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Investigation of Progressive Collapse Occurrence under Main and Aftershocks Earthquake

Armin Karimi*

[1] Structural Engineer (MSc), Neptune Technical and Engineering Office, Abbas Abad, Mazandaran Province, Iran; eng.arminkarimi723@gmail.com.

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Abstract

When a critical load such as (floods, explosions, earthquakes, or fire) enters the structure, the severity of the critical load depends on the strength of the load-bearing elements of the structure, whether the structure is completely destroyed or remains intact. Or some members are out of basic mode. Progressive failure is a nonlinear phenomenon that starts from the damage of a part of the structure and ends with the entire structure and its total destruction. Progressive failure is usually in the structure due to the loss of one of the main members, a column. A column in the structure at the node where the column is removed causes a location change with a seismic nature. This change of location has occurred, and the dynamic analysis's explanation of the force caused by removing the column on other elements is desired. It is checked that the side columns of the removed column can bear the explained load, or after explaining the load on the columns, they are destroyed due to buckling and breakage. This study uses the Alternative Path (AP) method, independent of the failure factor proposed by GSA and DoD. Nonlinear dynamic analysis is also used. Two types of column removal scenarios have been used. Since the AP method is independent of the failure factor, two concentrated loads with a very large size and opposite direction (more than the capacity of the column) are instantly entered into the ground floor column (shearing the column) and cause column failure. The research was conducted in three models on the 5, 10, and 15 floors, and the results are discussed.

Keywords: Robustness index, Progressive failure, Aftershock and aftershock, Linear and nonlinear static analysis.

1 | Introduction

The structure must be designed in such a way that the behaviour of its elements is not compromised against critical loads. They should function in such a way that the overall behavior of the structure is not compromised. Each element fulfils its task; in other words, the correct performance of each element against a critical load prevents endangering the entire structural system. It prevents the gradual chain failure of the entire structure .Therefore, the strength of the structure has become one of the important variables in the

Corresponding Author: eng.arminkarimi723@gmail.com

design. Sturrock gave an interesting answer to this question in 2009 [1]. A structure will be strengthened when the initial damage of a part of the structure does not lead to the disproportionate collapse of the whole structure. Biandini et al. [2] define structural strength as a system's ability to withstand cross-sectional damage with disproportionate causes. British Safety Committee (1994scoss) [3] has defined structural strength as the ability to withstand disproportionate falls .

Gradual collapse is the most dramatic and frightening form of failure in structural engineering. This state usually occurs unexpectedly and usually has high damage. Robustness, on the one hand, depends on the internal characteristics of the structure, such as uncertainty and plasticity, and the characteristics of progressive failure behaviour and connections, and key elements, on the other hand, depend on the type of scenarios in which an unforeseen critical event can lead to damage and collapse. Unforeseen critical incidents are divided into several categories: accidental events, floods, fires, and unwanted differences between design and structural defects.

McGuire [4] stated that considering alternative load paths to increase cross-sectional strength is not the best choice to prevent structural collapse, but all the structure needs should be considered. Ellingwood and Leyendecker [5] stated three main ways to achieve progressive collapse: incident control, indirect design, and direct design. Our studies were all qualitative. Asprone et al. [6], in 2015, at the beginning of their research, stated that the increase in the complexity of Jose Shahri was one of the important factors in checking the correct design of structural elements.

They continued to investigate various aspects of structural robustness under the effect of very large critical events. The innovation of their work in evaluating the robustness index of presenting a robustness index was accepted and appropriate. They compared their proposed method with other methods of evaluating the stout index. They concluded that their proposed methodology is more developed and more applicable for prefabricated industrial buildings compared to the works of others.

The first research about the follower destruction after the collapse of a part of the Ronan building was proposed [6]. In this report, the authors investigated the progressive damage in the Ronan building. But the most effective phenomenon was the bombing of the Mora Federal building in Al-Ahma city in 1995, which killed and injured nearly 1,000 people and caused damage to more than 300 buildings. Assessments showed that more than 80% of the casualties were due to the destruction of the building and not the explosion itself. This building had a bearing beam on the front facing the explosion, which transferred the load of the 10 upper columns to the five-floor columns. The explosion destroyed one of the first floor's external columns and damaged several others. Removing one column on the first floor means the failure of the three columns above it, and the lack of an alternative load path causes the front half of the building to start moving [7]. Sturrock completed the most complete classification of progressive damage based on the destruction mechanism. Other researchers have commented on this field.

Ellingwood [7], in 2006, states that strength is the fundamental property of structural systems to prevent damage propagation and reduce the risks of disproportionate failure and gradual collapse. In 2016, Weng et al. [8] proposed progressive failure modelling to simulate the collapse of the entire structure of a structural system or a part of it. To manage an accurate progressive failure modelling, it is necessary to identify the damaged members and the failure propagation process correctly. Therefore, appropriate failure evaluation criteria are considered vital for modulating the analysis. Also, due to reliable criteria placement, structural members' collapse mechanisms can be accurately simulated [5]. Powel [9] compared the results of linear static, nonlinear static and nonlinear dynamic analyses and found that if the load factor of two is included in the static analysis, the results will be very conservative.

On the other hand, a pushdown analysis based on the energy available in the structural members, which needs to be developed transfer loads, is performed on all the effective linear spans above the column removed in the building frames. Therefore, only one displacement force response curve is required in the above analysis. As a result, many complications and problems are reduced by using this method. Finally, they implemented the proposed method with a case example on a steel bending frame structure. The study results for this model indicate that the response values of key elements (such as maximum vertical displacement, maximum twisting of the plastic hinge, maximum axial force, and maximum bending moment) in the nonlinear static analysis based on the energy method proposed in this paper generally align with dynamic analyses. The nonlinearity in the time history matches with reasonable accuracy. Therefore, the proposed nonlinear static analysis offers a suitable balance of speed and accuracy for estimating progressive failure [9].

