



Assessment of Progressive Collapse in Buildings with Special Steel Moment Frames Considering Sudden Column Removal Time

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Abstract

In the technical literature of structural and earthquake engineering, the phenomenon of progressive collapse is not directly incorporated into calculations; analysis, design processes, and many researchers consider this phenomenon as an additional load. Despite the importance of this issue in the seismic behavior of structures, design codes ignore the consideration of this phenomenon. Therefore, this paper analyzes the nonlinear behavior of a 4-story structure containing a special moment-resisting frame under progressive collapse loads. In this regard, three types of scenarios for the removal of corner, middle, and peripheral columns on the ground floor are considered, with 32 different column removal times including very rapid times and times close to the natural period of the structure applied to the studied model in the nonlinear OpenSees software. The results indicate that as the column removal time increases, the maximum stress values in beams and columns, vertical displacement, beam rotation, and residual vertical displacement decrease. On the other hand, the difference in maximum displacement values at maximum and minimum times for scenarios 1, 2, and 3 is 81%, 92%, and 37% respectively, and for residual displacements, it is 61%, 82%, and 34%. Meanwhile, the difference in maximum rotations generated in the beam at maximum and minimum times for scenario 1 is nearly 5 times.

Keywords: Progressive Collapse, Nonlinear Dynamic Analysis, Column Removal Time, Steel Moment Frame, Special Steel Moment Frames

1. Introduction

Ensuring the performance of structures has always been a principle in the minds of engineers and researchers. The phenomenon of progressive collapse is one of the topics that has gained attention

in recent decades. This phenomenon often arises due to the application of unusual loads and human errors in buildings, potentially leading to the failure of part or all of a structure. It can be claimed that the progressive collapse was first known due to the localized failure of the Ronan Point building in London in 1968. The explosion at the World Trade Center on September 11, 2001, significantly impacted the discussion of this phenomenon. Although other failures in buildings such as the Ambience Plaza in Connecticut, USA, the Alfred P. Murrah Federal Building in Oklahoma City, USA, in 1995, and the Trust Bank building in New York City have also contributed to the initiation of research on the phenomenon of progressive collapse (Dehcheshmeh et al., 2024). Studies have shown that the occurrence of progressive collapse in structures poses a potential risk. Additionally, severe earthquakes can also lead to progressive collapse in buildings. This is because lateral forces and severe overturning can cause the removal of supports from corner columns, the removal of several main structural members, the occurrence of fires, and explosions due to gas, etc., leading to structural failure (Ghassemieh et al., 2021). A comprehensive investigation is presented by Bagherzadeh Azar and Sari (2025) for evaluation of the seismic response of a historical arch Halilviran bridge using a macro-modelling technique in ABAQUS. In this regard, 3D finite element models with damage plasticity behavior is modeled considering the masonry units and the Mohr-Coulomb (M-C) material model. This study reveals that failure of the spandrel walls and the inner section of the arches heightens the risk of progressive collapse of the bridge (Bagherzadeh Azar and Sari, 2025).

Nowadays, preventing progressive collapse in structures is essential to avoid the recurrence of past disasters in buildings. This issue is addressed in various codes around the world in different ways to mitigate the effects of progressive collapse. However, this topic has not yet been seriously considered in many seismic design codes, such as Iranian code. Regarding the consideration of the

phenomenon of progressive collapse, some researchers have proposed suggestions. For example, Kaewkulchai and Williamson (2004) considering the uncertainties related to the progressive collapse of structures, proposed a solution that includes a beam element formula and a method for the dynamic analysis of progressive collapse, providing necessary guidance for further studies in this area. Kim et al., (2007) discussed the performance of welded beam-column connections in moment frames against failure using cyclic and monotonic analysis methods. They selected three types of FEMA connections, including unreinforced welded connections, reduced-section beam connections, and welded connections with top flange and bottom flange plates as analytical models to obtain local and overall behavior after the removal of the specified column. They found that the demand for rotation in the RBS connection is greater than in the WUF-W and WFP connections. Lee et al., (2009) examined two simple and accurate methods for analyzing progressive collapse in steel moment frame structures with welded connections. In this study, while employing linear and nonlinear static analysis methods, they presented a relationship between vertical gravitational loads and maximum dynamic rotation with the concept of a damage spectrum for two-span steel moment frames with welded connections. This method is designed for the rapid assessment of maximum demands for nonlinear deformations. Kim and An (2009) investigated the effect of chain action on the potential for progressive collapse in steel moment frame structures. Nonlinear static and dynamic analyses were performed on 3- and 6-story structures with and without braces based on the alternative path method proposed by GSA2003. The results indicated that the contribution of chain action in preventing structural failure increases with the number of spans and floors, and the performance of chain action in braced frames significantly increases when deformation in the beam-to-column connection is prevented, allowing the beam on both sides of the removed column to reach its full tensile capacity. Kim and Kim (2009) also examined the capacity of moment

frames against progressive collapse using the alternative load path method with linear static and nonlinear dynamic analysis. They found that linear static analysis is more conservative than nonlinear dynamic analysis. Liu (2010) studied the reinforcement of beam-to-column connections in simple building frames to prevent progressive collapse. This study showed that by reinforcing connections and initiating chain action in the connection, progressive collapse can be prevented. It is noteworthy that if the beam-to-column connection is reinforced, additional moments may be transferred from the beam to the column, potentially forming a hinge at the top and bottom of the column if the column is weak due to the additional moments. Naji and Irani (2012) focused on developing a simple and approximate model for calculating the displacement of the upper point of the removed column in progressive collapse analysis. This study aimed to ensure that the proposed model is both sufficiently accurate and simple enough to provide quick responses through manual calculations. Additionally, by plotting the capacity curve of the structure and comparing it with the axial force of the removed column, the study assessed the resistance of the structure against progressive collapse. Comparing the results obtained from the proposed method with those from nonlinear dynamic analysis indicated that the proposed method has acceptable accuracy. Song and Sezen (2013) compared the results obtained from linear static and nonlinear dynamic analysis with experimental results from a real four-story steel moment frame building (Ohio University building) under the removal of four columns on the first floor. The results of this research showed that three-dimensional modeling can yield better results in comparing progressive collapse than two-dimensional modeling, and that static analysis is considered a conservative method compared to nonlinear dynamic analysis, which can align well with reality.

Tavakoli and Kiakojouri (2013) proposed a new method for simulating the dynamic removal of columns in steel frame systems. They used this method to assess the structural response of a five-

story steel building frame under various column removal scenarios, accounting for the nonlinear effects of materials and geometry. The results indicate that the potential for progressive collapse fundamentally depends on the position of the column removal, and the proposed method offers computational ease and practicality for simulating dynamic column removal in frame structures.

Cyclic behavior of reusable side plate cold-formed steel semi-rigid moment-resisting connections have been evaluated by Ibrahimov et al., (2025). In this regards, eight large-scale tests are performed on cantilever side plate cold-formed steel (CFS) semi-rigid moment-resisting connections. The results show that lipped side plate connections reached more energy dissipation and moment by up to 120% and 60%, respectively, compared with those of the unlipped connections. All the tested connections are classified as semi-rigid while the bolted connections show up to 1.9 times higher level of elastic stiffness than the screwed connections. For reusable design, the lipped side plate bolted connection is more suitable (Ibrahimov et al., 2025).

