Progressive collapse evaluation of steel structures considering the effects of uncertainties and catenary action

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Abstract:

Various collapse modes have been observed so far in the phenomenon of the progressive collapse. This study examines the collapse modes in damaged structures using pushdown analysis for two scenarios of removing interior and exterior columns. A special moment-resisting frame is selected as a structural model. The effect of catenary action is considered under three damage states including light, moderate and severe. In addition, the effect of uncertainty parameters such as yield strength, modulus of elasticity, dead and live loads is investigated on the structural responses using probabilistic analysis. Monte Carlo simulation method is used to perform probabilistic analysis. Latin Hypercube sampling method is used to generate random realizations to achieve good accuracy. Then, a sensitivity analysis is performed. The results showed that a more precise and realistic estimation of the structure resistance and collapse modes will be achieved for the structure under progressive collapse when the effect of catenary action and uncertainties are considered. The catenary action effect is more significant when the damage in the structure increases. Also, increasing the axial force in the beams causes that the bending moment decreases in the case of moderate damage. The results of sensitivity analysis showed that the yield strength of members is the most effective parameter of uncertainty on changing the axial force demand of the interior column after removing the exterior column.

Keywords: Progressive collapse; collapse mechanisms; catenary action effect; buckling; sensitivity analysis

1. Introduction

Local damage in a structure may cause a failure in a part or the entire structure, which cannot be resisted by the structural system with inherent integrity and ductility. Such disastrous failure can be referred to as the progressive collapse. Abnormal loads including gas explosions, possible errors in the design and implementation of the structures, accidents caused by vehicle collisions to the key structural members could be triggered by a progressive collapse phenomenon. The effects of these loads are not taken into account in conventional design of the structures and may cause significant damage to them [1].

In progressive collapse, the removal of a load-bearing member in a structure imposes excess forces on the adjacent members. Hence, redistribution of forces in the structure may cause the member forces to exceed from their load-carrying capacity [2]. The propagation of failure in the structure may vary depending on the collapse mechanisms after the initial damage to the structure. Since the structure loses its static stability due to these failures, it is necessary to examine how to establish an alternative static equilibrium in the structure. An important mechanism to achieve an alternative static equilibrium is the catenary action in damaged structures. Since beams cannot merely sustain vertical loads with bending moment reactions, excessive resistance can be achieved in sufficiently large deformations against the applied loads by developing axial forces in beams and creating catenary action [3].

As mentioned above, many researchers have so far examined various aspects of the issue of progressive collapse. For instance, Kiakojouri et al. [4] Investigated the behaviour of the moment-resisting frames subjected to the progressive collapse using static and dynamic incremental analyses. The results of their research showed that the potential of the progressive collapse in the structures with more floor number is lower. Also, removing the column on the upper floors will cause larger vertical displacements compared to the lower floors.Tavakoli and Moradi [5] investigated the potential of the structures against the progressive collapse using a robustness index for structures with different lateral load-bearing systems. They proposed a simple energy-based method for conducting robustness analysis. Liqiang and Jihong [6] also evaluated the robustness of the steel structures subjected to the unpredictable events. They identified key elements and the most likely collapse mode in damaged structures under progressive collapse using vulnerability analysis. Then, they quantified the failure consequences corresponding to the collapse mode.Tavakoli and Kiakojouri [7] evaluated the role of initial failure location and the number of floors for the progressive collapse potential of structures. They also comprehended that the weight of structure above damaged area has the most important effect on the analysis results. Tavakoli et al. [8] conducted a progressive collapse analysis on the structural frames to evaluate resistance of the structure under seismic progressive collapse. They adopted a method that localized failures and prevented the spread of damage to the intact parts. Gerasimidis et al. [9] worked on simulating the spread of damage. They stated that when a column removes from a structure, buckling may occurs in columns adjacent to removed column. Various methods of retrofitting steel moment-resisting frames under progressive collapse can be found in the work done by Abdollahzadeh and Shalikar [10]. They found that the vertical movements of the damaged columns were reduced using a vertical bracing system. Then, new paths were created to redistribute forces to the other elements after that the columns were buckled. They also noted the impact of catenary action on the collapse of structures subjected to the fire loadings.

Most of the previous studies have been carried out on the progressive collapse within the deterministic framework, while a lot of uncertainties can be considered for more accurate analysis. Uncertainties of the material specification and the gravity loads in both live and dead loads can be considered [11]. Rodríguez et al. [12] evaluated the progressive collapse of the steel frames with bolted-angle connections using fragility and sensitivity analysis. They considered the uncertainties in the material and geometrical properties and then used the Monte Carlo simulation for probabilistic analysis and the pushdown method for damage analysis. The results of their analysis showed that random variables affect the behavior of structures under progressive collapse. Moradi et al. [13] conducted a probabilistic assessment on the the collapse time of a steel structure under the fire event. They also estimated the probability of failure of an intact structure and a previously damaged structure for a specified failure time under fire. Naghavi and Tavakoli [14] made a probabilistic prediction of failure in column of a steel structure under progressive collapse. They used the response surface and artificial neural network methods to construct the limit state function required for probabilistic analysis. Javidan et al. [15] suggested a new method to evaluate the collapse of structures in the probabilistic framework under extreme loads including vehicle impact loads. They assessed the collapse behavior of steel frame structures via fragility curves along the weak and strong axes. They confirmed that the uncertainties regarding the loading and geometric characteristics had no significant effect on the output responses. Ding et al. [16] performed a progressive collapse analysis based on probabilities for steel frame structures under blast loads by applying the uncertainty to the explosion and gravity loads as well as material properties. They considered different scenarios by changing the location of the initial failure.

In the present study, a probabilistic analysis is carried out using Monte Carlo simulation method in a special moment-resisting frame. The Monte Carlo simulation method is one of the most effective methods adopted in probabilistic analysis. This method is used to calculate the probability of failure using random variables. Since this method has a high computational time due to the large number of input samples, other alternative methods can be used that save computational time [17, 18]. This alternative method can be a combination of the response surface or neural network method with the Monte Carlo simulation to obtain an explicit limit state function [14, 19]. For damage analysis, the columns of the structure in the interior and the exterior position are removed under gravity loads. Then, the pushdown analysis is applied for considering progressive collapse of the structural model in different damage states. Finally, the structural capacity is estimated in considered damage states. Furthermore, the effect of the formation of catenary action in different damage states and the role of its in collapse modes are investigated by taking into account the uncertainties. Ignoring uncertainties and catenary action may lead to very conservative estimation of the collapse modes. In this study, type of phenomenon which causes the progressive collapse is not important. But, more focus is on the propagation of damage in the structure. The focus of this research is on factors affecting the identification of collapse modes when the vertical load-bearing elements is removed from the structure. This study deals with the combined effect of catenary action and uncertainty on the collapse mechanisms and strength of the structures under progressive collaps due to the importance of the issue that has not been considered in previous research. Therefore, the determination of collapse mechanisms under local damage due to progressive collapse can help to simulate strength and stiffness degradations induced by initial damage in structures for successive events in the future.

