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# Assessment of Eccentrically Braced Frames Strength Against Progressive Collapse

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**Abstract**

One of the most important and effective factors of structural strength against the risk of progressive collapse is the type of lateral load bearing system of a building. In this research, strength of dual steel moment frames equipped with a variety of eccentric bracings against progressive collapse was evaluated by using nonlinear static alternate path method. 6-floored building samples were designed with steel frame using a dual steel moment system together with 3 different types of bracing, including inverted eccentrically V-shaped bracing (chevron bracing), eccentrically V-shaped bracing and eccentrically X-shaped bracing, each with two different kinds of arrangement of bracings in the structural plan, in form of alternate and neighbor. The effects of sudden removal of columns on different floors of these buildings were examined. These studies showed that dual steel moment frames equipped with eccentric bracings generally exhibited desirable strength against progressive collapse. A change in the type of bracing resulted in significant changes in the system capacity in the progressive collapse. Among the different types of braces assessed, chevron type eccentrically brace showed higher strength against progressive collapse. Also, that alternate arrangement of bracings in structure plan demonstrated better performance than neighboring arrangement.

**Keywords:** progressive collapse, eccentrically braced steel frames (EBFs), alternate path method, nonlinear static analysis

## 1. Introduction

Structural safety has always been one of the main concerns for the design of civil engineering projects. One of the mechanisms of structural failure which has attracted much attention in recent decades is progressive collapse. Progressive collapse is defined as the spread of an initial local failure from element to element, resulting eventually in the collapse of an entire structure or a disproportionately large part of it. The phenomenon of progressive collapse under the influence of various factors occurs in structures. Plane impact, car collision and gas explosions are a few examples of the hazards which can produce such an event. An effect of this localized failure is that, without an increase in external load, redistribution emerges in internal forces of structures, and other parts are placed under additional forces, with a consequence of one or more other parts being damaged and more redistribution in forces occurring. Thus the damage spreads throughout the entire structure and may lead to collapse of the entire structure.

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The progressive collapse phenomenon, because of its catastrophic consequences and the high probability of its occurrence, is gradually taken into consideration in the design standards. The attention of engineers were drawn to the progressive collapse by destruction of a part of Ronan Point building located in London in the year 1968. The context of structural response to abnormal events drew more attention after the collapse of the World Trade Center towers on September 11, 2001. Rethinking and revising standards related to the design process of progressive collapse drew the attention of researchers at different institutions. For example, technical studies in this regard were conducted by the United States Department of Defense DOD or UFC (2010) and GSA (2005) and editors of European regulations.

Kim and Kim (2009) studied the progressive collapse-resisting capacity of steel moment frames by using alternate path method (APM) recommended in the GSA and UFC guidelines and observed that when a nonlinear dynamic analysis was conducted, it led to larger structural responses. Furthermore, they observed that the potential for progressive collapse was highest when a corner column was suddenly removed. The research also concluded that the progressive collapse potential decreased as the number of stories increased.

Kim and Kim (2009) suggested that the performance of buildings using cover plate connections turned out to be most effective in resisting progressive collapse, especially in structures located in moderate-seismic regions. Fu (2009) declared that under the same general conditions, a column removal at an upper story will induce larger vertical displacement than a column removal at ground level.

Khandelwal *et al*. (2009) in a study reviewed the progressive collapse strength in two different bracing system, including eccentrically braced frames (EBF) and special concentrically braced frame (SCBF), using simulation software. Simulation results revealed that although both models used seismic resistant frame around the building, eccentrically braced frames had less destruction against progressive collapse in comparison to Frames with special CBF system.

Kim *et al*. (2011) reviewed the probability of progressive collapse occurrence in a variety of concentrically braced steel frames using GSA guidelines in two methods of nonlinear static and nonlinear dynamic. For this purpose, 8 different types of frame bracing system were designed and their performances were compared with a special moment frame that was designed for the same amount of loads. The results of the nonlinear static analysis showed that in case of removing the column from braced span on the first floor, apart from the frame with K-shaped bracing, most braced frames designed based on criteria of current regulations were able to resist progressive collapse. However, most of the modeled structures possessed brittle behavior because of buckling of braces and columns in the event of collapse. Among the braced frames, only -shaped braced frame (chevron) exhibited a ductile behavior in the event of a progressive collapse.

Sun *et al*. (2012) studied the influence of bracing systems on the capacity of steel frames to resist progressive collapse under a localized fire. Couto *et al*. (2013) evaluated the buckling length of columns and the elastic range of loads in braced and unbraced structural frames exposed to fire using analytical solution. They demonstrated that when the temperature of a compressive element increases, its buckling length decreases resulting in the reduction of frame's elastic load. In closing, they proposed buckling lengths of 0.5 and 0.7 L for the intermediate and last story of a heated braced frame, respectively.

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 by Rezvani and Asgarian (2014). The equation of progressive collapse safety as a function of bracing member capacity was presented in this study.

Kazemzadeh Azad *et al.* (2017) have done a review study on researches that has been done on EBF systems. They stated that research on the progressive collapse of EBFs is very limited and future research on this topic is essential.

