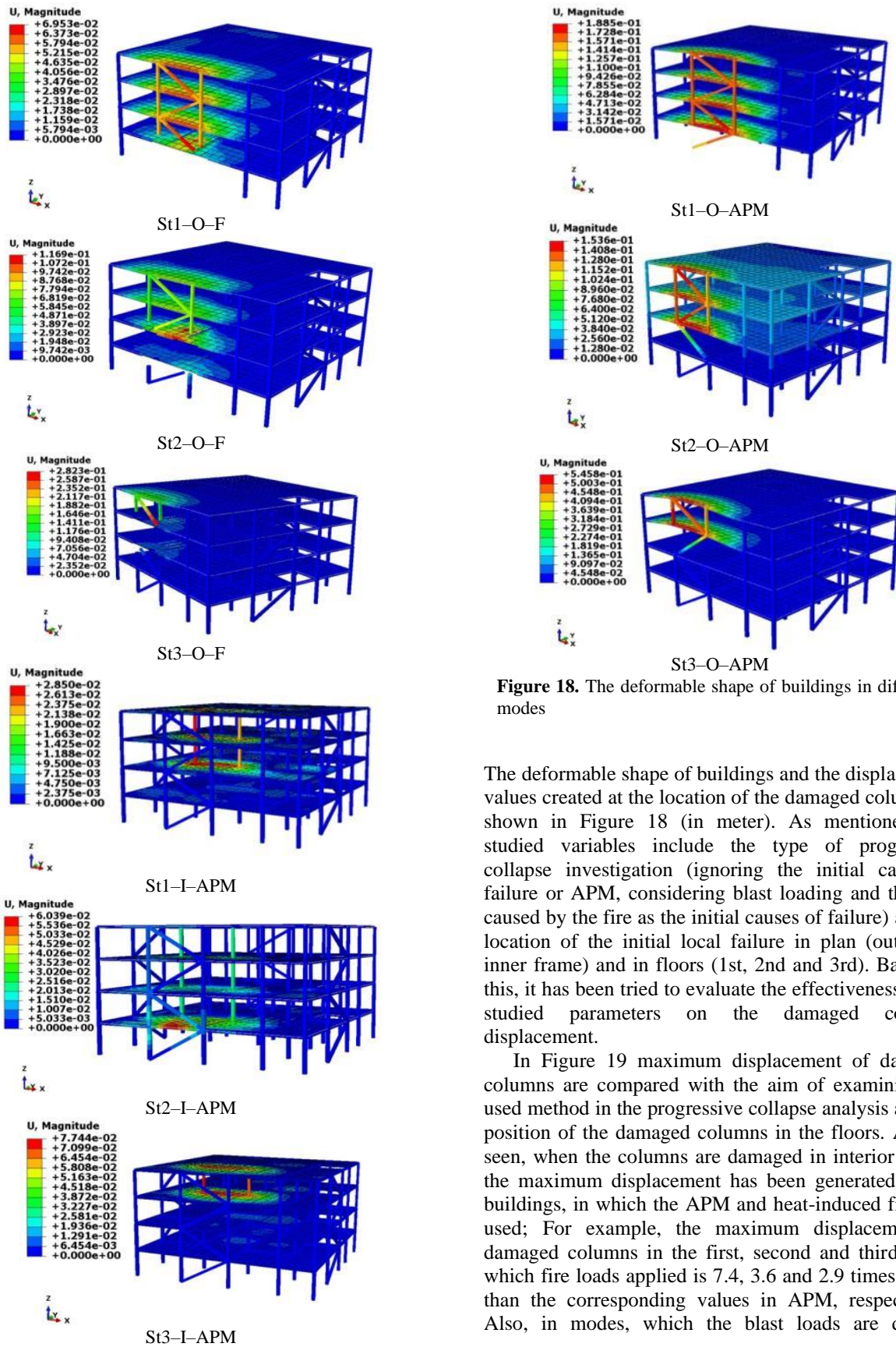


Figure 18. The deformable shape of buildings in different modes

The deformable shape of buildings and the displacement values created at the location of the damaged columns is shown in Figure 18 (in meter). As mentioned, the studied variables include the type of progressive collapse investigation (ignoring the initial cause of failure or APM, considering blast loading and the heat caused by the fire as the initial causes of failure) and the location of the initial local failure in plan (outer and inner frame) and in floors (1st, 2nd and 3rd). Based on this, it has been tried to evaluate the effectiveness of the studied parameters on the damaged columns displacement.