2. Model Validation

To validate the structural model, a specimen tested by Sadak et.al [20] was used to investigate the progressive collapse by removing a middle column in a steal frame with two spans. Diagonal braces with cross section of w14 × 109 used in this test to consider the effects of the upper floors. A hydraulic ram with a capacity of 2669 kN was utilized to simulate applied vertical load to the middle column. The cross section of w24 × 94 for beam and w24 × 131 for column were used. Modeling was performed in OpenSees software by considering the properties of the material in the form of elastic perfectly plastic according to the original reference. Details of the setup test are shown in Figure 1. The results of the exprimental test are compared with the numerical model in Figure 2 for the axial force induced due to large displacement in the beams after the removal of the middle column. As can be seen from Figure 2, the axial force developed in the beams is 2450 kN, which activates the catenary action effects in beams to resist against the large deformations due to removal of the column. Also, the result of the laboratory test is in a good agreement with the result of the numerical model.

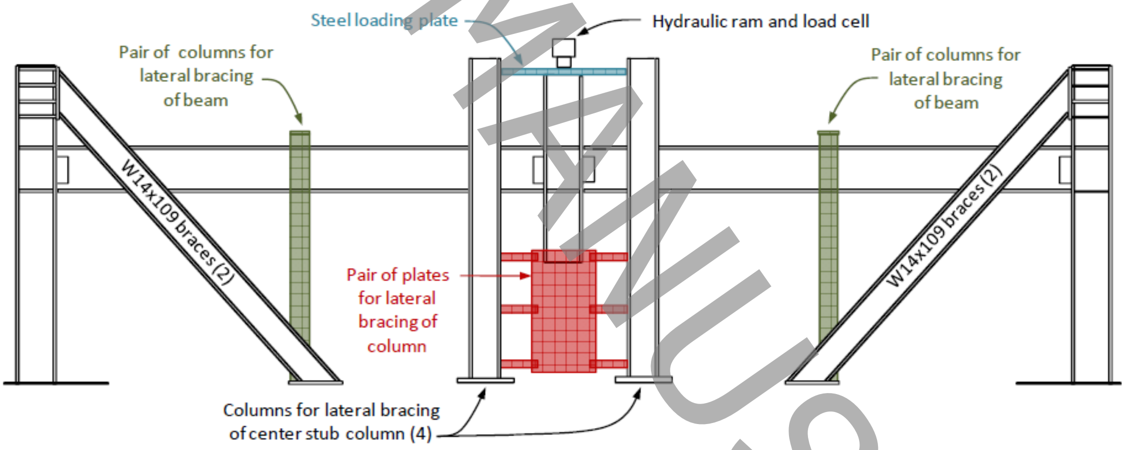


Fig. 1. Details of test setup for specimen [20].

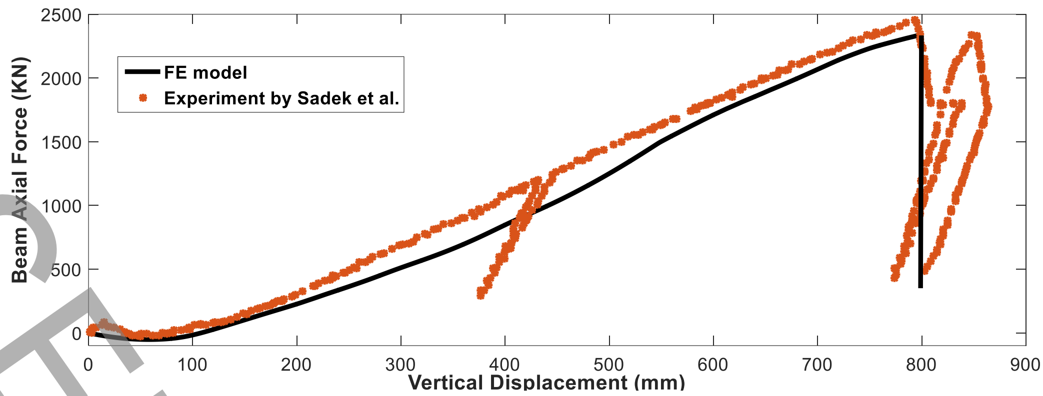


Fig. 2. Comparison of the results of the numerical model and laboratory test for the axial force created in the beams.

2.1. Description of modeling structure and material

The structural model used in this paper is an eight-story building system with four spans in the longitudinal and four spans in the transverse direction of same lengths of 9.14 m. The height of the first floor is 4.57 m, and the heights of the other floors are equal to 3.66 m. The peripheral special moment-resisting frames were considered as the lateral load-resisting system and the interior frames were regarded as the gravity load-resisting system. Plan view of the structural model are shown in Figure 3. The dead loads applied to floors and roof are respectively 5 kN/m^2 and 3 kN/m^2, while the corresponding live loads are 2.4 kN/m^2 and 0.96 kN/m^2, respectively. This structural model was designed as a standard office building located in an area close to Los Angeles, based on the soil type used during design of the structure with site class C, S_S=2.48g, and S_1=1.02g. The steel having yield stress 288 MPa, and modulus of elasticity 2×10^5 MPa was used to model all beams and columns throughout the analysis of the structure. Details of beam and column sections are presented in Table 1. More details of structural model can be found in reference [21].

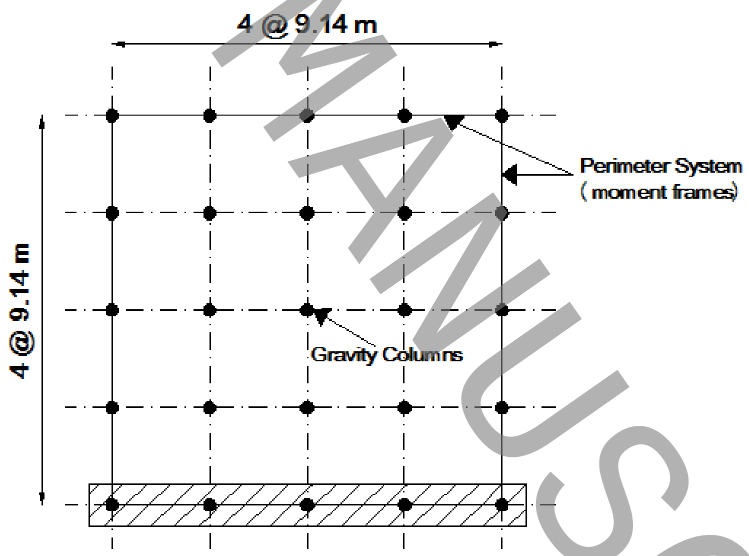


Fig. 3. Plan view of the 8-story model[21].