Alternate path method (APM) is the prevailing analysis approach in structural disproportionate collapse. Although this method proposes a practical tool for the evaluation of structural robustness, the complete column loss scenario is a rather unrealistic idealization of damage initiation. Pantidis et al., (2020) expands the previous studies about structural robustness and applies a partial distributed damage method (PDDM) in 3D multi-story steel framed and concrete composite buildings. Similar to common output of real extreme events, this method assumes that damage is distributed in more than one component (Pantidis et al., 2020). In the safety evaluation of civil buildings, the definition of dynamic collapse limit states is still known as a challenge in progressive collapse field. Hence, Ding and Chen (2025) summarizes and compares three kinds of collapse failure criteria in literature for reinforced concrete (RC) multi-story frames under column removal scenarios, including the displacement-based criterion, the resistance-based criterion, and the

energy-based criterion. In the stochastic analysis, random inputs were the depth of the concrete cover and the key material mechanical properties for both concrete and reinforcing steel because uncertainties have significant effect on the collapse limit states. The findings reveal that different collapse limit states can result by the different structural designs and the uncertainties in structural parameters. These states are strongly related to the specific failure modes or paths during the progressive collapse. Finally, the energy-based criterion is less affected by the different failure modes or paths and has the best performance in determination of the collapse limit states (Ding and Chen, 2025).

Zhang et al. (2020) examined multi-column removal in a six-story steel moment frame system. The results showed that the simultaneous removal of four columns is generally avoidable in practical designs, while the removal of pairs and triplets of columns should be specially considered. The potential for failure of consecutive columns is less than that of columns placed in rectangular openings. Under minimum gravitational loading, corner columns and under maximum gravitational loading, middle columns exhibit greater resistance to progressive collapse. Safari Honar et al. (2023) investigated the potential for progressive collapse in a four-story steel structure subjected to the impact of a truck with variable weight and speed colliding with the ground floor columns. This four-story steel structure (with an intermediate moment-resisting frame system and special concentric braces in one direction and an intermediate moment-resisting frame system in the other direction) was modeled and designed using ETABS software. To simulate the structure under impact and progressive collapse scenarios, code-compliant methods were employed using the ABAQUS finite element software. One significant finding of this research indicated that the results from vehicle impact could demonstrate much more critical outcomes compared to the column removal method (the conventional code-based progressive collapse).

In 2024, the numerical simulation and design of progressive collapse was examined in composite steel structures, considering the effect of floor collapse. For this purpose, a three-dimensional spatial composite structural model was created using the ANSYS/LS-DYNA finite element software, and experimental results were used to validate the model. The findings of this study showed that structural collapse depends on the type of beam-to-column connection, floor level, and the type of removed component. It was also identified that collapse is more likely to occur when internal columns, non-resistant lateral columns, and lower floor columns are removed, as after the failure of a column, the path of vertical loads is primarily transferred to the internal columns within the collapse zone. The area potentially affected by collapse is also subject to severe impact due to floor collapse. Based on the collapse scenarios and the path of internal force transfer, an innovative design method for preventing collapse was proposed. This method avoids additional modeling and nonlinear computations in collapse prevention design (Guo et al., 2024).

In general, the removal of load-bearing elements such as columns can quickly lead to structural failures and jeopardize the stability of the building. These failures may begin with minor damage but ultimately result in catastrophic events. The importance of this issue is even greater for irregular structures. Therefore, the resistance to progressive collapse was studied in irregular steel structures after an earthquake. The results indicate that earthquakes increase the risk of progressive collapse. Furthermore, an increase in recess of up to 13.33% improves the structure's performance against progressive collapse, reducing responses by an average of 21% compared to conventional structures. However, at higher percentages of recess, responses increase by an average of 82%, and structural resistance decreases (Saeedi et al., 2024). In the same year, an experimental study on the progressive collapse behavior of a full-scale three-dimensional steel frame substructure with reinforced concrete floor slabs was presented. In this regard, the alternative load path method was

considered by removing an exterior column before testing. The results indicate that in steel structures with rigid or semi-rigid connections that possess good flexural resistance but limited rotational capacity, reinforcing the beam connections during large deformations may be a solution to increase the structure's resistance to progressive collapse (Ren et al., 2024).

For RC buildings, Ding et al. (2025) evaluated the dynamic sensitive failure mode (DSFM) in the progressive collapse of these structures under column removal scenarios. This study showed that Reinforced concrete (RC) structures are often unsafe when considered in dynamic analysis due to their sudden dynamic behavior, even though they may be considered safe based on a static analysis. For this reason, DSFM should be identified in progressive collapse analysis. In this regard, a method for determining the DSFM window without performing dynamic analysis has been proposed and verified in this research. The Finite Element Method (FEM) and Applied Element Method (AEM) in analyzing progressive collapse of a RC building have been compared by Yuzbasi and Arslan (2025). This study evaluated the effect of various slab thicknesses, types, and damping ratios on failure analysis. The results showed AEM has more rigidity and damping within the first second while FEM states this time between 2 and 2.50 seconds. AEM can be considered as an alternative to FEM, especially in scenarios involving multiple element removals. Given the significant effects of the progressive collapse phenomenon on the seismic performance of structures and the lesser attention given to this issue by design codes, there is a need for a more comprehensive evaluation of this phenomenon in the behavior of structures during earthquakes, especially in seismic zones. Therefore, the present paper aims to assess progressive collapse through nonlinear analysis of a four-story structure containing a special steel moment frame against progressive collapse loads, considering the timing of sudden column removal. In this regard, three types of scenarios for the removal of corner, middle, and peripheral columns on the

ground floor are considered, with 32 different column removal timings. Flowchart of the research methodology is shown in Figure (1).

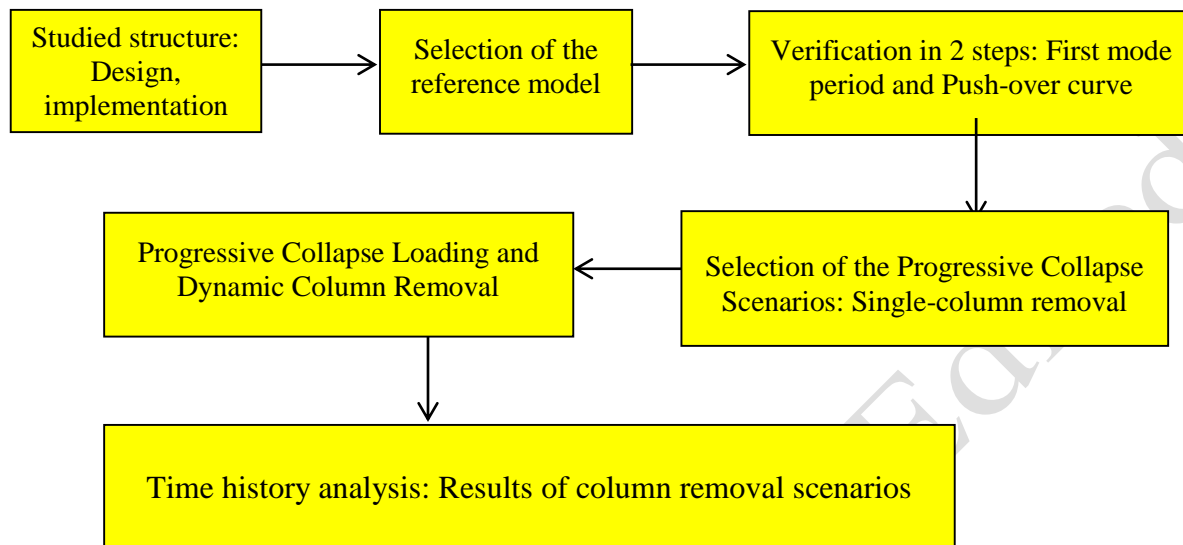


Figure (1) Flowchart of the research methodology

2. Research Method

2.1 Studied Structure

In this study, a four-story structure containing a special steel moment frame was examined. This structure with residential landuse which has been located in Tehran, on soil type II, designed using the LRFD method in accordance with the fourth edition of the 10th National Building Regulations. According to Figure (2), the building's lot coverage is 576 m² and has 6-meter spans in X direction and 6-meter spans in Y direction. The story height is considered to be 3.2 meters.

