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Evaluating the Effect of Buckling-Restrained Braces in Steel Buildings Against Progressive Failure using Different Simulation Strategies

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A B S T R A C T

Ignoring the primary damage to structural components due to blast load or fire is the alternate load path (APM) method's weakness in progressive failure analysis. The new technique used in this study examines the structure's more realistic responses by considering the initial cause of the failure. Also, buckling-restrained braces (BRBs) are applied to diminish the potential for progressive failure in braced steel buildings. Variables include the type of primary local loading (APM, blast loading, and heat caused by fire), the position of column removal in the plan (inner and outer frame), the type of brace (BRB and CB), and the number of stories (3, 5, and 8 stories). The buildings were simulated using ABAQUS. The results showed that BRBs in steel buildings under blast load, compared to conventional braces, reduce the potential of progressive failure. The use of BRBs provides much more energy absorption than conventional bracing systems due to brace buckling prevention.

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1. INTRODUCTION 1

The utilization of buckling-restrained braces (BRBs) in braced steel building and its effect on improving structure behavior against progressive failure is the most important aim of this study.

Extensive studies have been conducted on progressive failure [1-6] and BRBs [7-10]. Each of them examines a part of this event. Palmer et al. [11] examined braced steel frames' performance and built two double story frames with a span in two different modes (with conventional braces (CBs) and BRBs). The results showed that BRBs compared to CBs showed more stable response against lateral loading. Akbarnia et al. [12] examined the effect of column removal on a three-story steel building equipped with BRBs and compared its performance with a three-story building with a flexural frame. The results showed that BRBs elements make steel structures well perform against external loadings such as earthquakes [12].

Yang et al. [13] investigated the role of composite slabs against progressive failure. They showed that the ratio of the dimensions of the composite slab is effective on the bending frame behavior. Mashhadi and Saffari [14] investigated the effect of members' secondary stiffness ratio on dynamic load coefficient in the nonlinear analysis of structures under column removal. They showed that the span length and number of floors in short and medium steel moment-resisting frames significantly affect the dynamic load coefficient.

Tavakoli and Hasani [15] investigated the effect of seismic parameter characteristics on the progressive failure potential in steel moment-resisting frames. The analysis results showed the dynamic response of the removed member under the seismic load is entirely dependent on the seismic characteristics such as the energy applied to the structure, the maximum ground acceleration, and the frequency content. Lin et al. [16] presented a new method for evaluating steel moment-resisting frames against blast loads. This method was

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compared with other common methods. They proved that the onset of damage in the first floors has a more practical effect on the failures' chain than the other floors. Naghavi and Tavakoli [17] investigated the effect of columns' response to progressive failure. For this purpose, a neural network method was used, and sensitivity analysis was performed. The results of this analysis can be used to estimate the response of steel structures to progressive failure. Ryu et al. [18] conducted finite element modeling for the progressive failure analysis of steel stiffened-plate structures in fires. They showed that fire consequences should be quantified accurately for the quantitative fire risk assessment. Zhou et al. [19] compared design methods for beam string structure based on reliability and progressive collapse analysis. The results showed that the representative beam string structure designed with fixed load partial factors and optimum resistance factor, which varies with cases, had high performance of anti-progressive collapse. Zheng et al. [20] studied the progressive collapse mechanism in braced and tied-back retaining systems under deep excavations. They showed that the progressive failure path extends from struts or anchors to piles and will lead to large-scale collapse. Musavi and Sheidaii [21] compared the seismic and gravity progressive collapse in dual systems with special steel moment-resisting frames and braces. The results showed that structures had better performance under seismic progressive failure than models under gravity loads because of more resistance, ductility, suitable load redistribution, and more structural elements in load redistribution.

In general, the studies mentioned above can be divided into three general categories. In the first group, researchers assessed the progressive failure of steel buildings with different load-bearing systems using the column removal method (alternate load path method or APM). In the second category, the method and type of analysis were evaluated. In the third category, new strategies for strengthening steel buildings against progressive failure were assessed. According to the above general classification in the present study, the type of progressive failure analysis method and the performance of buckling-restrained braces (BRBs) in reducing the progressive failure potential were evaluated. Three different methods were used to analyze the progressive failure. In the first method, APM, which is a common method, was used. In the second method, the progressive failure was evaluated by removing several columns and the heat caused by the fire. In the third method, progressive failure analysis was performed by direct simulation of blast waves. The overall purpose of presenting these three methods was that the primary cause of the progressive failure in the APM method is not very important. However, loads such as heat from fires and blast waves can create more critical states than the APM. On the other hand, BRBs, due to the combined performance of concrete and steel and buckling prevention, can help the load-bearing of columns in sudden and unusual loads. Therefore in this work, the performance of this type of braces against progressive failure was investigated.

2. PROCEDURE

Variables were the type of primary local loading (APM, blast loading, and heat caused by fire), the position of primary local failure in the plan (inner and outer frame), the type of brace (BRB and CB), and the number of stories (3, 5, and 8 stories). Table 1 presents the studied modes. Steel braced frame buildings with CBs and BRBs (3, 5, and 8-story) were first analyzed using Sap2000 [22] which is based on Iran national building regulations. The plan of the buildings was similar in all cases. The length of each span and the dimensions of the stairs were considered 6 and 4×6 meters. The building's lateral load resisting system was braced frame in both directions (CB and BRB). St37 building steel specifications were used to define steel. Box sections with a thickness of 10 mm were used to simulate the core and steel sheath of the BRBs. The compressive strength of concrete used in the BRBs was considered 24 MPa. Dead and live loads of the stories were 335 and 200 kg/m^2, respectively. Also, dead and live loads of the roof were 200 and 150 kg/m^2, respectively. ABAQUS [23] was used to simulate the buildings for progressive failure potential. Finally, the response of the structure to both blast and fire loadings was compared with the APM. Studies of progressive failure showed that removing columns at the lowest stories can create a more critical situation for the buildings [24-26]. Therefore, to evaluate the progressive failure in building frames, removing two columns in the lowest story was assessed. The results of the structural design were displayed in Table 2.

