Considering the Effect of Semi-Rigid Connection in Steel Frame Structures for Progressive Collapse

Fooad Karimi Ghaleh Jough, Mohsen Soori

***Abstract*—**Today, the occurrence of progressive failure in structures has become a challenging issue, requiring the presentation of suitable solutions for structural resistance to this phenomenon. It is also necessary to evaluate the vulnerability of existing and under-construction buildings to progressive failure. The kind of lateral load-resisting system the building and its connections have is one of the most significant and influential variables in structural resistance to the risk of progressing failure. Using the "Alternative Path" approach suggested by the GSA2003 and UFC2013 recommendations, different configurations of semi-rigid connections against progressive failure are offered in this study. In order to do this, the Opensees program was used to model nine distinct semi-rigid connection configurations on a three-story Special Area of Conservation (SAC) structure, accounting for the impact of connection stiffness. Then, using nonlinear dynamic analysis, the effects of column removal were explored in two scenarios: corner column removal and middle column removal on the first level. Nonlinear static analysis results showed that when a column is removed, structures with semi-rigid connections experience larger displacements, which result in the construction of a plastic hinge. Furthermore, it was clear from the findings of the nonlinear static analysis that the possibility of progressive failure increased with the number of semi-rigid connections in the structure.

***Keywords*—**Semi-rigid, nonlinear static analysis, progressive collapse, alternative path.

I. Introduction

In the realm of structural engineering, the resilience and integrity of steel frame structures against catastrophic events are paramount considerations. Progressive collapse, characterized by the disproportionate spread of local failure to adjacent members, presents a critical challenge in ensuring the safety and functionality of these structures under extreme loading scenarios. While extensive research has been conducted to enhance the robustness of steel frames, the role of semi-rigid connections in mitigating progressive collapse remains a topic of significant interest and investigation. Progressive failure is a chain reaction or damage dispersion. Local failure caused by specific reasons at a small portion of the structure may diffuse to other elements and eventually cause total failure. In brief, the local failure of an element may spread throughout the entire structure. Possible hazards and unconventional loads that can lead to progressive failure are defects in design or construction, fire, gas explosions, unexpected overload, vehicle accidents, bomb explosions, etc. Since the occurrence probability of these hazards is low, they are neither considered in the design nor included in secondary measures. Most of them are imposed in a short time span and result in dynamic responses. Researchers first became interested in progressive failure in the 1970s following the local building catastrophe in Ronan Point, England. The September 11, 2001 terrorist attacks on the World Trade Center further complicated the examination of increasing failure. Most building rules are based on the general principle that designs should be based on the maximum loads that a structure can support over its lifetime. General recommendations to offset the effect of progressive failure for structures loaded over their design load can be found in most popular codes [1].

The traditional approach to steel frame design often assumes idealized rigid connections between structural members. However, the reality of construction and material behavior introduces complexities that challenge this assumption. Semi-rigid connections, which exhibit intermediate levels of stiffness and rotational capacity, are frequently encountered in practice due to factors such as fabrication tolerances, material variability, and construction imperfections. Understanding the influence of these connections on the progressive collapse resistance of steel frames is essential for advancing both design standards and structural safety. Semi-rigid connections in steel frame structures play a crucial role in resisting progressive collapse. Progressive collapse refers to the failure of a primary structural element leading to the collapse of adjoining elements, often triggered by extreme events like explosions or earthquakes. Semi-rigid connections exhibit a certain degree of rotational flexibility, which can absorb and redistribute loads more effectively compared to rigid connections.

Progressive failure is a phenomenon whereby a building's entire structure collapses or a significant component of it collapses due to local failure or an initial flaw of one element spreading to other elements. This raises a number of questions regarding the new regulations' inadequacy in preventing building collapse. Common constructions have a minimal chance of collapsing since they are often designed to withstand seismic, nonstructural, and gravity loads. However, failure entails significant social and financial damage. Numerous failures, both local and global, have been reported as a result of impact, car crashes, explosions, poor design and implementation, terrorist strikes, and unintentional overloads. Kim et al. [2] use push-down analysis to evaluate the moment frames' resilience to increasing failure. According to this research, the vertical load at the relevant spans rises as a result of an increase in vertical displacement at the location of the

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deleted column. It has been demonstrated that the likelihood of progressive failure rises with decreasing story counts and increasing gaps. Increasing the span number, on the other hand, strengthens the resistance against progressive failure. They came to the conclusion that the maximum load coefficient at dynamic analysis is rather lower than their alternatives from push-down analysis after comparing the load-displacement relation at push-down and nonlinear dynamic analysis. This suggests that when estimating the innate ability of the developing resistances toward progressive failure, the push-down analysis might be more conservative. By increasing the seismic design load, the structure's resistance toward progressive failure rises; on the other hand, the seismic load contribution to the progressive failure resistance of the building diminishes by decreasing the floor numbers and enlarging spans.

Using the FCM-PSO Method, Jough and Şensoy [3] examined steel moment-resisting frame Reliability via Interval Analysis to improve accuracy and save execution time while computing seismic fragility curves. Using finite element simulations, Jough and Golhashem [4] examined the out-of-plane motion of informal brick constructions, the self-weight axial deformation of walls constructed using the lightest masonry materials available today was decreased. Jough and Aval [5] employed the fuzzy C-means algorithm-based adaptive neuro-fuzzy inference framework to build an Special Moment Resisting Frame (SMRF) structure's seismic susceptibility curve. This allowed for the incorporation of epistemic uncertainty and increased computation accuracy. Ghasemzadeh et al. [6] examined the aspects that define and place infrastructure projects in context in order to point out and illustrate the present shortcomings of using BIM for infrastructure projects. The unpredictability of epistemic knowledge using a group-based data processing method in contrast to the previously outlined techniques, Jough et al. [7] implemented vulnerability in order to keep the same computation time while increasing power and output precision along with accuracy. Jough and Ghasemzadeh [8] presented the unknown interval assessment of steel the moment the framework by building of 3d-fragility curvature towards optimized fuzzy methods investigation, which aimed to improve precision and reduce execution time during driving the 3d-fragility curvature.