As shown in Figure 3, a peripheral two-dimensional special moment-resisting frames is drawn and then used for the nonlinear analysis in the open-source platform OpenSees [22]. The "nonlinearBeamColumn" element has been applied to model the beam and column elements with 5 integration points and 2% of post-yield stiffness. The geometric transformation used in beams with catenary action (C) was "Corotational", while it was "Linear" for beams without catenary action (NC). In addition, this geometric transformation for columns has been adopted using the "PDelta" option.

Since a two-dimensional frame has been used in the nonlinear analysis, and hence the results of analysis may be affected by the gravity frames, a leaning column carrying the weight of interior gravity system has been implemented to consider the P-Delta effects. The connection of the leaning column to the frame is carried out through axially rigid trussed elements.

Table 1. Sections of the structural members.

Floor	Beam	External column	Internal column
Roof	W18×60	W14×99	W14×132
7	W21×83	W14×99	W14×132
6	W21×93	W14×109	W14×176
5	W27×102	W14×109	W14×211
4	W30×108	W14×132	W14×233
3	W30×116	W14×145	W14×257
2	W30×116	W14×159	W14×257
1	W30×124	W14×283	W14×342

3. Probabilistic analysis of the progressive collapse

3.1. Progressive Collapse Analysis

In this paper, a nonlinear static pushdown analysis is used with removing a column in the exterior and the interior positions of the structure under gravity loads. This method takes into account some considerations such as vertical load intensity levels, plastic rotation demand of the structural elements and catenary action effect in beams under large displacement. The structure is pushed in the vertical direction by increasing the vertical load incrementally until the vertical displacement at the top node of removed column reaches the limit states that control the damage levels [23]. This study considers these limit states as work done by Conrath et al. [24] for structures under extreme loads as given in Table 2. Two column removal scenarios considered are including the interior column C_{12} and the exterior column C_{11} in the first floor as shown in Figure 4.

Table 2. The Limit states of the rotation angle in radian for damage of the steel structures subjected to the extreme loads [24].

Element type	Light	Moderate	Severe
Beam	0.05	0.12	0.25

According to General Services Administration [25], the gravity load combination for bays away from removed column is (1.2dead+0.5live). Likewise, the following increased gravity load combination is considered for those bays above the removed column as shown in Figure 4.

$$G_N = \Omega_N \left(1.2DL + 0.5LL\right) \tag{1}$$

Where G_N, DL and LL are respectively the increased gravity, dead and live loads. Ω_N is a dynamic increase factor for considering the dynamic effects of column removal in nonlinear static analysis. This factor is calculated as follows:

$$\Omega_N = 1.08 + 0.76 \big/ \left(\theta_{pra} / \theta_y + 0.83\right) \tag{2}$$

Where θ_{pra} is the plastic rotation angle given by the acceptance criteria tables in ASCE 41 [26] for life safety level in the present study:

$$\theta_{pra} = 0.0337 - 0.00086 \times \left(h_{beam} / 2\right) \tag{3}$$

Where h_{beam} is the height of beam section, and θ_y is the yield rotation that can be determined for steel via ASCE 41 as follows:

$$\theta_y = Z_{pl} \times f_y \times L / 6 \times E \times I \tag{4}$$

Where Z_{pl}, f_y, L, E and I are respectively the plastic section modulus, material yield stress, beam length, modulus of elasticity, and the beam moment of inertia.

To calculate the value of Ω_N for the entire structure, the lowest ratio of θ_{pra} / θ_y is used. Based on the above equations, the value of 1.33 has been obtained for Ω_N factor for deterministic analysis in the present study.

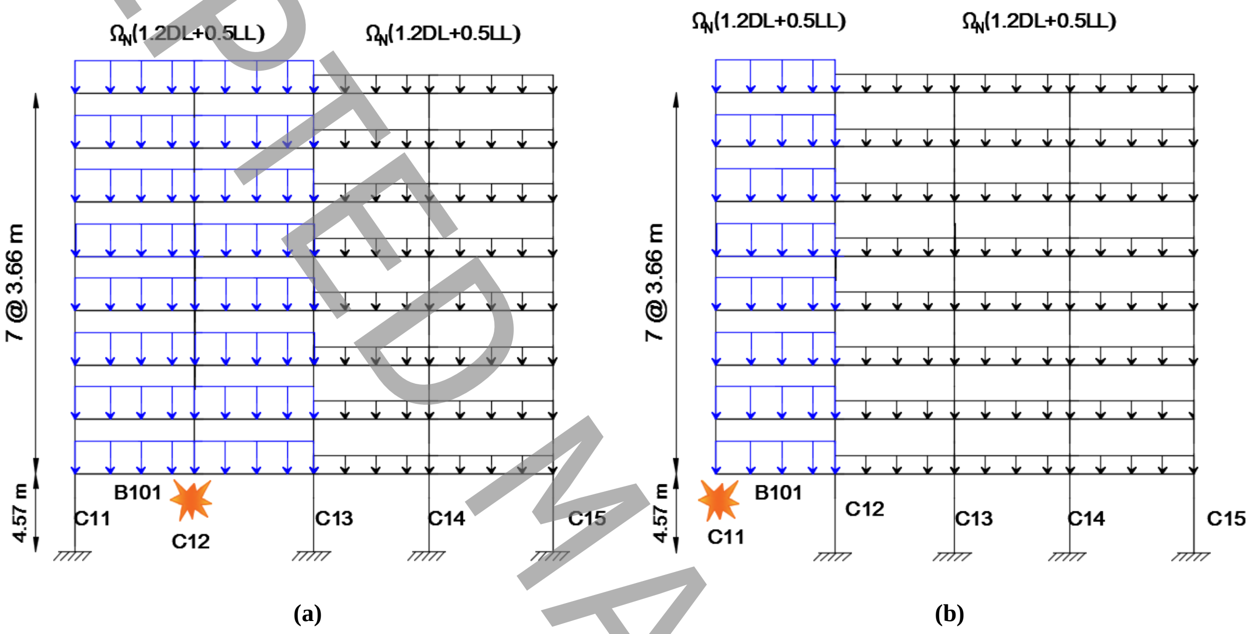


Fig. 4. Load combination in pushdown analysis: (a) interior column removal; (b) exterior column removal.

3.2. Probabilistic analysis method

It should be noted that the resistance of structures under progressive collapse is affected by the uncertainty parameters in the specification of materials and gravity loads, which are not reflected in the existing guidelines. The uncertainties considered in this study include yield strength of the structural elements, live load, dead load and the elastic modulus. The correlation coefficient between elastic modulus and yield strength is assumed to be 0.2. The subscript n for mean values in Table 3 expresses their nominal values. The statistical characteristics of these random variables such as mean and coefficient of variation (CoV) are given in Table 3. CoV is the ratio between the standard deviation and the mean. The Monte Carlo simulation method is used to perform the probabilistic analysis of the progressive collapse. In the Monte Carlo method, inputs are a set of random variables that lead to different structural responses. The statistical data of these random variables are usually specified [27]. These random variables are generated by Matlab software according to their distribution function as given in Table 3, then placed in OpenSees finite element software as input data to obtain structural responses. The accuracy of estimating the limit state function used for Monte Carlo simulation depends on the sampling method. The Latin hypercube sampling method (LHS) is used to generate the random variables required in the Monte Carlo simulation. This method is one of the methods to reduce the number of samples, which is based on reducing variance. In this method, the CDF diagram is divided into N regions. Then a sample from each section is randomly selected. [28].

