Seismic progressive collapse analysis of concentrically braced frames through incremental dynamic analysis

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SUMMARY:
Progressive collapse is a catastrophic structural phenomenon that can occur because of human-made or natural hazards. In progressive collapse mechanism, a single local failure may cause a significant deformation which may then lead to collapse of a structure. Researches on progressive collapse of structures generally focus on gravity and blast loadings where the design objective is to increase the redundancy and robustness of structures to prevent progressive collapse. After reviewing present scientific resources, it can be inferred that the progressive collapse behavior of structures due to earthquake loads has not received much attention as compared to those collapse triggered by sudden removal of columns due to blast loading. During a seismic event, redistribution of load carried by damaged or failed structural members to adjacent members may lead to overstress or exceeding load resistant capacity of the other members resulting in spread or propagation of damage until development of collapse mechanisms in seismic progressive collapse of structures. To study the progressive collapse phenomenon of structures during earthquakes, two concentrically braced frame buildings were designed and their outer frames were studied while subjecting to loss of some structural elements. In the above buildings some braces were removed and the effect of such scenarios on the dynamic behavior of the structure during earthquake was investigated. In this research potential and capacity of the buildings for occurrence of seismic progressive collapse and their failure modes were determined using nonlinear time history and incremental dynamic analyses as well as performance based analysis according to FEMA 356 criteria. From these analyses it is possible to detect the most probable failure modes for retrofitting purposes, which can lead to more reliable structures in seismic regions.

1. INTRODUCTION

Various structures are subjected to different dangers which may lead to element loss and in some cases to entire collapse. The potential hazards and abnormal loads including vehicular collision, aircraft impact, gas explosions, etc. can produce progressive collapse (NIST, 2007) which refers to the action resulting from the failure of one element and leads to the failure of further similar elements (ASCE, 2005). Although the disproportion between cause and effect is a defining and common feature, there are various differing mechanisms that produce such an outcome. Structures are not usually designed for abnormal events which can lead to element loss and eventually to catastrophic failure. Most of the building codes have only general recommendations for mitigating the effect of progressive collapse in the structures that are overloaded beyond their design loads. The American Society of Civil Engineering (ASCE, 2005) is the only mainstream standard which addresses the issue of progressive collapse in some detail. The guidelines for progressive collapse resistant design are noticeable in US Government documents, e.g. General Service Administration (GSA, 2003) and Unified Facility Criteria (UFC, 2009). The GSA guidelines have provided a methodology to diminish the progressive collapse potential in structures based on Alternate Path Method (APM). It defines scenarios in which one of the building's columns is removed and the damaged structure is analyzed to study the system responses. The UFC methodology, on the other hand, is a performance-based design approach, and is partly based on the GSA provisions.
Recently, element loss analysis of steel frames has been the subject of several studies. Kim et al. (2009) studied the progressive collapse resisting capacity of steel moment frames by using APM recommended in the GSA and UFC guidelines, and observed that the nonlinear dynamic analysis led

