Evaluation of Progressive Collapse in Steel Moment Frame with Different Braces

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ABSTRACT

Progressive collapse is a phenomenon that can happen as a consequence of natural and artificial dangers. In progressive collapse mechanism, a local damage (such as removing a column) leads to a comprehensive and significant damage, which results in structure collapse. Research studies on progressive collapse of structures usually focus on gravitational and explosive loadings, in which the design goal is to increase structure robustness in order to prevent progressive collapse. During an earthquake, redistribution of the load borne by structural damaged members to the adjacent ones might lead to excessive tension or exceed resistant capacity of other members' load, which is the result of damage expansion and dispersion. In order to study progressive collapse of structures during earthquakes, three steel moment frame buildings with CBF, EBF and BRB have been selected, which have similar plans along with 8 stories, while lacking some structural members. In the above mentioned buildings, some braces have been removed, so that the influence of such scenarios on the structure's dynamic behavior during the earthquake could be studied. In this research, the buildings' potential and capacity for seismic progressive collapse as well as their damage modes were determined *via* Incremental Dynamic Analysis (IDA). Therefore, analysis can find the most probable damage modes for improvement goals, which in turn lead to structures with higher reliability in seismic regions.

KEYWORDS: Progressive collapse, Concentrically braced frame (CBF), Eccentrically braced frame (EBF), Buckling restrained brace (BRB), Incremental dynamic analysis (IDA).

INTRODUCTION

Various structures are exposed to different dangers that might in some cases cause the absence of a member or element and in other cases total collapse. Potential dangers and abnormal loads, such as vehicle collision, airplane collision, gas explosion,… etc., may cause progressive collapse, which indicates an action, which results from a member's damage and leads to the damage of other similar elements (NIST, 2007).

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Usually, structures are not designed for abnormal events that might damage a structural element or lead to general destruction. Most building codes only have general recommendations to decrease the impact of progressive collapse in structures which have overload in design loads. American Society of Civil Engineers (ASCE) (2005) is the only main standard which deals with progressive collapse in detail. Guide books for resistant design against progressive collapse are significant in American government's documents (e.g., (UFC, 2009; US GSA, 2003)). GSA guide books have offered a methodology to decrease progressive collapse potentiality in structures based on Alternate Path

Method (APM), which defines scenarios where one of the building columns is removed and the damaged structure is analyzed for studying system responses. On the other hand, UFC methodology is a performance-based design method in accordance with GSA conditions.

Recently, numerous studies have dealt with element absence in steel frames. Kim et al. (2009) studied resistive capacity against progressive collapse of steel moment frames *via* APM, which is recommended in GSA and UFC. They saw that non-linear dynamic analysis leads to bigger structural responses. In addition, they witnessed that progressive collapse potentiality is at its highest rate when the corner column is suddenly removed. Furthermore, it was concluded that progressive collapse potentiality decreases as the number of floors increases. Khandellwal et al. (2009) concluded that a same-centered braced frame is less vulnerable against progressive collapse than a specially-braced one. Kim and Kim (2009) showed that dynamic enlargement could be bigger than 2, which is recommended by UFC and GSA. FU (2009) stated that under similar general conditions, removing a column at a higher level causes greater vertical movement than removing a column at ground level. Liu (2010) analyzed twist and curvature action, showing that this effect can significantly decrease bending moment by binding the beam axially. Furthermore, two methods have been suggested to improve and reinforce the beam-column connection of tall steel frame structures, which are exposed to terrorist explosions. England et al. (2008) studied the importance of vulnerability evaluation of a structure on unexpected events, also dealing with the nature of such events. Moreover, a structural vulnerability theory which studies the simple form for determining the most vulnerable stage of the damage has been explained. Pujol and Smith-Pardo (2009) proposed that a floor system can be designed to revive sudden removal of one of its bases by symmetrizing the system. To begin with, by using the results from a common linear static analysis, a model could be generated which prevents column removal and exceeding the coefficient of 1.5; secondly, by giving sufficient details to be certain that the system can achieve general resistance to deformations more than 1.5 times of the bigger limit of dependent deformations. Asgarian and Hashemi Rezvani (2012) studied the behavior of a steel braced frame without some structural elements, proposing the relation of the number of frame stories with their resistance against progressive collapse.