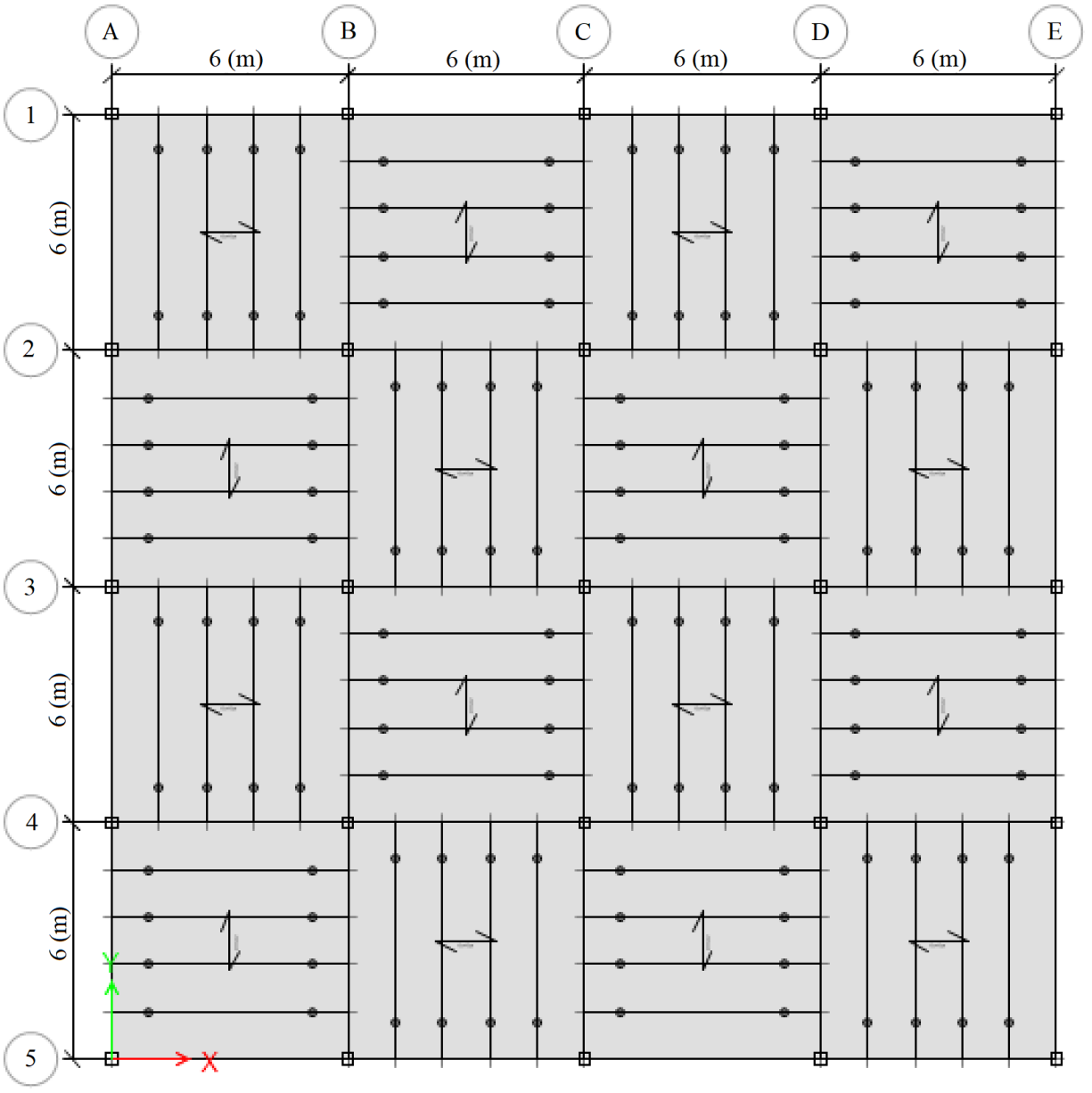


Figure (2) Plan of the studied structure

The loading of the building is in accordance with the 6th National Building Regulations, as detailed in Table (1). For seismic design, spectral analysis method was selected based on Standard 2800. Table (2) reports the section classification. To implement the studied model in the Opensees software, *Force-Based beam-column elements* with nonlinear materials were considered for the beams and columns, and *zero-length elements* were selected for the springs equivalent to the

composite roof. *Fiber sections* were used for the structural elements (beams, columns, and springs), with square sections for the springs, I-shaped sections for the beams, and hollow sections for the columns. Fiber sections consider extensive plasticity along the element, while the use of concentrated plastic hinges assigns nonlinear behavior to the ends of the structure. The actual behavior of the structural elements is extensive plasticity; however, in some studies such as (Mohammadi Dehcheshmeh et al., 2022), concentrated plastic hinges are used to speed up the analysis. It is worth noting that P- Δ effects are considered through three-dimensional modeling and loading on the nodes, with the local-to-global coordinate transformation (*geomTransf*) defined as *corotational*. P- δ effects are applied by dividing the structural members into several parts in the model based on Mohammadi Dehcheshmeh et al. (2022). The materials assigned to the sections of the columns and beams are *Steel02*, which is used for steel materials like Giuffr -Menegotto-Pinto, featuring kinematic hardening and optional isotropic hardening described by a nonlinear evolution equation (Mazzoni et al., 2009). Since the compressive and tensile failure ranges in the stress-strain curve of these materials are not defined in Opensees software library, strain in the members is limited with *MinMax* materials because these materials consider the compressive and tensile failure strains similarly to *Steel02* materials. Moreover, the uniaxial nonlinear *ElasticPP* materials were assigned to the spring's equivalent to the composite roof. The force in these springs is limited based on the failure and buckling of the floor members using *MinMax* materials. The yield stress and modulus of elasticity of the consumed steel were considered to be 255 MPa and 170 GPa, respectively. In this study, to ensure the adequacy of the model's performance, a two-step validation process was conducted. In the first step, the results of numerical model were compared with experimental (as reference model) specimen. According to Figure (3), the experimental

specimen, based on the study by Tsitos (2009), contains a two-dimensional frame with two spans and three stories.

Table (1) Gravity loading of the studied model

Blade load around	Snow load	Floor live load	Live partitioning load	Floor dead load	Floor
kgf/m	kgf/m^2	kgf/m^2	kgf/m^2	kgf/m^2	
190	105	150	0	550	Roof
550	0	200*	100	485	Other floors

Table (2) Sectional specifications of a 4-story building

Beam	Column	Floor
B-w350*10-f200*12	BOX 350*350*12	First and Second
B-w350*10-f150*12	BOX 275*275*10	Third and Fourth

The effective parameters for defining *Steel02* materials in Opensees are reported in Table (3). According to Figure (3), the middle column was removed, and the load was gradually applied to the top of the removed column on the second story. Ultimately, the amount of applied force along with the displacement of the removed column was obtained. Figure (4) compares the load-displacement curves of the reference model and the implemented model. According to this figure, the maximum error is 5%, and there is a satisfactory correlation between the results. In the second step of validation, the first mode period of the structure was compared between the Opensees and ETABS software. The first mode period of the structure in ETABS was found to be 1.13 (s) in both directions, while in Opensees, it was 1.18 (s) in both directions. Given the negligible difference between these two values, the results indicate a suitable accuracy of the model implementation method.