to larger structural responses. Furthermore, they observed that the potential for progressive collapse was highest when a corner column was suddenly removed. Besides, it was concluded that the progressive collapse potential decreased as the number of stories increased. Khandelwal et al. (2009) concluded that a eccentrically braced frame is less vulnerable to progressive collapse than a special concentrically braced frame. Kim et al. (2009) depicted that the dynamic amplification can be larger than 2 which is recommended by the GSA and UFC guidelines. Fu (2009) declared that under the same general conditions, a column removal at a higher level will induce larger vertical displacement than a column removal at ground level. Kim et al. (2009) deduced that among the braced frames, the inverted-V type braced frame shows superior ductile behavior during progressive collapse. Liu (2010) analyzed catenary action and showed that it can reduce the bending moment significantly through axially restraining the beam. Also, two schemes were proposed for retrofitting the fin plate beam-to-column connection of tall steel framed structures subjected to a terrorist blast. England et al (2008) studied the importance of assessing the vulnerability of a structure to unforeseen events and examined the nature of unforeseen events. Besides, a theory of structural vulnerability which examines the form of the structure to determine the most vulnerable sequence of failure events was described. Pujol et al (2009) proposed that, a floor system can be designed to survive the sudden removal of one of its supports by proportioning the system. This can be achieved firstly, by using the results from a conventional linear static analysis of a model that excludes the column to be removed and a load factor exceeding 1.5 or secondly, providing adequate detailing to ensure that the system can reach deformations exceeding 1.5 times greater than the deformation associated with the development of its full strength. Asgarian et al. (2012) and Hashemi Rezvani et al. (2012) have investigated the behaviour of steel braced frame while losing some structural members and proposed the relation of number of frame stories with their strength against progressive collapse.
Researches on progressive collapse of structures generally focus on gravity and blast loadings and the design objective is to increase the redundancy and robustness of structures to prevent progressive collapse. Studying present scientific resources, it can be inferred that progressive collapse behavior of structures due to earthquake loads has not received as much attention. During seismic response, redistribution of load carried by damaged or failed structural members to adjacent members may lead to overstress or exceeding load resistant capacity of the other members resulting in spread or propagation of damage until development of collapse mechanisms in seismic progressive collapse of structures. Considering this fact that application of split-X braced frames allocates more area to architectures contrasting to other concentrically bracing systems, such structural system is used widely by designers as lateral load resisting system. Reviewing the subject literature, it can be seen that the behavior of such system after element loss is not fully focused and there is no answer to some important matters such as potential of seismic progressive collapse occurrence, effect of loss location and number of frame stories on it and determination of probable failure modes and associated loads. According to above mentioned matters, this research aims to answer to these questions in order to enhance the structural safety of these frames. For this purpose, first, the subject frames were designed and then, according to various scenarios related to number of frame stories and loss locations, the seismic behavior of structures after element loss was investigated. For studying such behaviour, incremental dynamic analysis was used. Besides, for comparing these analyses results with results obtained from displacement-based control actions, seismic performance of structures according to FEMA 356 (2000) was performed. By using these results, good understanding of seismic behavior of concentrically braced frames subjected to element loss is achieved and the structural height effect of such system on structural safety against progressive collapse was studied.

2. INVESTIGATED STRUCTURES

To investigate the seismic behavior of steel braced frames after loss of some braces, two split-X braced frame buildings, comprising of four and eight-story buildings respectively, as representative of low-rise and semi-high-rise buildings were designed for a site (Tehran, Iran) which represents a very high seismic zone. Lateral force was applied according to UBC (2007) for Zone 4 and soil category of S_c. The buildings were square in plan and consisted of five bays of 6.0 m in each direction and the story height of 3.2 m as shown in Fig. 1 and Fig.2. Gravity loads were supposed to be similar to common residential buildings. For member design subjected to earthquake, equivalent lateral static forces were applied at all the story levels. The dead and live loads of 6.5 and 2 $\mathrm{kN/m^2}$ respectively, were used for gravity load for all stories except the roof where the gravity loads consisted of 6.0 and 1.5 $\mathrm{kN/m^2}$ for dead and live loads respectively (2006). Table 1 gives the cross sections for all structural members.

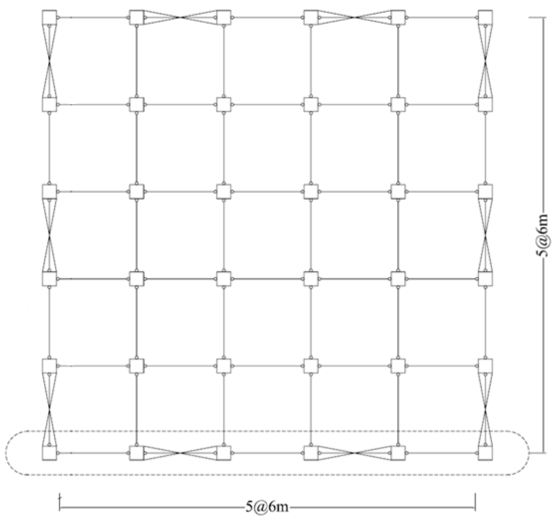


Figure 1. Plan view of buildings

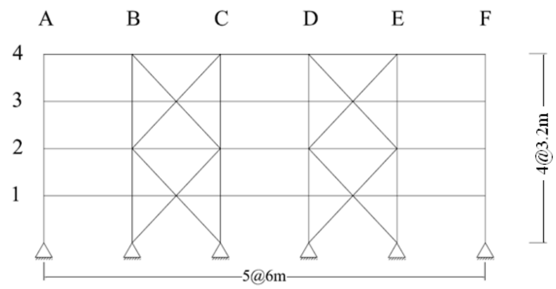


Figure 2. elevation and spans of frames

Table 1. Cross section for all members (B: Box Section in mm).