Tsai [10], in 2012, proposed a different loading simulation method to analyze the progressive failure of building frames when they undergo column removal. Based on this, a double-headed beam (with columns on both sides) and middle support with nonlinear behaviour were considered in 8 different buildings. The results show that if the force-displacement response is from the linear loading method, the load capacity may be lower, and the displacement demand is higher than the loading method proposed in this research. The error created in the force-displacement response of the linear loading method is more dependent on the basis and foundations of the pseudo-static simulation in the linear loading method. Also, the difference in the force-displacement response of the two methods mentioned above will increase by increasing the amount of plasticity or the number of floors of the building frame. As a result, this research proposed an experimental and validated formula using displacement error estimation. This formula may help increase the accuracy of practical tools in linear loading and the loading proposed in this article to analyze the progressive failure of regular building frames [11].

2 | Construction and Grid of Geometric Model

After drawing the geometric model of the desired element of the structure, we must assign the type of materials used and their environmental and physical characteristics. In general, the modelling should be done in such a way that the nodes of the adjacent members match each other. And apply loads to each other so that we can see the node's behaviour at the desired point in case of an element failure. An example of modelling is shown in *Fig. 1*.

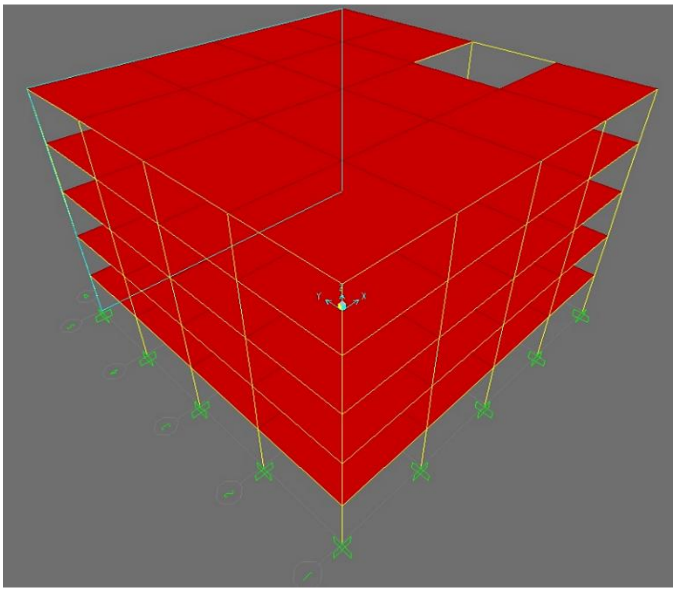

Fig. 1. Example of modelling.

The occurrence of severe earthquakes during the last decade and observing their harmful effects have once again revealed the defects and deficiencies in designing and implementing structures. The current method of designing structures is based on the resistance method, which includes estimating the base cut in the structure and its distribution in height, as well as determining the required resistance of the components against this load. Regardless of the shortcomings that exist in this method, expressing the behaviour of structural components through a single resistance parameter does not seem logical in many cases.

Determining the capacities does not determine the structure's real behaviour because the structure's behaviour is a combination of its components, and the components of the structure each have different behavioural characteristics depending on the intended efficiency and their type. In this way, it is necessary to find a method compatible with these characteristics, as it is very important to consider the parameters expressing the performance and capability of the structure. For structures, ductility means carrying significant non-elastic deformations before member destruction. A tensile member must first withstand large inelastic deformation without significantly reducing its resistance. Secondly, it should be able to absorb and dissipate a significant amount of earthquake energy through stable behaviour cycles.

In steel buildings, for the members to withstand the alternating loads caused by earthquakes in several consecutive cycles, they are designed in such a way that the behaviour of the members exceeds the elastic limit. Deformations occur in the members; in this case, if the members and connections can resist these deformations, the frame will act as a shock absorber of alternating loads. It will absorb a large amount of earthquake energy.

In this research, we will study the seismic behaviour of structural components. Behavior analysis is a type of nonlinear static analysis. The causes of the nonlinear behaviour of the structure can be divided into three categories: state change, geometric nonlinear behaviour and the nonlinear stress-strain relationship in the nonlinear behaviour of materials. Many factors affect the behaviour and stress-strain characteristics of the material. These factors include loading history in the form of elastoplastic response, environmental conditions such as heat, and loading duration. A lot of energy enters the structure during an earthquake. This energy is applied to the structure in kinetic and potential, which is absorbed or consumed. If the structure has no damping, its vibration will be continuous, but the vibration will decrease due to damping in the materials.

The concept of formability is the ability of the structure to withstand large and nonlinear shape changes without appreciable reduction in its resistance. The higher the structure's acceptance rate of inelastic deformation, the higher its formability.

A structure can be designed with a lateral resistance far less than the elastic state; the necessary condition to satisfy the functional goal in severe earthquakes is to provide the necessary formability. When the conditions for creating nonlinear shape changes are provided for a structure, the maximum lateral force during an earthquake is determined by the lateral resistance of the structure itself. Formability limits are one of the important issues in seismic design; two formability limits are defined as existing sub-formability and required formability.

The available formability is related to the structure and depends on the configuration, material characteristics, type of cross-section, gravity loads, reduction of hardness and resistance in reciprocating loads, etc. The required formability is the result of earthquake effects. It depends on parameters such as the earthquake's magnitude, the type of ground movement, soil effect, the structure's natural period versus the movement period, the number of round-trip cycles, etc. If the required ductility exceeds the available flexibility, the structure can change shape during the earthquake without reducing resistance.