3. FINITE ELEMENTS SIMULATION

Finite element simulation was performed with ABAQUS software [23]. Structural elements include beam, column, brace, and roof, which are three-dimensional and deformable types. Materials include steel and concrete (the concrete core of BRBs and roof). BRBs consist of steel sheath (outer wall), concrete core, and central core. Shell, solid, and beam elements were used to simulate the box-shaped steel sheath, concrete core, and central core, respectively.

Table 1. Introducing the modes in the present study

Name	Frame	Initial local loading type	Brace type	Number of stories	No.
3st-No R-CBr	----	No Remove	CB	3	1
3st-B-I-CBr	Inner	Blast			2
3st-B-O-CBr	Outer				3

3st-F-I-CBr	Inner	Fire			4
3st-F-O-CBr	Outer				5
3st-APM-I-CBr	Inner	APM			6
3st-APM-O-CBr	Outer				7
3st-No R-BRB	----	No Remove	BRB		8
3st-B-I-BRB	Inner	Blast			9
3st-B-O-BRB	Outer				10
3st-F-I-BRB	Inner	Fire			11
3st-F-O-BRB	Outer				12
3st-APM-I-BRB	Inner	APM			13
3st-APM-O-BRB	Outer				14
5st-No R-CBr	----	No Remove	CB	5	15
5st-B-I-CBr	Inner	Blast			16
5st-B-O-CBr	Outer				17
5st-F-I-CBr	Inner	Fire			18
5st-F-O-CBr	Outer				19
5st-APM-I-CBr	Inner	APM			20
5st-APM-O-CBr	Outer				21
5st-No R-BRB	----	No Remove	BRB		22
5st-B-I-BRB	Inner	Blast			23
5st-B-O-BRB	Outer				24
5st-F-I-BRB	Inner	Fire			25
5st-F-O-BRB	Outer				26
5st-APM-I-BRB	Inner	APM			27
5st-APM-O-BRB	Outer				28
8st-No R-CBr	----	No Remove	CB	8	29
8st-B-I-CBr	Inner	Blast			30
8st-B-O-CBr	Outer				31
8st-F-I-CBr	Inner	Fire			32
8st-F-O-CBr	Outer				33
8st-APM-I-CBr	Inner	APM			34
8st-APM-O-CBr	Outer				35
8st-No R-BRB	----	No Remove	BRB		36
8st-B-I-BRB	Inner	Blast			37
8st-B-O-BRB	Outer				38
8st-F-I-BRB	Inner	Fire			39
8st-F-O-BRB	Outer				40
8st-APM-I-BRB	Inner	APM			41
8st-APM-O-BRB	Outer				42

Table 2. Design results

Building	Storey	Beam	Column	Brace
3st	First	2IPE 300	Box 350 × 350 × 20	Box 200 × 200 × 20
	Second	2IPE 300	Box 350 × 350 × 20	Box 200 × 200 × 20
	Third	2IPE 270	Box 350 × 350 × 20	Box 200 × 200 × 20
	Fourth	2IPE 270	Box 350 × 350 × 20	Box 200 × 200 × 20
5st	First	2 IPE 300	Box 400 × 400 × 20	Box 200 × 200 × 20
	Second	2 IPE 300	Box 400 × 400 × 20	Box 200 × 200 × 20
	Third	2 IPE 270	Box 400 × 400 × 20	Box 200 × 200 × 20
	Fourth	2 IPE 270	Box 300 × 300 × 20	Box 200 × 200 × 20
	Fifth	2 IPE 270	Box 300 × 300 × 20	Box 150 ×150 × 20
	Sixth	2 IPE 270	Box 300 × 300 × 20	Box 150 × 150 × 20
8st	First	2 IPE 300	Box 450 × 450 × 20	Box 200 × 200 × 20
	Second	2 IPE 300	Box 450 × 450 × 20	Box 200 × 200 × 20
	Third	2 IPE 300	Box 450 × 450 × 20	Box 200 × 200 × 20
	Fourth	2 IPE 300	Box 450 × 450 × 20	Box 150 ×150 × 20
	Fifth	2 IPE 270	Box 450 × 450 × 20	Box 150 × 150 × 20
	Sixth	2 IPE 270	Box 350 × 350 × 20	Box 150 × 150 × 20
	Seventh	2 IPE 270	Box 350 × 350 × 20	Box 150 × 150 × 20
	Eighth	2 IPE 270	Box 350 × 350 × 20	Box 150 × 150 × 20
	Ninth	2 IPE 270	Box 350 × 350 × 20	Box 150 × 150 × 20

The explicit dynamic method in numerical solution was used for the analysis. Tie interaction constraint was utilized to define the interaction between members in all cases. The convergence examination method of the responses is used to specify the optimal mesh size in simulation. Also, concrete was simulated using concrete damage plasticity.

4. THE USED METHODS

Methods for investigating the progressive failure of the studied buildings include the APM, blast loading, and heat caused by fire, respectively. In this section, each of these methods is described.

