Steel and Composite Structures, Vol. 12, No. 3 (2012) 231-248
DOI: http://dx.doi.org/10.12989/scs.2012.12.3.231

Element loss analysis of concentrically braced frames considering structural performance criteria

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(Received June 10, 2011, Revised November 05, 2011, Accepted December 27, 2011)

Abstract. This research aims to investigate the structural behavior of concentrically braced frames after element loss by performing nonlinear static and dynamic analyses such as Time History Analysis (THA), Pushdown Analysis (PDA), Vertical Incremental Dynamic Analyses (VIDA) and Performance-Based Analysis (PBA). Such analyses are to assess the potential and capacity of this structural system for occurrence of progressive collapse. Besides, by determining the Failure Overload Factors (FOFs) and associated failure modes, it is possible to relate the results of various types of analysis in order to save the analysis time and effort. Analysis results showed that while VIDA and PBA according to FEMA 356 are mostly similar in detecting failure mode and FOFs, the Pushdown Overload Factors (PDOFs) differ from others at most to the rate of 23%. Furthermore, by sensitivity analysis it was observed that among the investigated structures, the eight-story frame had the most FOF. Finally, in this research the trend of FOF and the FOF to critical member capacity ratio for the plane split-X braced frames were introduced as a function of the number of frame stories.

Keywords: element loss, progressive collapse, concentrically braced frame, pushdown, vertical incremental dynamic analysis, failure mode, performance-based analysis, overload factor.

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 (NISTIR 7396 2007) which refers to the action resulting from the failure of one element and leads to the failure of further similar elements (ASCE 7-05 2005). Although the disproportion between cause and effect is a defining and common feature, there are various differing mechanisms that produce such an outcome. Their main characteristics include initial failure of vertical load-bearing elements; partial or complete separation and fall in a vertical rigid body motion of components; transformation of potential energy into kinetic energy; redistribution of the forces carried by these elements in the remaining structure; impact of the separated and falling structural components on the remaining structure; impulsive loading due to the suddenness of the initial failure; failure of other vertical load-bearing elements due to the impact loading; instability of the elements in compression; collapse progression in the vertical or horizontal directions (Staroseek 2007).

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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 7-05 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 an eccentrically braced frame is less vulnerable to progressive collapse than a special concentrically braced frame. Kim *et al.* (2009) depicted that the dynamic amplification factor 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.

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 progressive collapse occurrence, effect of loss location and number of frame stories on it and determination of probable failure modes and associated loads. Besides, the relation of force-based action approaches used in most articles related to progressive collapse analysis with standard regulations for structural performance study is not precisely investigated. According to above mentioned matters, this research aims to answer to these questions in order to enhance the structural safety of these frames as well as saving time and analysis efforts. For this purpose, at first, the subject frames were designed and then, according to various scenarios related to number of frame stories and loss locations, the behavior of structures after element loss was investigated. For studying such behavior, various types of nonlinear analyses such as THA, PDA and VIDA were used. Besides, for comparing these analyses results with results obtained from displacement-based control actions, the PBA of structures was performed. By using these results, good understanding of structural 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 behavior of steel braced frames after loss of one column and connected brace, four split-X braced frame buildings, comprising of four, six, eight and ten-story buildings respectively, as representative of low-rise, mid-rise, semi-high-rise and high-rise buildings were designed for a site (Tehran, Iran) which represents a very high seismic zone. Lateral force was applied according to UBC (1997) 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 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 kN/m^2 for dead and live loads respectively (Minimum building loads 2006). Table 1 gives the cross sections for all structural members.

3. Modeling of the structures

OpenSees (Mazzoni *et al*. 2007) finite element program was used to model and analyze the structures. A series of nonlinear dynamic and static analyses were performed for external frames of the