Reciprocating loads can greatly affect the behaviour of the structure and its members. Drawing the load diagram against the change of location when the load is reciprocating is called a hysteretic diagram. The ductile response of a steel structure is dependent on achieving steel yielding. The non-deformable response of steel systems is the result of failure or instability. As a result, the key parameter in the design of deformable structures for maximum yielding in the elements of steel frames is the delay in the onset of instability and failure. To realize this issue, the first step is to choose the places where the elements flow in the frames, called plastic joints. These places include the ends of beams in bending frames. Seismic design in steel structures consists of ductility design and capacity design.

In order to design modern systems, for a suitable earthquake-resistant design, one should try to minimize the amount of wasted hysteretic energy in the main members of the structure. There are two important points of view on achieving this goal. The first point of view includes plans in which we try to reduce the energy input to the structure, including base isolation systems. The second point of view is focused on the mechanisms of energy loss in the structure itself. For this purpose, we use a series of equipment. These equipment are designed so that they waste part of the energy entering the structure, and as a result, the damage to the main structure caused by hysteretic energy loss is reduced. The nonlinear static analysis method (Pushover) is used to perform the analysis. This method gradually increases the load on the structure according to a certain pattern. By increasing the amount of load, the structure's weak points and failure modes are revealed to the extent of the destruction of the structure.

The loading is one-way, but the behaviour is cyclic, and the one-way force-displacement function correction and damping estimation consider the reciprocating mode of the load. Observing the structure's shifted geometry makes it possible to examine the linear shape of the structure and the place of plastic joint formation. In addition, by using a pushover curve, it is possible to observe the curve of base shear-roof displacement and the structure's spectral acceleration-spectral displacement. SAP2000 software can calculate the structure's performance point coordinates based on a specific demand spectrum. The amount of shear and displacement corresponding to the yield point is actually the maximum base shear and displacement expected per earthquake, generating a certain spectrum.

3 | Selecting the Earthquake Record

In this section, to check the impact of consecutive earthquake records, we must first have access to the earthquake database, so we will use the Peer database to receive the raw data of earthquakes. To select an earthquake we select an earthquake by examining the research conducted in the field of consecutive earthquakes, and then by accessing the Peer database, the raw data of the earthquake is received. The Coalinga, California earthquake of May 1983 at 11:42 pm with a magnitude of 5.6 on the Richter scale occurred along the San Andreas fault. In this earthquake, 6 people were killed, the centre of the city of Gualinga, California, was destroyed, and the oil fields caught fire. This earthquake seriously endangered only the north and northwest of the city. The focal depth of this earthquake was estimated to be about 10 km.

In *Fig. 2*, the red line shows the centre of the Koalinga anticline, and the red dots show the centre of the earthquake. Quaternary slates are pale blue, ternary stones are green, and gypsum stones are brown.

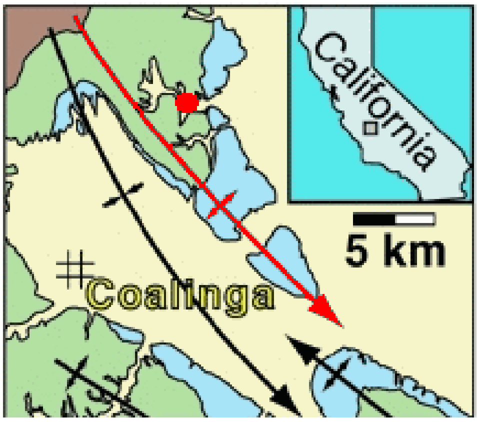

Fig. 2. Coalinga earthquake.

4 | Designing Models

In this section, the design of the investigated structures, which includes five-story, ten-story, and fifteen-story steel structures, is discussed despite the progressive deterioration. For the design of the samples, the dead load of the floors is 650 kg/m2, and the live load of the floors is 200 kg/m2. Also, the side wall load is 700 kg/m, assuming a wall of 22 cm. For columns, box sections according to the relevant tables and standard I-shaped sections have been used for beams. The desired side-bearing system in these structures is a steel bending frame. The sixth topic for dead and live load is the National Building Regulations and the earthquake load, the fourth edition of the 2800 regulation, and the structure conditions for the second type of soil and the region with high seismicity. Also, the regulations of the 10th topic of the National Building Regulations, 1392 edition, have been used for the design of the structure.

In this thesis, the unit of length is meter, and the unit of force is Newton. For the design of the sections, the maximum capacity was tried so that the design could be done economically. For example, some examples of the stress percentage of sections are provided. Progressive failure in the structure occurs when the partial (local) failure of one or more main structural members leads to the failure of adjacent members and eventually causes the rupture of a part of the structure or the failure of the entire structural system.

Progressive failure is a dynamic phenomenon that usually causes large structural deformations .In this phenomenon, the damaged system is replaced by load paths to survive.

One of the most important characteristics of progressive failure is that the final failure is not proportional to the initial failure. In conventional structures, energy input is absorbed by creating plastic joints in steel members, expanding cracks in concrete structures or failure in unreinforced masonry structures. This method of energy absorption has caused permanent deformations in the gravity-bearing members, which will destroy the structure or the very high cost of re-providing the operating conditions. In all the structures examined in this research, one of the internal columns of the first floor has been removed to consider the effects of progressive damage. See *Tables 1-4* that show this models.

Table 1. Specifications of the models investigated in this research.

The Number of Openings	Number of Floors	Model Name
4X4	5	MODEL-1
4X4	10	MODEL-2
4X4	15	MODEL-3

Table 2. Sections designed for MODEL-1.

The Cross-Section of the Beams	The Cross-Section of the Columns	The Floor
IPE-360	BOX-30X30T1. 8	First and second
IPE-330	BOX-25X25T1. 4	Third and fourth
IPE-300	BOX-20X20T1. 2	The fifth

Table 3. Sections designed for MODEL-2.