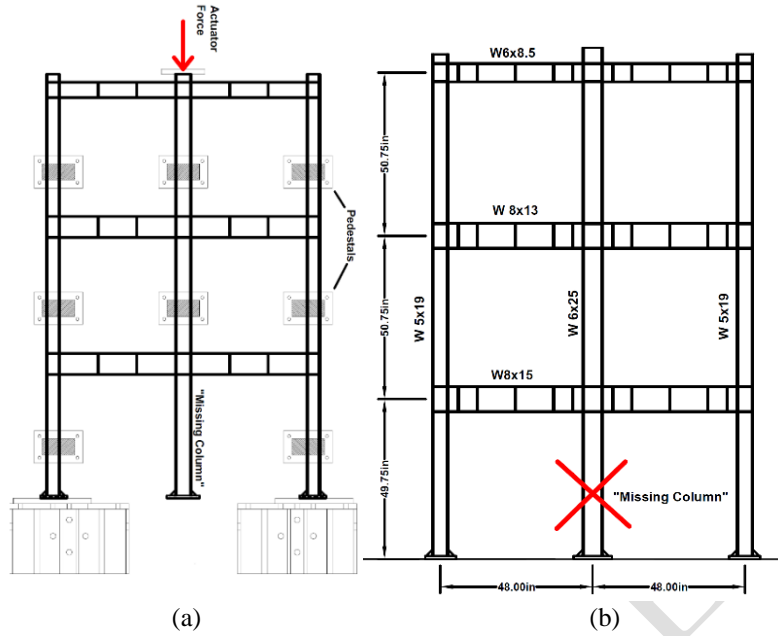


Figure (3) View of the experimental (Reference) specimen for validation (a) Geometric specifications and cross-sections of the frame members, (b) experimental modeling and loading application (Tsitos, 2009)

Table (3) Specifications of *Steel02* Materials for Implementing and Validating the Model in Opensees

F_y (Pa)	E (Pa)	b	R_0	cR_1	cR_2	a_1	a_2	a_3	a_4
200E ⁶	1.95 E ¹¹	0.003	20	0.925	0.15	0.065	1	0.045	1

2.2. Progressive Collapse Scenarios

To investigate various progressive collapse scenarios using Opensees software, the columns examined in the building plans were divided into three groups: corner, peripheral, and central, as shown in Figure (5) for the removal case. In technical literature, it is commonly suggested that structures have been analyzed under single-column removal to simulate progressive collapse (Fu et al., 2019; Fu and Tan, 2019; Fu et al., 2018; Mehdizadeh and Karamodin, 2018). Therefore, this study also follows this pattern. This figure showed the considered progressive collapse scenarios in this study. The used load combination in the nonlinear dynamic analysis for all frames is based

on the UFC and GSA regulations, and other researches such as Ferraioli et al., (2022); Yaghoubi et al., (2021) and Vinay et al., (2020) as expressed in Equation (1):

$$G = 1.2 D + (0.5 L \text{ or } 0.2 S) \quad (1)$$

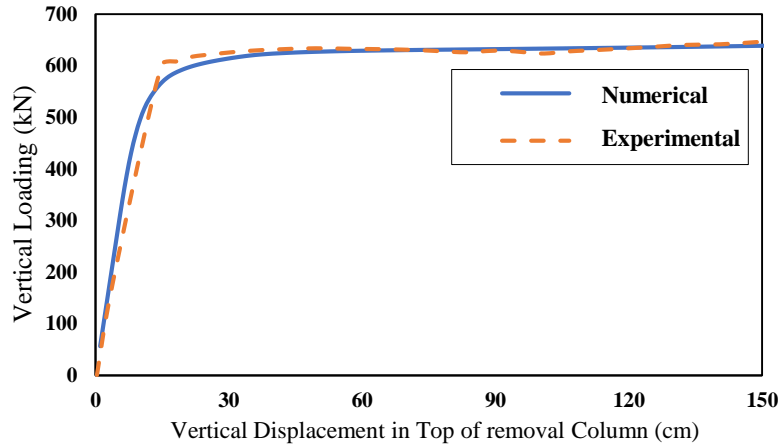


Figure (4) Force-displacement curves of the reference and the numerical model

In this study, the Opensees software was used to perform the nonlinear dynamic. The structure was loaded with the above combination, and then internal forces such as shear, moment, and axial force of the concerned column were recorded before its removal. These internal forces were then substituted into another model where the column had been removed. To simulate the sudden removal of the column, as shown in Figure (6), the internal forces applied instead of the removed column (V-M-P) are removed after a short period. In this figure, w represents the gravitational load applied to the structure according to Equation (1) (Kim and Kim, 2009).

2.3. Spring Equivalent to the Composite Roof in the Vertical Direction

The performance of the structure during progressive collapse is determined by the performance of the roof, infill, and main members such as beams and columns. To account for the performance of the ceiling, the desired states must first be modeled with the main members, and progressive collapse must be analyzed by removing a column and applying gravitational load at the location

of the removed column. Thus, a simplified bilinear curve can be considered according to publication 360 and the Federal Emergency Management Agency regulations, as shown in Figure (7-a). It should be noted that the springs equivalent to the composite roof are defined in the vertical direction at the location of the removed column, and due to the load transfer path, *equalDOF* constraints are used in the vertical direction to align the end constraints of the spring with the surrounding nodes.