In Figure 19 maximum displacement of damaged columns are compared with the aim of examining the used method in the progressive collapse analysis and the position of the damaged columns in the floors. As it is seen, when the columns are damaged in interior frame, the maximum displacement has been generated in the buildings, in which the APM and heat-induced fire was used; For example, the maximum displacement of damaged columns in the first, second and third floors which fire loads applied is 7.4, 3.6 and 2.9 times higher than the corresponding values in APM, respectively. Also, in modes, which the blast loads are directly

applied to the columns and their adjacent structural members located on the inner frame, the maximum displacement of damaged columns in the first, second and third floors is 66, 71 and 65 percent higher than the corresponding values in APM, respectively. According to mentioned values, it can be concluded that in the APM, which the initial failure does not consider, the maximum displacement of damaged columns in the inner frame is not correctly estimated. So that the displacement values obtained by the APM have a significant difference with the displacement values obtained from other methods. Despite the fact that the maximum displacement of damaged columns located in the outer frame is completely different from the corresponding values in the inner frame and the APM exhibits more displacement compared to the blast and fire loading methods. So, it can be stated that depending on the position of the damaged column in the internal and external frames, the initial cause of failure in structure can create different responses.

On the other hand, according to Figure 20, it can be seen that in all three methods, causing damage in the columns located on the upper floors leads to more displacement. In other words, removing the columns and causing local damage on the upper floors is much more dangerous and can make the behavior of the structure more critical.

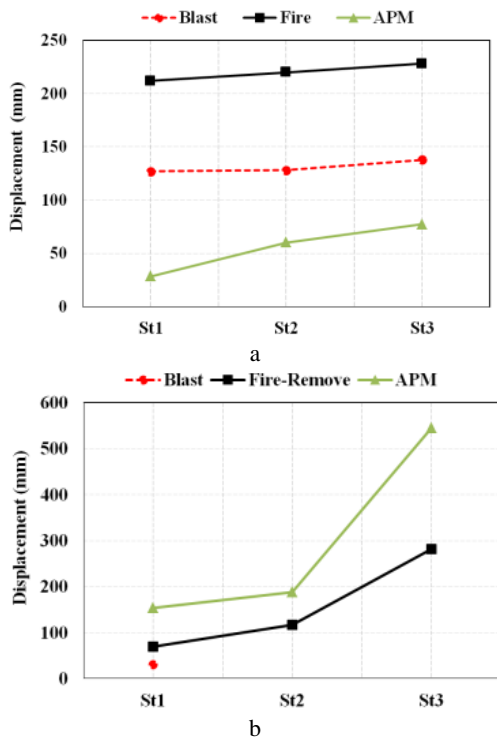


Figure 19. Maximum values of displacement generated at the damaged columns in different modes with the aim of examining the used method in progressive collapse analysis and the damaged column location a: Inner frame b: Outer frame

In Figure 20 maximum displacement of damaged columns is compared with the aim of examining the position of them in outer and inner frames. By considering explosion and fire as the initial cause of failure, the displacement of the damaged members placed in the inner frame in most cases is greater than the corresponding values of the outer frame. This is while the maximum displacement of the removed columns in the outer frame is much greater than the displacement of the columns removed in the interior frame when APM is used. So it can be concluded that the assumptions of each three used methods in the present study to predict the behavior of the structure against progressive collapse can provide different responses. As the APM states that the column removal in the outer frames creates more displacements; this is while applying fire and explosion as the initial local damages in the internal frame creates more displacements.