Story	Columns			Beam	Brace
	A and F axes	B and E axes	C and D axes		
			Eight-story frame		
8	B175 × 175 × 15	B175 × 175 × 15	B175 × 175 × 15	IPE330	B175 × 175 × 15
7	B175 × 175 × 15	B175 × 175 × 15	B175 × 175 × 15	IPE360	B175 × 175 × 15
6	B200 × 200 × 15	B225 × 225 × 20	B225 × 225 × 20	IPE360	B200 × 200 × 20
5	B200 × 200 × 15	B225 × 225 × 20	B225 × 225 × 20	IPE360	B200 × 200 × 20
4	B300 × 300 × 20	B350 × 350 × 25	B350 × 350 × 25	IPE360	B225 × 225 × 20
3	B300 × 300 × 20	B350 × 350 × 25	B350 × 350 × 25	IPE360	B225 × 225 × 20
2	B375 × 375 × 30	B400 × 400 × 35	B400 × 400 × 35	IPE360	B225 × 225 × 20
1	B375 × 375 × 30	B400 × 400 × 35	B400 × 400 × 35	IPE360	B225 × 225 × 20
			Four-story frame		
4	B175 × 175 × 15	B175 × 175 × 15	B175 × 175 × 15	IPE330	B175 × 175 × 15
3	B175 × 175 × 15	B175 × 175 × 15	B175 × 175 × 15	IPE360	B175 × 175 × 15
2	B200 × 200 × 15	B225 × 225 × 20	B225 × 225 × 20	IPE360	B200 × 200 × 15
1	B200 × 200 × 15	B225 × 225 × 20	B225 × 225 × 20	IPE360	B200 × 200 × 15

3. MODELING OF THE STRUCTURES

OpenSees (2007) finite element program was used to model and analyze the structures. A series of nonlinear dynamic were performed for external frames of the designed buildings which are shown in Fig. 2 with dotted lines. To model the steel behavior, a bilinear Kinematic stress-strain curve was assigned to the structural members using steel02 material from the library of materials introduced in OpenSees. A transition curve was provided for this material at the intersection of the first and second tangents to avoid any sudden changes in local stiffness matrices formed by the elements and to ensure a smooth transition between the elastic and plastic regions. A strain hardening modulus of 2% E and the ultimate stain of 4% was considered for the member behavior in inelastic range of deformation. This behavior together with the structural steel properties is shown in Fig.3. For the beams, columns and braces, nonlinear beam-column elements in combination with fiber cross sections were used to model the cross sections as accurately as possible. Also, the plastification of elements over the member length and cross section was considered. Moreover, large displacement effects were accounted for through the employment of corotational transformation of the geometric stiffness matrix. All the frame members, i.e. beams, columns (At the foundation level) and braces, were considered as pin-ended. An initial mid span imperfection of $L/1000$ was applied for all braces and columns as depicted in Fig. 4.

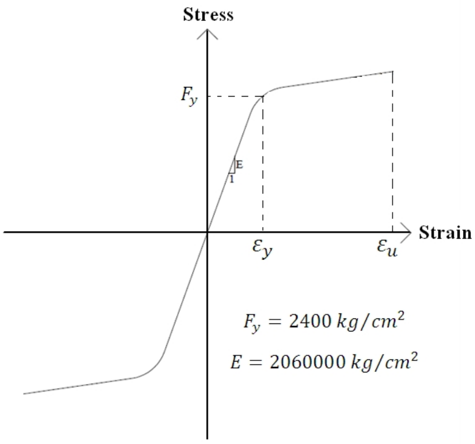


Figure 3. Structural steel behavior

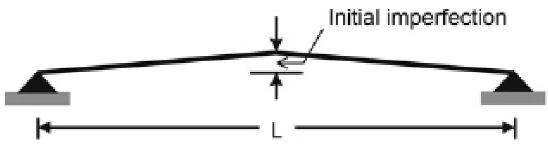
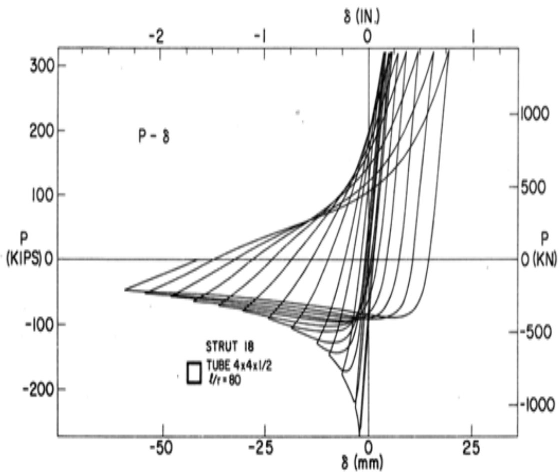