Jough [9] looks at how steel wallposts affect structural concrete walls' out-of-plane action in attempt to produce wallposts for masonry walls with lower adjustment elements. To decrease the environmental pollution during manducating process, sustainable CNC machining operations is studied by Soori et al. [10]. Soori and Jough [11] evaluate the use of artificial intelligence in the optimization of steel moment frame structures in order to improve the performance of these structures under operating conditions. Jough and Ghasemzadeh [12] have created SMRF reliability prediction based on the combination of neural network and incremental dynamic analysis, with the goal of improving performances by reducing random uncertainty in steel structures. Soori et al. [13] evaluated uses of smart robotics systems to improve productivity in industry 4.0. Prediction of seismic collapse risk in steel moment framed structures by metaheuristic algorithm is implemented by Jough [13] to enhance the performances of steel moment framed structures.

In 2011, Liu et al. [14] approach to estimate the occurrence probability of progressive failure and then conducted a range of analyses at UCF, including linear static, nonlinear static, and nonlinear dynamic. According to a 2D model analysis, the linear static approach is more cautious. Conversely, the nonlinear static and dynamic techniques lead to a more cost-effective design. Additionally, it showed that structures should be constructed for progressive failure because those that are gravitationally optimized in accordance with seismic design requirements are more vulnerable to it. The impact of the steel filler plate on the resisting frame during the progressive failure was examined by Kim [15]. According to the findings of the nonlinear static analysis, these plates lessen the likelihood of progressive failure. It was also observed that increasing the thickness of the steel filler plate increases the resistance toward progressive failure.

This paper aims to explore the effect of semi-rigid connections on the progressive collapse behavior of steel frame structures through a comprehensive review of existing literature, numerical simulations, and experimental investigations. By elucidating the mechanisms governing the response of semi-rigid connections under extreme loading conditions, valuable insights can be gleaned to inform more accurate predictive models and design guidelines. To achieve this, nine different semi-rigid connection configurations on a three-story SAC structure were modeled using the Opensees tool, taking the effect of connection stiffness into consideration. The impacts of column removal were then investigated in two scenarios—corner column removal and center column removal on the first level—using nonlinear dynamic analysis. The findings of the nonlinear static analysis demonstrated that constructions with semi-rigid connections undergo greater displacements upon the removal of a column, leading to the formation of a plastic hinge. Moreover, the results of the nonlinear static analysis made it evident that the number of semi-rigid connections in the structure raised the likelihood of progressive failure.

II. Progressive Failure Analysis Methods

In order to assess the building subjected to progressive failure with the guidelines have suggested three methods: linear static, nonlinear static, and nonlinear dynamic analysis. In this simple method (Linear Static Method), to assess the elements under progressive failure, a coefficient known as Margin of Load Increase Factor (MLIF) is chosen to include the effect of the element's geometry and the dynamic load at the gravitational loading of the elements [16]. By taking the structure type, such as steel, concrete, etc., and the type of connections into consideration, a coefficient called the magnification factor (MLIF) is evaluated for each of the elements. The smallest magnification factor corresponds to the elements connected to beams. In the nonlinear static method, the coefficient obtained for the dynamic loading is added to the gravitational loading of the elements above the removed element. This coefficient is determined by the type of building and the type of its connection and is chosen based on the tables in the guidelines. Magnification dynamic load factor (MDF) for nonlinear static analysis is obtained from the recommended provisions of ASCE 41-06. One of the primary tasks in nonlinear static analysis is defining the load model of the element's deformation [17]. Fig. 1 schematically portrays the load-deformation of the flexural elements with its acceptable value for each of the performance levels: immediate occupancy (IO), life safety (LS), and collapse prevention (CP). The pushdown analysis (vertical nonlinear static analysis) is conducted in the following steps: 1) the static analysis is conducted under the permanent loads imposing the structure and the evaluated internal load of the pre-eliminated element. 2) The model is modified by replacing the removing element with its internal loads, accompanied by the existing permanent loads. 3) A unit force is vertically applied to the end of the eliminated element. This force is gradually increased step by step until the control node reaches the designated target displacement or the structure collapses. In this regard, displacement-control nonlinear static analysis with initial conditions to include the tension generated from the permanent loading of the building was used. The P-Δ effect is also considered in this analysis [18]. Afterward, the axial force-vertical displacement graph of the eliminated element's upper node is obtained. The evolutionary parameter, called the load factor, is introduced by dividing the axial force of this graph by the axial loading from the first step, where the eliminated element is subjected to the permanent loading. As this parameter reaches one, the probability of progressive failure due to eliminating the load-resisting element increases. In addition, if this value is less than one, progressive failure occurs at the building resulting from the elimination of a load-resisting element [5].

