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Effect of seismic design level on safety against progressive collapse of concentrically braced frames

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Abstract. In this research the effect of seismic design level as a practical approach for progressive collapse mitigation and reaching desired structural safety against it in seismically designed concentric braced frame buildings was investigated. It was achieved by performing preliminary and advanced progressive collapse analysis of several split-X braced frame buildings, designed for each seismic zone according to UBC 97 and by applying various Seismic Load Factors (SLFs). The outer frames of such structures were studied for collapse progression while losing one column and connected brace in the first story. Preliminary analysis results showed the necessity of performing advanced element loss analysis, consisting of Vertical Incremental Dynamic Analysis (VIDA) and Performance-Based Analysis (PBA), in order to compute the progressive collapse safety of the structures while increasing SLF for each seismic zone. In addition, by sensitivity analysis it became possible to introduce the equation of structural safety against progressive collapse for concentrically braced frames as a function of SLF for each seismic zone. Finally, the equation of progressive collapse safety as a function of bracing member capacity was presented.

Keywords: progressive collapse; structural safety; failure; performance; seismic load factor; vertical incremental dynamic analysis

1. Introduction

Progressive collapse refers to the action resulting from the failure of one structural element and leads to the failure of further similar elements which may result in entire structural collapse in some cases (ASCE 7-05 2005). The main characteristics of such an event 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 forces carried by these elements in the remaining structure; impact of separated and falling structural components on the remaining structure; 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). Vehicular collision, aircraft impact and gas explosions are some examples of the potential hazards and abnormal loads which can produce such an event (NIST 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 building codes have only general recommendations for mitigating the effect of progressive collapse in structures that are overloaded beyond their design loads. The American Society of Civil Engineering (ASCE) 7-05 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 (U.S. GSA 2003) and Unified Facility Criteria (UFC 2009). The GSA guidelines have provided a methodology to diminish progressive collapse potential in structures based on the 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.

Progressive collapse analysis of steel frames has recently been the subject of several studies. 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. Kim *et al*. (2009) studied the progressive collapse-resisting capacity of steel moment frames by using alternate path methods 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 can be larger than two; 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 (2011) deduced that among different types of braced frames, the inverted-V type braced frame shows superior ductile behavior during progressive collapse. Pujol and Smith-Pardo (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. 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. Asgarian and Rezvani (2012) studied the influence of number of braced bays on mitigating progressive collapse of concentrically braced frames and concluded that a frame with two braced bays had more robustness, at least to the rate of 17.21% comparing to a frame with three braced bays.

Though progressive collapse of different types of structural frames is primarily considered as vertical motion, researchers have suggested that seismic design of buildings leads to mitigating such an event (Khandelwal *et al*. 2009). However, studying present scientific resources, it can be said that the quantitative influence of such application was not fully focused. On the other hand, looking through steel structures designed and constructed, it is obvious that a great percentage of them use concentric bracing as their lateral load resisting system. So, this research aims to investigate the effect of seismic design level of the structures on mitigating progressive collapse and reaching desired safety levels against it in seismically designed concentric braced frame buildings. Toward this aim, several steel concentrically braced frame buildings were designed for five seismic zones according to UBC (1997), and by applying various seismic load factors. Their outer frames were studied for collapse progression as well as determining their safety levels against it by computing Failure Overload Factor (FOF). In addition, by sensitivity analysis it became possible to select a specific Seismic Load Factor (SLF) in order to reach a desired safety level without performing element loss analysis for the cases in which there was a high risk of progressive collapse.

2. Investigated structures

To investigate the effect of seismic design level on mitigating progressive collapse and reaching desired safety level against it, fifteen 4-story concentrically braced frame buildings were designed for five seismic zones and the soil category of S_c according to UBC 97 and based on

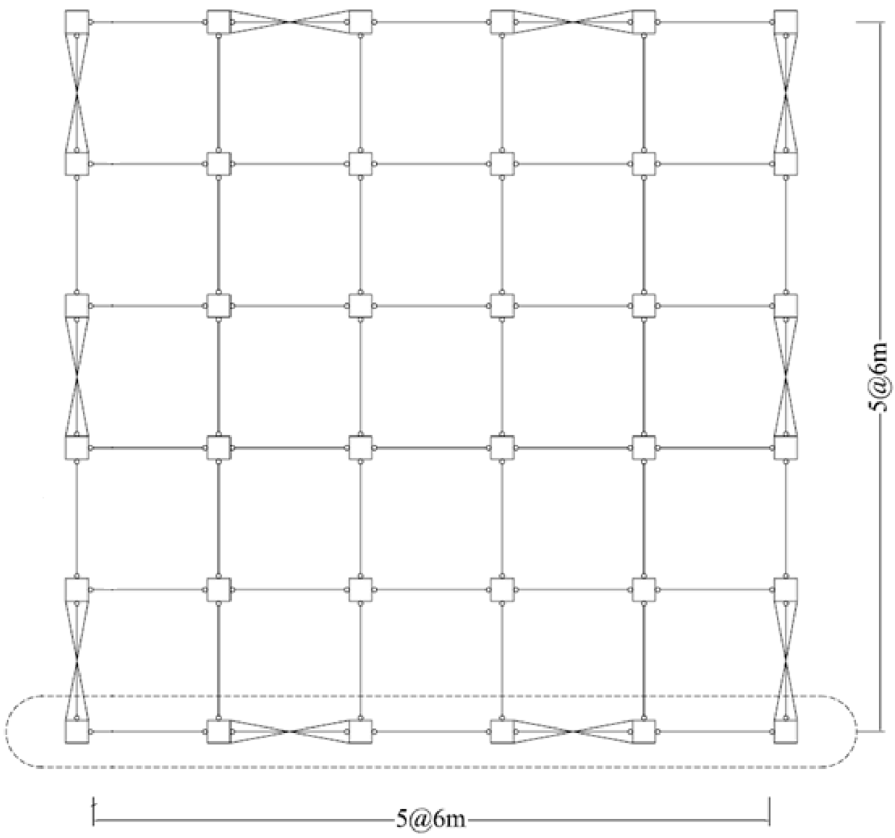


Fig. 1 Plan view of buildings

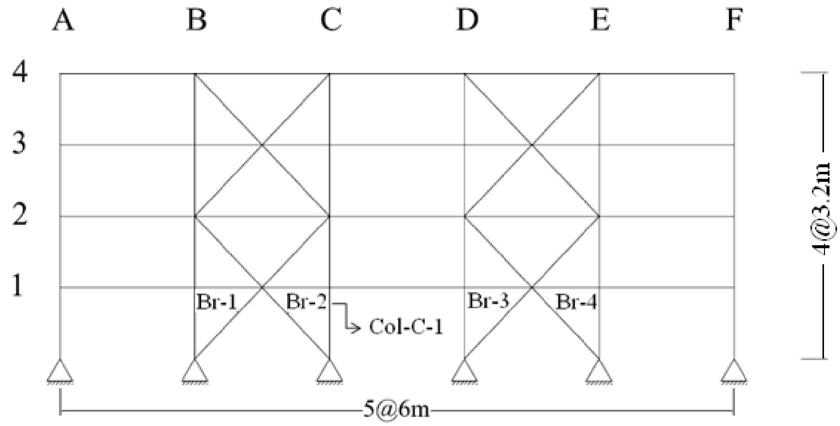


Fig. 2 Elevation view of building frames

different SLFs. The buildings were square in plan and consisted of 5 bays of 6 m in each direction and the story height of 3.2 m. The plan and elevation view of the frames are shown in Figs. 1 and 2. In the design process, gravity loads were supposed to be similar to common residential buildings. For member design subjected to earthquake, equivalent lateral static forces were applied on all the story levels. The dead and live loads of 6.5 and 2 kN/m^2, respectively, were used as gravity loads for all stories except the roof, where the gravity loads consisted of 6 and 1.5 kN/m^2 for the dead and live loads, respectively (INBCSD 2006).