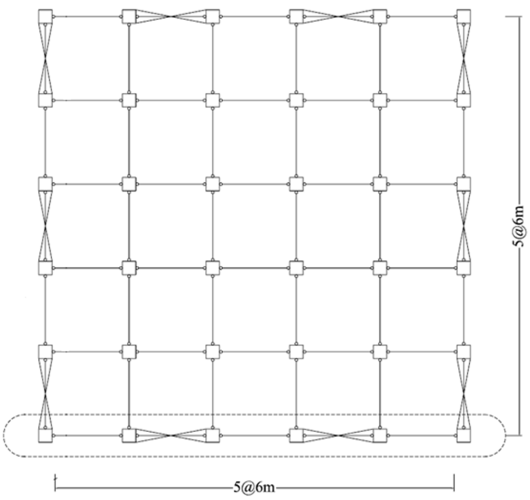


Fig. 1 Plan view of buildings

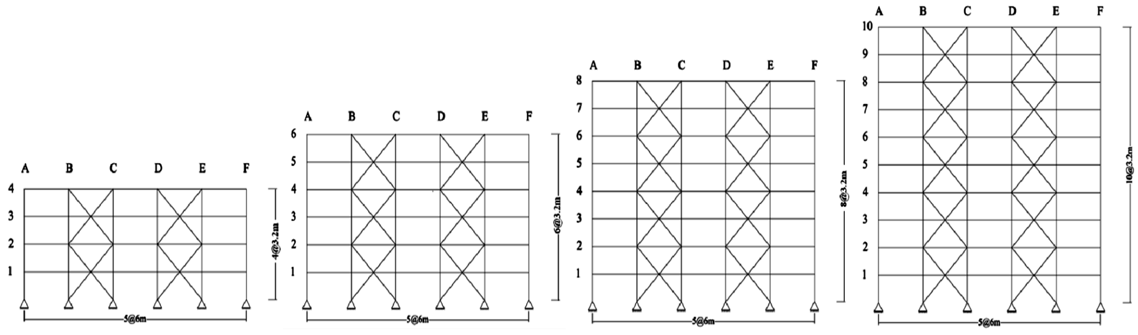


Fig. 2 Elevation and dimensions of frames

designed buildings which are shown in Fig. 2. To model the steel behavior, a bilinear Kinematic stress-strain curve was assigned to the structural members using steel 02 and fatigue 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 a maximum ductility of 15 were 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.

3.1 Model verification

Though several verification exercises of the developed model and its structural elements can be found in (Asgarian *et al*. 2008-2010, 2012) in this research the result of the experimental study on a square tube, Strut 17 (*TS*4 × 4 × 0.25), under reversed cyclic loading conducted by Black *et al.* (1980) was compared to the result of the numerical model, developed in this research. This was to verify the buckling and post buckling behavior of bracing members. Fig. 5(a) shows the experimental response of the axial force-axial displacement relationship while Fig. 5(b) illustrates the numerical result. Note that the model represented the buckling strength and post-buckling stiffness of the tested specimen as accurately as possible.

Table 2 gives the comparison of the analytical buckling loads of three bracing members with those calculated in accordance with UBC (1997) as their expected bucking loads. According to this table it can be inferred that the developed model is able to successfully simulate the buckling load of the members with various slenderness ratios.

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

Story	Columns			Beam	Brace
	A and F axes	B and E axes	C and D axes		
			Ten-story frame		
10	B175 × 175 × 15	B200 × 200 × 15	B200 × 200 × 15	IPE330	B125 × 125 × 12
9	B175 × 175 × 15	B200 × 200 × 15	B200 × 200 × 15	IPE360	B125 × 125 × 12
8	B175 × 175 × 15	B200 × 200 × 15	B200 × 200 × 15	IPE360	B175 × 175 × 15
7	B175 × 175 × 15	B200 × 200 × 15	B200 × 200 × 15	IPE360	B175 × 175 × 15
6	B225 × 225 × 20	B275 × 275 × 20	B275 × 275 × 20	IPE360	B200 × 200 × 20
5	B225 × 225 × 20	B275 × 275 × 20	B275 × 275 × 20	IPE360	B200 × 200 × 20
4	B300 × 300 × 25	B350 × 350 × 30	B350 × 350 × 30	IPE360	B250 × 250 × 20
3	B300 × 300 × 25	B350 × 350 × 30	B350 × 350 × 30	IPE360	B250 × 250 × 20
2	B375 × 375 × 30	B400 × 400 × 40	B400 × 400 × 40	IPE360	B250 × 250 × 20
1	B375 × 375 × 30	B400 × 400 × 40	B400 × 400 × 40	IPE360	B250 × 250 × 20
			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 × 15
5	B200 × 200 × 15	B225 × 225 × 20	B225 × 225 × 20	IPE360	B200 × 200 × 15
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
			Six-story frame		
6	B175 × 175 × 15	B175 × 175 × 15	B175 × 175 × 15	IPE330	B150 × 150 × 15
5	B175 × 175 × 15	B175 × 175 × 15	B175 × 175 × 15	IPE360	B150 × 150 × 15
4	B200 × 200 × 15	B225 × 225 × 20	B225 × 225 × 20	IPE360	B200 × 200 × 15
3	B200 × 200 × 15	B225 × 225 × 20	B225 × 225 × 20	IPE360	B200 × 200 × 15
2	B300 × 300 × 20	B350 × 350 × 25	B350 × 350 × 25	IPE360	B200 × 200 × 15
1	B300 × 300 × 20	B350 × 350 × 25	B350 × 350 × 25	IPE360	B200 × 200 × 15
			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

4. Preliminary element loss analysis

4.1. Analysis procedure

The applied load to structures for studying their behavior after structural element loss consisted of dead, live and lateral loads according to Eq. (1) where DL is the dead load, LL is the live load and $0.002\Sigma P$ is the lateral load in which ΣP is sum of the gravity loads acting on only one floor (UFC 2009)