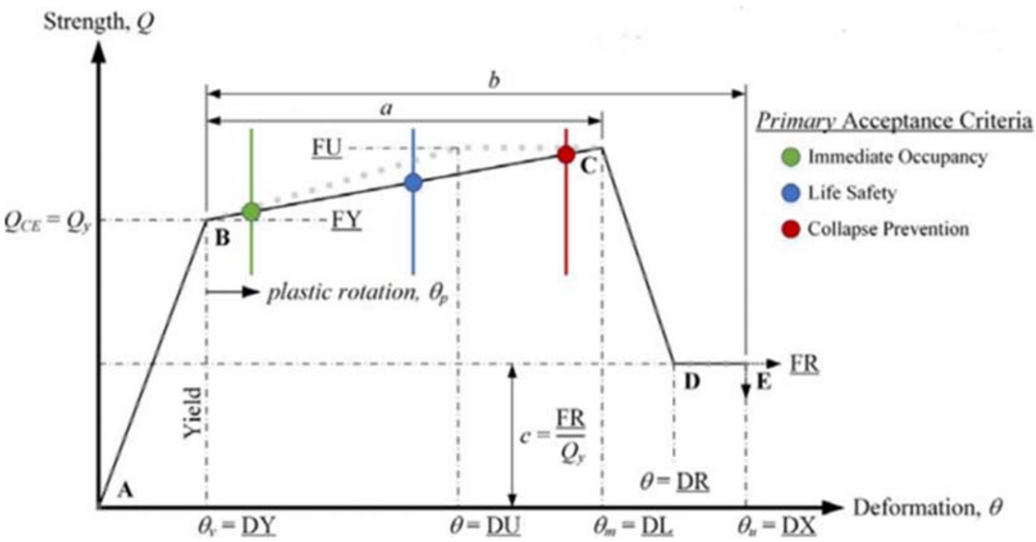


Fig. 1 Generalized component backbone curve

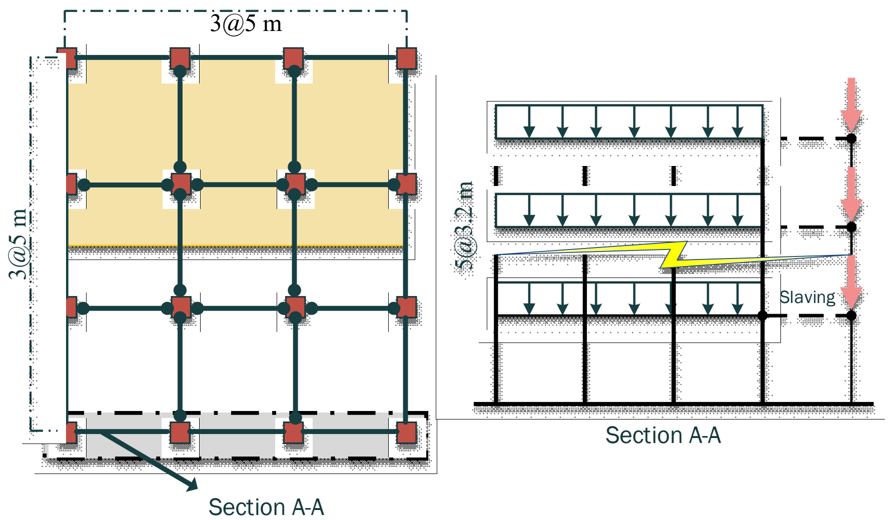


Fig. 2 Five story case study: plan view and elevation view

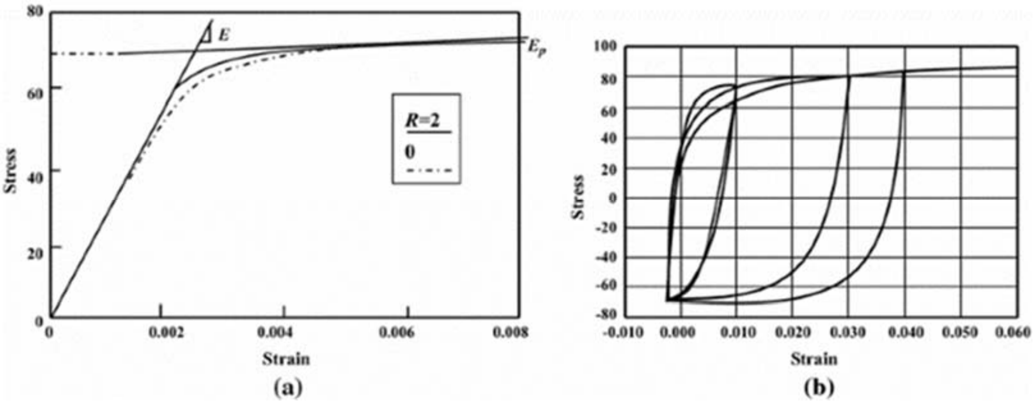


Fig. 3 Steel02 material model of OpenSees: (a) Stress-strain curve with monotonicity; (b) Hysteretic behavior of Steel02

TABLE I
DESIGN SECTIONS FOR CONSIDERED STRUCTURE

Story	Column external	Column internal	Beam internal	Beam internal
1	BOX 180x180x1.6	BOX 200x200x1.6	IPE 300	IPE 330
2	BOX 180x180x1.6	BOX 200x200x1.6	IPE 300	IPE 330
3	BOX 180x180x1.6	BOX 200x200x1.6	IPE 300	IPE 330

III. CASE STUDY

In this study, three story building was investigated by taking the panel zone into consideration. Fig. 2 displays plan and elevation of the analyzed frame. Steel used in beams and columns are ASTM A992 with module of elasticity and yield tension of 2.1e6 and 345 Mpa [19]. The W-sections used for beams and columns for all the models are depicted in Table I. As previously indicated, the alternative path of the UFC code is used for progressive failure analysis. The number of scenarios for elimination is according to the UFC. According to the UFC, identifying critical columns for elimination and assessing the performance level of the building subjected to progressive failure should be as follows:

1. Eliminated columns from the plan include the corner and middle columns.
2. After identifying the critical columns in the plan, removing columns within the height should be as follows: a) column of the first story from the ground, b) column of the story beneath the roof, c) column of the story in the middle of the building.