3. Modelling of the structures

OpenSees (Mazzoni *et al*. 2007) finite element program was used to model and analyze the structures subjected to structural member loss. A series of nonlinear dynamic analyses were performed for the external frames of the designed buildings which are shown in Fig. 1 (with dotted lines) and Fig. 2. To model the steel behavior, a bilinear kinematic stress-strain curve was assigned to the structural elements using Steel02 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 change 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 the inelastic range of deformation. This behavior together with the structural steel properties is shown in Fig. 3. For the beams, columns and braces, beam-column elements in combination with fiber cross sections were used to model the cross sectional areas 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 frame members, i.e. beams, columns (at foundation level) and braces, were considered as pin-ended with the gravity loads sustained mainly by the columns. An initial mid span imperfection of L/1000 was applied to all braces and columns as depicted in Fig. 4.

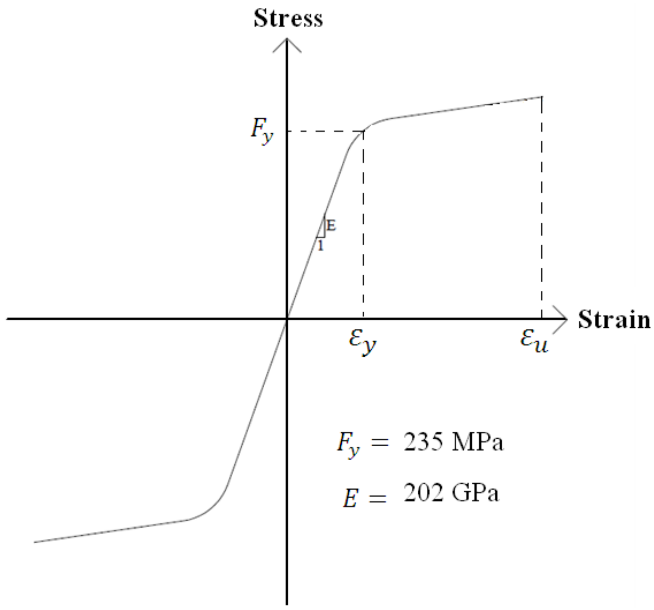


Fig. 3 Structural steel behavior

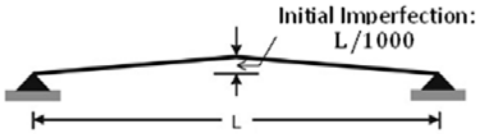
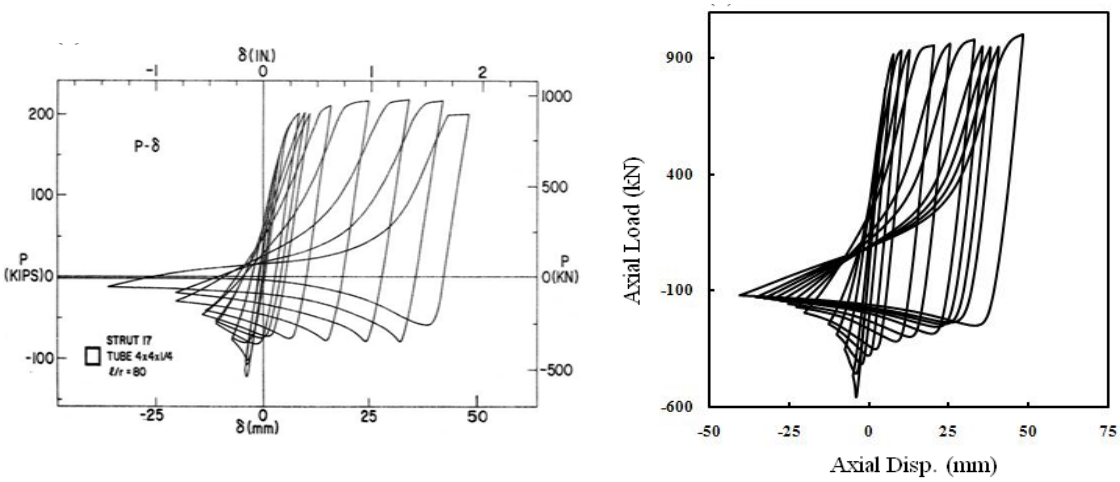


Fig. 4 Initial imperfection

3.1 Model verification

Though several verification exercises of the developed model and its structural elements can be found in the researches conducted by Asgarian and Rezvani (2012) and (Asgarian and Shokrogozar 2008, Asgarian *et al*. 2010), 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.

As another measure to verify the behavior of the employed structural elements, according to Table 1, the analytical buckling loads of 3 bracing members were compared with their expected bucking loads calculated in accordance with UBC 97. 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.



(a) (b)

Fig. 5 Initial imperfection

Table 1 Analytical and UBC 97 buckling loads

Section	Slenderness ratio	Analytical buckling load (kN)	UBC 97 buckling load (kN)
B250 × 250 × 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

*B: Box section in mm

Table 2 Cross section for all members (SLF =1) (B: Box Section in mm)

Seismic zone	SLF	Story	Columns			Brace
			A and F axes	B and E axes	C and D axes	
0	1	3,4	B100 × 100 × 10	B100 × 100 × 10	B100 × 100 × 10	B100 × 100 × 10
		1,2	B125 × 125 × 10	B150 × 150 × 12	B150 × 150 × 12	B100 × 100 × 10
1	1	3,4	B100 × 100 × 10	B125 × 125 × 10	B125 × 125 × 10	B125 × 125 × 10
		1,2	B125 × 125 × 12	B150 × 150 × 15	B150 × 150 × 15	B150 × 150 × 10
2	1	3,4	B125 × 125 × 10	B125 × 125 × 12	B125 × 125 × 12	B125 × 125 × 12
		1,2	B150 × 150 × 12	B150 × 150 × 15	B150 × 150 × 15	B150 × 150 × 12
3	1	3,4	B125 × 125 × 12	B150 × 150 × 12	B150 × 150 × 12	B150 × 150 × 12
		1,2	B175 × 175 × 15	B200 × 200 × 20	B200 × 200 × 20	B175 × 175 × 15
4	1	3,4	B150 × 150 × 10	B150 × 150 × 15	B150 × 150 × 15	B150 × 150 × 15
		1,2	B200 × 200 × 15	B225 × 225 × 20	B225 × 225 × 20	B175 × 175 × 15

*SLF: Seismic Load Factor

4. Preliminary element loss analysis

In order to investigate the structural behavior of concentrically braced frames when structural members are lost, one column and related brace were selected to be omitted suddenly in the first story. On this basis, at first, 5 split-X braced frames were designed in accordance with the requirements of seismic zones 0 to 4 (UBC 1997) for preliminary analysis. The sections selected for the mentioned frames are listed in Table 2. To simplify the discussion, the columns and braces located in the first story of the investigated frames are portrayed and coded according to Fig. 2. Table 3 presents the list of scenarios considered in this analysis together with the members that were removed.