4.1. APM The general idea of APM is that the building should be designed so that if the normal load transfer paths are removed or damaged. There are other alternative paths for transferring the load to the ground. Therefore, structures are designed to remove columns or specific walls. The loads applied to the models under study are applied according to the Equations (1) and (2): [27]:

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

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

where, G_{LD} is increased gravity load for the upper floors of the column or removed wall, and G is gravity load for floors above a column or the wall, which is not removed. This load combination must affect the span that is not loaded with a combination of G_{LD} load. In equations 1 and 2, D, L, and S are dead, live, and snow loads, respectively. Ω_{LD} is load increase coefficient for calculating deformation-controlled actions.

4.2. Fire load application method One of the issues that affect structures such as residential buildings, factories, offices, and industrial complexes is fire. This is especially important for steel structures because, in steel members, fire reduces strength and stiffness due to their high thermal conductivity and low thickness. After axial loading on the column, the fire was applied. Equation 3 was used to define fire heat. In this regard, T is the temperature change, and t is the duration of the fire. The mentioned equation is based on ISO834 [28, 29].

$$T = \mathrm{Log}_{10}(8t + 1) + T_0 \quad (3)$$

4.3. Blast load application method CONWEP model [30] was used to apply blast loads. A possible blast can have different intensities. The blast loading intensity applied to the column will increase as the explosive increases and the distance from the blast center decreases. For this purpose, by defining the scaled distance (Z) of Equation 4, the overpressure produced by the blast (Ps), TNT material in kg (W) at R distance from the explosive center can be calculated from Equation 5.

$$Z = \frac{R}{W^{\frac{1}{3}}} \quad (4)$$

$$X = \mathrm{Log}_{10}(Z) \quad (5)$$

$$\mathrm{Log}_{10}(Z)[\mathrm{Log}_{10}P_s] = -0.1319X^2 - 0.2331X + 0.4644 \quad (6)$$

The weight values of explosive (w), collision angle (α), and distance of the blast center (R) were first determined. Then maximum pressure values and blast continuity time were determined for different modes. For this purpose, it was placed in the Friedlander blast load equation [31]:

$$P_{(t)} = P_{so}e^{-\frac{t}{t^*}}(1 - \frac{t}{t^*}) \quad (7)$$

In this Equation, P_{so} is the maximum pressure caused by the blast, $P_{(t)}$ is the pressure value in time t, and t* is the continuation time of the blast (when the pressure reaches to zero). To apply blast loads, an explosive weighing 500 kg TNT equivalent, for the external blast, is placed near the corner of the structure and at a distance of 5 meters from it, and in the case of internal blast, it is placed at a distance of 5 meters from the middle column.

5. VERIFICATION

5.1. Verification of progressive failure analysis using APM The verification of progressive failure analysis with APM was performed using the method developed by Zhang et al. [32]. In the selected laboratory study, a one-story steel frame was constructed. This frame had one bay in one direction and two bays in the other direction. The beams and columns of the frame were made of H100×67×4.5×6 and H200×200×6×8, respectively. A 60 mm thick concrete slab was made on the beams. This slab was installed on the beams using shear studs. The length of the larger bay was 2.4 m, and the lengths of smaller bays were 1.8 m. According to Figure 1a, eight steel blocks with a total weight of 148 kN were placed in the middle of the span to simulate a gravitational load on the slab. One of the columns at the edge of the slab was suddenly removed using a knocking hammer, and its response was measured in the form of vertical displacement. More details about this experiment are provided in the literature [32]. The experimental frame, deformable shape, and displacement diagram of finite element and experimental specimen are shown in Figure 1. The experimental study's maximum displacement is about 10.7 mm. The maximum displacement in the finite element method is 10.9 mm. The difference between the two displacements is about 1.86 percent. Therefore, it can be stated that the hypotheses used to simulate the APM in the present study had good accuracy.

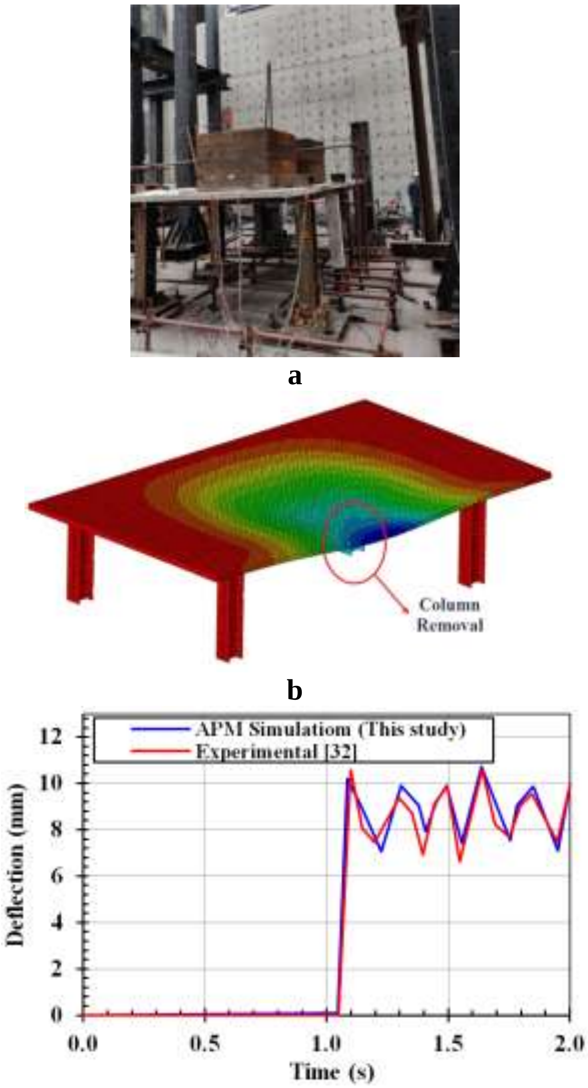

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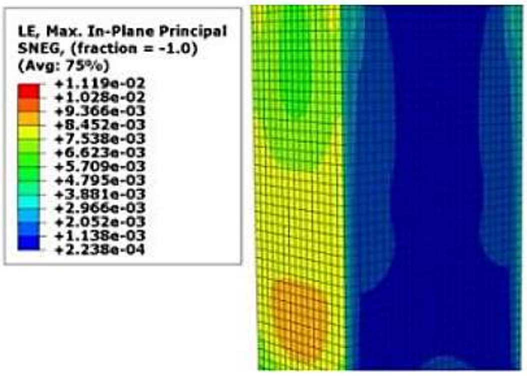
Figure 1. Verification of progressive failure analysis using APM a: Experimental specimen [32] b: FEM c: Comparison displacement diagrams