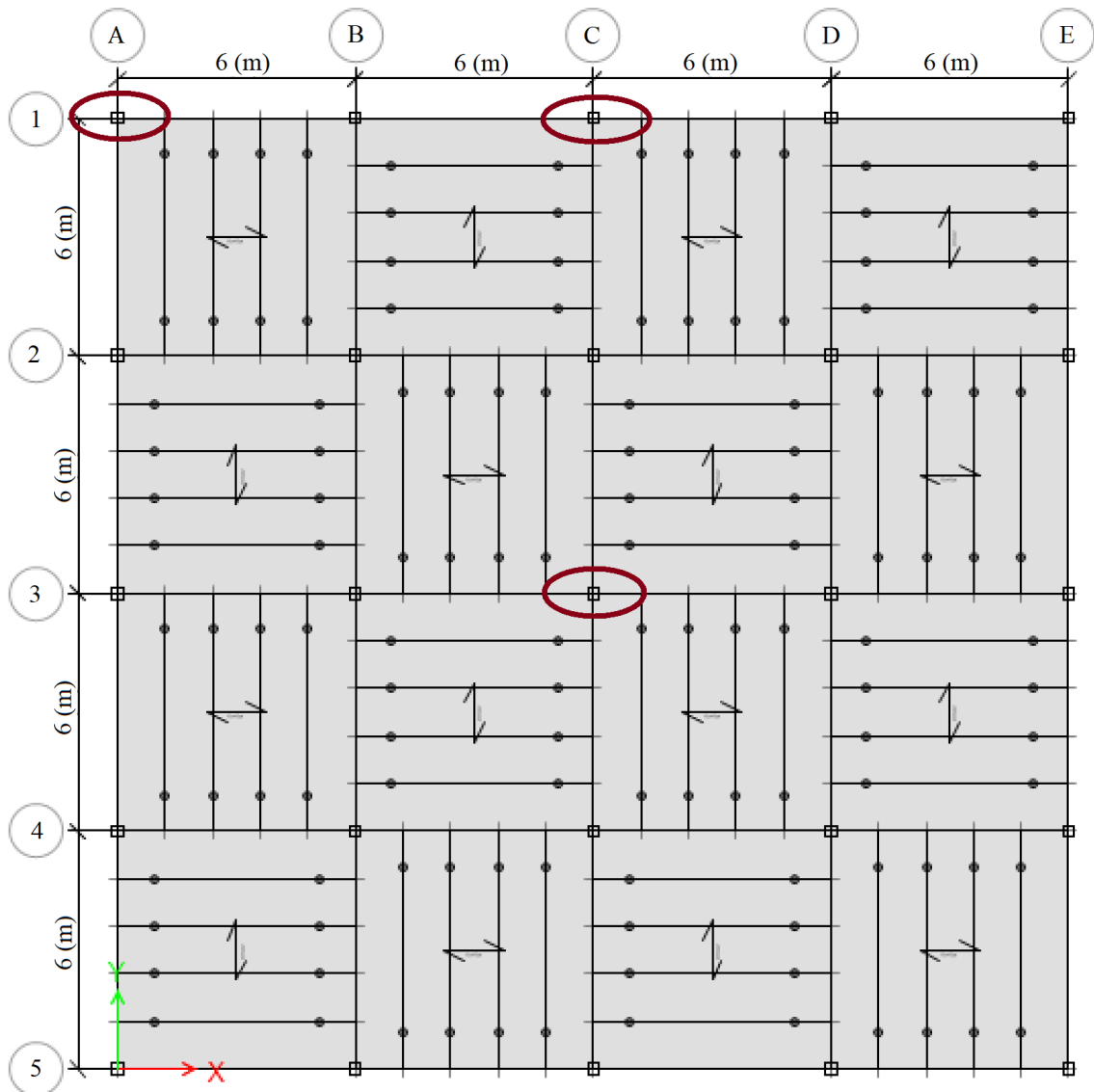


Figure (5) Progressive collapse scenarios through the removal of corner, peripheral, and central columns

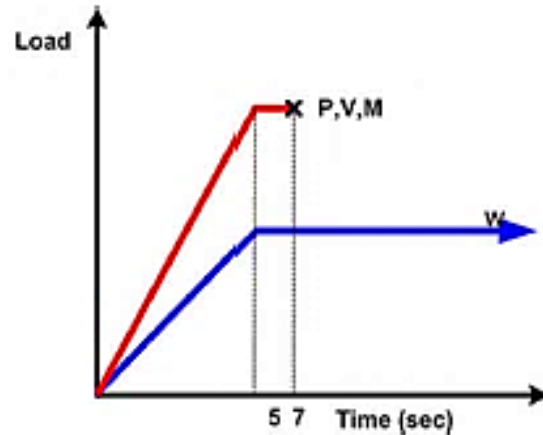


Figure (6) Dynamic removal of a column (Kim and Kim, 2009)

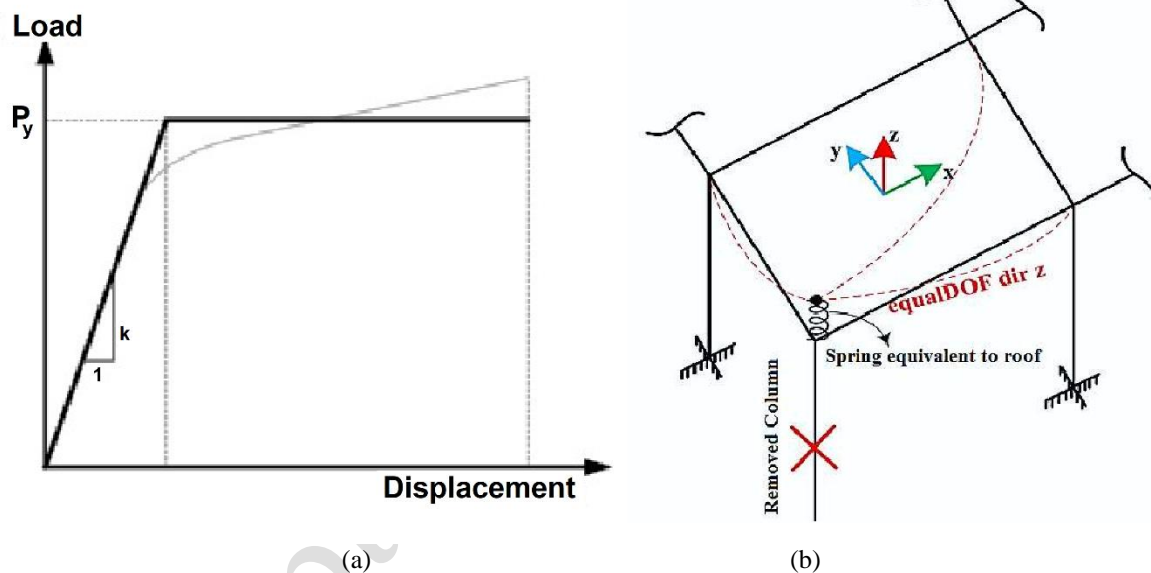


Figure (7) a- Simplified bilinear force-displacement curve, b- Definition of the spring equivalent to composite roof (Broujerdian et al., 2020)

3. Results of Column Removal Scenarios

Due to the redistribution of force, the displacement response has expanded in the stories and lateral nodes. Since this development is a clear result and consistent with previous research (such as Mohammadi Dehcheshmeh et al., 2022; Kim and An, 2009), it is not explicitly discuss in the section. Instead the results consist of displacement curves at the time of column removal, the sensitivity of maximum displacement and residual displacement to the time of column removal,

the sensitivity of maximum rotation and maximum stress of beam members to the time of column removal, and the sensitivity of beam and column members to the time of column removal are reported in detail.

3.1. Displacement at the Time of Column Removal

In studies related to progressive collapse, it is suggested that the removal of a column be considered at a time equal to 10% of the structure's period (GSA, 2016). In this study, 32 different column removal times have been considered for each scenario, as shown in Table (4).

Table (4) Selected column removal times in this study

	1	2	3	4	5	6	7	8	9	10
Removal Column Time (RCT)	0.005	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
Number	11	12	13	14	15	16	17	18	19	20
Removal Column Time (RCT)	0.1	0.12	0.14	0.16	0.18	0.2	0.25	0.3	0.35	0.4
Number	21	22	23	24	25	26	27	28	29	30
Removal Column Time (RCT)	0.45	0.5	0.55	0.6	0.65	0.7	0.75	0.8	0.85	0.9
Number	31	32								
Removal Column Time (RCT)	0.95	1								

In this regard, the selected range for these times has been selected from the minimum possible (very rapid column removal) to longer times (approximately equal to the structure's period). For shorter column removal times, smaller intervals have been selected, and as the selected column removal time increases, the steps become larger. The displacements at the locations of the removed columns for the three scenarios under consideration are shown in Figure (8). It can be seen that as the column removal time increases, the maximum and residual displacements decrease. This means that the results approach a static state with increasing column removal time, and the dynamic load effects, which are more critical, are ignored. It should be noted that in Scenario 3, which pertains to the middle column, a vertical displacement is observed in the first region of the curve. This displacement is influenced by the significant gravitational loads on the structure and represents the initial vertical deformation of the structure.