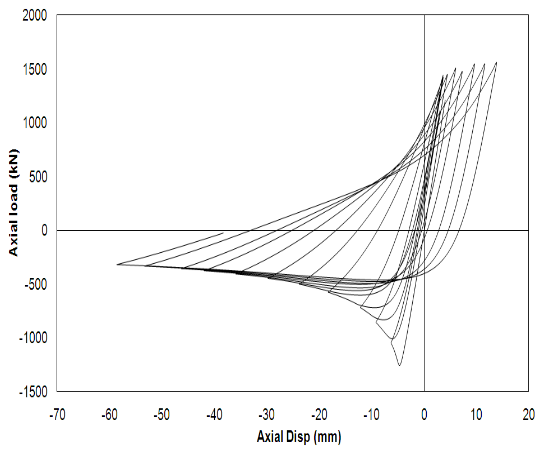
Figure 4. Initial imperfection

3. 1. MODEL VERIFICATION

Several verification exercises of the developed model and its structural elements can be found in researches of Asgarian et al. (2008-2010). But in this research to verify the buckling and post buckling behavior of bracing members, the result of the experimental study on a square tube, Strut 18 (TS $4 \times 4 \times 0.5$), under reversed cyclic loading conducted by Black et al. (1980) was compared to the result of the numerical model, developed in this research according to specifications mentioned in report of Black et al. (1980). Fig. 5 shows a comparison between results of the numerical model and those obtained from experimental study. According to this figure, the model represented the buckling strength and post-buckling stiffness of the tested specimen as accurately as possible.



(a) Experimental analysis



(b) Numerical analysis

Figure 5. Verification of brace behavior

4. SEISMIC PROGRESSIVE COLLAPSE ANALYSIS
4.1. ANALYSIS PROCEDURE

In order to investigate the behaviour of concentrically braced frames while losing some structural elements during earthquakes, at first, the gravity loads were applied to the structures. Afterward the predefined braces were removed from the structure and then the earthquake acceleration was applied and the subsequent response of the braced frames was investigated. The simulations were conducted with 5 % mass and stiffness proportional damping. In order to investigate the structural behavior of the concentrically braced frames when critical members are lost, one or two braces in the first story of the investigated structures were selected to be omitted suddenly in accordance with the symmetry in the first story. In Fig. 6 the columns and braces located in the first story of the investigated frames are portrayed and coded for simplifying the future discussions. Table 2 presents the list of loss cases studied in this research together with the members that were removed in each case. For each loss scenario, incremental dynamic analysis (Vamvatsicos, 2002) was performed using several time history analyses in order to determine the maximum inter-story drift and the seismic performance level of structure. The results of such scenarios are compared to the result of the intact structures in order to investigate the seismic performance of structures which may lose some of their lateral load bearing elements in earthquakes. In Fig. 7 the IDA curve of the four-story frame is illustrated.

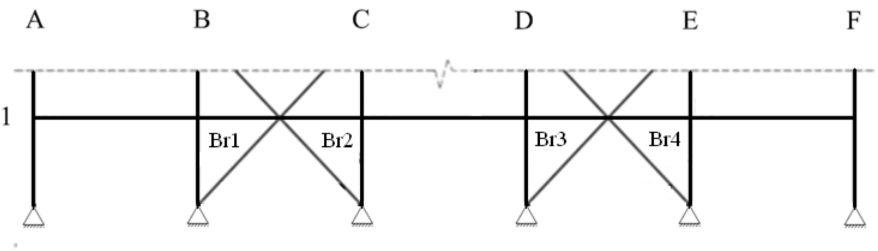


Figure 6. Numbering of frame elements

Table 2. Loss cases (scenarios)

Loss scenario	Frame Type	Removed Element(s)
1	four-story frame	-
2	four-story frame	Br2
3	four-story frame	Br1 and Br2
4	eight-story frame	-
5	eight-story frame	Br2
6	eight-story frame	Br1 and Br2

According this figure it is apparent that loss of lateral load bearing elements leads to decrease in seismic load capacity of the investigated frames. According to Iranian Seismic load the limit state of inter-story drift is 2.5%. In figure 7 it is shown that while in case 1, intact structure can sustain the PGA of 0.7 g, in loss case 2 and 3 these values are 0.66 g and 0.55 g, respectively. Besides, for the inter-story drift of 4.0%, it can be observed that the intact structure can sustain the PGA of 1.22 g while in the four-story frame, loss scenarios lead to decrease this value to 0.71 g and 0.51 g in cases 2 and 3, respectively.