To model the beam and column elements, the disp-Beam-Column element was used. This element is nonlinear and considers plasticity distribution along the length of the element. To model the behavior of the steel used in the beam and column elements, the bilinear kinematic tension-strain diagram was assigned to the elements. Steel02 was used for all elements, which contains a transitional curve at the intersection of the initial and secondary slopes in its strain stress diagram. This curve prevents the element's local stiffness matrix from sudden change and provides a soft transition from the elastic to the plastic region. 2% stiffness hardening and a maximum of 15% ductility were assigned to the plastic phase. Stress-strain diagram of the utilized steel is provided in Fig. 3 [20].

The fiber section was assigned to the sections where they are divided into smaller potions, and the stress-strain response of the material is the summation of their responses. In addition, the effect of large displacement is accommodated in the geometrical stiffness matrix with co-rotational transformation. The fiber section only models the uniaxial force-deformation and the moment-curvature behaviors; therefore, the combined section is used to include the force-shear deformation behavior. Acceptable seismic performance of the moment-resisting frame not only relies on its beams and columns, but appropriate behavior at the connections is also required. As a result, designers should prevent connection failures in buildings where ductile behavior and super-elastic deformation with negligible strength reduction are expected. Such an objective in steel buildings may be addressed by preventing failure and deformation of the columns' flange, local yielding and buckling of it, and panel zone failure. The connection zone is under combined moment, shear, and uniaxial loads. The panel zone in the analytical model of this study is modeled by rigid boundaries, as presented in Fig. 4 [21]. Panel zone deformation is controlled by two bilinear springs located at its corners. Fig. 4 illustrates how the trilinear behavior of the connection zone is modeled with two bilinear springs. As portrayed in Fig. 4, the first slope after the rupture represents the connection zone behavior after the rupture initiates and the plastic capacity is fully reached. As the plastic capacity is obtained, a small slope (α = 1%) according to Fig. 4 can be selected. The Steel02 material was used to model each of these springs. The force-displacement behavior of the Steel02 is depicted in Fig. 3 [19].

One type of semi-rigid connection, according to Table II, in addition to a rigid one, was used to assess the effect of the connections on the steel frame. Regarding the recommendations of the European guideline, the connection stiffness can be obtained by:

$$q_p = \frac{M_{pb}}{(E_b I_b)/(5 d_{be})}, k_{sup} = \frac{25 E_b I_b}{L_b} \tag{1}$$

In these equations, L_b is the beam length and d_{be} is the depth of the web. The responses are compared for these nine different cases in different configurations, as shown in Fig. 5.

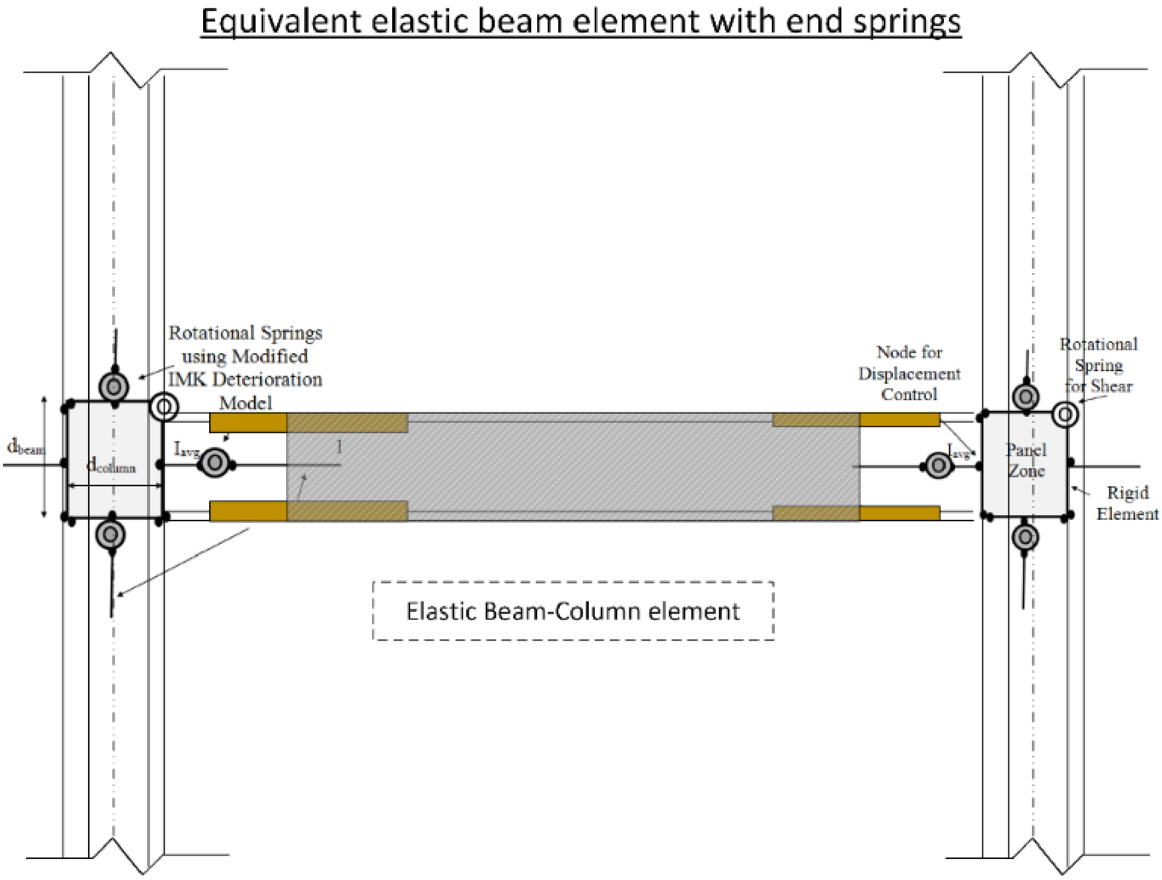


Fig. 4 The model of the sample structure with spring rotation.