The Cross-Section of the Beams	The Cross-Section of the Columns	The Floor
IPE-360	BOX-45X45T2. 4	First and second
IPE-360	BOX-40X40T2. 2	Third and fourth
IPE-330	BOX-35X35T2	Fifth and sixth
IPE-330	BOX-30X30T1. 8	Seventh and eighth
IPE-270	BOX-25X25T1. 4	Ninth and tenth

Table 4. Sections designed for MODEL-3.

The Cross-Section of the Beams	The Cross-Section of the Columns	The Floor
IPE-450	BOX-55X55T2. 8	First and second
IPE-400	BOX-50X50T2. 6	Third and fourth
IPE-360	BOX-45X45T2. 4	Fifth and sixth
IPE-360	BOX-40X40T2. 2	Seventh and eighth
IPE-330	BOX-35X35T2	Ninth and tenth
IPE-330	BOX-30X30T1. 8	11th and 12th
IPE-270	BOX-25X25T1. 4	13th, 14th and 15th

The 3D structural model for model 1 representation in ETABS software that shown in *Fig. 3* also you can see the stress percentage in the designed sections of model 1 in *Figs. 4-8*.

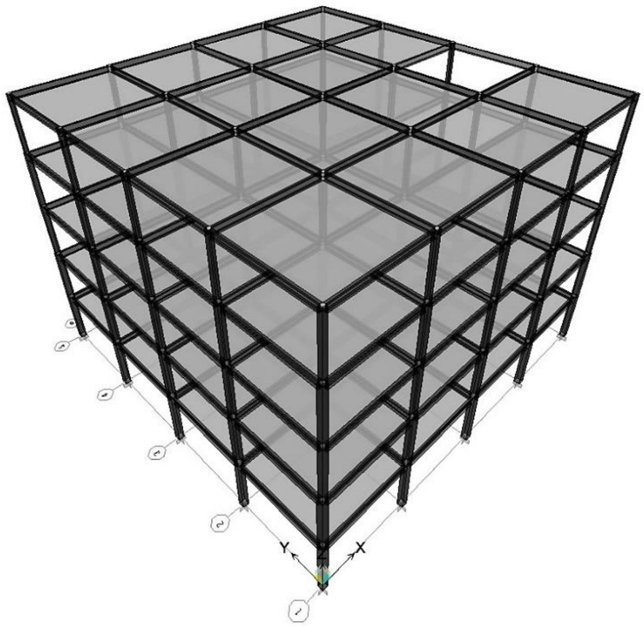

Fig. 3. Modelling of model 1 in ETABS software.

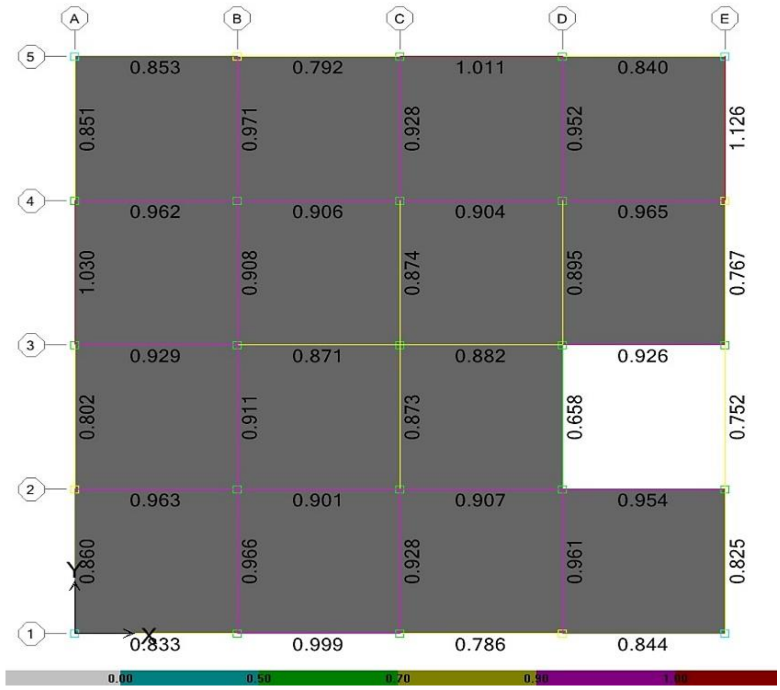

Fig. 4. The stress percentage in the designed sections of model 1 in ETABS software.

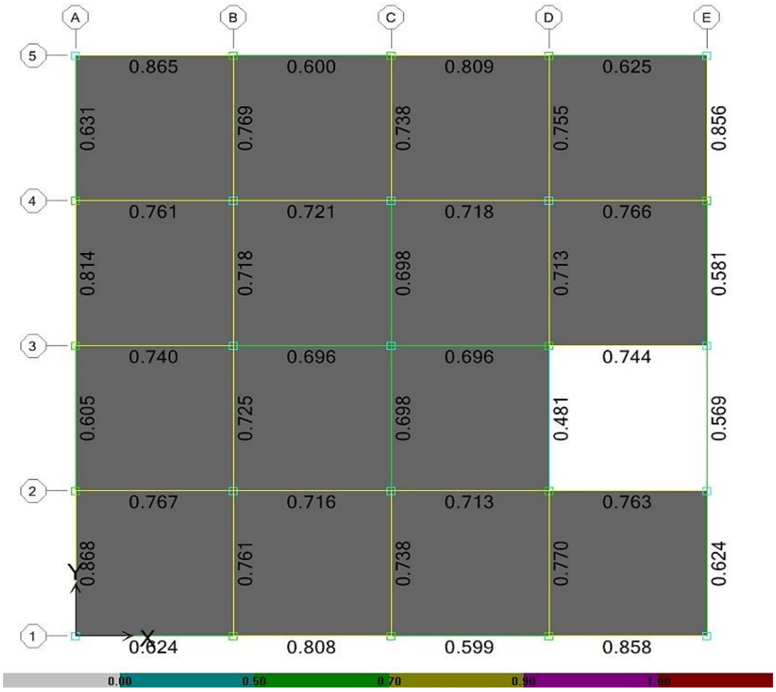

Fig. 5. The stress percentage in the designed sections of model 1 in ETABS software.