4.1 Analysis procedure

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

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

In this analysis, the gravity loads were linearly increased during 5 seconds to reach their final values, and after that, they were kept unchanged for 2 seconds to avoid exciting dynamic effects. Once the gravity loads have been fully applied at the end of the seventh second, the elements related to the specific scenario were removed suddenly, and afterwards the subsequent response of the braced frame was investigated. Sudden removal of predefined elements could be performed during the dynamic analysis through which new stiffness matrix was formed and the analysis continued. The simulations were conducted with 5 % mass and stiffness proportional damping. In this analysis, for each removal scenario, primarily, the response of the structure while removing its elements was investigated by performing a nonlinear dynamic analysis, and secondly, the peak value of each structural element effort was checked against its nominal capacity. If the peak value exceeded the capacity calculated from the code (UBC 1997), 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 removal but solely according to the imposed loads.

Table 3 APM analysis cases (scenarios)

APM case/scenario	Seismic zone	Elements removed
1	0	Col-B-1 and Br-1
2	1	Col-B-1 and Br-1
3	2	Col-B-1 and Br-1
4	3	Col-B-1 and Br-1
5	4	Col-B-1 and Br-1

4.2 Analysis results

In Figs. 6 to 10, time history responses of critical columns and braces' axial forces for predefined scenarios are illustrated. In Table 4, the summary of preliminary analysis of the investigated structures is shown in accordance with the parameters described in Fig. 11. According to this table, in the first scenario (seismic zone 0) the axial force of Col-C-1 spiked from 663.28 kN to a peak value of 1530.64 kN which is more than its nominal capacity (1414.49 kN) by