Nowadays, the use of dual steel moment frames with eccentric braces are taken into consideration as a lateral seismic system efficient in seismic rehabilitation in order to increase the lateral strength and stiffness of buildings against earthquake in concrete and steel structures. The results of tests conducted on this lateral seismic system under seismic loads showed high stiffness, sufficient strength, appropriate ductility and high energy dissipation. Dual steel moment frame systems with eccentric braces in addition to structures under construction are also used in seismic retrofitting of existing buildings. Therefore, due to the use of these systems in seismic resistant design, it is essential that the strength of these kind of systems be evaluated carefully against progressive collapse. The type of eccentric brace and bracing arrangement in the structure plan can be of importance and serve as effective factors in the structures against applied loads. In this study, the effect of these factors on the strength of dual steel frames against progressive collapse has been evaluated by using nonlinear static alternate path analysis based on UFC guidelines.

## 2. Analytical Modeling Progressive Collapse

In this article, the strength and performance of dual steel moment frame systems equipped with a variety of eccentric bracings against sudden removal of a column has been evaluated. UFC guideline recommends the usage of dynamic load increase factor of $\Omega_N$ in increased gravity load combination to those bays immediately adjacent to the removed element and at all floors above the removed element, as shown in Fig. 1 (Mohamed 2015).

According to UFC guidelines (Tables 3-5), dynamic increase factor is calculated using the following equation (e.g., Eq. (1)):

$$\Omega_N = 1.08 + 0.76(\theta_{pra}/\theta_y + 0.83) \quad (1)$$

where, $\theta_{pra}$ is the plastic rotation angle given in the acceptance criteria tables in ASCE 41 and UFC guideline for the appropriate structural response level (Collapse Prevention or Life Safety, as specified in UFC) for the particular element, component or connection; $\theta_y$ is the yield rotation. A flowchart of alternate path method and push-down analysis is shown in Fig. 2.

In nonlinear static alternate path procedure, the performance of structures against progressive collapse is evaluated in a way that the load of structures gradually increases until the structure eventually reaches the level of collapse under the suggested load pattern of regulations. For analyzing nonlinear progressive collapse, the nonlinear behavior of materials is defined by assigning a plastic hinge to elements in SAP2000 software.

Generally, for the studied structural models, four types of plastic hinge are defined:

(1) M3 flexural hinge for beam elements that are under

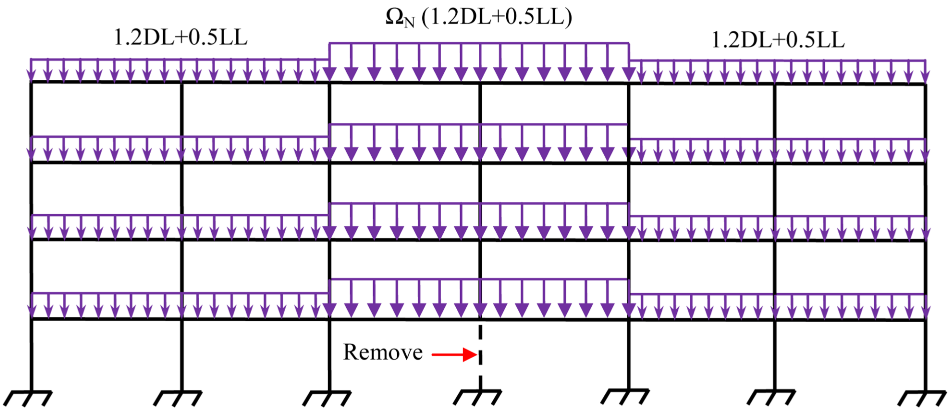



**Figure 1.** Imposed loads for progressive collapse static analysis.

Modeling and design of structure using SAP2000

↓

Obtaining the dynamic increase factor $\Omega_N$ according to UFC guideline

↓

Applying the following increased gravity load combination ($G_N$) to those bays immediately adjacent to the removed column and at all floors above the removed column
$G_N=\Omega_N(1.2DL+0.5LL)$

↓

Applying the following gravity load combination (G) to those bays not loaded with $G_N$
G=(1.2DL+0.5LL)

↓

Defining plastic hinges in beams, columns, braces and link beams

↓

Removing the intended column

↓

Performing a nonlinear static analysis with deformation-controlled actions

**Fiure 2.** Flowchart of alternate path method and push-down analysis.

bending in both ends of the beam.

(2) P axial hinge for bracing elements that are under pure axial force in the middle of the brace.

(3) P-M2-M3 interactional axial-flexural hinge for beam-column members that are under the combination of bending and axial forces in both ends and middle of the column.

(4) V2 shear hinge for eccentrically braced link beam in both ends of the link beam.

For non-linear analysis, UFC guidelines determine the maximum amount of rotation of the plastic hinges (Kim *et al*. 2009). Table 1 shows the acceptance criteria for different structural members.