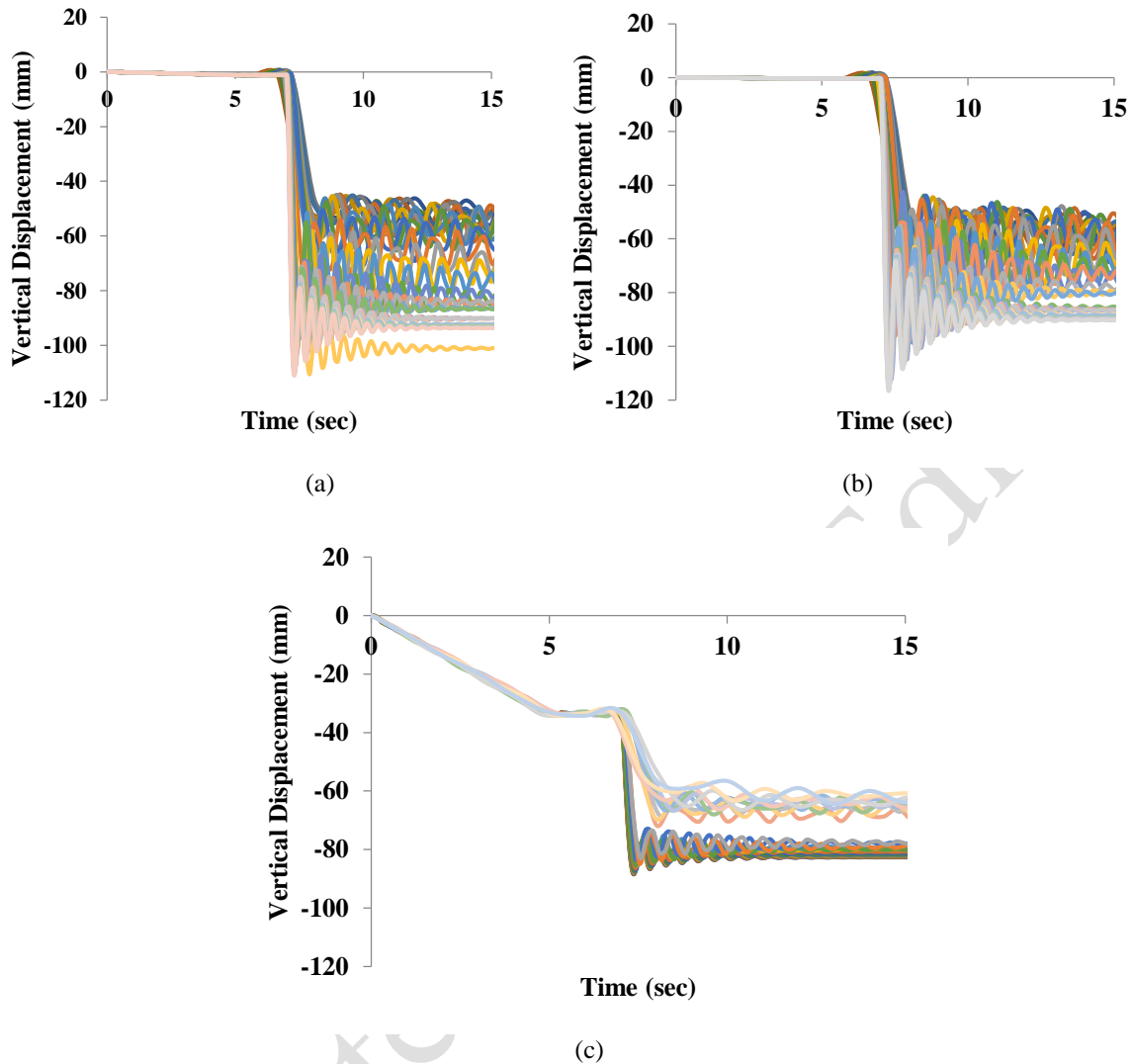
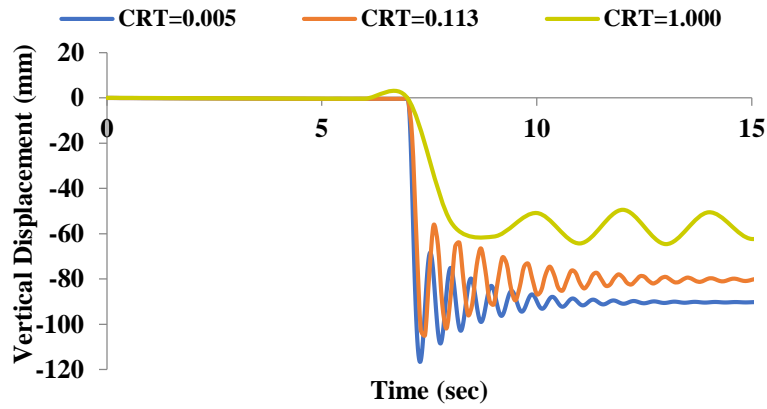
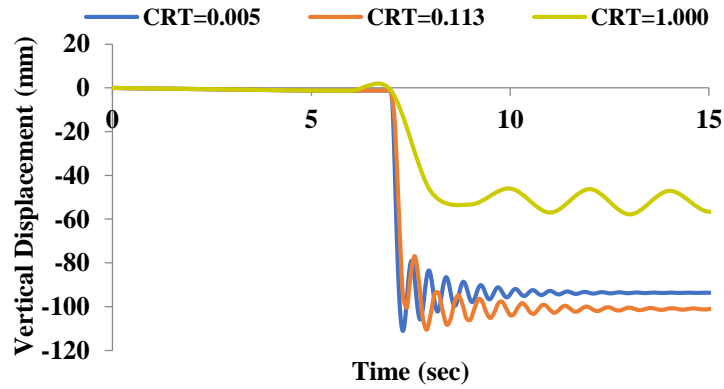


Figure (8) Displacements of the removed column at different removal times (a) Scenario 1, (b) Scenario 2, and (c) Scenario 3

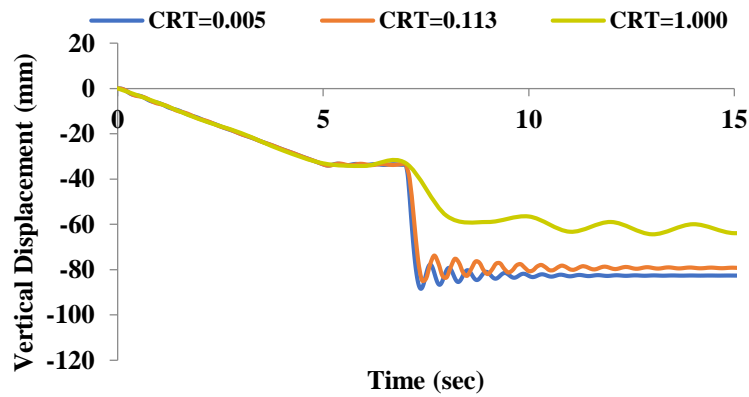
In such cases, as the column removal time increases, the graph deviate from their actual state due to the stair-step nature of the deformations. To better comparison the results, the maximum and minimum displacements of the removed column for the three scenarios under consideration are shown in Figure (8) and for three different column removal times, including 0.005 seconds, 0.113 seconds (equivalent to ten percent of the structure's period), and 1 second in Figure (9). Meanwhile, the error values for residual displacement will be 12, 10, and 4, respectively.