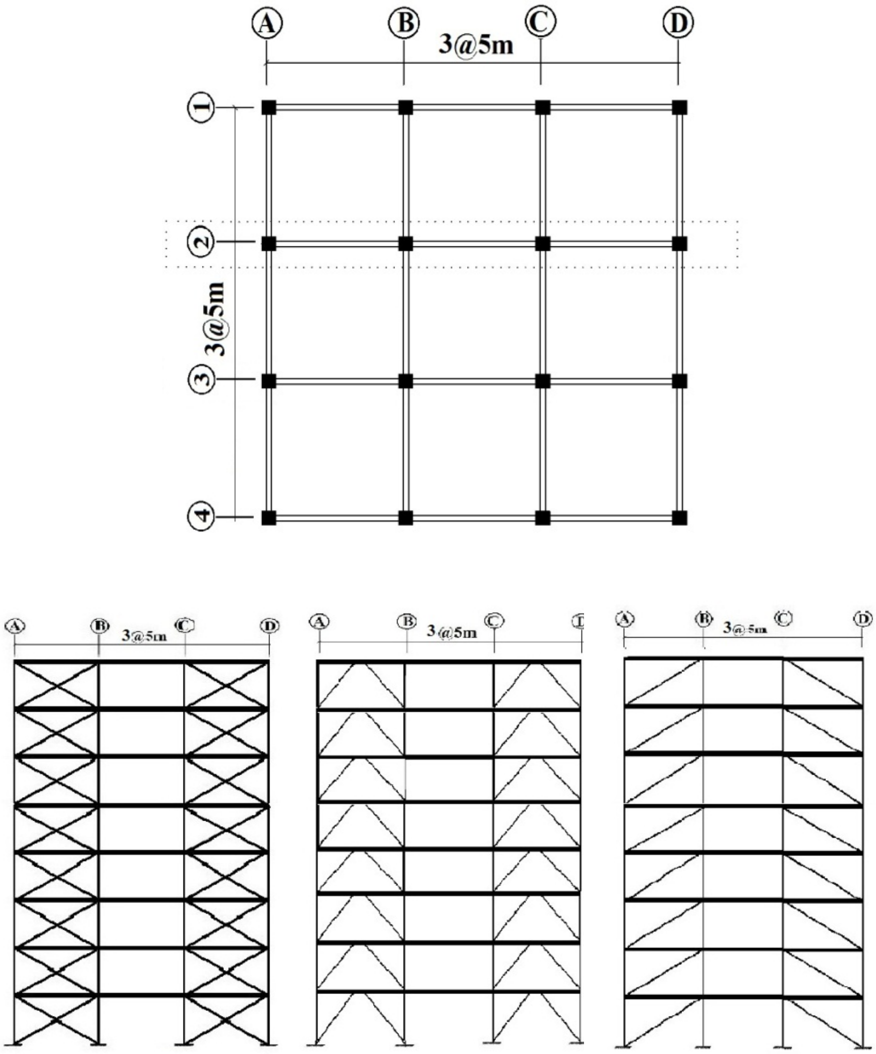
Studies on structures' progressive collapse usually focus on gravitational and explosive loading with the aim of increasing the structures' variety and robustness to prevent progressive collapse. By studying the existing scientific sources, one can conclude that structures' progressive collapse due to earthquake load has not attracted much attention. During seismic response, redistribution of the load borne by damaged structural members may increase the tension or resistive capacity of other members, which is a consequence of damage expansion and circulation. Using moment frames with CBF, EBF and BRB provides much space to architects to limit bracing systems. By surveying the technical literature, one can observe that these systems' behavior after element absence is not completely centralized and that there is no response for important issues such as the potentiality of seismic progressive collapse, the number of frame stories, determining potential damage modes and dependent loads. Based on what was aforementioned, this research aims at answering these questions for increasing structure safety of these frames. Accordingly, the considered frames are designed at first and then, they are studied based on different scenarios concerning the number of frame stories and structures' seismic behavior after the absence of the element. In order to study such behavior, IDA is used. Furthermore, in order to compare these results, the analysis is carried out with the results from control effects based on relocating structures' seismic performance in accordance with FEMA 356 (2000). By using these results, a good understanding is gained of seismic behavior of EBFs in column absence and the impact of structure height of such systems on structure safety against progressive danger is studied.