5.2. The verification of the used method in simulation of fire loading The verification of the used method in simulation of fire loading was performed using method developed by Jiang et al. [33]. Three different frames were studied. The dimensions of the sections and the temperature applied to them were different in different modes.

The dimensions of hollow rectangular sections were 50×30×3 cm and 60×40×3.5 cm for columns and beams, respectively. The middle column of the first floor was warmed up using a furnace.

Figure 2 shows the plastic strain distribution, strength-time curves of the model, and the laboratory specimen. As can be seen, the highest and the lowest values of heat generated in the finite element modeling method in this study have a good precision compared to the experimental results. Therefore, this software has the appropriate and acceptable accuracy in modeling the heat load caused by fire.



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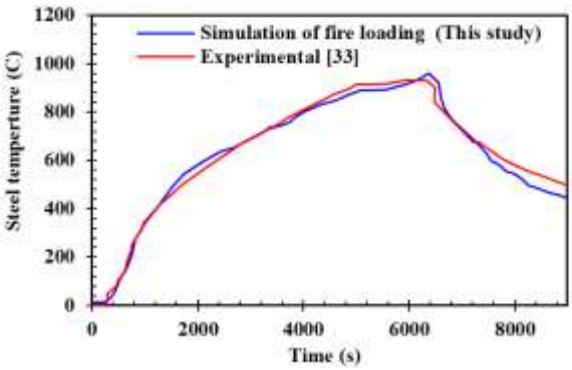

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Figure 2. Verification used in progressive failure analysis using fire load application method a: Plastic strain b: Comparison of strength-time curves of finite element model and experimental specimen

5.3. The verification used in blast load simulation

This validation, based on an experimental data reported by Lawver et al. [34], was performed by applying a blast load to a single column (Figure 3). Several steel columns were subjected to the blast load, and only the column with W360×122 specifications was selected for verification. Figure 3 shows the single column under blast load after analysis. Also, this figure compares the results of the study of Lawver et al. [34] and the simulated model. According to the obtained values, it is observed that the results are very close to each other. Therefore, the simulation method of the finite elements used to model the blast load has good accuracy.

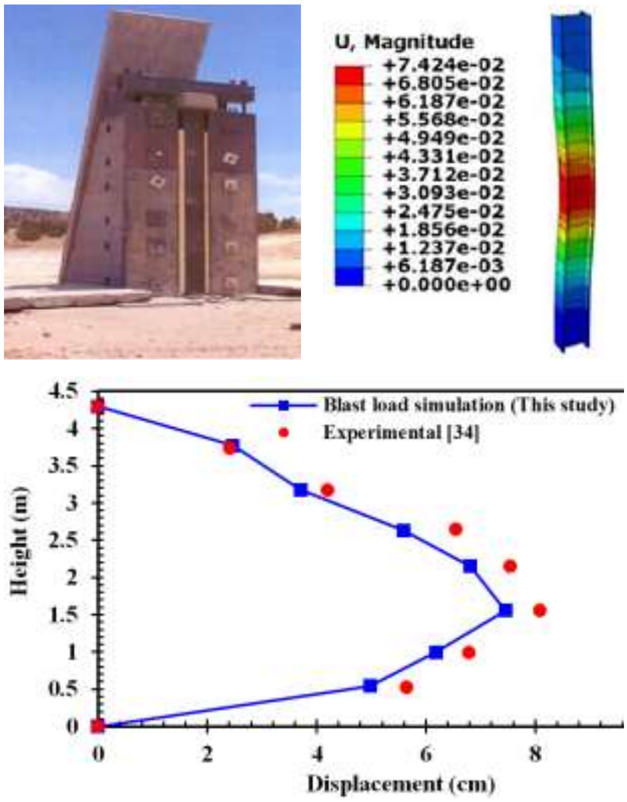


Figure 3. The verification used in the progressive failure analysis using the blast load application method a: Single column under blast load [34] b: Finite element model c: Comparison of column displacement (Numerical and Experimental)

6. INTERPRETING THE RESULTS

Although several studies on buildings' behavior against progressive failure are available, researchers still believe that more research is needed to examine their behaviors against various parameters that may affect them. Experimental tests of these parameters' effect required considerable cost, time, and equipment, which in some cases is impossible or very difficult. Analytical models can be effective and have appropriate accuracy in predicting structural response. The simulation method introduced in the previous sections was used to perform sensitivity analysis of the desired variable parameters.

When various factors locally damage buildings, columns, as one of the most important members of the structure in preventing structural failure, have an important role in transferring and redistributing loads. Therefore, researchers' redistribution criterion of axial forces is one criterion that researchers have always considered in studies related to progressive failure [33-37].