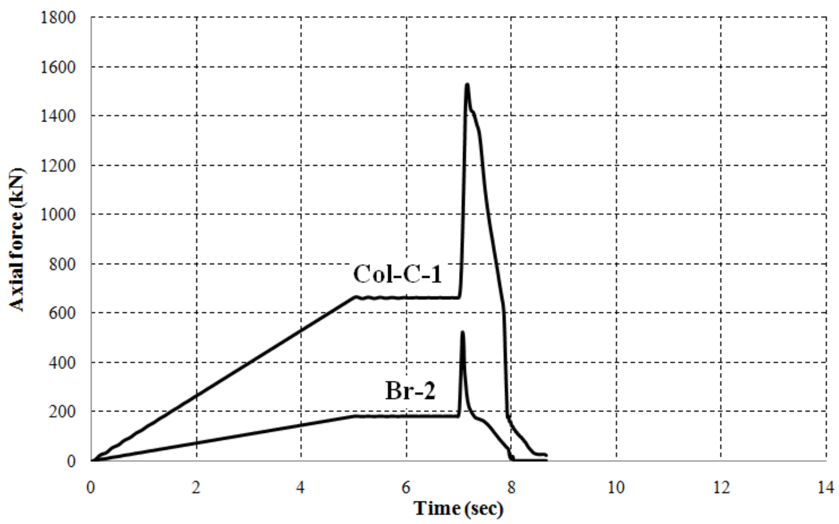


Fig. 6 Time history of critical structural members – Seismic zone 0 – SLF = 1

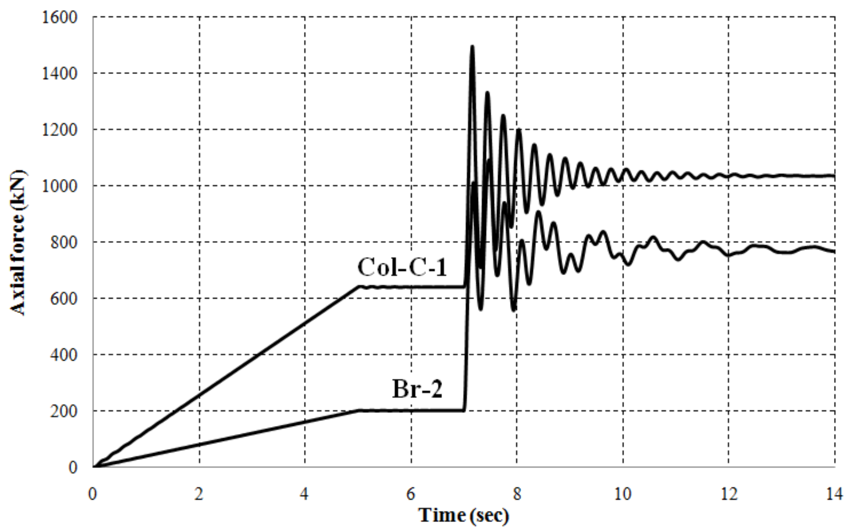


Fig. 7 Time history of critical structural members – Seismic zone 1 – SLF = 1

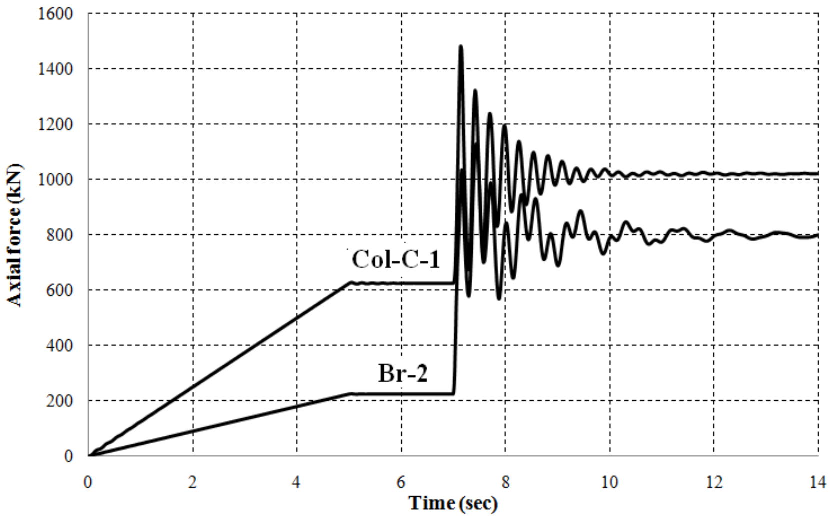


Fig. 8 Time history of critical structural members – Seismic zone 2 – SLF = 1

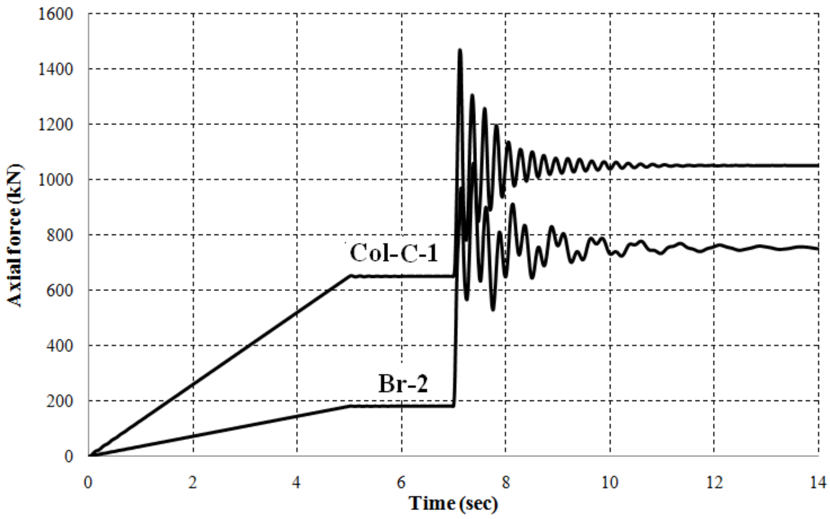


Fig. 9 Time history of critical structural members – Seismic zone 3 – SLF = 1

assuming an effective length factor of, $K = 1.0$, and when combined with the moment generated on the column, implies that the column was overloaded and resulted in instability of the frame as well as collapse progression. However, for other seismic zones the simulation results demonstrated that the system was able to successfully absorb the loss of structural members predefined in Table 3. For such cases a large distribution of forces was observed to take place. For example, for the same element, in the second scenario, the axial force spiked from 641.99 kN to a peak value of 1498.71 kN before settling down at a steady value of 1037.59 kN, which is less than its nominal capacity (1722.61 kN), and while combined with the relatively small moment generated on the columns,

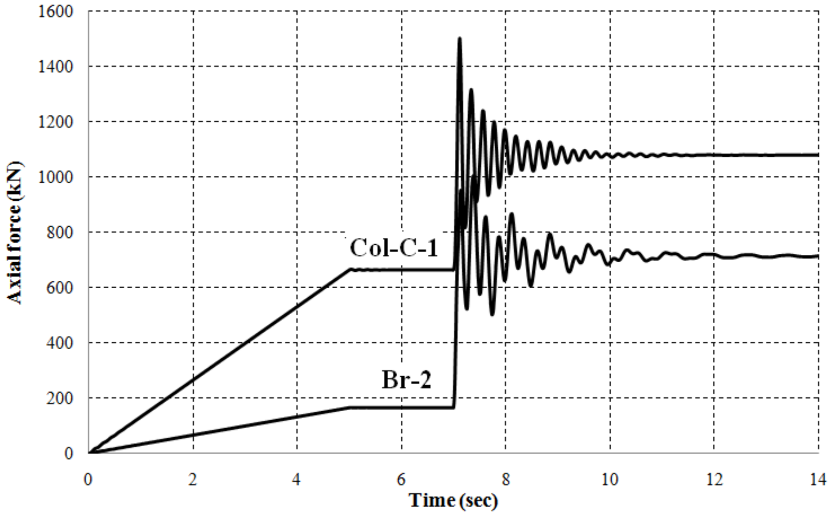


Fig. 10 Time history of critical structural members – Seismic zone 4 – SLF = 1

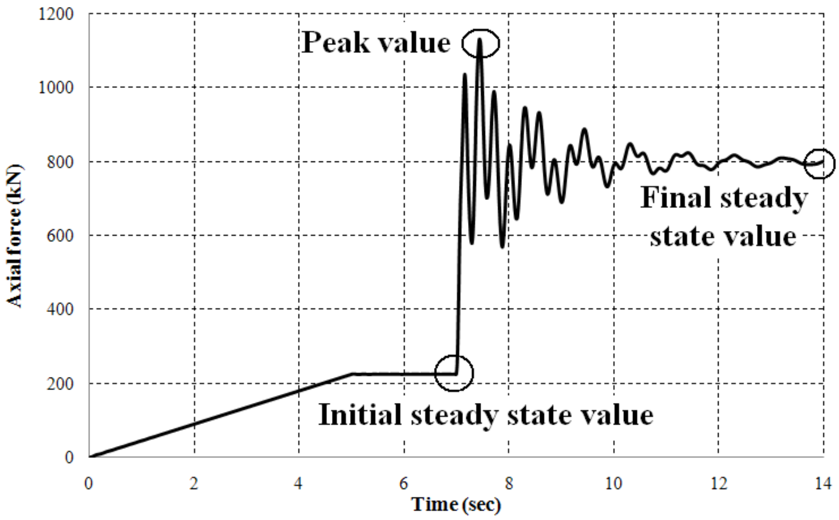


Fig. 11 Description of parameters introduced in Table 3

implies that the column was not overloaded. There was the same story for braces, since for seismic zone 0, the axial force of Br-2 spiked from 180.81 kN to a peak value of 521.50 kN and in the second scenario the axial force of the same brace spiked from 201.85 kN to a peak value of 1095.07 kN before settling down at a steady value of 775.01 kN. By assuming an effective length factor, K = 1.0 the capacities of those braces were 449.96 kN and 1093.07 kN, respectively. This implied that although there was no further failure in the split-X braced frame designed for seismic zone 1, there was a high risk of progressive collapse in it. So, in order to monitor the safety border against failure progression, in the last column of Table 4 the peak value to capacity ratios of the

Table 4 Summery of time history analysis (force values unit: kN)

Structural element	APM case	Initial steady state value	Peak value	Final steady state value	Peak value/member capacity
Col-C-1	1	663.28	1530.64	-	0.92
	2	641.99	1498.71	1037.59	1.15
	3	624.25	1480.92	1019.99	1.16
	4	651.52	1470.59	1052.59	2.18
	5	665.44	1504.54	1081.41	2.46
Br-2	1	180.81	521.50	-	0.96
	2	201.85	1095.07	775.01	1.00
	3	224.45	1130.67	799.92	1.14
	4	181.30	1060.52	754.45	1.85
	5	165.45	1005.43	714.68	1.95

critical members were illustrated. According to these ratios, the structure designed for seismic zone 0 collapsed progressively after the loss of predefined structural members in the first story, while in the second scenario the peak value of the critical brace's axial force was almost equal to its capacity, which demonstrated a high potential for collapse progression. For other structures, designed for seismic zones 2 to 4, simulations predicted no collapse progression after elements removal, although peak value to member capacity ratios were below 1.5 and 2.0 for third and fourth/fifth scenarios, respectively.