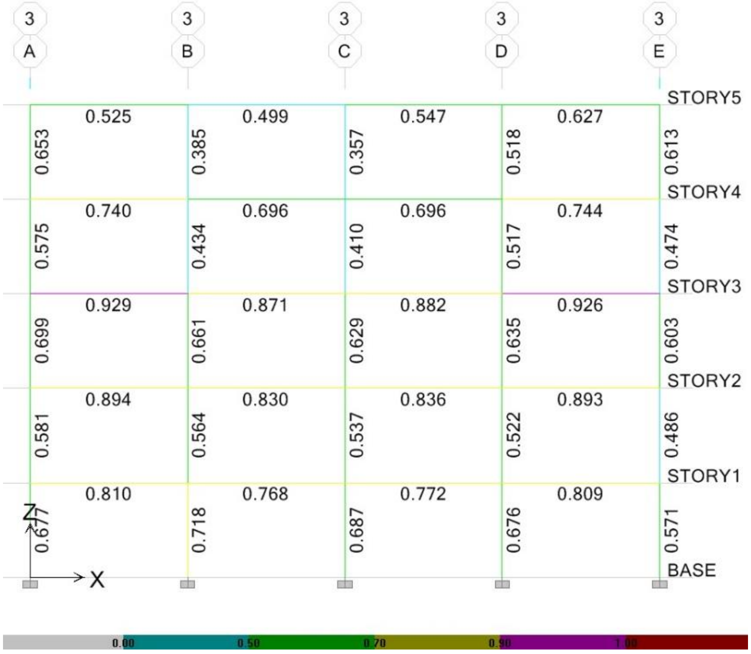

Fig. 6. The stress percentage in the designed sections of model 1 in ETABS software.

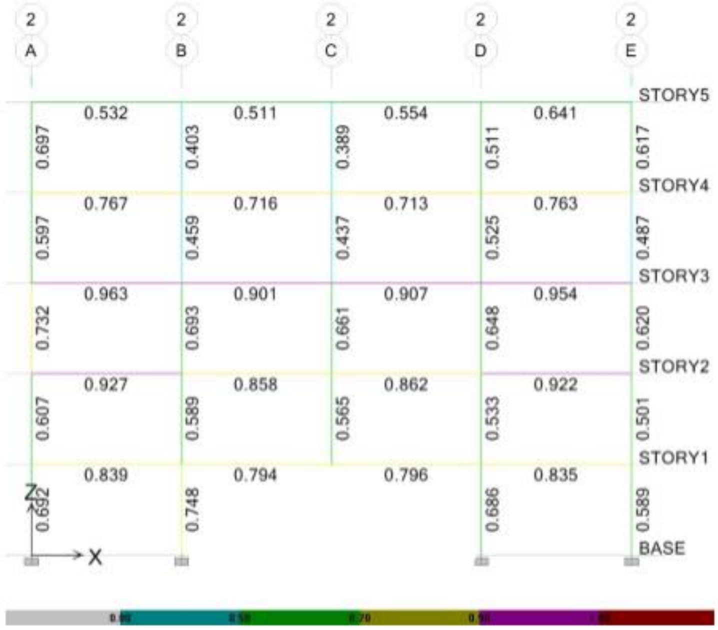

Fig. 7. The percentage of stress in the designed sections of model 1 in ETABS software and with progressive failure.

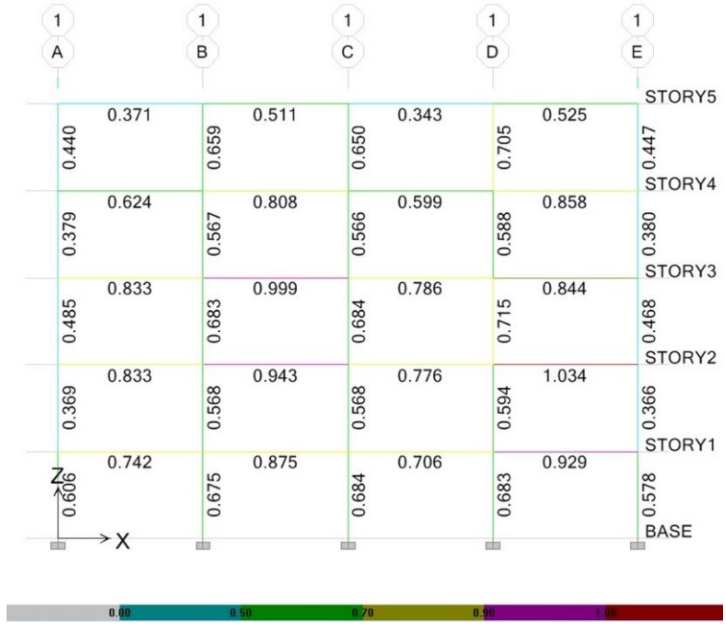

Fig. 8. The stress percentage in the designed sections of model 1 in ETABS software.

Fig. 9 and *Fig. 10* illustrate the three-dimensional models developed for this study using ETABS software. The models represent two different structural configurations that were analyzed to evaluate their response to progressive collapse under various load conditions. Both models were designed to replicate the realistic structural behavior of multi-story buildings under seismic and other dynamic loads. The accurate modeling of such systems enables a detailed investigation into the resilience of the structures and the potential failure mechanisms. These models form the basis for nonlinear static and dynamic analyses, aiding in understanding how different failure scenarios can unfold in practical engineering environments.