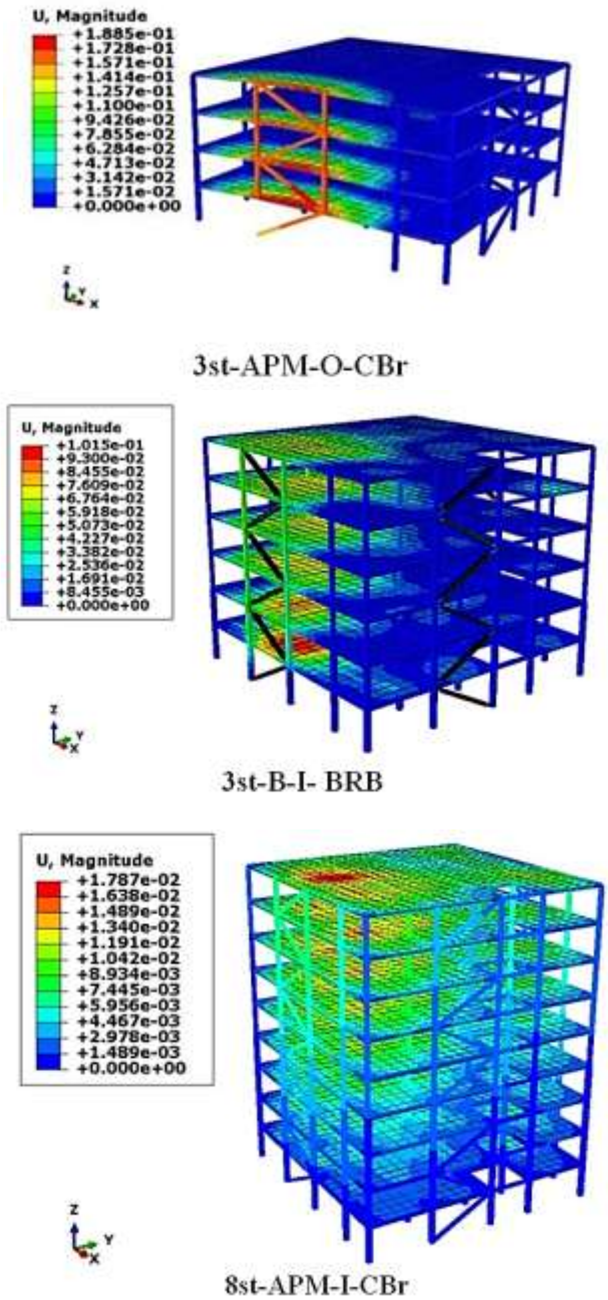


Figure 4. The deformable shape of several buildings under study

The axial force changes of the columns of 3, 5, and 8-story simulated buildings will be examined. The values of the axial force changes of the columns around the local load application location (column removal location), compared to their values in the non-applied local load state according to the desired variables (BRB and ordinary brace), type of progressive failure analysis method (direct blast load application method, fire load application method and APM) and the position of the initial local load application (inner and outer frames) and the number of stories (3, 5 and 8 stories) have been calculated. The axial forces' values are the maximum axial force extracted from the diagrams related to the axial force of the columns. The resulting values are presented and analyzed separately in different graphs to understand better and compare the desired modes and examine the introduced variable parameters. After reviewing the maximum values of axial force in adjacent members to the local load application location, the effect of each of the studied parameters on the axial force changes of the columns is investigated in this section. Also, the deformable shape of several buildings under study is shown in Figure 4.

6.1. Investigating the type of used method

According to Figure 5, when the direct blast load method is used, an increase in axial forces is much more significant than in the APM modes. For example, in 3, 5, and 8-story steel buildings braced by CBs where the local load is applied as a blast load (inner frame), the maximum axial force of the adjacent columns to the load location is 3.6, 2.3, and 1.7 times more than their corresponding values in the column removal method (APM) (Figure 5a). On the other hand, in 3, 5, and 8 story steel buildings braced by CBs, local load, and column removal were applied on the outer frame, the maximum axial force of the adjacent columns to the load application location. In contrast, the direct blast load application method was used is 1.01, 2.19, and 1.88, times more than their corresponding values in the column removal method (APM) (Figure 5b). Also, according to Figure 5c, in 3, 5, and 8-story steel buildings braced by BRBs that the local load and column removal are applied on the inner frame, the maximum axial force of the adjacent columns to load application location, while the direct blast load application method was used, is 200, 59 and 49% more than their corresponding values in the column removal method (APM).

According to Figure 5d, in 3, 5, and 8-story steel buildings braced by BRBs that the local load and column removal are applied on the outer frame, the maximum axial force of the adjacent columns to load application location, while the direct blast load application method was used, is 200, 67, and 64% more than their corresponding values in the column removal method (APM). Therefore, it can be concluded that when explosive is considered as the initial cause of the failure in the progressive failure analysis, the maximum force created in the adjacent columns to the impact area will be significantly higher than other methods. Because of the sudden pressure caused by blast applied to the columns and the stresses that are applied to the beams, the axial force of columns in the blast load application method is more than the APM. The APM in evaluating the axial load of the column has smaller values.

Although in the APM, several columns of the structure are completely removed and lose their function, the loads caused by the blast were applied in an impacting manner and create a more critical state in the structure. Therefore, it can be stated that the primary cause of the local failure and considering it during structure analysis against progressive failure can have much more accurate predictions of the actual behavior of the structure against the initial local failure.