Studied Structures

In order to study the seismic behavior of braced steel frames after the absence of some braces, three buildings with CBF, EBF and BRB were selected, which had 8 stories and were designed on a site (Tehran, Iran) with very high seismicity. Secondary forces were used based on UBC (2007) for S_C categorization of the soil. Fig. 1 shows the story plan as well as the considered frames. The gravitational load includes both live and dead loads. The dead load of the stories was 550 $\frac{kg}{m^2}$, the live load was 200 $\frac{kg}{m^2}$ and the roof load was 150 $\frac{kg}{m^2}$ with other types of load, such as wind load or snow load, not taken into account. Moreover, structure-soil interaction was not taken into consideration and the columns' base was assumed to be in the floor. Story height was 3.2 m. Compressive strength of the concrete roof was 210 $\frac{kg}{cm^2}$ and the concrete slab thickness in the stories was 15 cm. Table 1 shows the column and beam sections used. Bracing members in all stories have similar cross-sections and material characteristics. These bracing members are considered to be square hollow sections with a width of 15cm and a thickness of 4mm; therefore, all of them have medium thinness with their effective length coefficient assumed to be one.

In BRBs, the nucleus should be designed in a way that it will give up in both compression and tension. In order to prevent general tension in compression, the nucleus is placed within a steel tube, with the space between the tube and the steel nucleus filled with mortar or concrete. In this research, the size of the nucleus, tube and the thickness of the BRBs' crust are 153×19 (mm^2), 200×71 (mm^2) and 3 mm. The central nucleus is made of steel tip ST37 and the surrounding steel crust is made of high-strength steel ST52. The used concrete is the same as the normal one with compression strength of 21Mpa. There is 2.5mm of empty space between the central nucleus and mortar/concrete from each side. This distance is in fact the thickness of the separating layer, so that the nucleus, based on the inflicted force, can enter higher modes and, consequently, the brace can show better behavior in cyclic loadings. The middle concrete and the steel crust are continually in touch (Abdullahzadeh and Faghihmaleki, 1014).

The structures considered were calculated using SeismoStruct V.6 (SeismoSoft, 2012). Each member is modeled with a column-beam element, while the frames have constant rigid connections as well as abutments. This element makes a good balance between calculative accuracy and costs (Fragiadakis and Papadrakakis, 2008). The effects of gravitational forces and secondary impacts are considered and studied in accordance with non-linear geometrical considerations. Modeling scheme of steel behavior assigns a cinematic stress-tension curve for structural members by means of steel materials in SeismoStruct. A transition curve has been presented for these materials at the first and second tangent intersections in order to prevent sudden changes in local hardness matrices, which are made by the elements and for certainty of a straight and smooth transition among elastic and plastic regions. Strain hardening module is 2%E and an ultimate strain of 4% is considered for member behavior within non-elastic deformation region. Fig. 2 illustrates the behavior as well as structural steel characteristics. For beams, columns and braces, non-linear column and beam elements have been used at cross-sections for their accurate modeling. Additionally, the impacts of huge transition for changing corotational matrix of geometrical hardness have been taken into account.



a) CBF **b) EBF** **C) BRB**

Figure (1): Story plan and the frames considered (frame no. 2)

Table 1. Sections used in the structures considered

Story	Column	Beam	Story	Column	Beam
1	2IPE600	IPE450	5	2IPE500	2IPE360
2	2IPE600	IPE450	6	2IPE500	2IPE360
3	2IPE600	IPE450	7	2IPE450	IPE360
4	2IPE500	2IPE360	8	2IPE450	IPE360