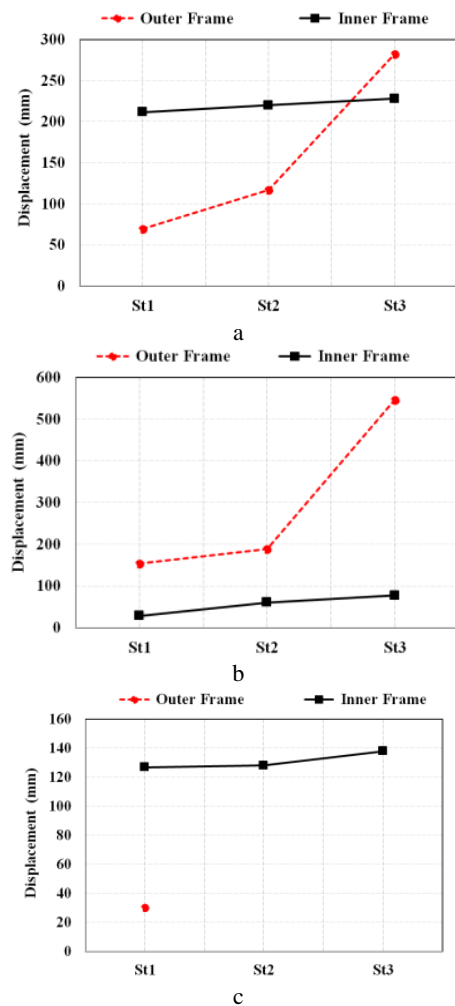


Figure 20. Maximum values of displacement generated at the damaged columns in outer and inner frames a: Fire loading b: APM c: Blast loading

One of the proposed criteria in the progressive collapse analysis is the force criterion that has been evaluated in this section. After removing the columns in different modes, the load is distributed across the adjacent members, and these members must have the ability to withstand excessive force [14]. For this purpose, the force redistribution of members can be seen using distributing axial forces of the columns adjacent to the removal site before and after removing columns.

In Figure 21 maximum axial forces of columns adjacent to damaged columns are compared with the aim of examining the used method in the progressive collapse analysis and the position of the damaged columns in the floors; As it is seen, when the explosion load affects the two columns and their adjacent members in inner frame the axial forces of the adjacent damaged columns are more than the corresponding values in APM and fire loading.

According to Figure 22, applying the blast load to the columns in the inner frame has led to increase the maximum axial force around the initial local failure to 317, 318, and 230 percentages in the first, second and third floors, respectively. However, in the APM, removing two columns in the inner frame has led to increase the maximum axial force around column removal site to 15, 25, and 58 percentages in the first, second and third floors, respectively. Comparison of the axial force values obtained from the three methods used indicates that if the initial failure reason is explosive loading, the axial forces of columns around the removal site are very noticeable; which these forces are ignored in APM. Therefore, due to this significant difference, it is recommended to be considered the initial cause of failure during progressive collapse analysis of steel braced frame buildings; thereby building designing against progressive collapse will be accordance with the initial cause of failure. Since force transferring of the remaining structural members that have been affected by the local failure can certainly prevent progressive collapse, and if the correct prediction of failure is not initialized, the structure response against progressive collapse potential will not be appropriate. On the other

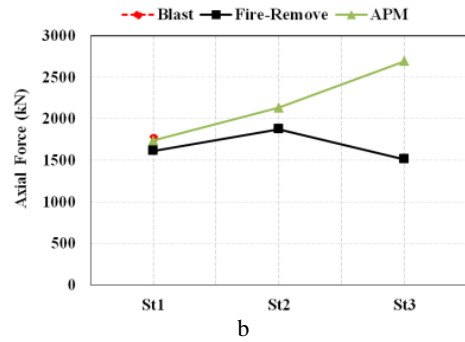
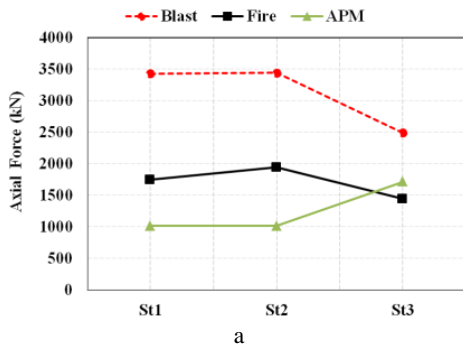


Figure 21. Maximum values of axial forces generated at the damaged columns in different modes with the aim of examining the used method in progressive collapse analysis and the damaged column location a: Inner frame b: Outer frame

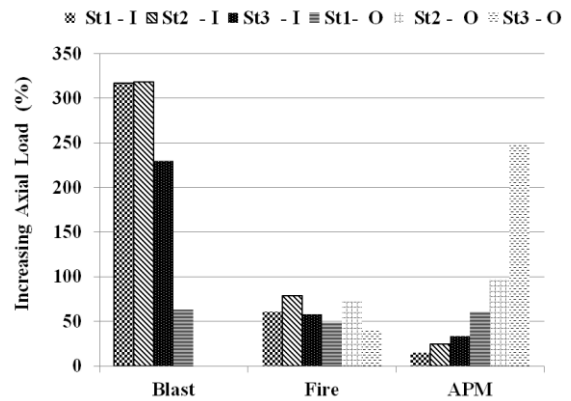


Figure 22. Percentage of axial force increase in the columns around the location of the local failure

hand, it is observed that applying the heat-induced by fire to the inner columns and their adjacent members of frames has caused more axial forces in the columns around the studied area compared to APM (column removal).