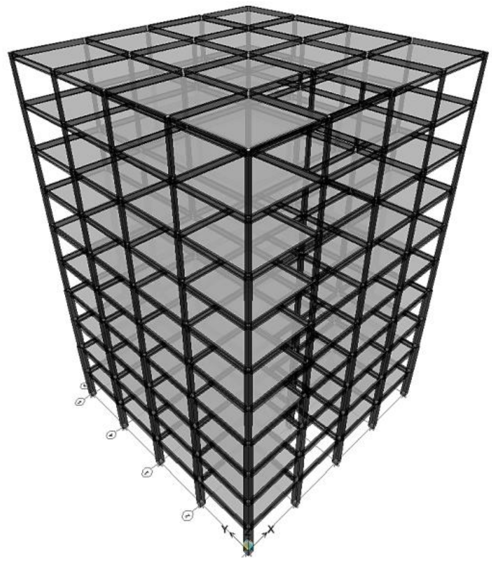

Fig. 9. Modeling model 2 in ETABS software.

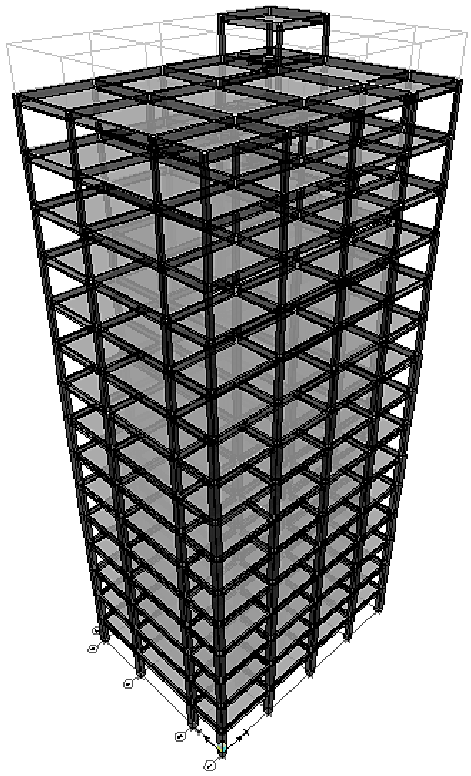

Fig. 10. Modeling model 2 in ETABS software.

5 | Column Deletion Scenario

This study employs the AP method, which operates independently of the failure criteria suggested by GSA and DoD, alongside nonlinear dynamic analysis. Two different column removal scenarios were considered. Since the AP method is not dependent on specific failure factors, two large, opposing concentrated loads (exceeding the column's capacity) are instantaneously applied to a ground floor column, resulting in its shear failure and subsequent collapse.

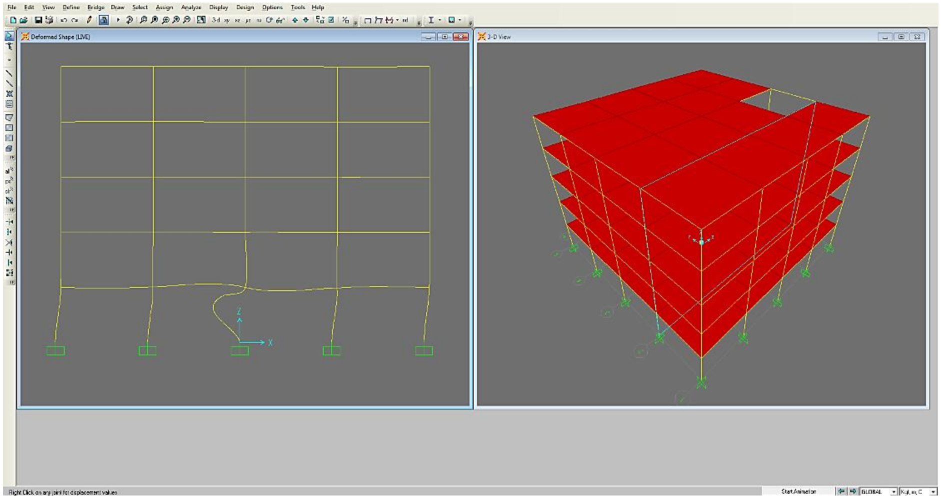

Fig. 11. Column damage in a 5-story structure.

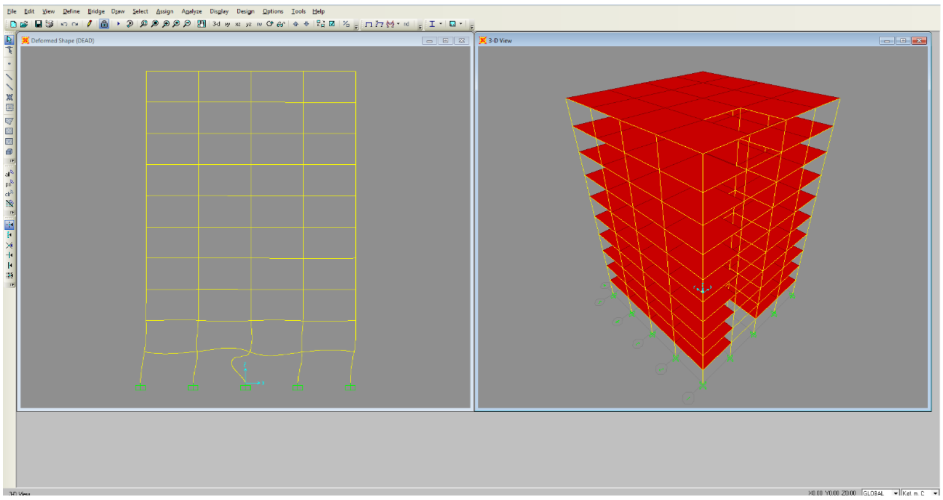

Fig. 12. Column damage in a 10-story structure.