This situation occurred because the buildings were designed to support the seismically induced forces and the extra capacity in which the columns connected to braces were to bear the magnified earthquake forces in seismic zones 3 and 4. 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 removal 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.

5. Effect of seismic design level on structural safety against progressive collapse

In this study, failure progression was evaluated according to the preliminary analysis performed in the previous section by determining whether internal forces of each structural element exceeded its nominal capacity after removal. This method did not consider determination of the residual capacity of the structure after such removals or failure overload factors. In addition, although the investigated structures designed for seismic zones 2 to 4 did not show a high risk of collapse progression, there might be an extra requirement for some critical buildings for which adequate structural safety against progressive collapse should be provided. On these bases, to reach desired safety level, the effect of seismic design level of the structures on structural safety against progressive collapse of concentrically braced frames was studied in this research. This could be an indirect method for mitigating the occurrence of progressive collapse. Such investigation was achieved through advanced element loss analysis, consisted of both nonlinear force-controlled and displacement-controlled actions by which it became possible to evaluate the structural robustness and performance by comparing the FOFs. The force-controlled action was done through Vertical Incremental Dynamic Analyses (VIDA), in order to estimate the residual capacity of a damaged structure and determine the probable failure modes for the investigated cases. On the other hand, the displacement-controlled action was performed in accordance with FEMA 356 (2000), by which the structure was loaded incrementally until the limit states stated in the standard occurred. To study the effect of seismic design level on structural safety against progressive collapse, 2 extra buildings were designed for each seismic zone by applying the SLFs of 2 and 3 which doubled and tripled the seismic load demand of the structures in the design process. The sections selected for the mentioned structures are listed in Table 5.

5.1 Vertical Incremental Dynamic Analysis (VIDA)

Regardless of collapse or failure progression in preliminary analyses, for each designed frame,

Table 5 Cross section for all members (SLF = 2 and 3) (B: Box section in mm)

Seismic Zone	SLF	Story	Columns			Brace
			A and F axes	B and E axes	C and D axes	
0	2	3,4	B100 × 100 × 10	B100 × 100 × 10	B100 × 100 × 10	B100 × 100 × 10
		1,2	B125 × 125 × 12	B150 × 150 × 15	B150 × 150 × 15	B125 × 125 × 10
	3	3,4	B125 × 125 × 10	B125 × 125 × 10	B125 × 125 × 10	B125 × 125 × 10
		1,2	B150 × 150 × 12	B175 × 175 × 15	B175 × 175 × 15	B150 × 150 × 12
1	2	3,4	B150 × 150 × 10	B150 × 150 × 15	B150 × 150 × 15	B150 × 150 × 15
		1,2	B150 × 150 × 15	B200 × 200 × 15	B200 × 200 × 15	B175 × 175 × 15
	3	3,4	B150 × 150 × 12	B175 × 175 × 15	B175 × 175 × 15	B175 × 175 × 15
		1,2	B200 × 200 × 15	B200 × 200 × 20	B200 × 200 × 20	B200 × 200 × 20
2	2	3,4	B150 × 150 × 15	B150 × 150 × 12	B175 × 175 × 15	B175 × 175 × 15
		1,2	B175 × 175 × 15	B200 × 200 × 20	B200 × 200 × 20	B200 × 200 × 20
	3	3,4	B175 × 175 × 15	B200 × 200 × 15	B200 × 200 × 15	B200 × 200 × 15
		1,2	B200 × 200 × 20	B250 × 250 × 20	B250 × 250 × 20	B225 × 225 × 20
3	2	3,4	B150 × 150 × 15	B175 × 175 × 15	B175 × 175 × 15	B175 × 175 × 15
		1,2	B225 × 225 × 20	B300 × 300 × 20	B300 × 300 × 20	B200 × 200 × 20
	3	3,4	B175 × 175 × 15	B200 × 200 × 20	B200 × 200 × 20	B200 × 200 × 20
		1,2	B300 × 300 × 20	B350 × 250 × 25	B350 × 250 × 25	B200 × 200 × 20
4	2	3,4	B175 × 175 × 15	B200 × 200 × 15	B200 × 200 × 15	B200 × 200 × 15
		1,2	B275 × 175 × 20	B325 × 325 × 25	B325 × 325 × 25	B225 × 225 × 20
	3	3,4	B200 × 200 × 15	B225 × 225 × 20	B225 × 225 × 20	B225 × 225 × 20
		1,2	B300 × 300 × 25	B375 × 375 × 30	B375 × 375 × 30	B275 × 175 × 20