TABLE II
DETAILS OF SEMI AND RIGID CONNECTION

Connection	Strength of connection	Stiffness of connection
Rigid	$1.2M_{PB}$	∞
Semi-Rigid	$0.6M_{PB}$	$0.8M_{PB}$

IV. NONLINEAR STATIC ANALYSIS RESULTS

In order to investigate the amount of damage, the displacement-load coefficient graph is plotted for nonlinear static analysis of all the models. Furthermore, a plastic hinge formation sequence is provided as well. Deformation in this graph is the vertical displacement of the upper node in the eliminated column. The load coefficient is the ratio of the imposed load at each step of the nonlinear static analysis to the total gravitational load of the progressive failure. Load coefficient – upper node displacement of the middle and corner columns of the first floors are depicted in Fig. 6 [14].

For example in Fig. 6, the load-displacement diagrams for the top node of the corner and middle columns on the first floor are shown. According to the mentioned figure, the first plastic hinge in the immediate occupancy performance occurs due to the removal of the corner column with a displacement of 18 cm in the beams of the first and second floors. In the scenario of removing the middle column on the second-floor beam, the displacement is 11 cm. The first plastic hinge in the life-safety performance level occurs with a displacement of 57 cm in the scenario of removing the corner column and 51 cm in the scenario of removing the middle column. In the collapse prevention performance level, for the removal of the corner column and the removal of the middle column, the displacements are 88 cm and 51 cm, respectively. In Fig. 6, the stages of plastic hinge development are illustrated for different performance levels for both scenarios. Observing the presented diagrams, it is noticed that the S3 model, with all its connections being moment-resisting, forms more plastic hinges in response to the removal of both the middle and corner columns compared to the S1 model. Additionally, comparing the two models reveals that in the S1 model, the first plastic hinge forms in the second-floor beam, while in the S3 model, this occurs simultaneously in both the first and second-floor beams. The stages of plastic hinge growth for various performance levels in both cases are shown in Fig. 6. Based on the presented diagrams, it is observed that the S9 model, compared to the S1 model with all its connections being moment-resisting, forms more plastic hinges in response to the removal of both the middle and corner columns, resulting in a greater displacement. The maximum displacement is observed in this condition, indicating the worst-case scenario. Additionally, by comparing these two models, it is evident that in the S1 model, the first plastic hinge forms in the second-floor beam, whereas in the S9 model, this occurrence takes place in the second-floor beam. The removal of the corner column with a displacement of 33 cm in the first-floor beam is what causes the first plastic hinge in the uninterrupted imitate occupancy performance, as shown in Fig. 6. The displacement is 13 cm in the case when the middle column on the second-floor beam is removed. Moreover, the development of the first plastic hinge in the life-safety performance level happens with a displacement of 52 cm in the scenario of removing the corner column and 45 cm in the situation of removing the middle column. The displacements in the collapse prevention performance level are 93 cm and 40 cm, respectively, for the removal of the center column and the corner column.