To compare the force-controlled actions with the displacement-controlled action, as another approach of element loss analysis, performance based analysis of such removals was carried out. Toward this aim, the limit states given in the FEMA 356 were mainly used to determine the failure mode of each loss case. Table 5-6 and 5-7 of FEMA 356 were used for modeling parameters and acceptance criteria for nonlinear dynamic procedures. For computing the updated yield rotation of the beams and columns under increasing load, in each step, the axial force of a structural member at the instant of computation was utilized. Besides, the columns with $P/P_{CL} > 0.5$, (P is the axial force in a member and P_{CL} is the lower-band axial strength of a column) were considered force-controlled which resulted in excluding some columns from the displacement-based controlled actions in higher dynamic overload factors. For the braces, the axial deformation at expected buckling load was the basis of determination of the limit states. In this analysis, for each step of increasing the applied load, the plastic rotations and acceptance criteria of the beams and columns were updated as a function of the yield rotation. Performing multiple analyses, the failure PGA related to FEMA 356 limit states were computed. In Table 3 performance based analysis results related to each limit state are listed for each scenario. In this table, LS and CP represent the Life Safety and Collapse Prevention respectively.

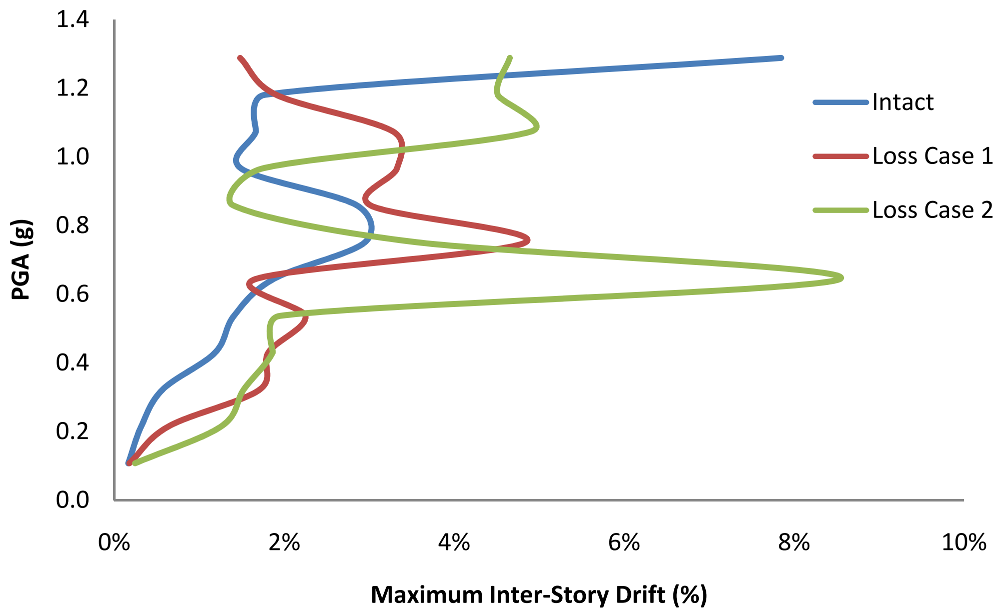
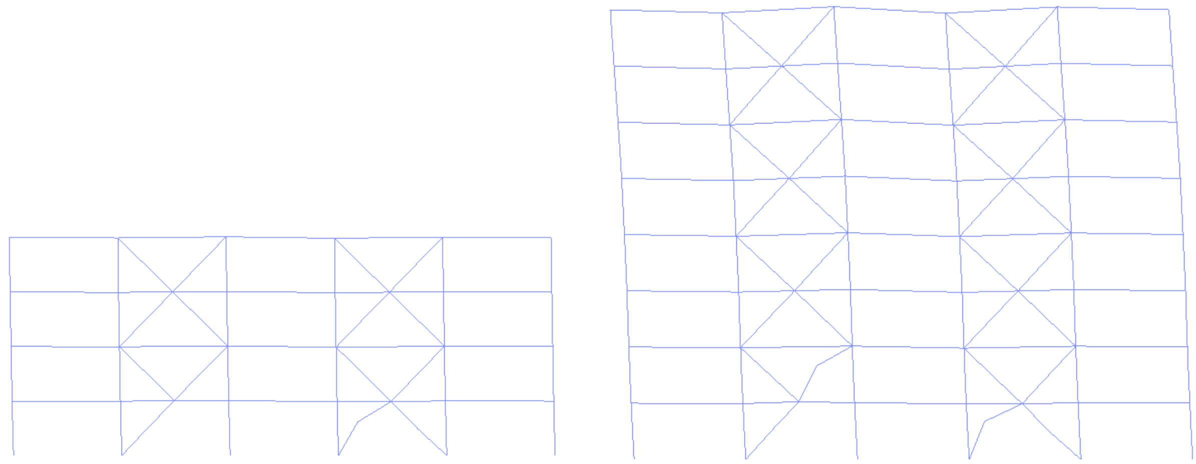