(a)



(b)



(c)

Figure (9) Comparison of maximum and minimum displacements of the removed column with typical column removal times (a) Scenario 1, (b) Scenario 2, and (c) Scenario 3

In Tables (5) and (6), the maximum and residual displacements for the three scenarios and the specified column removal times (0.005, 0.113, and 1 second) are reported, respectively. The

comparison of the results indicates that if the column removal times of 0.005 and 1 second are considered as the minimum and maximum times, the typical column removal times in researches can lead to significant differences in results. If the column removal time is set to a smaller value than the typical values, the error values for maximum displacement in Scenarios 1, 2, and 3 will be 11, 9, and 4, respectively. If the column removal time is greater than the typical values, the error values for maximum displacement and residual displacement in Scenario 1 will be 38 and 30, in Scenario 2 will be 43 and 40, and in Scenario 3 will be 24 and 22, respectively.

Table (5) Maximum displacements for various scenarios examined at three different column removal times

RCT	Maximum Disp. (mm)			Error (%)		
	S1	S2	S3	S1	S2	S3
0.005	117	111	88	11	9	4
0.113	105	102	85			
1	65	58	64	-38	-43	-24

Table (6) Residual displacements of different scenarios at three different column removal times

RCT	Residual Disp. (mm)			Error (%)		
	S1	S2	S3	S1	S2	S3
0.005	90	94	83	12	10	4
0.113	80	85	79			
1	56	51	62	-30	-40	-22

2.3. Sensitivity of Maximum Displacements to Column Removal Time

By examining the maximum vertical displacement in various scenarios, it can be stated that, as shown in Figure (10), the removal of middle columns is less sensitive to column removal time compared to the removal of corner and peripheral columns. Furthermore, Scenario 2 (peripheral column) is more critical than Scenario 1 (corner column). By analyzing the maximum and minimum column removal times, it can be claimed that the difference in maximum displacements at maximum and minimum times for Scenarios 1, 2, and 3 is 81%, 92%, and 37%, respectively.

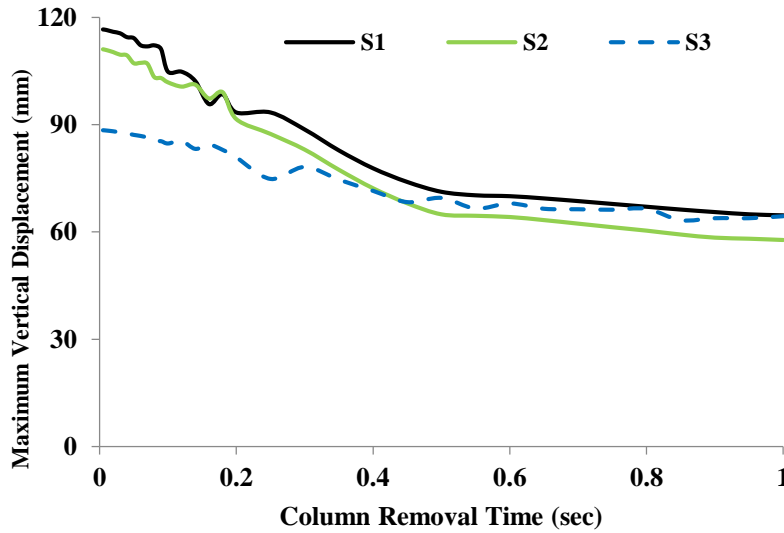


Figure (10) Maximum displacements of the removed column considering the different column removal times

3.3. Sensitivity of Residual Displacements to Column Removal Time

By examining the residual vertical displacements in various scenarios, it can be stated that the removal of peripheral columns (Scenario 2) is more sensitive to column removal time compared to the removal of corner and middle columns. Moreover, according to Figure (11), Scenario 1 (corner column) is more critical than Scenario 3 (middle column). By analyzing the maximum and minimum column removal times, it can be said that the difference in residual displacement values at maximum and minimum times for Scenarios 1, 2, and 3 is 61%, 82%, and 34%, respectively.

3.4. Sensitivity of Maximum Beam Rotations to Column Removal Time

By examining the maximum vertical rotations in various scenarios, it can be stated that the removal of corner columns (Scenario 1) is more sensitive to column removal time compared to the removal of peripheral and middle columns. Furthermore, according to Figure (12), Scenario 2 (peripheral column) is more critical than Scenario 3 (middle column). By analyzing the maximum and

minimum column removal times, it can be claimed that the difference in maximum rotations created in the beams at maximum and minimum times for Scenarios 1, 2, and 3 is 498%, 166%, and 63%, respectively.

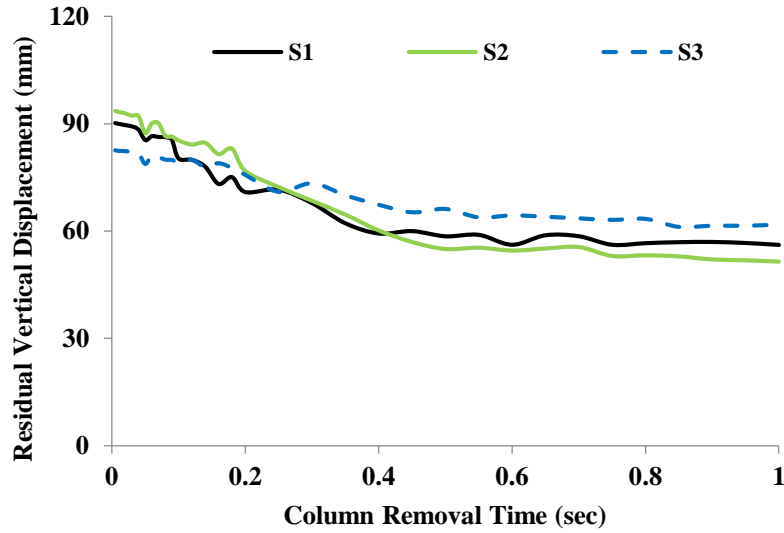


Figure (11) Residual displacements of the removed column considering the different column removal times

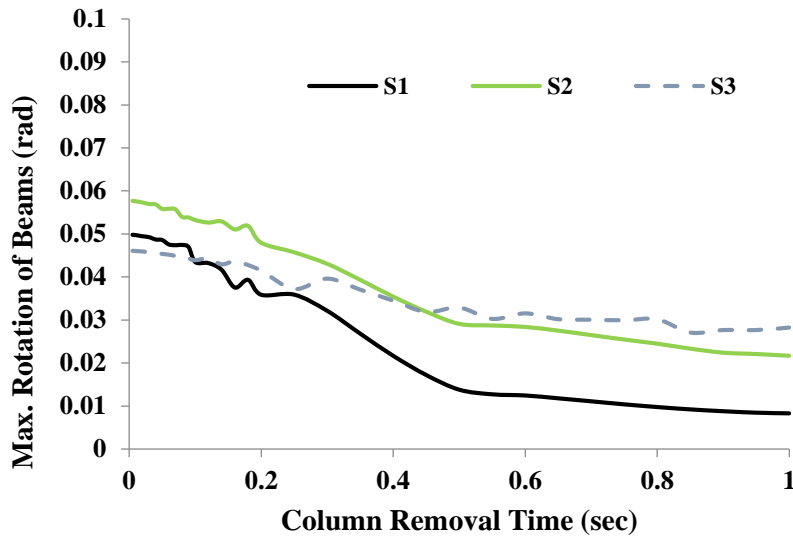


Figure (12) Maximum rotations of beams considering the different column removal times

3.5. Sensitivity of Maximum Stress in Beams and Columns to Column Removal Time

The examination of the maximum stresses of beams in various scenarios shows that (Figure 13), the column removal time does not significantly affect the differences in stress values in the beams. By analyzing the maximum and minimum column removal times, it can be claimed that the difference in maximum stress values generated in the beams at maximum and minimum times for Scenarios 1, 2, and 3 is 13%, 9%, and 5%, respectively. The examination of maximum stress in columns across various scenarios indicates that the removal of corner columns (Scenario 1) is more sensitive to column removal time compared to the removal of peripheral and middle columns. Additionally, according to Figure (14), Scenario 2 (peripheral column) is more critical than Scenario 3 (middle column). By analyzing the maximum and minimum column removal times, it can be claimed that the difference in maximum stress values generated in the columns at maximum and minimum times for Scenarios 1, 2, and 3 is 16%, 15%, and 5%, respectively.