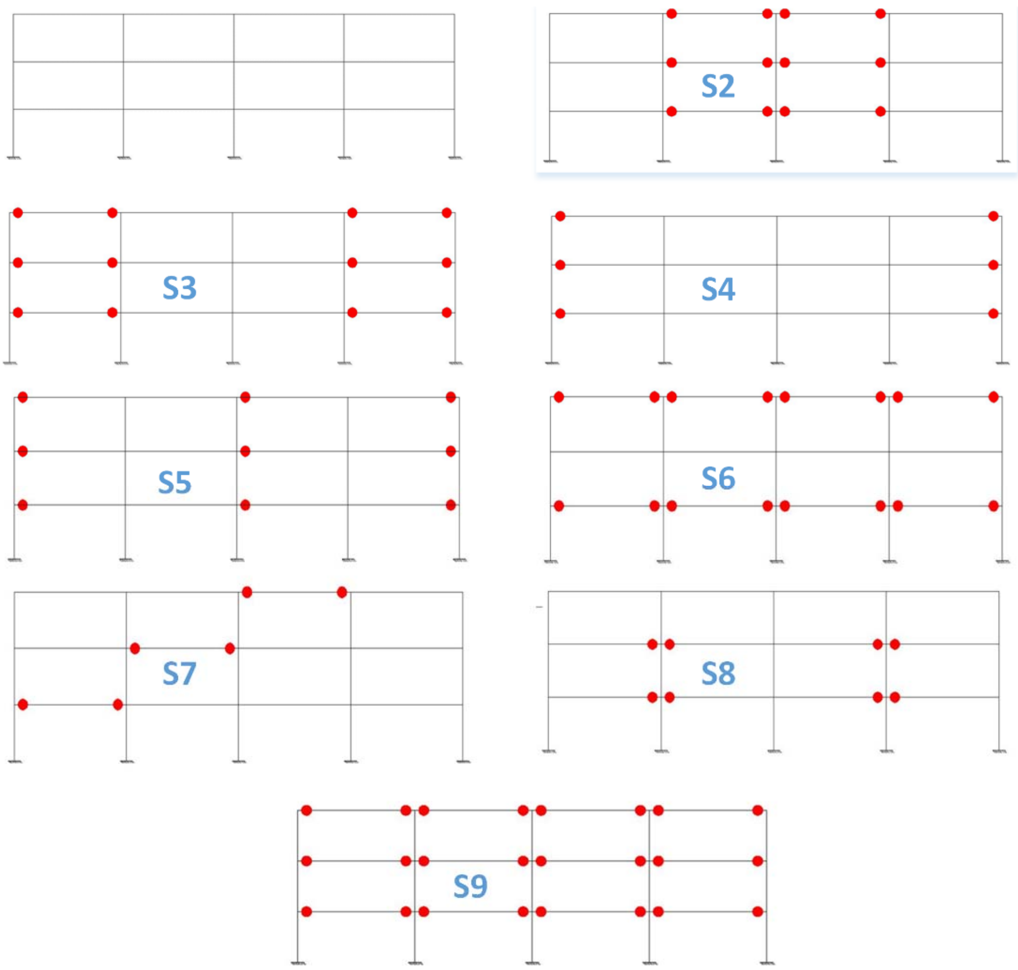
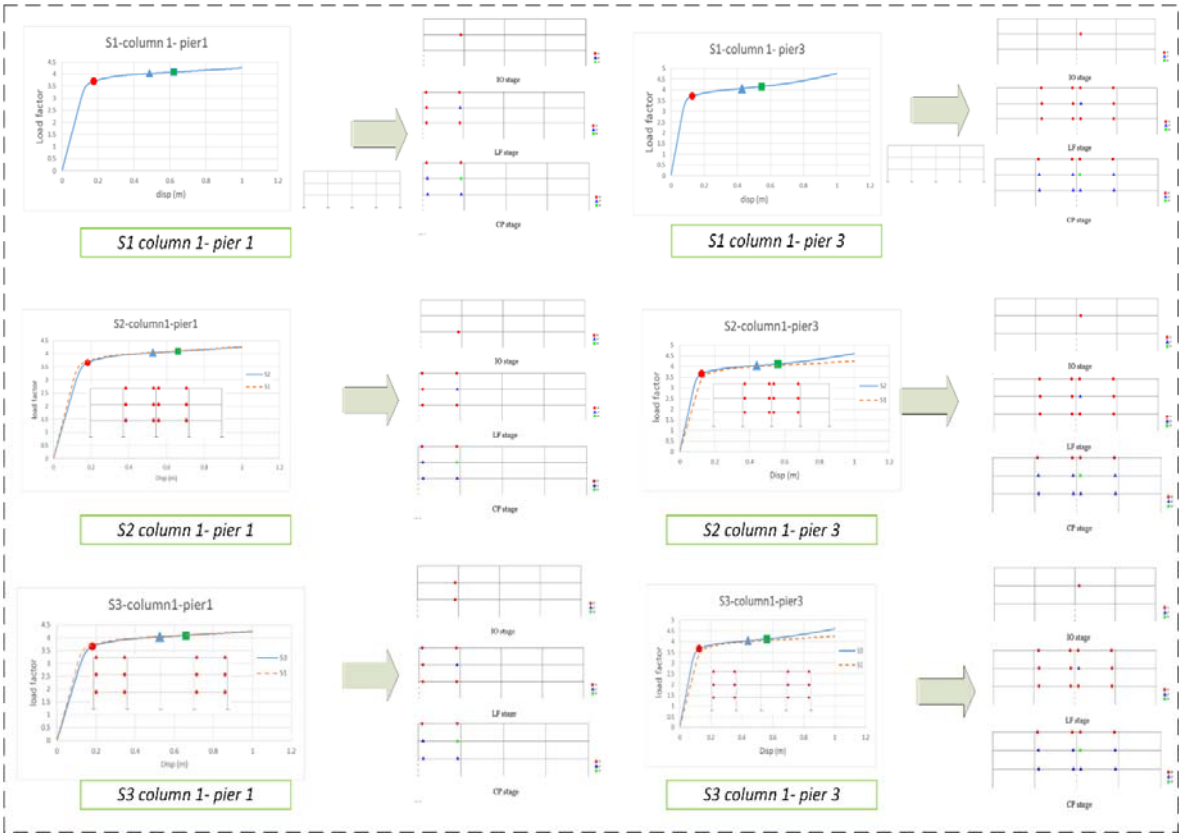


Fig. 5 Nine different scenarios for progressive analysis

V. Results and Discussion

In this study, the capacity of the steel moment frame with semi-rigid connections and the connection zone toward progressive failure was investigated using nonlinear analysis according to GSA. Two scenarios of column removal on the first floor were assessed for each of the models. In this regard, nine different buildings with distinct orientations of semi-rigid connections were modeled utilizing the 3-story model.

Nonlinear static analysis showed that at the corner column removal of the model S1 comprising rigid connections, in comparison with models consisting of semi-rigid connections, plastic hinges form at lower displacements. The S9 model, on the other hand, reaches the first sequence of plastic formation even at the most imposed displacement. It should also be pointed out that in the middle column removal scenario, S7 with the least displacement and S9 with the most displacement experienced their initial plastic hinge. Nonlinear static analysis reveals that in the middle column removal scenario, the first plastic hinge forms at the second story beam for all the models. However, in the corner column removal scenario of the S1 model, the first plastic hinge forms at the 2-story beam, and in the S2 model at the 1-story one. Initial hinge formation under this scenario for other models occurs simultaneously at the 1st and 2nd-floor beams. The hinge formation sequence also reveals that all the plastic hinges form at the beams, but none of them occur at columns.