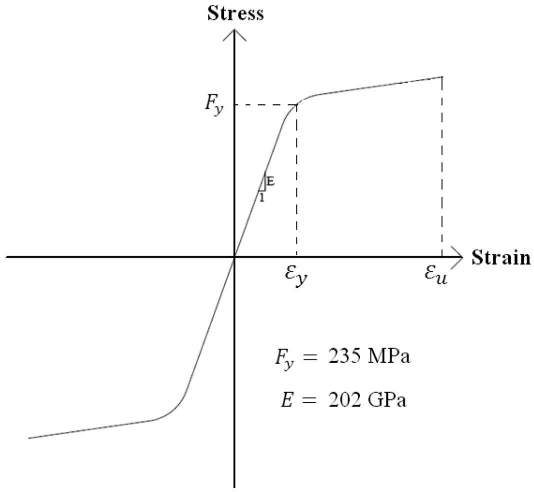


Fig. 3 Structural steel behavior

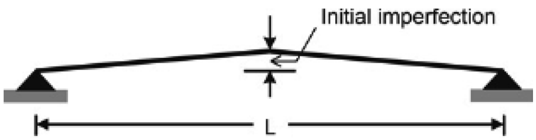
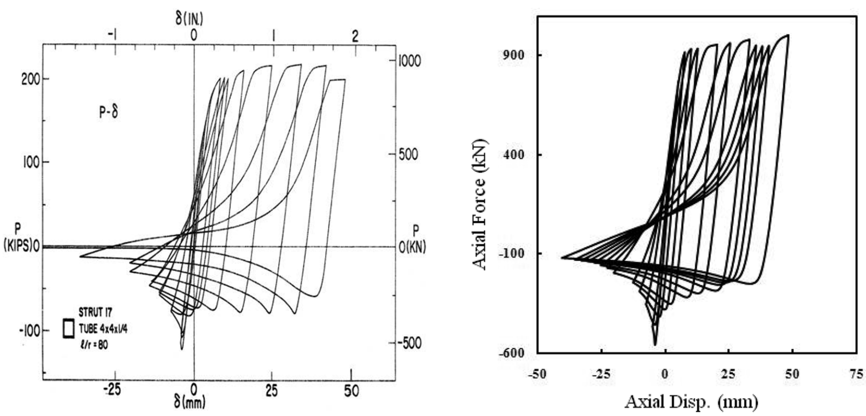


Fig. 4 Initial imperfection in compression members



(a) (b)

Fig. 5 (a) Model verification- Experimental analysis (Black *et al.* 1980), (b) Model verification - Numerical analysis

$$\text{Applied Load} = 1.2DL + 0.5LL + 0.002\Sigma P \tag{1}$$

In nonlinear dynamic analyses the gravity loads were linearly increased during 5 seconds to reach the final values. After that, they were kept unchanged for 2 seconds to avoid exciting dynamic effects.

Table 2 Analytical and UBC 97 buckling loads

Section	Slenderness ratio	Analytical buckling load (kN)	UBC 97 buckling load (kN)
B225 × 225 × 20	46.7	3862.42	4057.26
B150 × 150 × 15	79.3	1477.84	1558.17
B100 × 100 × 10	119.1	459.58	499.95

Once the gravity loads were fully applied at duration of 7 seconds, one column and related brace were suddenly removed, and afterward 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, two columns and related braces in the first story were selected to be omitted suddenly in accordance with the symmetry in the 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 3 presents the list of APM cases studied in this research together with the members that were removed in each case. For each removal scenario, the peak value of internal forces of each structural element was checked against its nominal capacity. If the peak value exceeded the capacity calculated from the code it means that the building is susceptible to progressive collapse and the analysis ends in that scenario. If not, it means that the structure is able to reach a static balance after element removals but solely according to the imposed loads.

4.2 Analysis results

In Fig. 7 to Fig.10 time history responses of axial load of columns and braces for critical APM cases

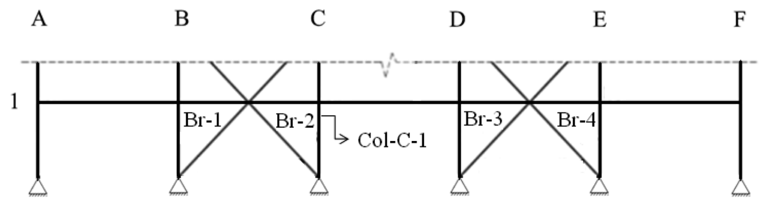


Fig. 6 Example of coding system of columns and braces

Table 3 APM analysis cases (scenarios)

APM case/scenario	Frame type	Removed elements
1	four-story frame	Col-B-1 and Br-1
2	four-story frame	Col-C-1 and Br-2
3	six-story frame	Col-B-1 and Br-1
4	six-story frame	Col-C-1 and Br-2
5	eight-story frame	Col-B-1 and Br-1
6	eight-story frame	Col-C-1 and Br-2
7	ten-story frame	Col-B-1 and Br-1
8	ten-story frame	Col-C-1 and Br-2