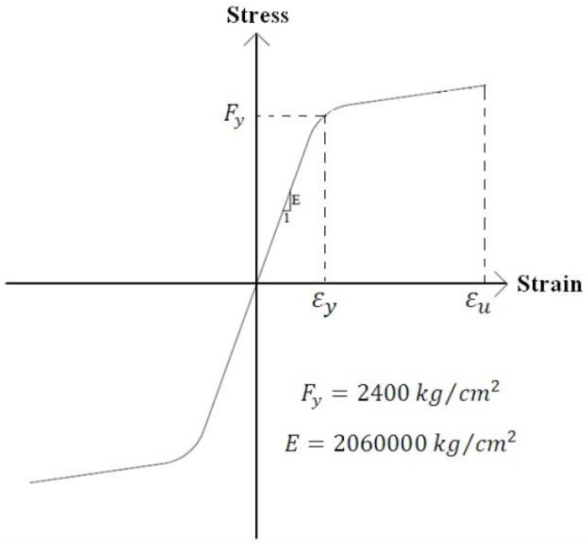
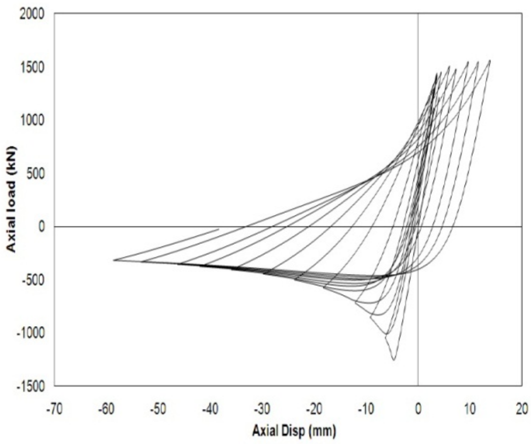


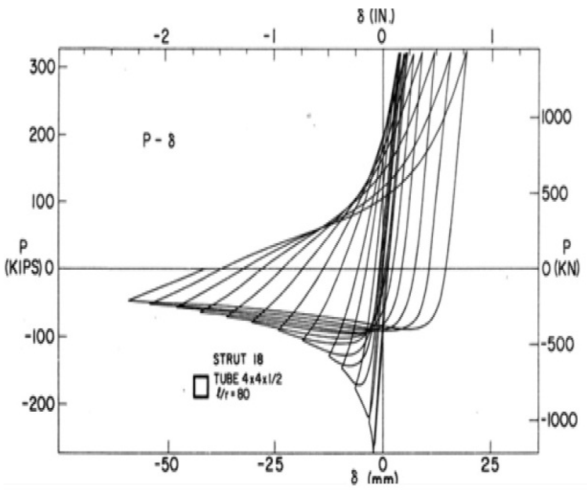
Figure (2): Structural steel behavior

Reviewing and Confirming the Model

Some exercises of reviewing the correctness of the expanded model as well as structural elements can be found in Asgarian and Hashemi Rezvani (2010) and in Asgarian and Shokrgoza (2008). Yet, for buckling and post-buckling analysis of bracing members, which is the result of an experiential study by Black et al. (1980) on a cube pipe strut 18 (TSS×4×0/5) under reverse cyclic loading, this research was expanded with the numerical model's results. In the current research, it was compared based on the mentioned characteristics of Black et al.'s report. Fig. 3 shows a comparison between the numerical model's results and the obtained results from the experiential study.

Accordingly, this figure shows the buckling resistance model and post-buckling hardness of the tested sample as accurate as possible.





a) Experiential results b) Numerical results

Figure (3): Brace behavior review and study

Seismic Progressive Collapse Analysis

Analysis Method

In order to review bracing frames with the considered braces and without some structural elements during earthquakes, at first gravitational loads were inserted into the structure, and then the pre-defined braces were removed from the structure. Afterwards, earthquake acceleration was performed and its consequent response was analyzed. Simulations were carried out with a relative hardness damping of 5%. In order to study the structural behavior of braced frames with considered braces during the absence of important and vital members, in all three studied frame models, one or two buckling braces on the first story were selected to be suddenly removed. Fig. 4 shows the columns as well as the buckling braces, located on the first story of the studied frames.