6. CONCLUSIONS

In this study, the effect of the initial local failure type on steel braced frame buildings against progressive collapse was investigated. For this purpose, variable parameters include the type of initial loading (regardless of the initial cause of failure, blast loading and the heat caused by the fire), the location of the initial local failure in plan (outer and inner frame) and floors (1st, 2nd and 3rd). A 4-story braced steel building was simulated using ABAQUS software and its response was compared using different methods. Also, the accuracy of the used FEM in simulating the models was evaluated and a suitable agreement between the results

was observed. The most important results showed that in the APM, which the initial failure does not consider, the maximum displacement of damaged columns in the inner frame is not correctly estimated. So that the displacement values obtained by the APM have a significant difference with the displacement values obtained from other methods. On the other hand, it can be stated that depending on the position of the damaged column in the internal and external frames, the initial reason for local failure in structure can create different responses.

Also, comparison of the axial force values obtained from the three methods used indicates that if the initial failure reason is explosive loading, the axial forces of columns around the removal site are very noticeable; which these forces are ignored in APM. Therefore, due to this significant difference, it is recommended to be considered the initial reason of failure during progressive collapse analysis of steel braced frame buildings; thereby building designing against progressive collapse is accordance with the initial reason of failure. damages in the internal frame creates more displacements.

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Effect of Initial Local Failure Type on Steel Braced Frame Buildings against Progressive Collapse

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در بسیاری از مطالعات، از روش مسیر بار جایگزین برای تحلیل خرابی پیش رونده استفاده می‌شود. در این روش یک یا چند ستون از ساختمان حذف می‌شوند و پاسخ ساختمان مورد ارزیابی قرار می‌گیرد. این روش عامل اولیه ایجاد خرابی در سازه را نادیده می‌گیرد و این موضوع می‌تواند پاسخ سازه را تحت تاثیر قرار دهد. بررسی روش مسیر بار جایگزین در نادیده گرفتن عامل اولیه ی ایجاد خرابی پیش رونده در قاب های فولادی مهاربندی شده به عنوان مهم ترین هدف مطالعه حاضر می‌باشد. متغیرهای مورد بررسی به ترتیب شامل نوع بارگذاری موضعی اولیه (بدون در نظر گرفتن علت اولیه خرابی، بارگذاری انفجار و حرارت ناشی از آتش سوزی)، موقعیت ایجاد خرابی موضعی اولیه در پلان (قاب بیرونی و داخلی) و در طبقات (طبقات ۱، ۲، ۳) می‌باشند. ساختمان‌های فولادی مهاربندی شده ۴ طبقه با استفاده از نرم افزار ABAQUS شبیه سازی شدند و پاسخ با استفاده از روش‌های مختلف با یکدیگر مقایسه شد. مهم ترین نتایج حاصل نشان می‌دهد در صورتیکه عامل ایجاد خرابی موضعی اولیه بارگذاری انفجاری باشد، نیروهای محوری بسیار چشمگیری در ستون‌های اطراف محل حذف به وجود می‌آید؛ که این نیروها در هنگام استفاده از روش APM نادیده گرفته می‌شود. از این رو و با توجه به این تفاوت چشمگیر توصیه می‌شود چنانچه طراحی یک ساختمان فولادی مهاربندی شده در برابر خرابی پیش رونده مورد نظر باشد، عامل اولیه خرابی در نظر گرفته شود تا متناسب با آن عامل، طراحی‌های لازم صورت پذیرد؛ زیرا مسلماً انتقال نیرو توسط اعضای سازه‌ای باقیمانده ساختمانی که دچار خرابی موضعی شده است، می‌تواند از خرابی پیش رونده جلوگیری نماید و چنانچه پیش‌بینی درستی از خرابی اولیه نشود، پاسخ سازه در برابر خرابی‌های احتمالی نیز مناسب نخواهد بود. بنابراین می‌توان بیان نمود که نوع بارگذاری اولیه نقش بسیار تاثیر گذاری بر پاسخ سازه دارد و عدم در نظر گرفتن خرابی اولیه می‌تواند منجر پیش‌بینی‌های نادرستی از پاسخ سازه شود.

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