*SLF: Seismic Load Factor

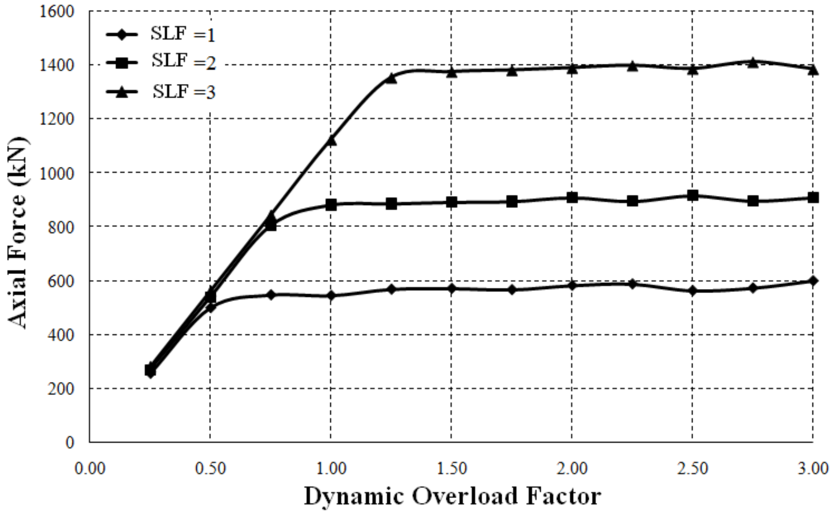


Fig. 12 VIDA curves of Br-2, Seismic zone 0

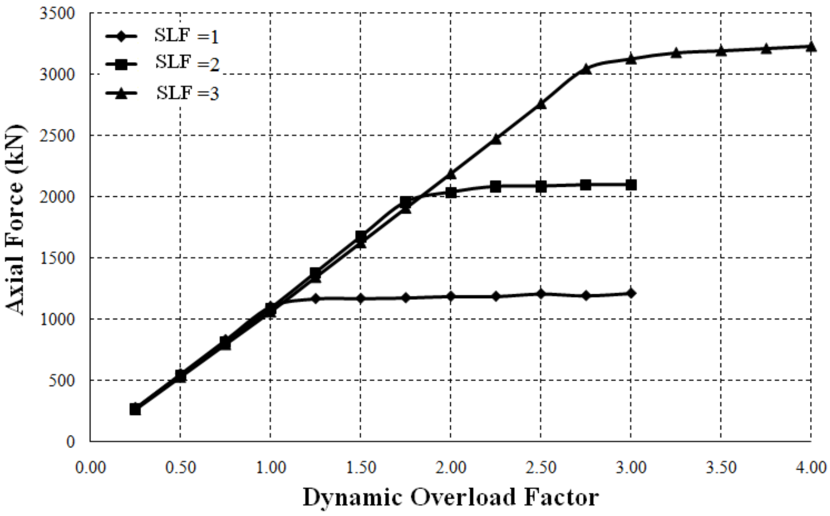


Fig. 13 VIDA curves of Br-2, Seismic zone 1

VIDA was performed. This analysis was similar to the 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 and Cornell 2002). In Figs. 12 to 16, the VIDA curves of the axial force of critical braces are illustrated as a function of dynamic overload factors. According to these curves, the Vertical Incremental Dynamic Overload Factors (VIDOFs) as the forced-controlled FOF were 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 (scenario).

5.2 Performance-based analysis (*PBA*)

To compare the force-controlled actions performed in the previous section with the displacement-controlled action, as another approach of element loss analysis, PBA of such

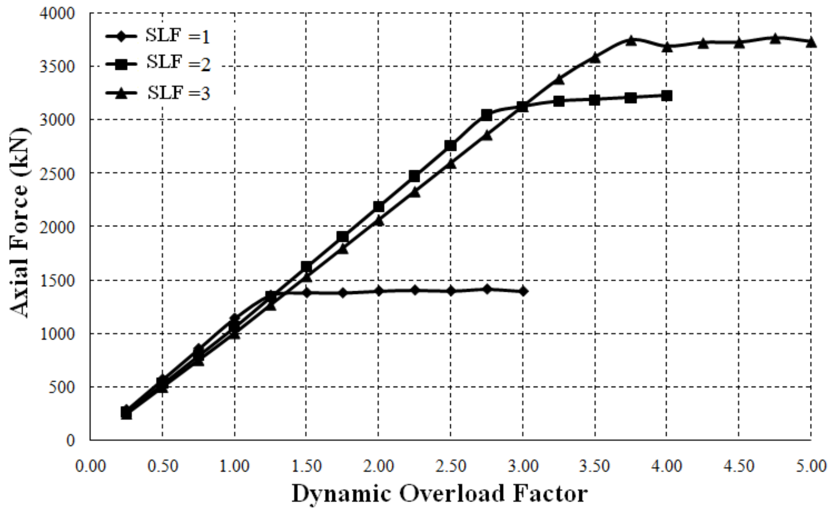


Fig. 14 VIDA curves of Br-2, Seismic zone 2

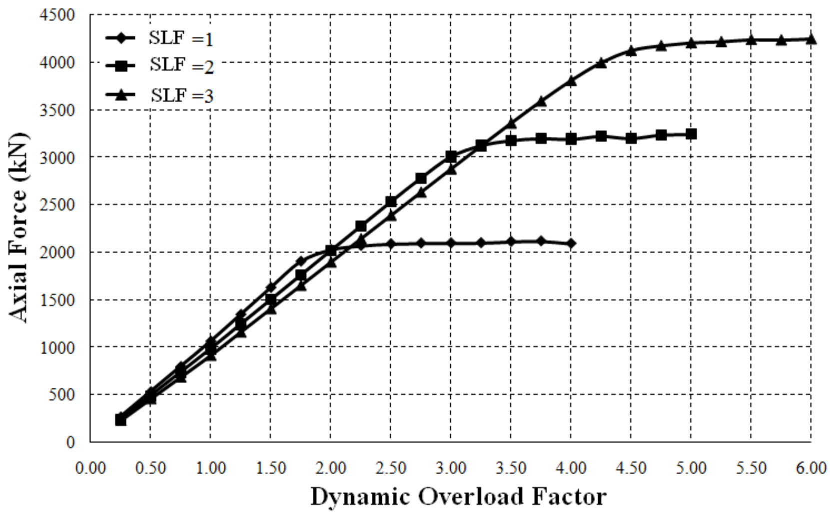


Fig. 15 VIDA curves of Br-2, Seismic zone 3

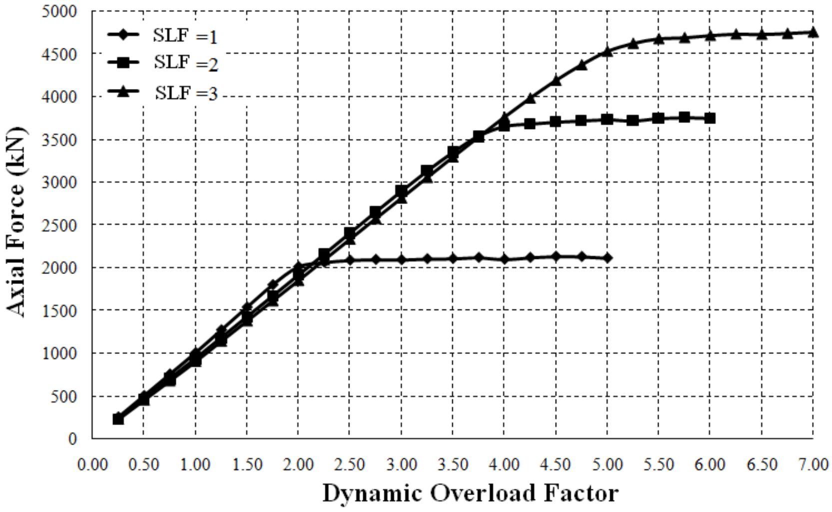


Fig. 16 VIDA curves of Br-2, Seismic zone 4

Table 6 Performance-based analysis results

Seismic Zone	SLF	Failure mode	Limit state	PBOF	Disp. (cm)
0	1	Br-2	IO	0.13	0.08041
			LS	0.60	1.51283
			CP	0.80	2.12345
	2	Br-2	IO	0.19	0.09716
			LS	0.85	1.91763
			CP	0.91	2.69668
	3	Br-2	IO	0.29	0.10726
			LS	1.31	2.11514
			CP	1.37	2.97206
1	1	Br-2	IO	0.25	0.10808
			LS	1.14	2.13551
			CP	1.19	2.98613
	2	Br-2	IO	0.50	0.12380
			LS	2.01	2.27987
			CP	2.07	3.21864
	3	Br-2	IO	0.75	0.12074
			LS	3.06	2.42187
			CP	3.10	3.25573
2	1	Br-2	IO	0.27	0.10601
			LS	1.28	2.13753
			CP	1.33	3.04041

Table 6 Continued

2	2	Br-2	IO	0.69	0.11611
			LS	3.06	2.44744
			CP	3.10	3.27947
	3	Br-2	IO	1.00	0.13386
			LS	3.82	2.42599
			CP	3.91	3.33043
3	1	Br-2	IO	0.50	0.12077
			LS	2.09	2.32466
			CP	2.19	3.14883
	2	Br-2	IO	0.76	0.11602
			LS	3.39	2.31594
			CP	3.51	3.22413
	3	Br-2	IO	1.25	0.13739
			LS	4.92	2.41409
			CP	5.09	3.42039
4	1	Br-2	IO	0.50	0.11412
			LS	2.19	2.23692
			CP	2.37	3.15108
	2	Br-2	IO	1.00	0.12475
			LS	4.17	2.37149
			CP	4.32	3.33598
	3	Br-2	IO	1.25	0.12169
			LS	5.69	2.48405
			CP	5.86	3.44733

*SLF: Seismic Load Factor
*PBOF: Performance-Based Overload Factor

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. Tables 5-6 and 5-7 of FEMA 356 were used to model the parameters and acceptance criteria for the nonlinear dynamic procedures. Since the distributed plasticity beam column element a was selected for structural modeling, every point along each element's length and across its cross section had the potential to enter the plastic region.