Table 3. Statistical properties of random variables.

Variables	Mean	Coefficient of variation	Probability distribution	References
Yield strength	$1.10F_{yn}$	0.06	Normal	[29]
Elasticmodulus	$0.993E_n$	0.034	Normal	[29]
Dead load	$1.05D_n$	0.1	Normal	[30]
Live load	L_n	0.25	Normal	[30]

The variability of the structural response to the random variables can be determined using sensitivity analysis. Tornado Diagram Analysis (TDA) is one of the most common sensitivity analysis methods. Structural responses corresponding to upper and lower bounds of each random variable are determined in TDA method as shown in Figure 5. The difference between the obtained responses is considered as a measure of sensitivity which is called swing. Larger swing expresses more effect the corresponding random variable on the structural response. In sensitivity analysis, the structural response is calculated for the mean values of the random variables and used as the base value for the tornado diagram at the first stage. Then, the variability of the structural response is measured for upper and lower bounds of the certain random variable while the other variables remain constant in their mean value.

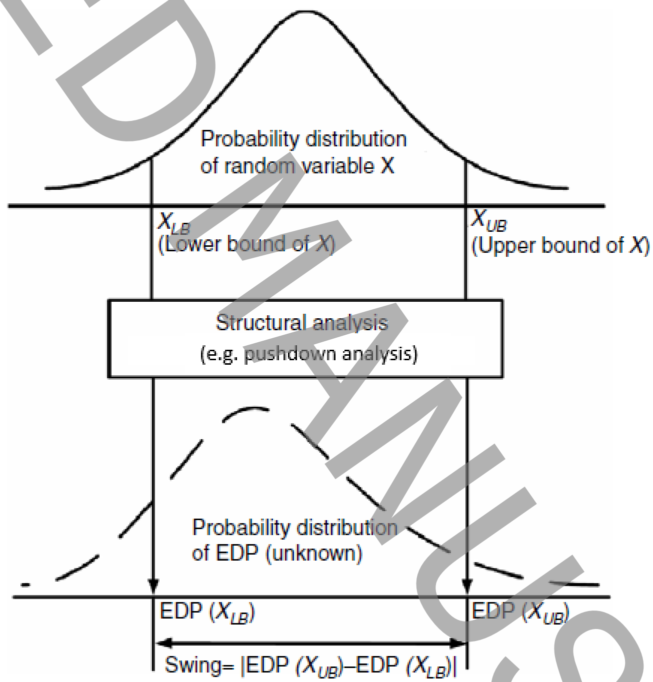


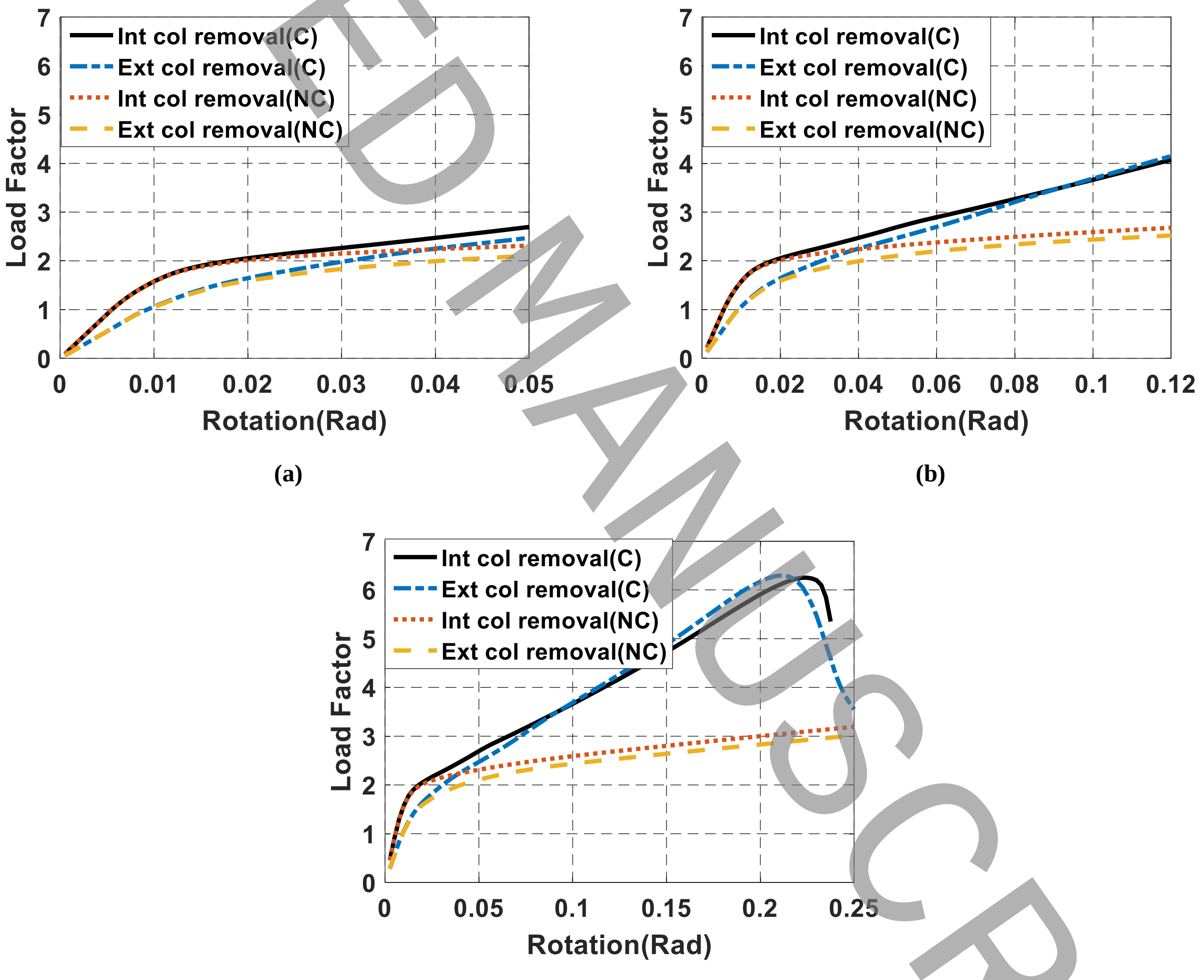
Fig. 5. The process of performing the tornado analysis [31].