Table 3 gives a list of absence scenarios studied in this research, along with the members that were removed in each scenario. For each absence, IDA (Vamvatsicos and Cornell, 2002) was carried out by means of many time history analyses to determine the maximum inter-story drift along with the structure's seismic performance. Table 2 demonstrates 15 accelerometries used for IDA. Results from such methods are compared with healthy structures' results so as to study the seismic performance of structures that might lose some of their lateral elements in the earthquake. Fig. 5 shows IDA curves of the considered frames.

Table 2. Characteristics of 15 natural records

Record ID	Earthquake Name	Station	PGA (g)	PGV (m/cm)	PGD (m)
P0810	Cape Mendocino	Rio Dell	0.385	43.9	22.03
P0345	Coalinga	Park Field	0.112	14.6	5.69
P0817	Landers	Morongo Valley	0.188	16.6	9.45
P0739	Loma Prieta	Halls Valley	0.134	15.4	3.3
P0889	Northridge	Beverly Hills	0.617	4.8	8.57
P0058	San Fernando	Lake Hughes	0.145	17.3	2.88
P0139	Tabas	Boshrooyeh	0.107	13.7	10.5
P0360	Coalinga	Vineyard Cany	0.23	27.6	6.21
P0352	Coalinga	Gold Hill	0.094	11.0	2.87
P0891	Northridge	Big Tujunga	0.245	12.7	1.12
P0933	Northridge	Sunland	0.157	14.5	4.29
P0166	Imperial Valley	Chihuahua	0.27	24.9	9.08
P0169	Imperial Valley	Cucapah	0.309	36.3	10.44
P0916	Northridge	La Crescenta	0.159	11.3	3.0
P0899	Northridge	Glendale	0.357	12.3	1.94

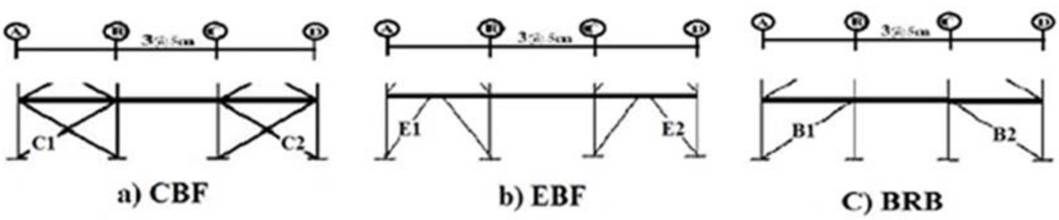


Figure (4): Numbering the frame's elements

Table 3. Scenarios of element absence

Loss Scenario	Frame Type	Removed Element
1	CBF	C1
2	CBF	C2
3	EBF	E1
4	EBF	E2
5	BRB	B1
6	BRB	B2

Based on this figure, it is obvious that the absence of lateral bearing elements decreases the seismic bearing capacity in the studied frames. In accordance with seismic load of UBC code, the limit state of inter-story drift is 2.5%. Fig. 5 shows that if in the first scenario the healthy structure can bear a PGA of 0.67g, in absence scenarios it becomes 0.62g and 0.51g, respectively. Furthermore, for an inter-story drift of 4%, it can be observed that the healthy structure can bear a PGA of 1.14g, whereas CBF scenarios element absence and lack result in a decrease in PGA to 0.53g and 0.69g for scenarios 2 and 3, respectively. It should be stated that a healthy structure means a model of special steel moment frame without any buckling brace suffering from structural damage.

In order to compare the controlling effects of the force with controlling impacts of transition, the transition was conducted based on the performance of such understandings. To do so, the presented limit state in FEMA 356 was used to model the parameters and acceptance criteria for non-linear dynamic methods. In order to calculate the updated failure rotation of columns and beams under load increase, in each step the axial power of one structural element was used at the moment of calculation.

Apart from columns with $P/_{PGL}$ greater than 0.5 (in which P is the axial power in a member and PGL the axial resistance of the string lower than a column), the force controlled was taken into consideration. For the buckling braces, the axial deformation in the considered strain load was the basis of determining the limit states. In this analysis, for each acted step of load increase, plastic rotation and acceptance criterion of the columns and beams were updated as a function of failure rotation. By having numerous analyses, the PGA damage related to limit states of FEMA 356 was calculated. Table 4 lists the performance-based analysis related to each limit state for each scenario, in which LS and CP are life safety and collapse prevention, respectively.