S1-column 1- pier1
S1 column 1- pier 1
S1-column 1- pier3
S1 column 1- pier 3
S2-column1-pier1
S2 column 1- pier 1
S2-column1-pier3
S2 column 1- pier 3
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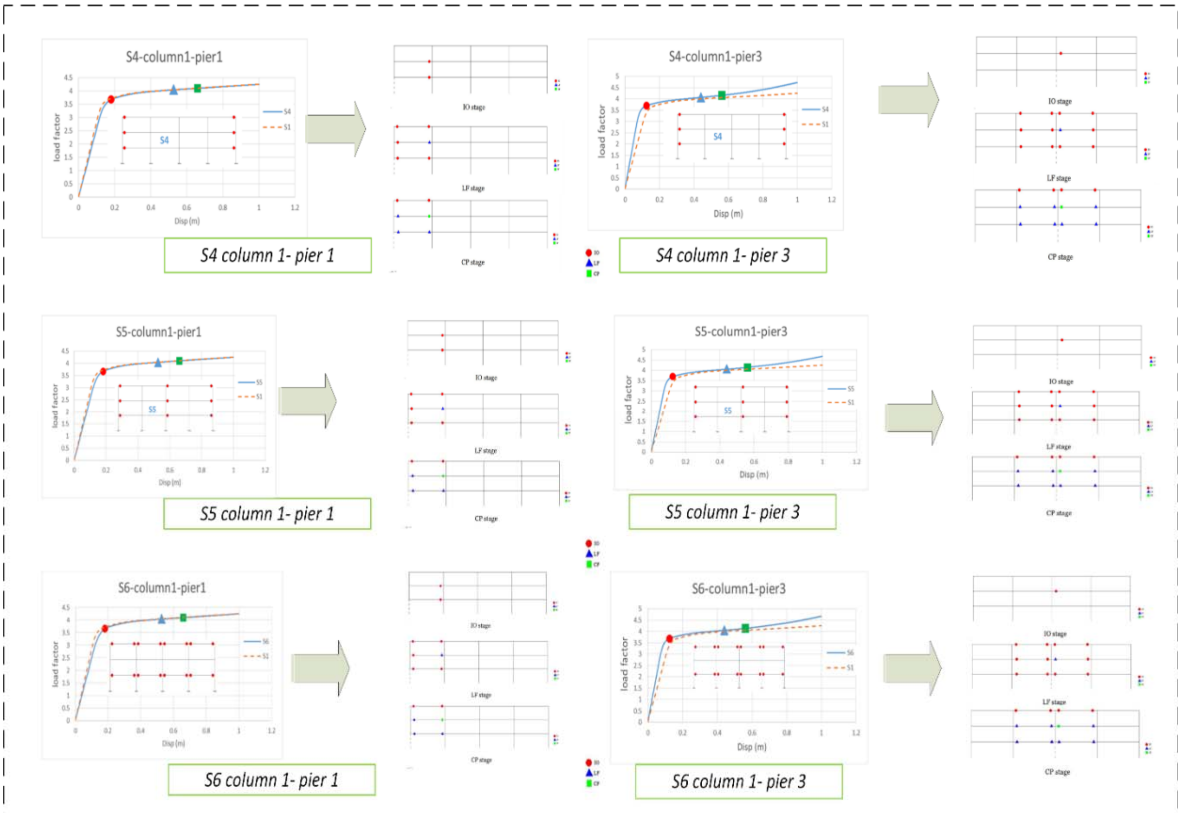
S3 column 1- pier 1
S3-column1-pier3
S3 column 1- pier 3

(a)