6.2. Position of column removal In this section, the position of column removal in the plan (outer and inner frame) on the redistribution of axial forces is examined. The maximum axial force is presented in Figure 6. As it can be seen, in buildings with fewer stories, the columns in the inner frame are in a much more critical position and bear more forces after applying the initial local load. However, as the height increases, the columns in the outer frame bear more forces. Therefore, the position of removing columns in the plan can depend on the number of stories. Changes in the columns' axial force in situations where the blast load was applied from the outside indicate that the columns, walls, and infills around the structure can play an imperative part within the structure reaction against impact waves. If the walls or infills are connected to the columns, a large amount of the pressure caused by the blast will reach the surrounding columns, which will cause them to be exposed to severe damage. However, suppose a solution can be found to separate the structural columns from the walls. In that case, the load transfer through the rigid diaphragm to the roof will be transferred to other columns and structural members. More members will bear the blast wave and the structure strength increases, and the potential for progressive failure decreases.

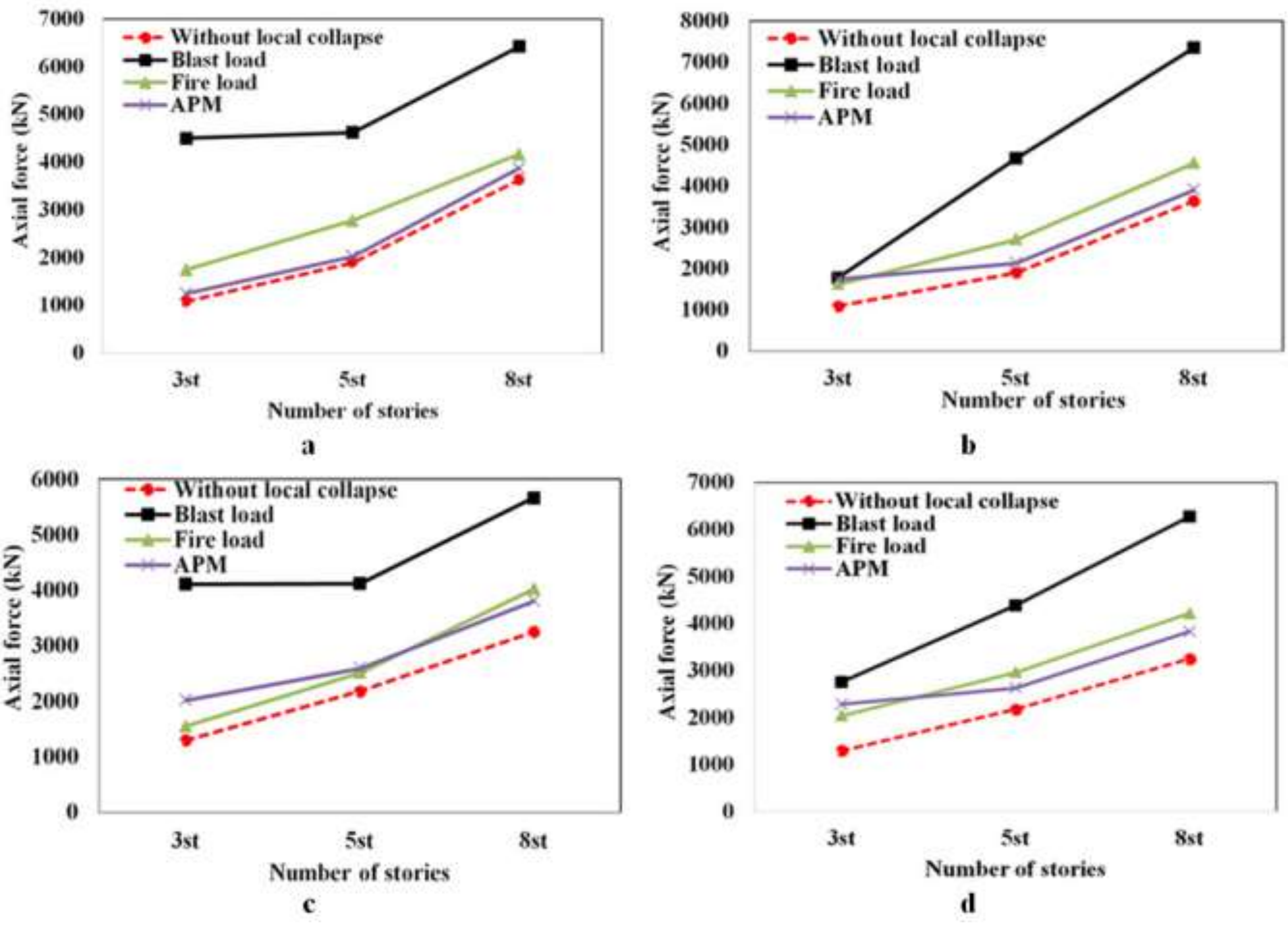


Figure 5. Comparison of methods used in progressive failure analysis a: Conventional brace - inner frame b: Conventional brace - outer frame c: BRB - inner framed: BRB - the outer frame

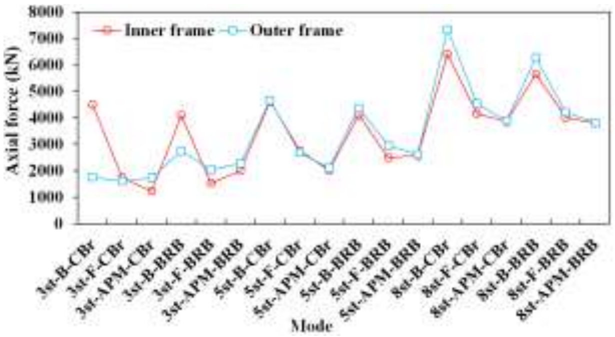
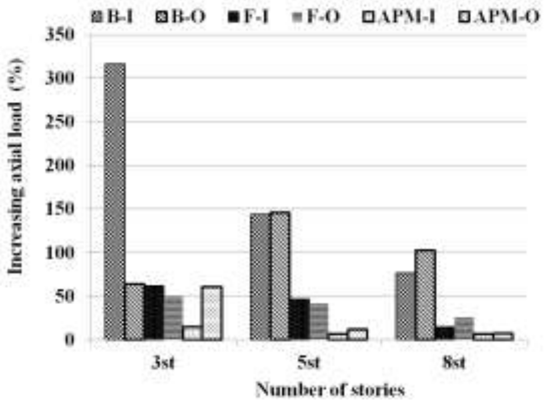