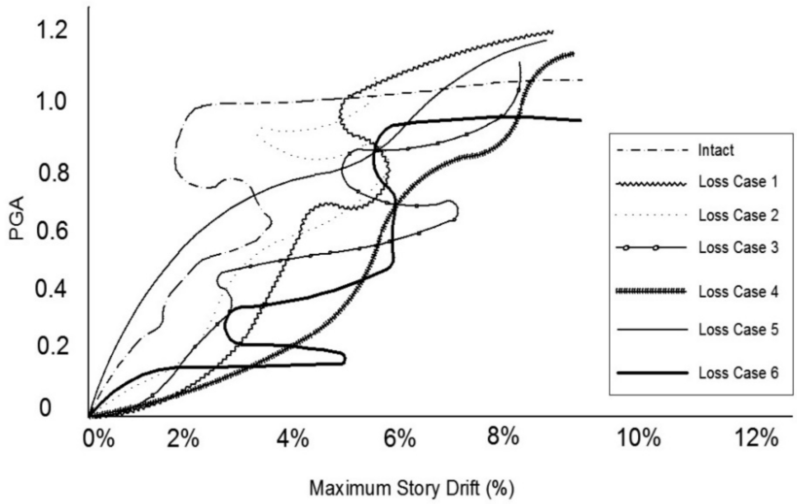


Figure (5): IDA curves for the considered frames

Table 4. Results from the performance-based analysis

Loss Case	Failure Mode	Limit State	PGA (g)	Axial Disp.
1	C1	LS	0.24	3.86
2	C2	CP	0.31	3.94
3	E1	LS	0.14	2.85
4	E2	CP	0.17	2.92
6	B1	LS	0.11	2.12
6	B2	CP	0.09	2.16

Based on Table 4, it can be noted that in the studied structures in all scenarios, buckling brace damage is the primary and essential damage mode. Additionally, it can be deduced that damage PGA of the same-center braced frames decreases as the number of structural elements removed increases.

For instance, the healthy structure in scenario 1 has a damage PGA of 0.24g in LS limit state, based on FEMA 356, whereas in absence scenarios 2 and 3, damage PGA is 0.13g and 0.17g, respectively. In addition, the healthy structure in scenario 4 has a damage PGA of 0.17g in LS limit state, while in absence scenarios 5 and 6 this rate is 0.11g and 0.09g, respectively.

CONCLUSIONS

This research studied progressive collapse of steel moment frame with CBF, EBF and BRB *via* Incremental Dynamic Analysis as well as performance-based analysis. For so doing, the numerical models were generated in SeismoStruct. In this research, some absence scenarios were studied, in which one or two bearing elements of the structure were removed during the earthquake. Results made clear that in the studied structures, the absence of one or two buckling braces decreases seismic performance.

In the studied absence scenarios, the absence of one lateral bearing element increases the maximum inter-story drifts in a PGA. On the other hand, in a specific drift percentage (for example 2.5%), absence scenarios 2 and 3 are the reason behind performed PGA decrease to 5.6% and 17.3%, whereas in drift percentage of 4%, these scenarios decrease the performed PGA to 39.6% and 43.7%. In order to have a deformation-based analysis, based on FEMA 356, absence scenarios 2 and 3 decreased damage PGA for LS limit state to 37% and 53%, respectively, whereas PGA for CP limit state was 34% and 42%, respectively. Furthermore, scenarios 5 and 6 decreased damage PGA for LS limit state to 23% and 42%, respectively, whereas PGA for CP limit state was 31% and 36%, respectively. Also, the research showed that the steel frames studied need reinforcing buckling braces in seismic regions, in which it is possible that the absence of other buckling braces decreases the potentiality of seismic progressive collapse occurrence. It should be noted that such conclusions are limited to the frames studied and it is necessary to have more analyses in order to expand the indicated points in the future.

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