4. The results of progressive collapse analysis

4.1. resistance of the structure using nonlinear static pushdown analysis

In recent years, pushover analysis method [32] has been used to estimate the response of structures under seismic loads. In this paper, pushdown analysis method is used to evaluate structures under progressive collapse. This method has two advantages compared to the pushover method. In this method, yield-type collapse mechanism which is the most probable mode of vertical failure occurs in damaged beams due to column removal under the gravity loads Also, most structural vibration occurs in the first vertical bending mode and the effect of higher modes is insignificant. Therefore is no concern regarding the modal combination of structural responses [23]. Figure 6 shows the results of nonlinear static pushdown analysis of the structural model under the various damage state caused by the removal of interior or exterior columns as given in Table 3. The exterior column C_{11} and interior column C_{12} are removed column as shown in Figure 4. The first digit of index C represents the floor number and the second digit represents the row of columns in the elevation of frame. In Figure 6, the vertical axis represents the load factor which is drawn against the rotation angle of the damaged beams during different damage states. The rotation angle of the beam is the ratio of maximum vertical displacement above the removed column to length of the beam in damaged bay which is considered as a measure for variability in damage levels. Afterwards, the maximum capacity tolerated by the structure is determined in different damage states using a load factor. This load factor is defined by ratio of the load corresponding to structural failure to the total gravity load applied to structure. Moreover, the effect of catenary action was considered in all limit states. It can be seen that, in the light limit state corresponding to rotation angle 0.05, no significant changes exist between two modes of with and without catenary action.

The load factors in the moderate and severe damage states are noticeably higher with considering catenary action (C) than the without catenary action (NC). Therefore, the resistance of the structure to the progressive deterioration increases with the activation of the catenary action behavior in the large deformation of damaged beams. Consequently, the structure cannot withstand against more damage and its resistance is suddenly reduced at an angle close to 0.25, corresponding to the severe damage state. In addition, there is no significant difference for load factor values between the two cases of exterior and interior column removals in the light and moderate limit states. Nevertheless, the resistance of the structure when catenary action is considered, decreases earlier in lower load factor in the case of exterior column removal as compared to the interior one.



(c)

Fig. 6. Load factor for the structural model in the removal of exterior and interior columns with catenary action (C) and without catenary action (NC) in different damage states: (a) Light; (b) Moderate; (c) Severe.

4.2. Collapse modes

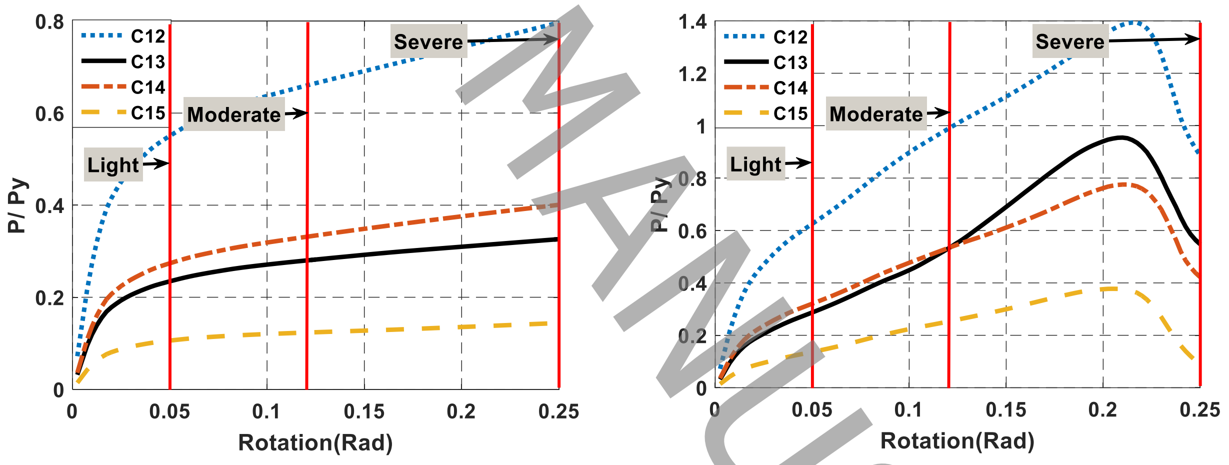
To investigate the collapse mechanisms in a progressive collapse, the collapse modes should be detected after the removal of column. The collapse modes of yielding-type and stability have thus been studied in this paper.

The yielding-type mechanism occurs in beams when the moment demands in two ends of the beam reach their flexural capacity. The flexural capacity of each beam is equal to the specified value that can be obtained from the following equation:

$$M_p = z \times f_y \tag{5}$$

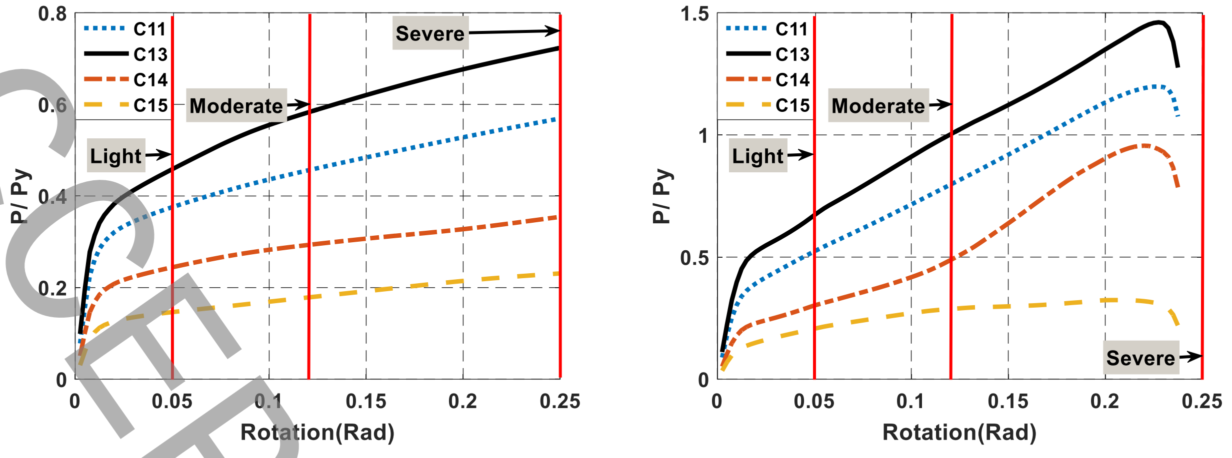
Where z is the plastic section modulus of beam, f_y the material yield stress, and M_p is the plastic moment capacity. Furthermore, stability mechanism is defined as to when the axial forces in columns adjacent to the removed column reach their axial capacity. Once a column is removed from the interior or exterior bay, the adjacent columns will be loaded additionally. Such distribution of load may lead to inelastic buckling in the columns adjacent to the removed one. In this case, the axial forces of columns reach yield capacity of A× f_y ; where A is cross-sectional area of the column and f_y is material yield stress. To reveal the buckling mode, it is necessary to consider imperfections in the modeling, even though they have no effect on structural responses. These values are considered to be equal to 0.001 times the vertical loads, which are applied at the level of each floor as a horizontal concentrated force [33].