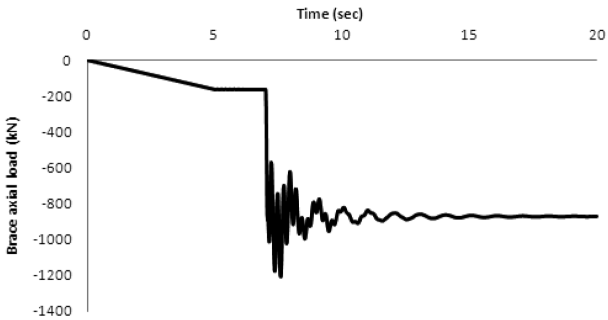


Fig. 7 Time history response of axial force of Br-2, APM case 3

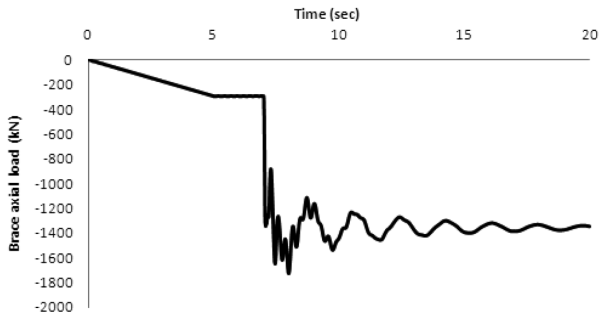


Fig. 8 Time history response of axial force of Br-1, APM case 8

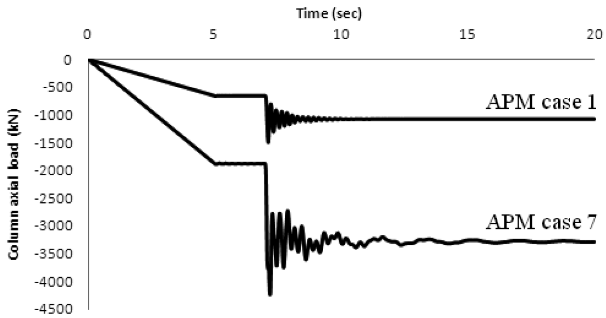


Fig. 9 Time history response of axial force of Col-C-1

are illustrated. As shown in these figures a large distribution of forces was observed to take place. For example in the APM case 1, the axial force of Col-C-1 spiked from 653.35 kN to a peak value of 1486.21 kN before settling down at a steady value of 1070.97 kN. And in the APM case 6, the axial force of Col-B-1 spiked from 1529.24 kN to a peak value of 3308.57 kN before settling down at a steady value of 2522.95 kN. By assuming an effective length factor, K = 1.0, the axial capacity of Col-C-1 in the first scenario is 36973.91 kN while a value for Col-B-1 in the sixth scenario is 11867.1 kN They are substantially more than the peak value computed in those columns which when combined with the relatively small moment generated on the columns, implies that the columns will not be

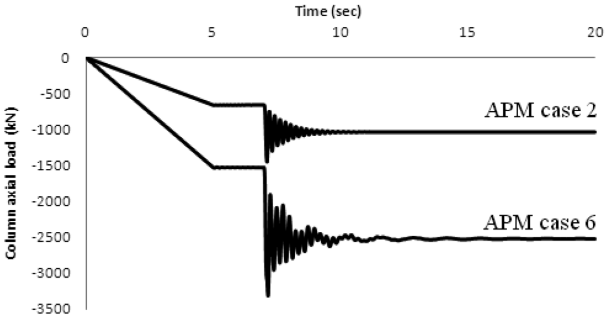


Fig. 10 Time history response of axial force of Col-B-1

overloaded. The same story would be for braces. For example, in the APM case 3, the axial force in Br-2 spiked from 163.44 kN to a peak value of 1206.26 kN before settling down at a steady value of 870.82 kN and for the axial force of Br-1 in the APM case 8 spiked from 295.17 kN to a peak value of 1730.11 kN before settling down at a steady value of 1348.85 kN. By assuming an effective length factor, K= 1.0, the capacity of those braces are 3041.69 kN and 4057.26 kN respectively.

For the eight scenarios investigated in this research the simulation results demonstrated that the system is able to successfully absorb the loss of members predefined in Table 3; hence, there would be no collapse prediction. This situation occurred because the buildings were designed to support the seismically induced forces and the extra capacity in which the columns connected to the braces are to bear the magnified earthquake forces. Therefore, the compression members were so massive that the frames were still able to successfully carry all the gravity loads. In such frames, bays influenced by element removals, derived their stability from intact bays and as a result they did not collapse. Transmission of loads between the damaged and intact bays took place through the gravity beams located in unbraced bays. Though these beams were under significant tension forces, the members were able to fruitfully transmit those loads.

5. Advanced element loss analysis

The preliminary analysis performed in the previous section determined whether the internal forces of each element exceed its nominal capacity after element removals. This method did not consider determination of the most critical places of element removal, probable failure mode or residual capacity of the structure after such removals. In order to determine the structural capacity, in this research advanced element loss analysis was performed. Such analysis consists of both nonlinear force-controlled and displacement-controlled actions by which it became possible to compare the results of the so-called subjects. The force-controlled action was done through PDA and VIDA considering increasing gravity loads in order to estimate the residual capacity of a damaged structure and the probable failure modes in the cases in which the structural system survives the loss of critical members. According to such an analysis, gravity load was increased until the failure leading to disproportionate collapse of the structure occurred. The load corresponding to this failure state was defined as the failure load. On the other hand, the displacement-controlled action was performed in accordance to FEMA 356 (2000) by which the structure was loaded increasingly till the limit states commenced in the standard