To compute 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 nd P_{CL} is the lower-band axial strength of a column) were considered force-controlled, which resulted in excluding some columns from the displacement-controlled actions in higher dynamic overload factors. For the braces, the axial deformation at the expected buckling load was the basis for the determination of the limit

Table 7 Failure overload factors

Seismic zone	SLF	Failure mode	FOF (Structural Safety Factor)		Diff. (%)	Enhance. (%)
			VIDOF	PBOF		
0	1	Br-2	0.65	0.60	7.69%	
	2	Br-2	1.00	0.85	15.00%	41.67%
	3	Br-2	1.35	1.31	2.96%	118.33%
1	1	Br-2	1.20	1.14	5.00%	
	2	Br-2	2.25	2.01	10.67%	76.32%
	3	Br-2	3.25	3.06	5.85%	168.42%
2	1	Br-2	1.25	1.28	2.40%	
	2	Br-2	3.25	3.06	5.85%	144.80%
	3	Br-2	3.75	3.82	1.87%	200.00%
3	1	Br-2	2.25	2.09	7.11%	
	2	Br-2	3.75	3.39	9.60%	62.20%
	3	Br-2	5.00	4.92	1.60%	135.41%
4	1	Br-2	2.50	2.19	12.40%	
	2	Br-2	4.50	4.17	7.33%	90.41%
	3	Br-2	6.00	5.69	5.17%	159.82%

*SLF: Seismic Load Factor
*VIDOF: Vertical Incremental Dynamic Overload Factor
*PBOF: Performance-Based Overload Factor
*Diff: difference of VIDOF and PBOF
*Enhance: enhancement of FOF (safety factor) by applying higher SLF

states. In this analysis, for each step of increasing the applied load, the plastic rotations and the acceptance criteria of the beams and columns were updated as a function of the yield rotation. By comparing the biggest plastic rotation of beam/ column elements and axial displacement of braces with those regulated on FEMA 356 in each step, hinges' status were determined.

Performing multiple analyses, the Performance-Based Overload Factors (PBOF) related to FEMA 356 limit states were computed. In Table 6, 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. UFC (2009) states that the limit state for structural elements in compression is LS; accordingly, this kind of limit state was the basis to determine the structural safety against progressive collapse of the investigated frames. According to Table 6, it can be inferred that the acceptance criteria in plastic rotations are not exceeded for the columns of the investigated frames, though encountering significant axial forces. The situation occurred in some cases 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 4-story frame designed for seismic zone 1 and the SLFs of 1 and 3 while losing the predefined structural elements considering various overload factors. 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.

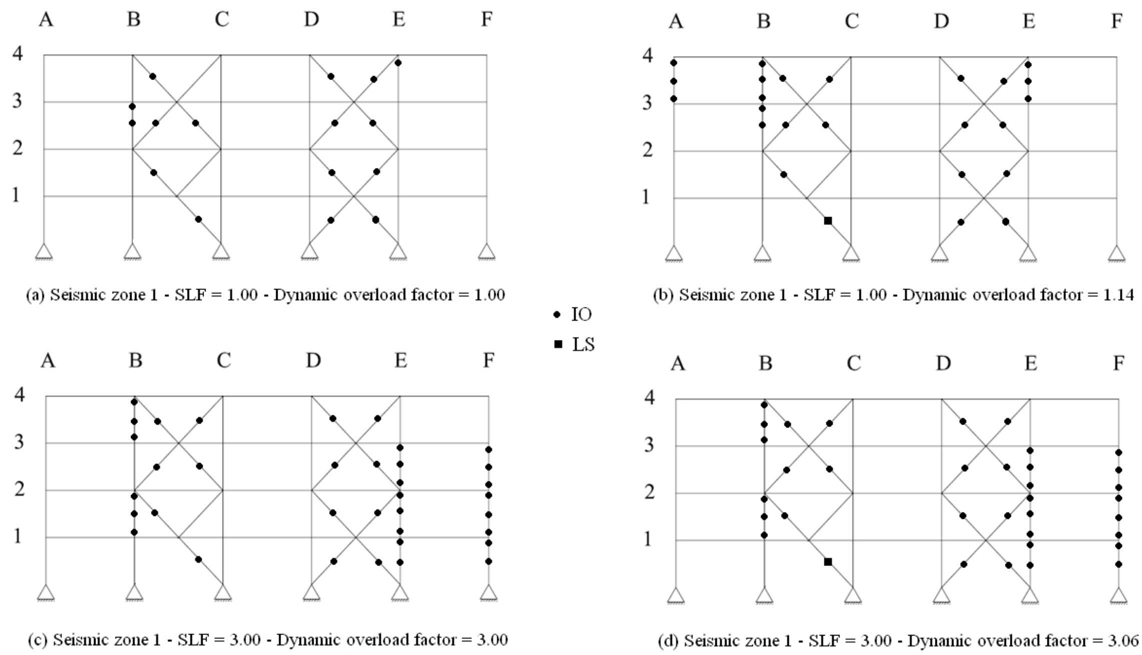


Fig. 17 Development of plastic hinges together with the monitored performance level

5.3 Structural safety against progressive collapse

In Table 7, VIDOFs and PBOFs are listed for each scenario according to different seismic zones and SLFs. The minimum of such FOFs in each scenario was selected as the structural safety factor against progressive collapse. According to this table, it can be inferred that except the first scenario in which the split-X braced frame designed for seismic zone 0 collapsed after the loss of one column and related brace in the first story, the minimum FOF for other cases designed for a SLF of 1 was 1.14, which implied that such a structural system can survive the loss of predefined elements against the loads 1.14 times greater than the UFC recommended load. Besides, it was shown that the collapse was initiated by buckling of the braces for each scenario which implied that the columns of such structures had adequate strength to survive the loss of one column and connected brace in the first story before initiation of the next failure. In seismic zones 3 and 4, this situation occurred because the buildings were designed to support the seismically induced forces and the extra capacity in which the columns connected to braces are to bear the magnified earthquake forces. Furthermore, it was observed that all kinds of FOFs increased as the SLF increased, which implied that the structural safety against progressive collapse increased. According to this table, it can be deduced that by applying the SLFs of 2 and 3, the structural safety against progressive collapse increased by 83.08% and 156.40%, respectively on average. In addition, it can be seen that structures designed according to the requirements of seismic zones 0 to 2 would be vulnerable to progressive collapse if one of their columns and connected brace were removed suddenly in the first story since they had a safety factor less than 1.50. So it would be effective to apply a higher SLF in design process as an indirect approach for mitigation progressive collapse. Besides, for seismic zone 0, as the most critical case, it was observed that the