The relationship of the axial force in columns adjacent to the removed column subjected to the different damage states and rotation angle of beam is shown in Figures 7 and 8. As can be seen from Figures 7(a) and 8(a), the ratio of the axial force demand to the inelastic buckling capacity (P/P_y) in the columns has not exceeded the value of 1 in all damage states without considering catenary action, which means that the columns around the removed column have not buckled.



(a) **(b)**

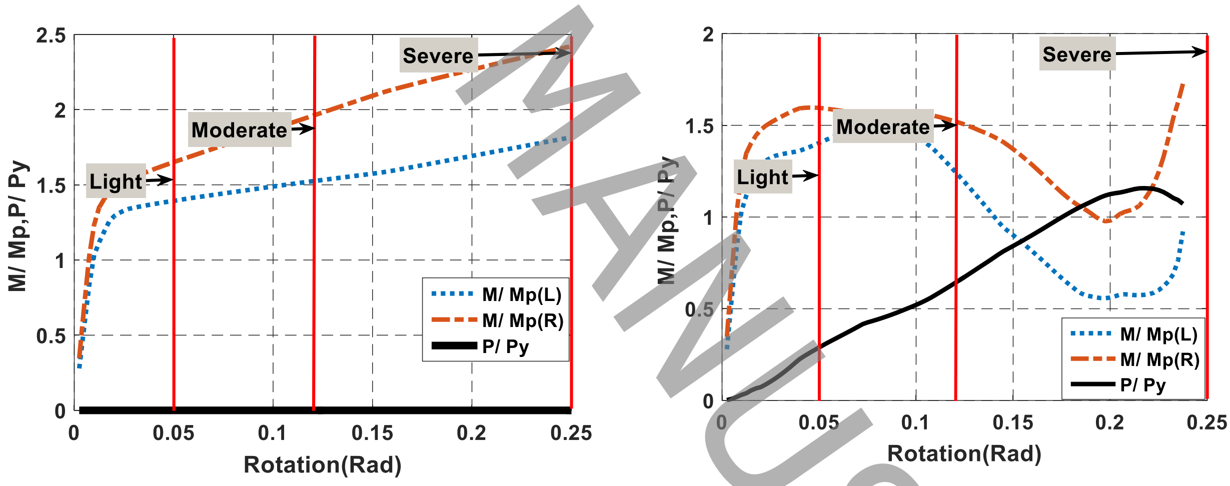
Fig. 7. The ratio of P/P_y in the columns adjacent to the exterior removed column C_{11} under three damage states: (a) Without catenary action; (b) With catenary action.



(a) (b)

Fig. 8. The ratio of P/P_y in the columns adjacent to the removed interior column C_{12} under three damage states: (a) Without catenary action; (b) With catenary action.

It can be seen from Figures 7(b) and 8(b) that, the buckling does not occur as before in the case of both the interior and exterior column removal in light and moderate damage states. But, after removing the interior column C_{12}, buckling occurs in C_{11} and C_{13} and after removing the exterior column of C_{11}, buckling occurs in C_{12} before reaching the severe damage state when the catenary action effect was considered. Also, the buckling mode was activated earlier in the adjacent columns in the case of the removal of the exterior column in comparison with the removal of the interior column.



(a) (b)

Fig. 9. The ratio M/M_P or P/P_y in beam 101 above the interior removed column C_{12} under three damage states: (a) Without catenary action; (b) With catenary action.

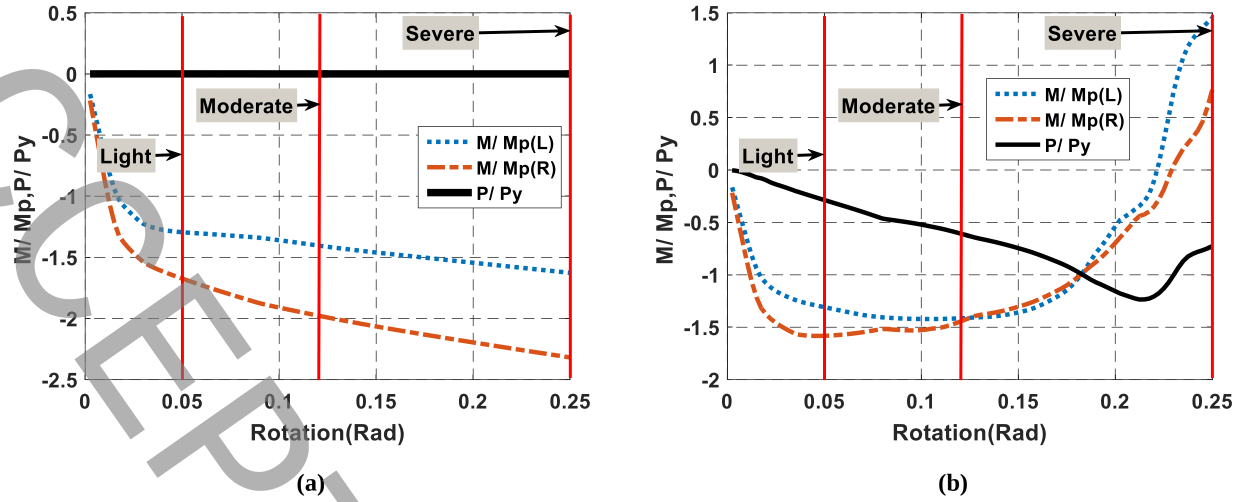


Fig. 10. The ratio M/M_P or P/P_y in beam 101 above the exterior removed column C_{11} under three damage states: (a) Without catenary action; (b) With catenary action.

Figures 9 and 10 show the ratio of the moment demand to the plastic capacity (M/M_P) and axial force demand to the axial capacity (P/P_y) on the left and right of the beam labeled 101 at top of the interior and exterior lost columns, which are plotted against the beam rotation as shown in Figure 4. From the comparison of Figures, it can be seen that considering the catenary action effect causes that the axial force are produced in the beam as shown in Figures 9(b) and 10(b), which leads to the moment reduction when vertical displacement of joint above the missing column reaches to the moderate damage state. Axial force produced in the beam with the activation of the catenary action effect is dominant to the bending moment relative to their capacity in the distance between the moderate to the severe limit state. These axial forces are almost zero when the catenary action effect was ignored as shown in Figures 9(a) and 10(a). This beam is yielded due to the exceedance of the moment demand from its plastic capacity in all cases. In Figures 9(b) and 10(b) can be seen that the axial force generated in the beam decreases faster in removal of exterior column than removal of interior column. When the structure is damaged in limit state of the moderate to severe, moment values drop by increasing axial loads and they increase by decreasing axial loads. Also, a reasonable agreement between the results achieved by the analysis can be observed in two ways of eliminating the column with and without considering the effect of the catenary action.