Figure 7. IDA curves for the four-story frame (Imperial Valley)

Table 3. Performance based analysis results (Imperial valley)

Loss case/Scenario	Failure mode	Limit state	PGA (g)	Axial Disp.
1	Br3	LS	0.28	2.33
		CP	0.29	3.31
2	Br3	LS	0.17	2.38
		CP	0.19	3.29
3	Br4	LS	0.13	2.51
		CP	0.14	3.64
4	Br3	LS	0.41	2.38
		CP	0.43	3.41
5	Br3	LS	0.29	2.42
		CP	0.30	3.47
6	Br4	LS	0.21	2.39
		CP	0.23	3.51

According to table 3 it can be observed that in the investigated structures, in all scenarios brace failure is the main and initial failure mode. Furthermore, it can be inferred that failure PGA of the studied concentrically braced frames decrease as the number of removed structural elements increases. For example, the intact structure in case 1 had the failure PGA of 0.28 g in limit state of LS according to FEMA 356 while in loss case 2 and 3 the failure PGAs are 0.17 g and 0.13 g, respectively. Furthermore, the intact structure in case 4 had the failure PGA of 0.41 g in limit state of LS while in loss case 5 and 6 the failure PGAs are 0.29 g and 0.21 g, respectively. In Fig. 8 the deformed shape of structures in loss scenarios 2 and 5 are illustrated.



(a) Loss scenario 2, PGA = 0.21 g

(b) Loss scenario 5, PGA = 0.32 g

Figure 8. Deformed shape of structures in loss scenarios 2 and 5 (Imperial Valley)

5. DISCUSSION OF THE RESULTS AND CONCLUSION

In this research seismic progressive collapse of concentrically braced framed through incremental dynamic analysis and performance based analysis was investigated. Toward this aim two concentrically braced frames were designed and their numerical models were built in Opensees software. In this research some loss scenarios were considered in which one or two lateral load bearing elements were removed from the structure during an earthquake. Analysis results revealed that in the investigated structures the loss of one or two braces lead to decrease of seismic performance. In the studied loss scenarios the loss of one or two lateral load bearing elements lead to increase of the maximum inter-story drifts in a specific PGA. On the other hand, in the specific drift ratio, for example 2.5 %, the loss cases 2 and 3 were the cause of decrease in applied PGAs to the values of 5.7% and 21.5% while at the drift ratio of 4% these scenarios lead to decrease the applied PGAs to the values of 41.8% and 58.2%, respectively. For the displacement-based analysis according to the FEMA 356 criteria, the loss scenario 2 and 3 decreased the failure PGAs for the limit states of LS to the values of 39% and 53%, respectively while these values for the limit states of CP were 34% and 52%, respectively. Besides, for the eight-story frame, scenario 5 and 6 decreased the failure PGAs for the limit states of LS to the values of 29% and 48%, respectively while these values for the limit states of CP were 30% and 49%, respectively. Accordingly, it was also concluded that the potential for seismic progressive collapse decreased as the number of frame stories increased. The research also showed that in the studied steel frames there is need to retrofit braces in seismic regions where there is probability of loss of other braces in order to minimize the potential of occurrence of seismic progressive collapse. It should be mentioned such conclusion is limited to the investigated frames and there is still need to perform much more analyses to generalize the points indicated.

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