Figure 6. Examining the position of the column removal (outer and inner frame) in the stories on the changes of the maximum forces of the columns near to the initial local damage position

An interesting point that can be indicated in Figure 6 is that when the column is removed in the outer frame, the APM shows much more force is more conservative in this respect. The reason for this is that when the column is removed in the inner frame, more members of the structure participate in bearing additional loads and redistribution of forces. This creates fewer axial forces around the removal location. When the column is removed in the outer frame, fewer members contribute to the load distribution, creating more force in the columns around the removal location.

6.3. Percentage of changes in the maximum axial forces around the local failure location Figures 7a and 7b compare the percentage increase of the maximum axial forces around the local failure location to investigate the method used and the position of the position column removal in the stories. According to the desired variables, the values of the axial forces around the local failure location are calculated relative to their values in the non-removal mode. It should be noted that the axial forces' values are the maximum axial force when the blast load affects two columns of the inner frame and the elements around them. The columns' axial forces around the location of the local failure increased more than APM and fire loading methods. As shown in Figure 7b, explosion-induced loads on inner columns caused maximum axial forces around the local failure location to increase by 317% in the mode of blast load application. However, in the APM, removing the two columns in the inner frame caused maximum axial forces around the local failure location to increase by 15%. The analysis results showed that if explosives cause the primary local failure, considerable axial forces are created in the columns around the primary local failure location. These forces are not considered when APM is exerted. According to these results, the initial cause of failure should be considered. Because the force transfer by the residual structural members of the building under local failure can prevent the progressive failure, and if the initial failure is not predicted correctly, the structure response against the possible failures would not be appropriate. Therefore, it can be stated that the type of primary local failure is very influential, and ignoring it may activate to wrong divinations of building behavior. The heat application due to fire caused the axial forces created in the columns to be much higher than the APM.

6.4. Investigating the effect of BRBs The performances of CBs and BRBs are compared in this section. Figures 8a and 8b compare the performance of CBs and BRBs on the structure response against progressive failure in modes affected by the blast load in the inner and outer frames. When the explosive was placed inside the structure, the BRBs showed much better performance in terms of axial force changes than CBs. The axial forces created in adjacent members to their blast location compared to the corresponding values in buildings with CBs are greatly reduced. This is also true to some extent in buildings that were placed under an external blast. For example, the percentage increase in the axial force of a 5-story building braced with BRB is about 30 percent less than the corresponding value in buildings braced with CB.



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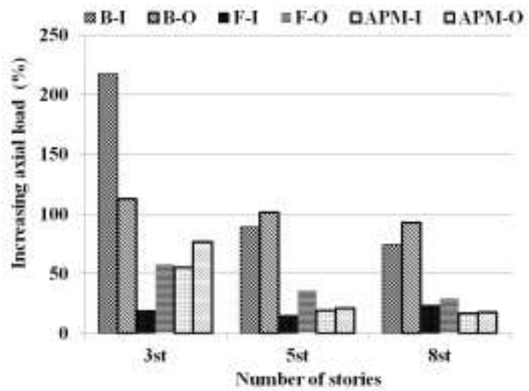

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Figure 7. Increasing axial loads of the columns around the removal location compared to the non-removal mode a: buildings with CBs b: buildings with BRBs