S4-column1-pier1
S4 column 1- pier 1
S4-column1-pier3
S4 column 1- pier 3
S5-column1-pier1
S5 column 1- pier 1
S5-column1-pier3
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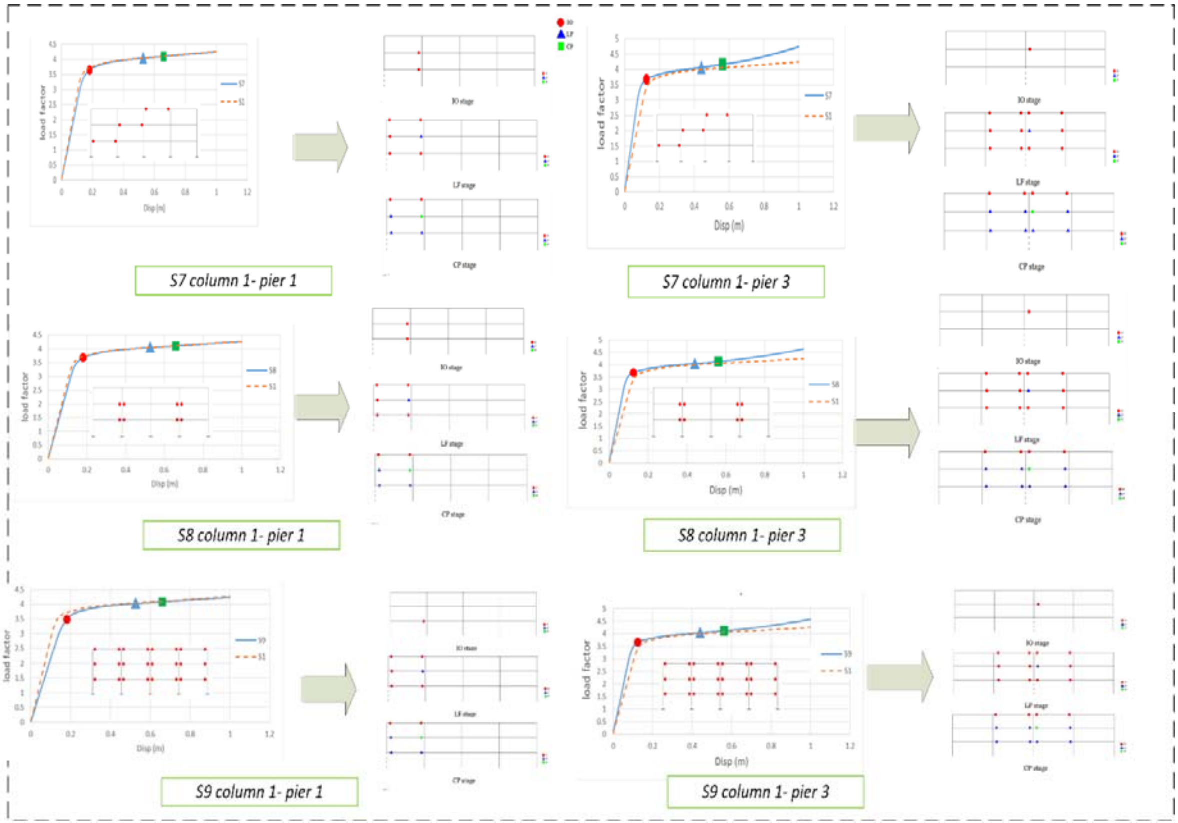
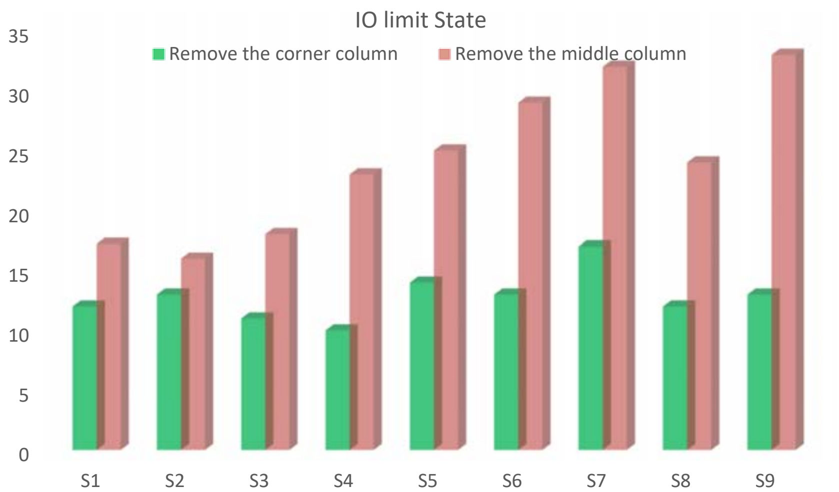
S6-column1-pier3
S6 column 1- pier 3

(b)

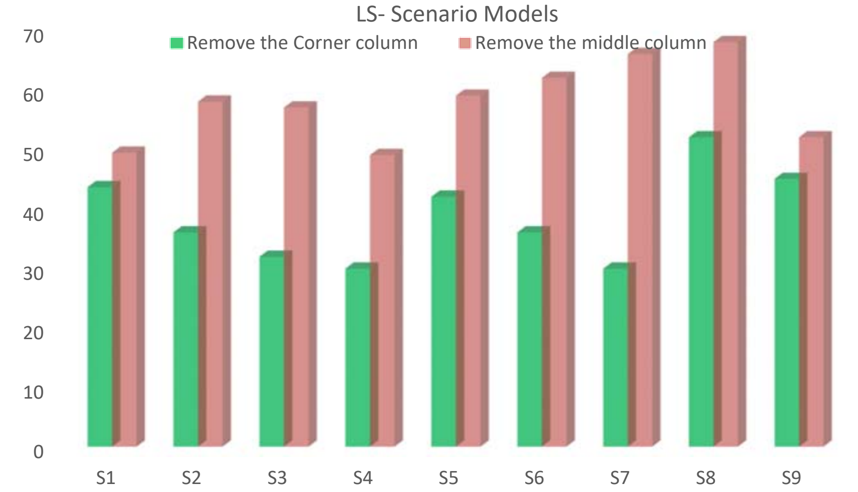


(c)

Fig. 6 Plastic hinge formation and capacity curve for nine different scenarios



(a)



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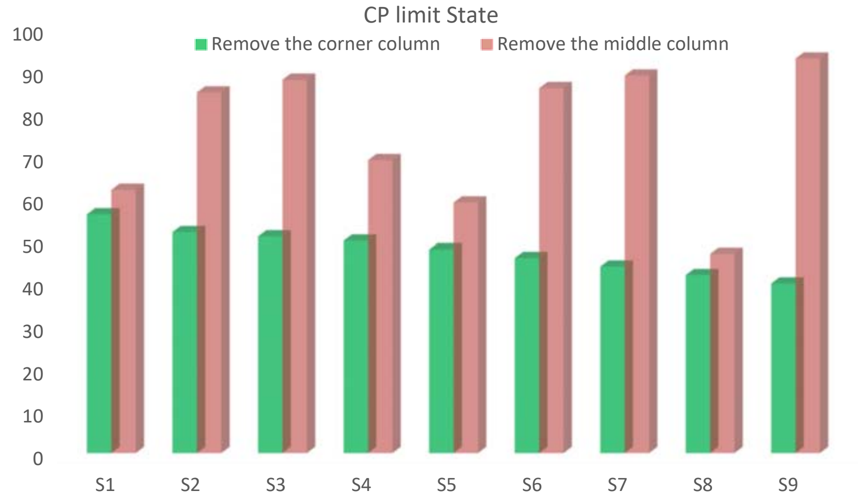

(c)

Fig. 7 The minimum capacity in each limit state for nine different scenarios

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