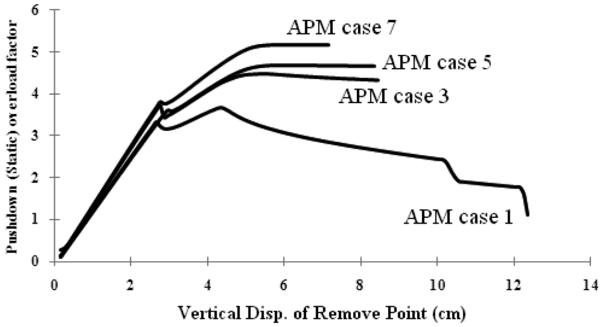


Fig. 11 Pushdown curves for the odd cases

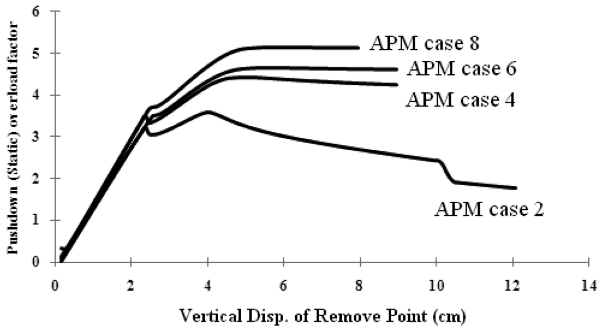


Fig. 12 Pushdown curves for the even cases

occurred.

5.1. Pushdown analysis (PDA)

After predicting no collapse in preliminary analyses, a PDA was performed for each scenario. To focus on the damaged bays, only the bay PDA was performed. In this method the gravity load was increased proportionally only in the bays that suffered the damage until instability occurred while the remaining part of the structure was only subjected to the nominal gravity loads. As a result of such an analysis it became possible to detect the probable failure modes and Pushdown Overload Factor (PDOF). The overload capacity of the structure in this analysis is expressed in terms of PDOF as demonstrated in Eq. (2). In Fig. 11 and Fig. 12 the pushdown curves of the investigated scenarios are depicted. According to these curves it became possible to detect the Failure Overload Factors (FOFs) and the most probable failure mode by determining the first fall (drop) in the mentioned curves.

$$\text{overload factor} = \frac{\text{Failure load}}{\text{Nominal gravity load}} \tag{2}$$

In Fig. 13 the critical PDOFs resulted from the minimum factor in each story (between the two scenarios) are depicted. As shown in this figure, the minimum PDOF is 3.22 for the four-story frame while the highest one is 3.51 for the eight-story frame.

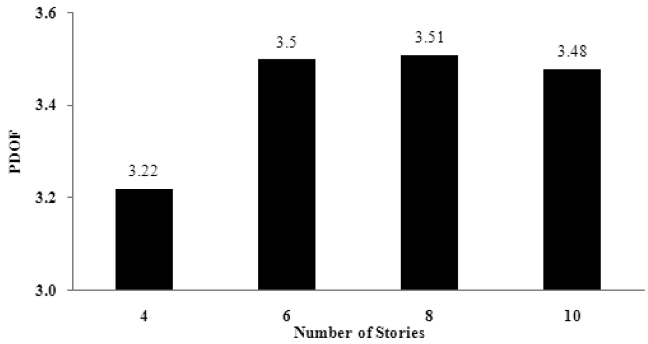


Fig. 13 Pushdown Overload Factors (PDOFs)

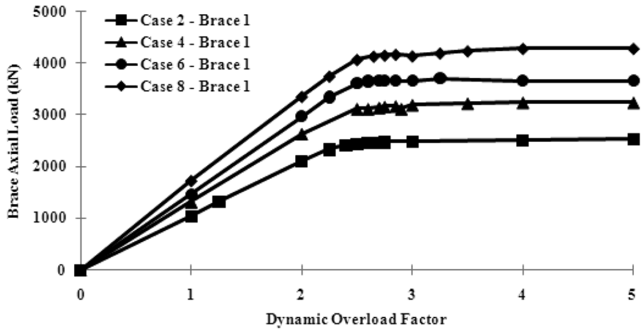


Fig. 14 Vertical Incremental Dynamic Analysis (VIDA) curves for the odd cases

5.2. Vertical incremental dynamic analysis (VIDA)

The VIDA, performed in this stage, was similar to nonlinear dynamic analyses conducted in preliminary analysis section but with one important difference, i.e., the gravity load in the damaged bays was increased incrementally after loss of elements up to a limit in which the first failure mode occurred. Multiple analyses with increasing gravity loads in the damaged bays might be required until an overload factor corresponding to the failure mode in the damaged bays was determined. This analysis method accounts for the dynamic effects and is similar to incremental dynamic analysis utilized in earthquake engineering (Vamvatsicos *et al.* 2002). In Fig. 14 and 15 the VIDA curves of the axial force of critical braces are illustrated as a function of Vertical Incremental Dynamic Overload Factors (VIDOFs). According to these curves, the VIDOF is determined by two approaches; when the internal force of a structural member decreases or remains constant by increasing the applied load. These curves were drawn for each structural element and for each predefined scenario and the lowest overload factors were selected as VIDOF in the investigated APM case.