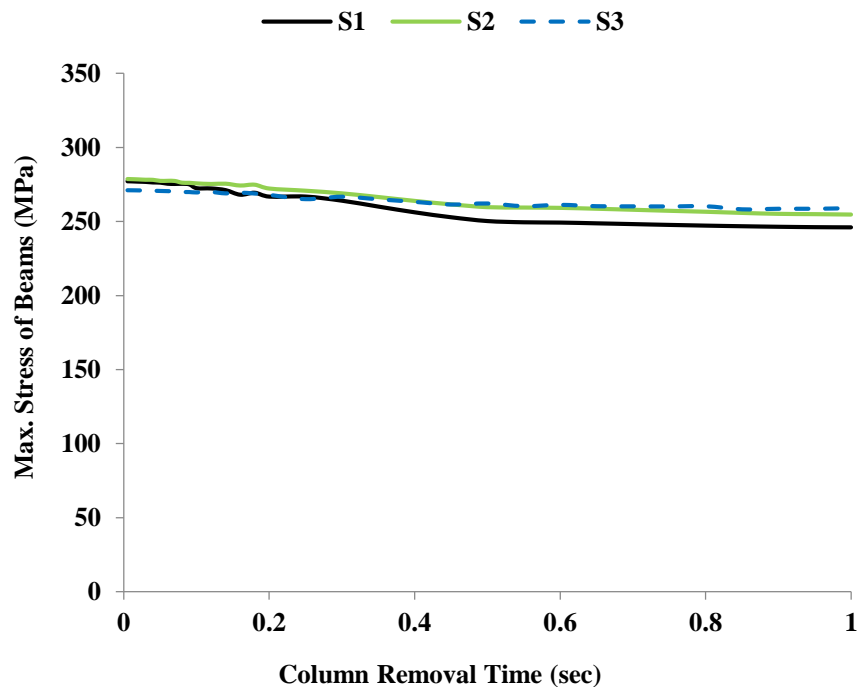


Figure (13) Maximum stress in beams considering the different column removal times

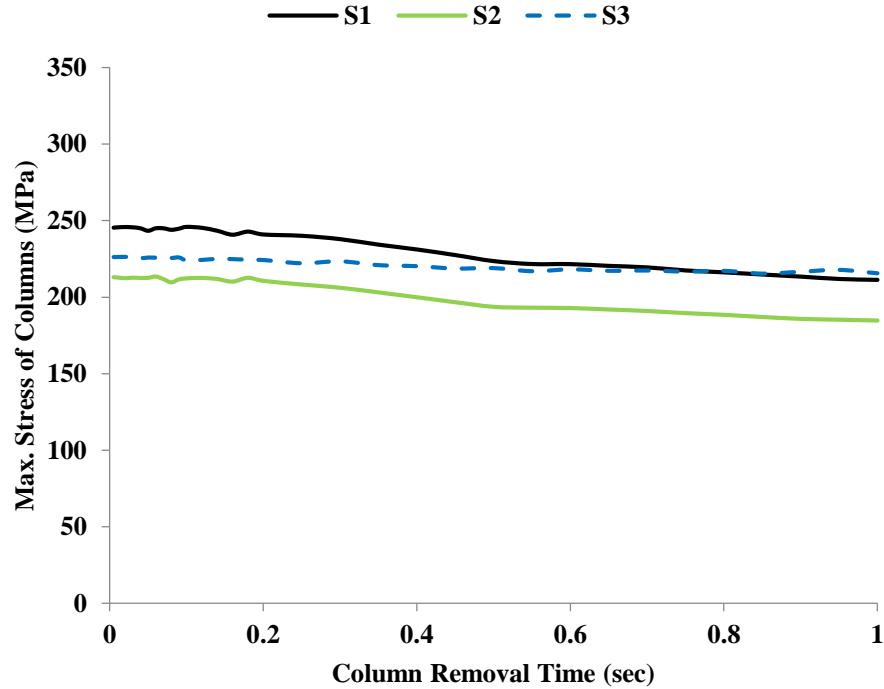


Figure (14) Maximum stress in columns considering the different column removal times

4. Results

In the phenomenon of progressive collapse, the redistribution of forces allows for the failure of structural members along any path, and with the change in load transfer path, the collapse process continues until the total collapse of the structure or a significant portion of it occurs. Thus, progressive collapse can have various scenarios. However, in the technical literature of structural and earthquake engineering, there is still a need for a more comprehensive examination of this phenomenon. Therefore, this paper analyzes the nonlinear behavior of a 4-story structure containing a special steel moment frame system under progressive collapse loads. In this context, three types of scenarios—removal of corner, middle, and peripheral columns on the ground floor—were selected with 32 different column removal times in the nonlinear Opensees software. Overall, the main results of the studied models related to a 4-story structure are as follows:

- By investigating the maximum vertical displacement in various scenarios, it can be said that the removal of middle columns is less sensitive to column removal time compared to the removal of corner and peripheral columns. However, for residual vertical displacement, the removal of peripheral columns (Scenario 2) is more sensitive to column removal time compared to the removal of corner and middle columns.
- Regarding the maximum vertical rotation in various scenarios, it was found that the removal of corner columns (Scenario 1) is more sensitive to column removal time compared to the removal of peripheral and middle columns. Meanwhile, the examination of maximum stresses generated in beams across various scenarios has shown that the column removal time does not significantly affect the differences in stress values in the beams and does not lead to substantial variations in results.
- The investigation of maximum stress in columns across various scenarios indicates that the removal of corner columns (Scenario 1) is more sensitive to column removal time compared to the removal of peripheral and middle columns.
- With an increase in column removal time, the maximum stress values in beams and columns, vertical displacement, beam rotation, and residual vertical displacement decrease.
- The difference in maximum displacement values at maximum and minimum times for Scenarios 1, 2, and 3 is 81%, 92%, and 37%, respectively, and for residual displacements, it is 61%, 82%, and 34%.
- The difference in maximum rotations created in the beams at maximum and minimum times for Scenario 1 is nearly 5 times.

Limitation: In this paper, capacity based design has been used to design the connections. Hence, connections have been assumed as perfectly rigid. Also, this paper analyzes the nonlinear behavior

of a 4-story structure containing a special steel moment frame system under progressive collapse loads. Increasing the height of the building and using the different lateral force resisting system can affect the results because of the different dynamic characteristics and gravity load distributions. Therefore, the results of this study should be used with caution.

Author contributions

Arzhang Yousefnejad: Conceptualization, Methodology, Formal analysis and investigation

Ehsan Darvishan: Conceptualization, Methodology, Writing - original draft preparation, Supervision, Review and editing.

Elham Rajabi: Conceptualization, Writing - original draft preparation, Supervision, Review and editing.

Ayoob Shakouri: Conceptualization, Writing - original draft preparation, Supervision, Review and editing.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability statement

The participants of this study did not give written consent for their data to be shared publicly, so due to the sensitive nature of the research supporting data is not available.

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