4.3. Probabilistic analyses

The effects of the probabilistic properties given in Table 3 on the load factor are evaluated by means of cumulative distribution function (CDF) curves. The probability of the values obtained for the structural responses exceeding a limit state is displayed in diagrams known the CDF. These curves are plotted using fitting responses with a lognormal distribution function [34]. A desirable failure probability can be obtained for load factor in structures subjected to the progressive collapse due to gravity loads. CDF curves are obtained for the load factors from Monte Carlo analysis by fitting these values with a lognormal distribution function. The structural model has been analyzed for the three limit states under two cases of removing exterior and interior columns. Figure 11 shows the probability of exceeding the light, moderate and severe limit states corresponding to rotations 0.05, 0.12 and 0.25 at the point above the removed column in both exterior and interior cases versus load factor, respectively. As shown in results mentioned above, the catenary action plays an effective role in the resistance of structures subjected to progressive collapse in large deflection. Such effect has therefore been considered in the probabilistic analysis for both moderate and severe limit states.

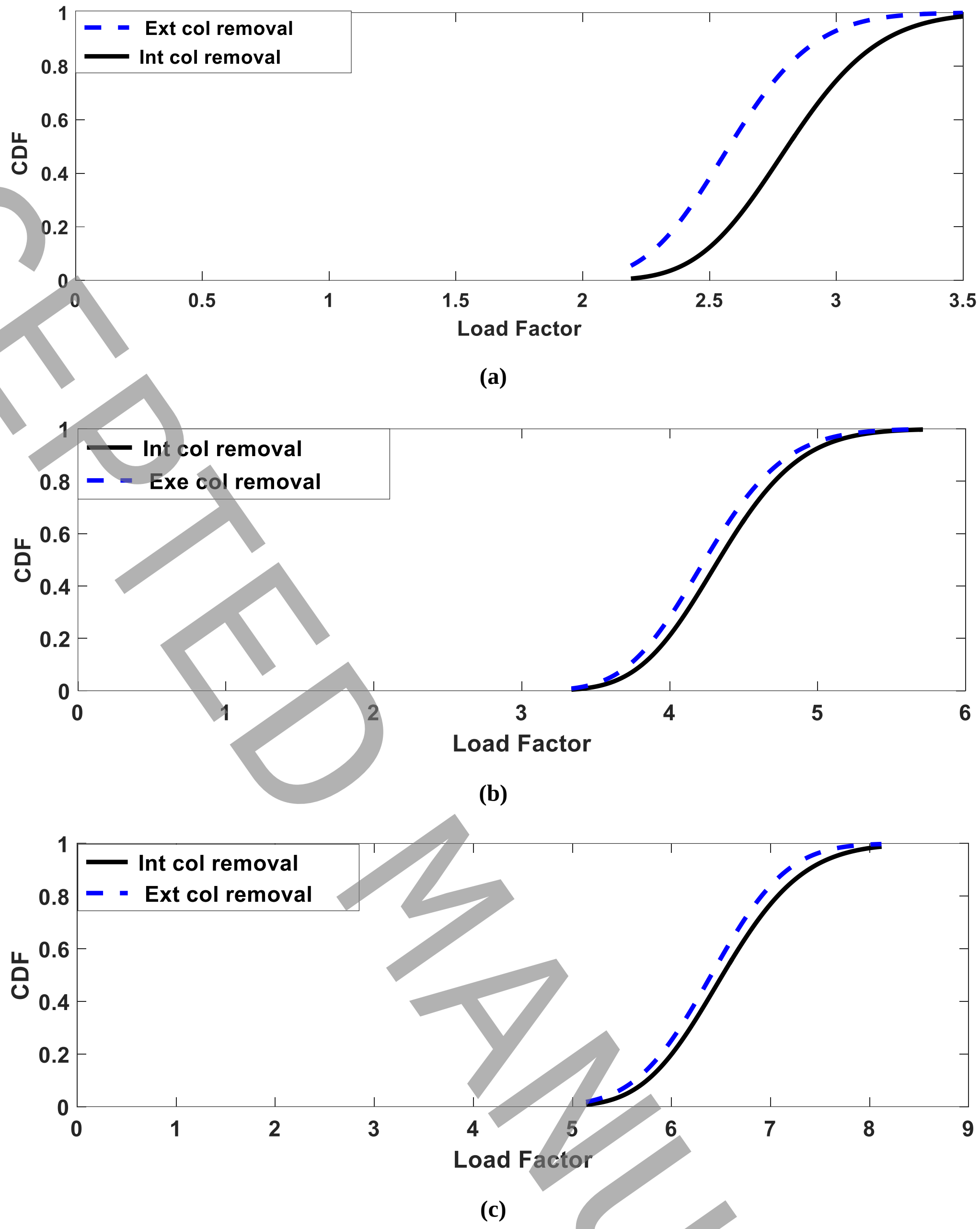


Fig. 11. Cumulative distribution function (CDF) curves of the load factor in limit states: (a) Light; (b) Moderate; (c) Severe.

It can be seen from Figure 11 that the structure with the removal of interior column has larger load factor than the one with the removal of the exterior column in a certain probability of exceedance of all limit states. Since the axial force created in the large displacement corresponding to beam rotations 0.12 and 0.25 is due to the activation of the catenary action in the beams above the removed column, difference between the load factors obtained for the structure with removal of the interior and exterior columns is more significant in light limit state than moderate and severe limit states. As shown in Figure 11, CDFs of the load factor in probability 50% are approximately the same as those obtained from using the mean values of the uncertainty parameters in deterministic analysis as shown in Figure 6.