In Fig. 16 the critical VIDOFs resulted from the minimum factor in each story are depicted. As shown in this figure, the minimum VIDOF is 2.75 for the four-story frame while the highest value is 2.95 for the eight-story frame.

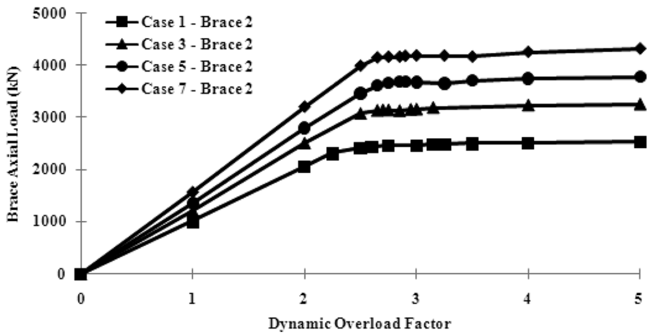


Fig. 15 Vertical Incremental Dynamic Analysis (VIDA) curves for the even cases

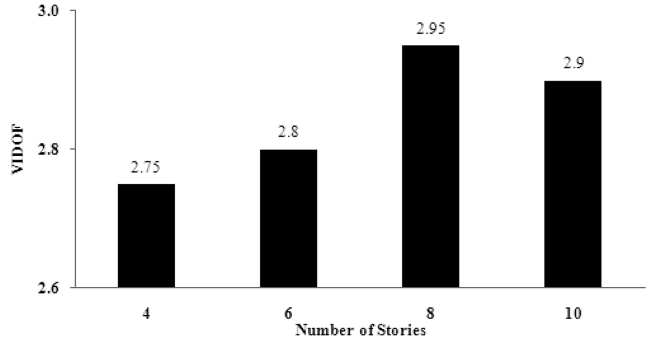


Fig. 16 Vertical Incremental Dynamic Overload Factors (VIDOFs)

5.3. Performance-based analysis (PBA)

To compare the force-controlled actions performed in the previous sections with the displacement-controlled action, as another approach of element loss analysis, PBA 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 APM 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 Performance-Based Overload Factors (PBOF) related to FEMA 356 limit states were computed. In Table 4, PBA results related to each limit state are listed for each scenario. In this table, IO, LS and CP represent the Immediate Occupancy, Life Safety and Collapse Prevention, respectively. According to Table 4, it can be inferred that the acceptance criteria in plastic rotations are not exceeded for the columns of the investigated frames, though encountering

Table 4 Performance-Based Analysis (PBA) results

APM case/Scenario	Failure mode	Limit state	Acceptance Criteria (cm)	PBOF	Axial Disp.(cm)
1	Br-2	IO	0.115652	0.57	0.11661
		LS	2.313045	2.66	2.33244
		CP	3.238263	2.84	3.26431
2	Br-1	IO	0.115652	0.56	0.11664
		LS	2.313045	2.66	2.32235
		CP	3.238263	2.86	3.24142
3	Br-2	IO	0.115023	0.63	0.11701
		LS	2.300465	2.78	2.35546
		CP	3.220651	2.98	3.23057
4	Br-1	IO	0.115023	0.59	0.11674
		LS	2.300465	2.73	2.31996
		CP	3.220651	3.03	3.23099
5	Br-2	IO	0.118002	0.67	0.11973
		LS	2.360049	3.00	2.39064
		CP	3.304069	3.24	3.30686
6	Br-1	IO	0.118002	0.61	0.11843
		LS	2.360049	2.96	2.39495
		CP	3.304069	3.25	3.31617
7	Br-2	IO	0.120074	0.65	0.12148
		LS	2.401479	2.94	2.45657
		CP	3.362071	3.14	3.40447
8	Br-1	IO	0.120074	0.59	0.12099
		LS	2.401479	2.98	2.41295
		CP	3.362071	3.26	3.36738

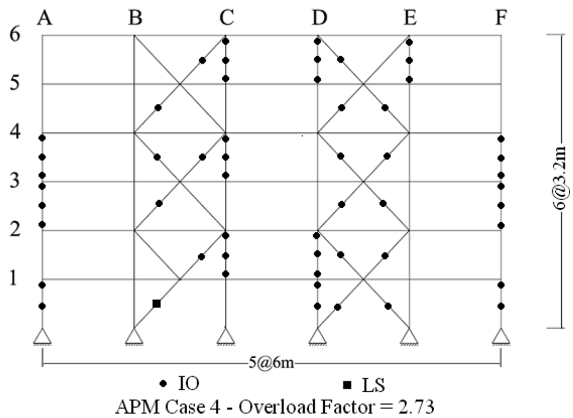


Fig. 17 Example of development of plastic hinges together with the monitored performance level

significant axial forces. The situation occurred because of the excluding rule dependant to P / P_{CL} ratio in higher dynamic overload factors. Fig. 17 depicts the development of plastic hinges together with the monitored performance level of the six-story frame while losing the predefined structural elements