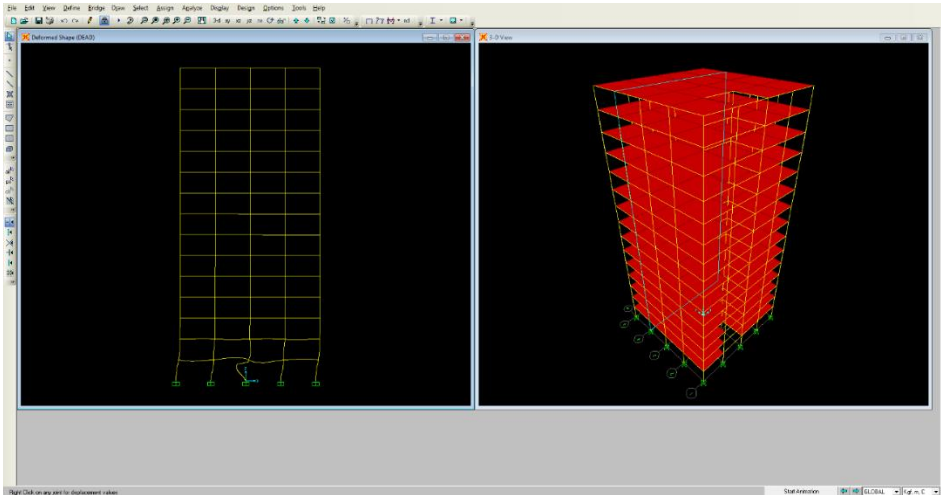

Fig. 13. Column failure in a 15-story structure.

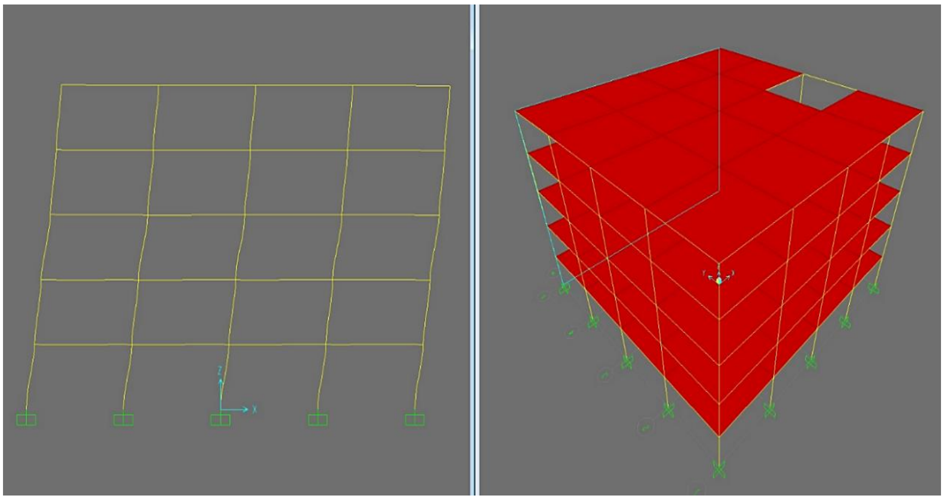

Fig. 14. The mechanism of forming plastic joints model 1 in SAP software.

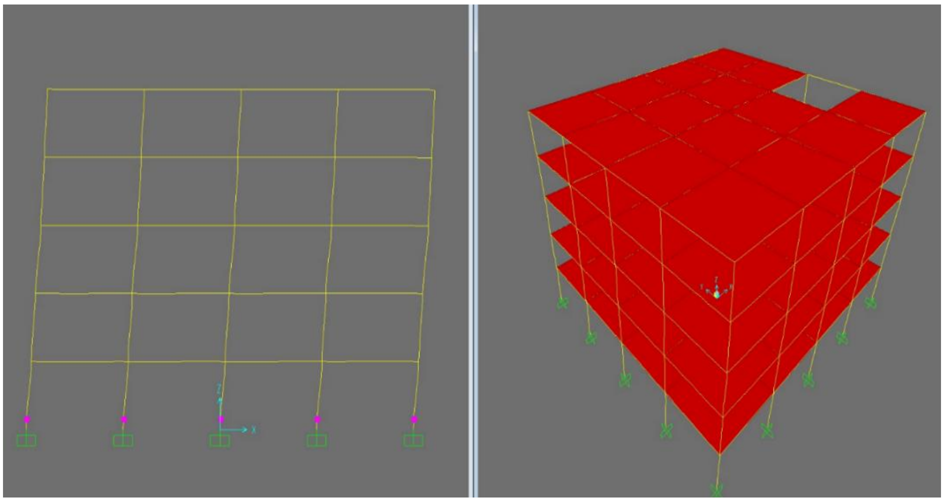

Fig. 15. The mechanism of forming plastic joints model 1 in SAP software.

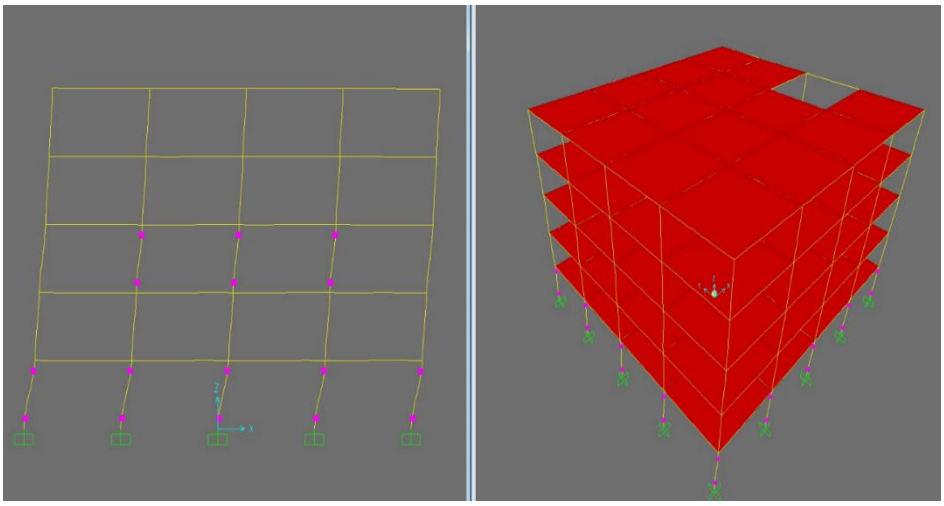

Fig. 16. Mechanism of forming plastic joints model 1 in SAP software.