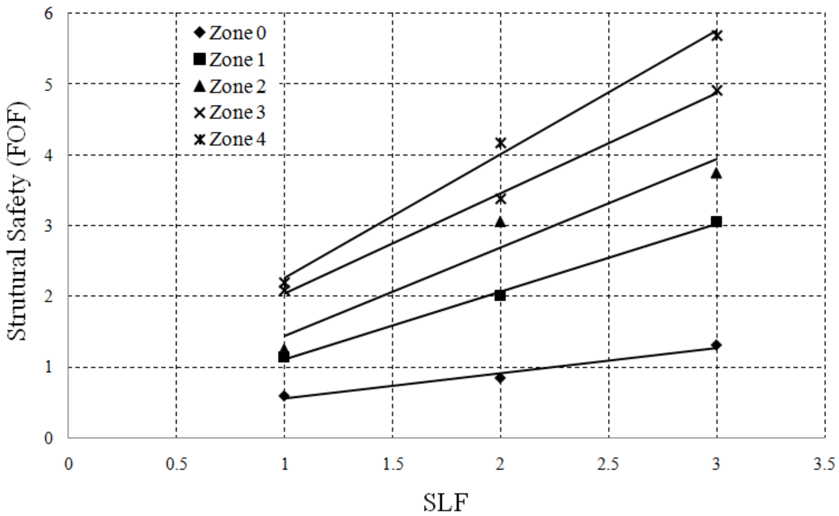


Fig. 18 Structural safety against progressive collapse

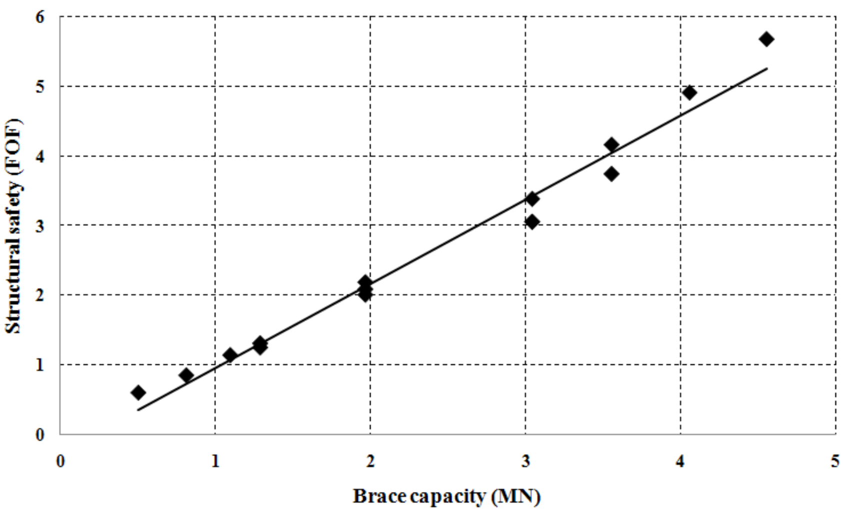


Fig. 19 Relation of Structural safety and bracing member capacity

structural safety against failure progression did not increase as expected though the structure was redesigned by applying higher SLFs (2 and 3). In Fig. 18, structural safety curves for each seismic zone are depicted. Using these curves, it became possible to design protective structures against progressive collapse with desired safety levels without performing element loss analysis. In Table 8, the safety equations for various seismic zones are listed via which it is possible for designers and scholars to select a specific structural safety factor and design structures for a specific seismic zone by applying a specific SLF regardless of further progressive collapse analysis for the cases in which the structures may lose one of their columns and connected brace in the first story. For example, a designer of a 4-story split-X braced frame building located in seismic zone 0 and the

Table 8 Structural safety equations

Seismic zone	Structural safety (FOF)	R^2
1	0.35SLF + 0.21	0.97
2	0.96SLF + 0.15	0.99
3	1.25SLF + 0.19	0.94
4	14SLF + 0.64	0.99
5	1.75SLF + 0.52	0.99

*SLF: Seismic Load Factor

soil category of S_c can calculate the design earthquake load factor of 3.63 in order to attain the structural safety of 1.5 against progressive collapse. Such equations can be applicable for the taller buildings since there is at most 11.27% difference among the FOFs of a 4-story split-X braced frame building and 6, 8 and 10-story ones (Rezvani and Asgarian 2012). Furthermore, in Fig. 19, the relation of structural safety as a function of bracing member capacity is illustrated. This relation followed Eq. (2) in which S.F. is safety factor, equal to FOF and C is bracing member capacity in MN. Using this equation, designers and researchers could calculate the structural safety against progressive collapse based on bracing member capacity which might differ according to building codes as well as lateral load demand in design process.

$$\text{S.F.} = 1.21C - 0.25 \quad \text{and} \quad R^2 = 0.98 \tag{2}$$

6. Conclusions

In this research the effect of seismic design level on structural safety against progressive collapse of seismically designed concentric braced frame buildings was investigated. Toward this aim, 15 split-X braced frame buildings were designed for seismic zones according to UBC 97 and by applying various SLF. Their outer frames were studied for collapse progression while losing one column and connected brace in the first story. Preliminary analysis results showed that structures designed for seismic zone 0 collapsed progressively after such removal, while in other cases there was no failure progression. To quantify the effect of seismic design level as an indirect approach for mitigating progressive collapse, advanced element loss analysis, consisting of VIDA and PBA, was carried out. It was observed that except for the structure designed for seismic zone 0, structures could survive member removal against loads at least 1.14 times greater than the UFC recommended load. Besides, it was shown that the failure was initiated by buckling of braces in each scenario, which implied that columns of such structures had adequate strength to survive one column loss in the first story before the occurrence of next failure. It was shown that by applying the SLFs of 2 and 3, the structural safety against progressive collapse increased by 83.08% and 156.40%, respectively on average. In addition, by sensitivity analysis it became possible to draw and present equations of structural safety curves, which were equal to FOFs for each seismic zone, as a function of SLF. Using such equations, it became possible to design protective structures by choosing a specific structural safety factor and design structures by applying a specific SLF regardless of further progressive collapse analysis for the cases in which the investigated structures might lose one of their columns and connected brace in the first story. Finally, for the differences in calculating the structural member capacity as well as the lateral load demand in design process, the equation of structural safety as a function of bracing member capacity was introduced.

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