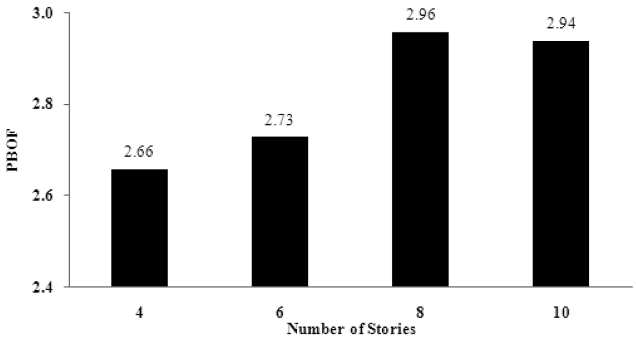


Fig. 18 Performance-Based Overload Factors (PBODs)

Table 5 Failure Overload Factors (FOFs) and failure modes

APM Case	Failure mode	FOF		
		PDOF	VIDOF	PBOF
1	Br-2	3.22	2.75	2.66
2	Br-1	3.24	2.80	2.66
3	Br-2	3.61	2.80	2.78
4	Br-1	3.50	2.85	2.73
5	Br-2	3.81	2.95	3.00
6	Br-1	3.51	3.00	2.96
7	Br-2	3.61	3.00	2.94
8	Br-1	3.48	2.90	2.98

considering the overload factor of 2.73. In this figure, it is apparent that a brace in the first story has exceeded its LS limit state while other structural members are at most in their IO state. Such conditions occurred for other APM cases, too. UFC states that the limit state for structural elements in compression is LS; accordingly, in Fig. 18 the critical PBOFs resulted from the minimum factor in each story is depicted. As shown in this figure, the minimum PBOF is 2.66 for the four-story frame while the highest one is 2.96 for the eight-story frame.

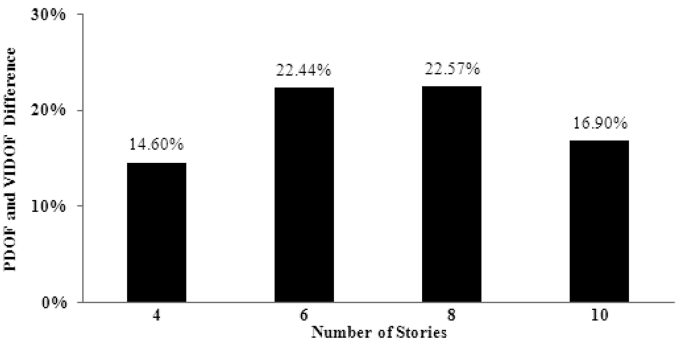


Fig. 19 PDOF and VIDOF difference

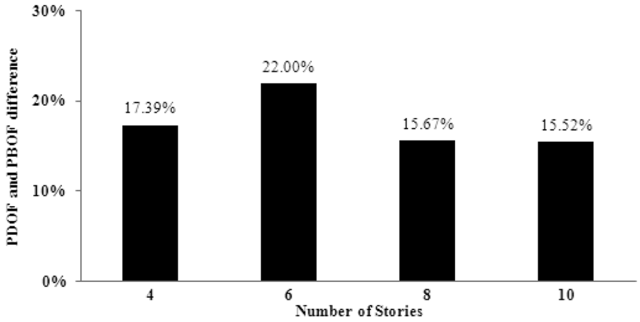


Fig. 20 PDOF and PBOF difference

6. Discussion of the results

In Table 5 various types of FOFs including PDOF, VIDOF and PBOF are listed for each scenario. According to this table, the collapse was initiated by buckling of braces for each scenario which implies that the columns of such structures have adequate strength to survive one element loss in the first story. It is due to designing of the investigated structures against high lateral forces. Furthermore, it can be seen that totally, all kind of FOFs increased as the frame height increased which implies that the structural safety against progressive collapse increased.

In Fig. 19 and Fig. 20 the difference between PDOF and VIDOF and PDOF and PBOF are depicted respectively. According to these figures it can be deduced that the highest value of difference between PDOF and VIDOF is 22.57% while the average value is 19.13%. Besides, the highest value of difference between PDOF and PBOF is 22.00% while the average value is 17.64%. In addition, it can be concluded that the determined VIDOFs and their detection method was suitable for the plane concentrically braced frames as there was only 1.87% difference between VIDOF and PBOFs. Besides, it reveals that FEMA 356 and its acceptance criteria which are generally used in seismic rehabilitation of buildings can accurately determine the FOFs related to element loss of the investigated split-X braced frames. According to these results, in order to save element loss analysis time an effort, for the studied structures and their design specifications, it will be more convenient to perform PDA and modify PDOF to VIDOF or PBOF by the highest value of difference obtained from results of this research. For other types of frame the difference value should be investigated.