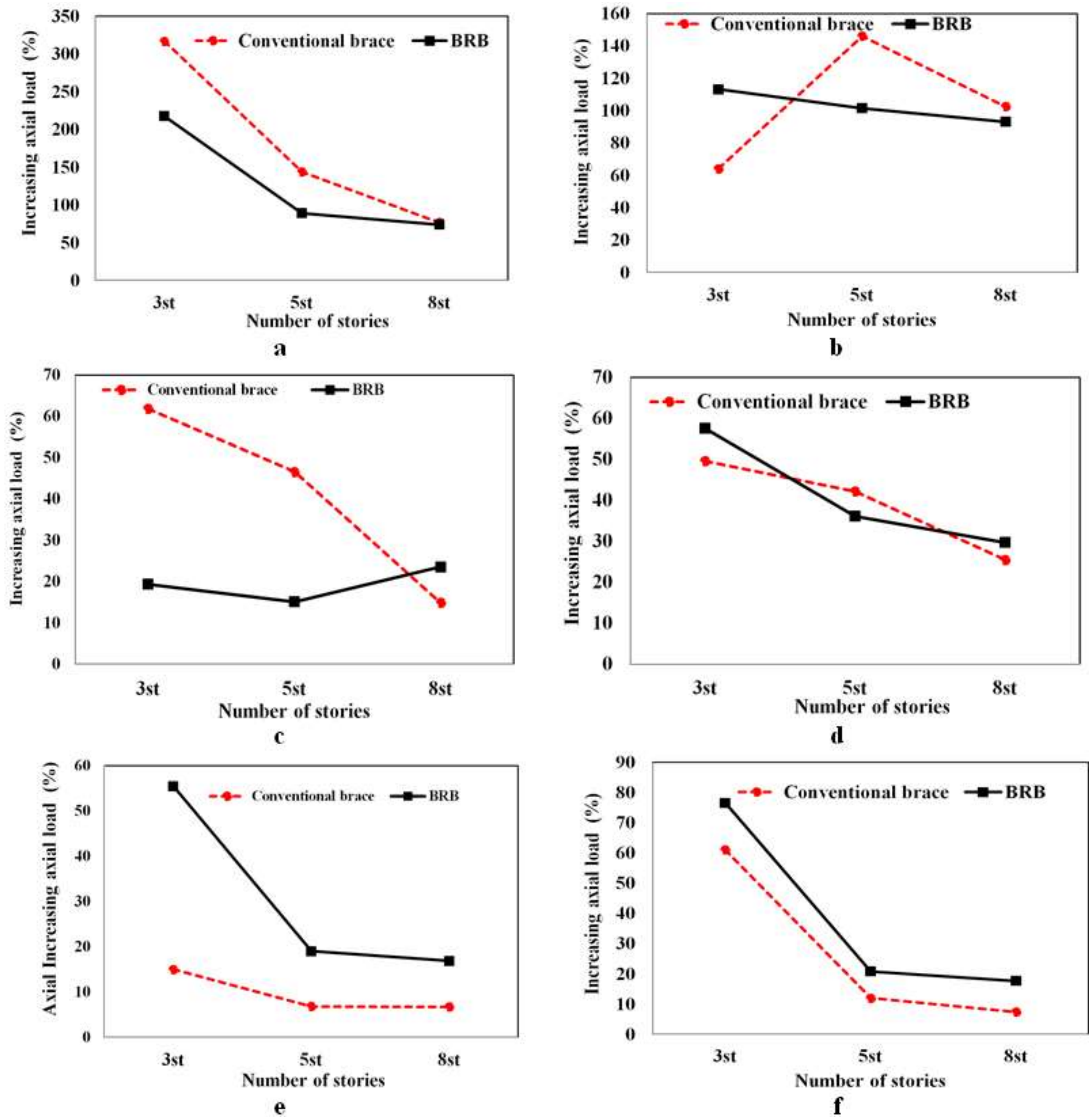

Figure 8. Comparison of the performance of CBs and BRBs on the structure response against progressive failure a: blast load-inner frame b: blast load-outer frame c: heat caused by fire-inner frame d: heat caused by fire-outer frame e: APM-inner frame f: APM-outer frame

The use of BRBs in steel buildings under blast load, compared to CBs, improves the structural performance and reduces the possibility of progressive failure. This is because CBs cannot buckle due to applied pressure and have similar behaviors in tension and pressure. However, BRBs can absorb higher energy by yielding brace in tension and pressure. Due to steel sheaths and the simultaneous effect of concrete and steel on tension and pressure, they have more energy loss capacity and more ductility than CBs. BRBs, with their compressive performance, can significantly contribute to bearing axial forces created by column removal to the adjacent columns of the blast location. Figures 8c to 8g show that the BRBs in some modes cause the difference between the axial force created in the removal and not removal methods to be reduced compared to CBs.

7. CONCLUSION

The results showed that APM is relatively easy without considering the loading type, but it is less accurate and unreliable for predicting progressive failure in the structures. The method that was used in this manuscript provides rather dependable prophecies of the failure caused by explosive and fire loads. The initial cause of the progressive failure and its application to the structure led to more realistic responses.

Also, due to the performance of BRBs in preventing the occurrence of buckling of the steel core (in order to allow the occurrence of compression yield phenomenon), absorbing more energy and covering the entire length of the steel core with concrete, it is expected that the potential for progressive failure in the structure is reduced.

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Persian Abstract

چکیده

ضعف روش مسیر بار جایگزین (APM) که یکی از روش‌های متداول تحلیل خرابی پیش رونده است، نادیده گرفتن خرابی اولیه یا آسیب اعضای سازه‌ای مجاور در اثر بار انفجار و یا آتش سوزی است. از این رو روش جدیدی که در این مطالعه مورد استفاده قرار گرفته است، با در نظر گرفتن عامل اولیه ایجاد خرابی پیش‌رونده و اعمال آنها به سازه، پاسخ های واقعی تری از سازه را مورد بررسی قرار می‌دهد. متغیرها شامل نوع بارگذاری موضعی اولیه (بدون در نظر گرفتن علت اولیه خرابی، بارگذاری انفجار و حرارت ناشی از آتش سوزی)، موقعیت حذف ستون در پلان (قاب بیرونی و داخلی)، نوع بادبند (معمولی و کمانش تاب) و تعداد طبقات (۳، ۵ و ۸ طبقه) می‌باشند. ساختمان‌ها با استفاده از ABAQUS به صورت سه بعدی شبیه سازی شدند. نتایج نشان داد 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. همچنین استفاده از مهاربندهای کمانش‌تاب در ساختمان‌های فولادی که تحت بار انفجار قرار گرفته‌اند، در مقایسه با مهاربندهای معمولی باعث بهبود عملکرد سازه می‌شود و احتمال وقوع خرابی پیشرونده را کاهش می‌دهد. دلیل این موضوع آن است که مهاربندهای معمولی قابلیت کمانش ناشی از فشار وارده را نداشته و دارای رفتار مشابه درکشش و فشار می‌باشند؛ این در حالیست که مهاربندهای کمانش تاب با تسلیم مهاربند درکشش و فشار توانایی جذب انرژی بالاتری دارند و به علت وجود غلاف فولادی و اثر همزمان بتن و فولاد درکشش و فشار ظرفیت اتلاف انرژی و شکل پذیری بیشتری نسبت به مهاربندهای معمولی دارند. در واقع مهاربندهای کمانش تاب با عملکرد فشاری خود می توانند در تحمل نیروهای محوری ایجاد شده ناشی از حذف ستون کمک قابل توجهی به ستون‌های مجاور محل انفجار داشته باشند.