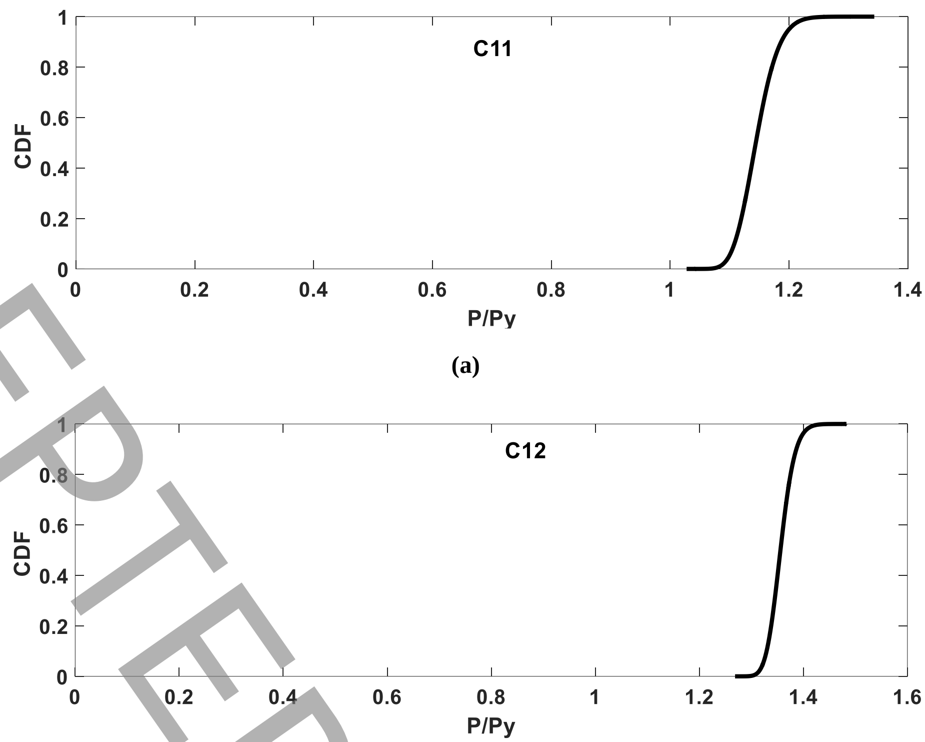


Fig. 12. Cumulative distribution function (CDF) curves of the ratio P/Py in the adjacent column to the damaged column in both the exterior and interior column removal scenarios in severe limit state: (a) column C_{11}; (b) column C_{12}

Figure 12 shows the CDF of the ratio P/Py in the adjacent column to the damaged column in both the exterior and interior column removal scenarios in severe limit state. This ratio is greater than the value of one, which means that yielding occurs in columns adjacent to the damaged column in both column removal scenarios and, the probability of the ratio P/Py less than 1 is zero. The maximum values of the ratio P/Py corresponding to the probability 100% in the CDF diagram are 1.5 for column C_{12} and 1.35 for column C_{11} in the exterior and interior column removal scenarios, respectively. Therefore, adjacent column to the exterior damaged column are more affected by increased axial effort than interior ones.

4.4. Sensitivity analysis

According to the results of the previous section, since the column C_{12} is more likely to be more vulnerable to the removal of the exterior column, sensitivity analysis is performed on this column. The sensitivity of the ratio P/Py in column C_{12} to the uncertainty parameters is investigated. Uncertainty parameters considered in this study include yield strength, modulus of elasticity, dead and live loads. The variability of the ratio P/Py of this column to the variability of random variables is shown in Figure 13 as a swing. Swings are sorted from large to small according to their size. The larger size of the swings indicates that the corresponding random variable is more effective on the P/Py ratio. It can be seen from Figure 13 that sensitivity of the ratio P/Py of the column C_{12} to the variability of the yield strength of the structural elements is higher than the other random variables in the exterior column removal. The second and third variables affecting the sensitivity of the structural response are elastic modulus and dead load, respectively. Also, change in the live load has a negligible effect on the variability of the ratio P/Py of the column C_{12} adjacent to the removed exterior column.

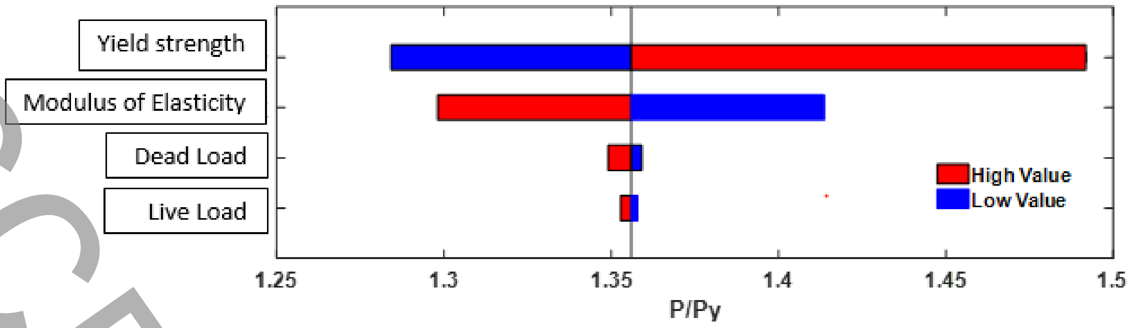


Fig. 13. Sensitivity of the ratio P/Py in column C_{12}

5. Conclusions

The present study evaluated the resistance of an 8-story special moment-resisting frame subjected to progressive collapse using pushdown analysis under load combinations of GSA 2013 guideline. Structural model was analyzed by removing a column in either of the interior or exterior positions in the first floor. Three limit states including light, moderate, and severe were implemented to carry out pushdown analysis. Moreover, the effect of catenary action was taken into account to examine a more accurate effect of the activation of axial forces in the beams of damaged spans as a consequence of column removal. Structural collapse mechanisms such as the yielding-type mechanism in beams, and the buckling in columns were investigated in the remaining structure after removing the column. Since the resistance of structure in progressive analysis is influenced by many uncertainties, the effects of these uncertainties have been applied to material properties and loadings, and their statistical properties have been utilized in the probabilistic analyses. In this study, the yield strength of beams and columns, the elastic modulus, and the dead and live loads were considered as random variables. The probability of exceeding the limit states for the load factor and the ratio P/Py of the column adjacent to removed column was investigated as a structural response. Finally, a sensitivity analysis was conducted to investigate the most effective parameter on the results of probabilistic analysis with TDA method. The results obtained from analysis are summarized as follows:

- The structure has maintained its strength against progressive collapse without any reduction even at severe limit state. This indicates that a special moment-resisting frame has a good resistance to the progressive collapse.
- Larger load factors were obtained when the effect of catenary action was considered, especially in the moderate and severe limit states. Activation of catenary action induces large axial forces in the beams of damaged spans due to the removal of column that leads to their greater resistance against larger loads. It was also observed that the strength of the structure against progressive collapse decreases earlier by removing the exterior column compared with the removal of the interior column when it approaches the severe limit state.
- Consideration of the catenary action has a great impact on identification of structural collapse modes subjected to progressive collapse. These effects are often ignored by researchers, which may result in unrealistic results. The buckling mode occurs in columns adjacent to the removed column as the axial force increases in beams of damaged spans to increase their resistance to larger rotations, while this buckling mode is not activated regardless of the catenary action effect.
- Investigations of the CDF curves extracted from Monte Carlo analysis demonstrated that load factors corresponding to a certain probability of exceeding all three limit states had smaller values in the case of exterior column removal than an interior ones. Also, the probability that the column adjacent to the removed column will remain safe in the severe limit state is zero.
- The most important parameter influencing the variability of structural response obtained from sensitivity analysis was the yield strength, while live load had a negligible effect on these changes in the structure under progressive collapse.
- The results of analysis will be provided more precise when the effects of the uncertainties are included in the probabilistic analyses, and the sensitivity of structural responses to all random variables is taken into account.

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