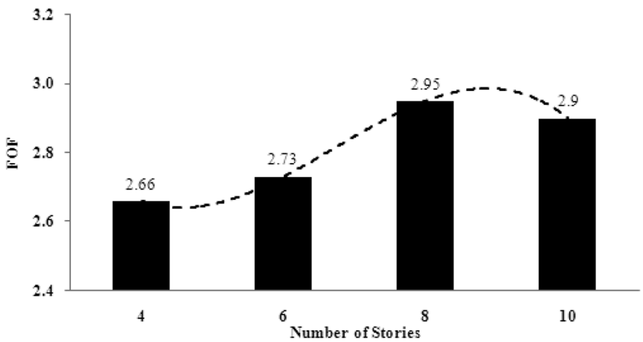


Fig. 21 Failure Overload Factors (FOF)

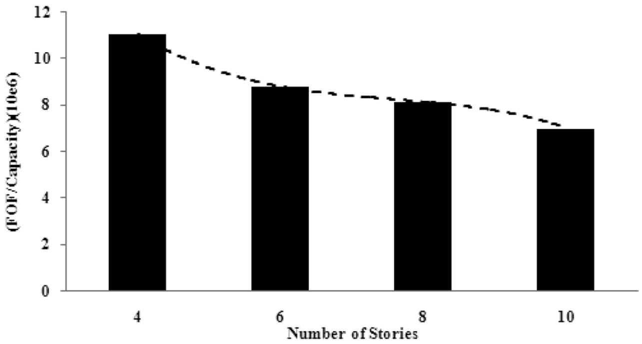


Fig. 22 FOF to bracing member capacity ratio

In Fig. 21 the FOFs, resulted of the minimum overload factors computed via PDA, VIDA and PBA are depicted. According to this figure, it is apparent that the minimum FOF for the investigated scenarios is 2.66 which means that the investigated structures can survive the loss of one column and connected brace in the first story against loads 2.66 times greater than the UFC recommended load. Furthermore, comparing FOFs, it can be concluded that for the frames studied in this research, where there is high risk of element loss, the eight-story frame have the most safety as it had the highest FOF. Besides, the FOF of such structural system subjected to element loss in the first story follows Eq. (3) by which it becomes possible for designers and scholars to calculate FOF without element loss analysis of concentrically braced frames studied in this research.

$$\mathrm{FOF} = -0.0088\mathrm{N}^3 + 0.1763\mathrm{N}^2 - 1.0625\mathrm{N} + 4.65 \text{ and } \mathrm{R}^2 = 1 \tag{3}$$

In which FOF is failure overload factor and N is number of frame stories.

In Fig. 22 FOF to critical member capacity ratio is illustrated. According to this figure this ratio decreases as the frame stories increased which implies that although increasing the frame stories leads to more structural safety against progressive collapse, the structural performance and robustness do not improve proportionally contrasting to the enhanced structural element capacity. This ratio, for the split-X braced frames investigated in this research, follows the Eq. (4) by which designers and researchers can calculate FOF based on the number of frame stories and the member capacity which may differ according to building codes as well as lateral load demand while designing.

$$FOF/C = -0.0432N^3 + 0.977N^2 - 7.6218N + 28.6820 \text{ and } R^2 = 1 \tag{4}$$

In which FOF is the failure overload factor, C is the bracing member capacity (KN) and N is number of frame stories. It should be noted that this ratio is multiplied by one million.

7. Conclusions

In this research the element loss analysis of concentrically braced frames was focused through various types of nonlinear analyses and considering structural performance criteria stated in FEMA 356 and UFC. That was to assess the potential and capacity of such structural system for occurrence of progressive collapse as well as determination of the failure overload factors and associated failure modes. Results showed that for the eight scenarios investigated in this research, the simulations predicted no collapse in preliminary analyses. Furthermore, it was showed that totally all kind of FOFs increased as the frame height increased. Also it was observed that the most difference between PDOF and VIDOF was 22.57%. Such value for PDOF and PBOF was 22.00% which can make it possible to save the analysis time and effort for element loss analysis of the investigated structures by performing PDA and calculating FOF by such difference values. In addition, it was revealed that FEMA 356 and its acceptance criteria which are generally used in seismic rehabilitation of buildings can accurately determine the FOFs and failure modes in the studied split –X braced frames while losing one of their columns and connected brace in the first story. In addition, such structures had the ability to successfully absorb the loss of predefined elements at least against the loads 2.66 times greater than its nominal applied loads before occurrence of progressive collapse. Furthermore, comparing the FOFs, it was concluded that among the studied frames, where there is high risk of element loss, the eight-story frame have the most safety as it had the biggest FOF. Finally, introducing the trend of FOF and the FOF to critical member capacity ratio of the studied structures as a function of number of frame stories, it was observed that this ratio decreased as the frame stories increased. It should be noted that the conclusion obtained from the present research are dependant to the symmetric mode of elements removing as well as the imperfection value considered in the numerical models. Although the potential of progressive collapse occurrence decreases as the number of frame elements increases, for further studies on progressive collapse behavior of the structures, there is a need to investigate different types of structural systems such as V and inverted-V braced frames, different types of moment resisting frames and eccentrically braced frames in order to make preference choosing one of them when there is a high risk of sudden loss of structural elements.

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