6 | Conclusion

6.1 | Nonlinear Static Analysis

According to the mechanism of rupture and deformation of structures with progressive failure under investigation, we see the proper and regular operation of the system in such a way that at the point of destruction of the structure, plastic joints are formed in more than 60% of the members, and this is a sign of the proper design of the system. We also see the structure's performance in the area of life safety.

Emphasizing the reduction of progressive failure and stressing the coverage diagram of the systems, we reach this important point:

I. With the help of the hardness and slope diagram obtained in this diagram, we find that the system has good hardness.

II. The surface below the load-displacement diagram shows a lot of energy absorption in the system.

III. Considering the yield points and final points, we see the appropriate plasticity of the system.

IV. According to the surface under the curve of the linear part, we see the high resistance coefficient of the system.

6.2 | Dynamic Analysis of Nonlinear Time History

Coalinga earthquake record in structures with progressive damage.

This earthquake occurred in 1980 with a maximum acceleration of 0.38 gravity acceleration, and 19 hours later, an aftershock with a maximum acceleration of 0.28 gravity acceleration occurred. The following results were obtained from the analysis.

According to the response of the system, it can be seen that we see a very large effect of a strong aftershock on the response of the system, so in the five-story structure, the system performed very poorly in 125 seconds and with a 28% decrease in PGA, we see the response of the system with more than We are 100% more system displacement.

However, in the ten-story structure, this effect has become weaker in such a way that with the decrease in the acceleration of the earthquake in the aftershock, we do not see a large increase in the response of the system, while the response of the system was greater in the aftershock. In the fifteen-story structure, this effect has been reduced so that the system's response is appropriate in the aftershock and less in the main earthquake.

According to the structure deformation mechanism, it can be seen that in the five-story, ten-story, and fifteen-story models, the structure enters the mechanism. Its operating point is in the linear region. We see a large displacement of the system with the lowest acceleration, so it can be said that the system has given up before reaching the safety zone and cannot be operated properly.

According to the extracted diagram for the foundation shear of the investigated structures, it can be seen that the foundation shear of the structures is proportional to the performance of the displacement of the roof of the structure, and the performance of the structures in the post-seismic phase is not suitable.

Imperial Valley earthquake record in structures with progressive damage.

This earthquake occurred in 1979 with a maximum acceleration of 0.39 gravity acceleration, and 3 minutes later, an aftershock with a maximum acceleration of 0.15 gravity acceleration, and the following results were obtained from the analysis.

According to the system's response, we see a very large effect of a strong aftershock on the system's response, so in the next 12 seconds, the system performed very poorly. With a 61% decrease in PGA, we see a response of the system with more than 100% more displacement of the system. However, in the ten-story structure, this effect has become weaker in such a way that with the decrease in the acceleration of the earthquake in the aftershock, we do not see a large increase in the response of the system, while the response of the system was greater in the aftershock. In the fifteen-story structure, this effect still exists, so we are not suitable for the system's aftershock response.

According to the structure deformation mechanism, it can be seen that in the five-story, ten-story, and fifteen-story models, the mechanism is entered. Its operating point is in the linear region. We see a large displacement of the system with the lowest acceleration, so it can be said that the previous system has given up on reaching the safety zone and cannot be used properly.

According to the extracted diagram for the foundation shear of the investigated structures, it can be seen that the foundation shear of the structures is proportional to the performance of the displacement of the roof of the structure, and the performance of the structures in the post-seismic phase is not suitable.

Mammoth Lakes earthquake record in structures with progressive damage

This earthquake occurred in 1980 with a maximum acceleration of 0.441 gravity acceleration, and 2 hours later, an aftershock with a maximum acceleration of 0.11 gravity acceleration, and the following results were obtained from the analysis:

According to the system's response, it can be seen that we see a very large effect of a strong aftershock on the system's response, so 32 seconds later, the system performed very poorly. With a 75% decrease in PGA, we see a response of the system with more than 100% displacement of the system. In the ten-story structure, this effect still exists, and with the reduction of the acceleration of the earthquake in the aftershock, we see a large increase in the system's response.

According to the structure deformation mechanism, it can be seen that in the five-story, ten-story, and fifteen-story models, the mechanism is entered. Its operating point is in the linear region. We see a large displacement of the system with the lowest acceleration, so it can be said that before reaching it, it has submitted to the safety zone and does not have a proper operable function. According to the extracted diagram for the foundation shear of the investigated structures, it can be seen that the foundation shear of the structures is proportional to the performance of the displacement of the roof of the structure and the performance of the structures in the post-seismic phase is unsuitable.

Author Contribution

The author, Amin Karimi was responsible for the conceptualization, methodology, investigation, and writing of the manuscript. The author also conducted the analysis of the progressive collapse occurrence under main and aftershock earthquakes, contributing to the technical assessment of the findings.

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Data Availability

The datasets generated and analyzed during this study are available from the corresponding author upon reasonable request.

Conflicts of Interest

The author declares no conflict of interest.

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