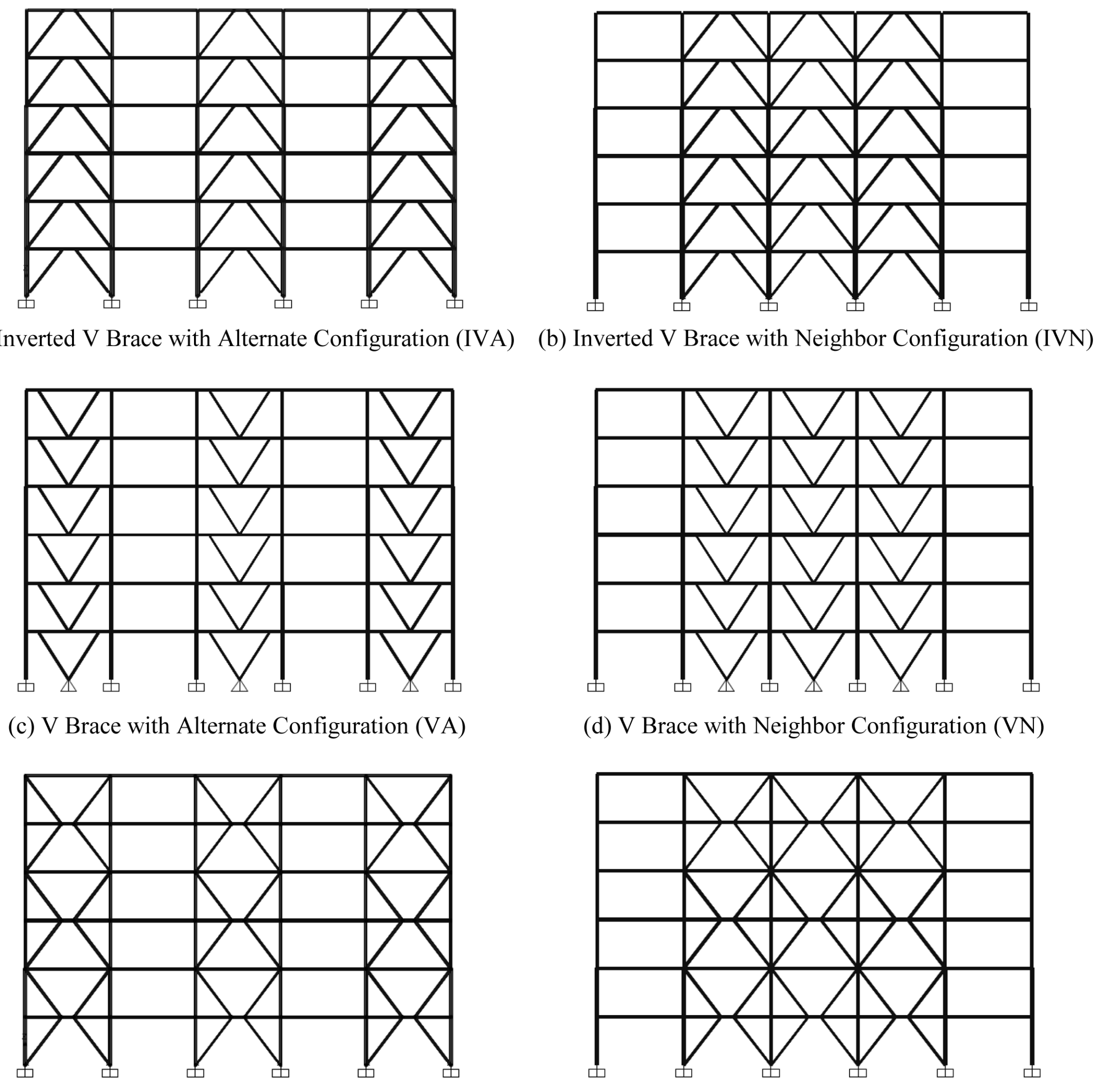
**Table 1.** Progressive collapse acceptance criteria for different structural members

| Component | Performance level (UFC2010) |
| --- | --- |
| Steel beams | CP |
| Steel columns | LS |
| Steel braces | LS |
| Shear links | CP |

## 3. Structural Analytical Models

One important and effective factor in the strength of structures against risk of progressive collapse is the type of lateral load resisting system. By changing the load path on the effect of removing a critical member such as column, the forces of the removed member, as a result of this event must be transmitted to other parts of the structure by neighboring members. In this study, six models of 6-story steel building, with 3 kinds of bracing and 2 kinds of bracing arrangements in structural plans have been designed. The building plan for all samples comprised 5 spans as symmetrically in two directions perpendicular to each other by a span of 7 m and the height of floors were considered to be 4 m. Vulnerability of these buildings against progressive collapse have been studied by using suggested UFC nonlinear static alternate path method. As shown in Fig. 3, to resist the gravity and seismic loads, special dual steel moment frame systems with inverted V-shaped, V-shaped and X-shaped eccentric braces, and two different types of bracing arrangement, including alternate (A) and neighbor (N) types in external frames of structure were considered. These structures have been designed in accordance with AISC (2010) guidelines and all seismic standards related to strength and drift limitations were controlled in these structures. Geometry of the studied sample structures are shown in Fig. 3 along with the relevant acronym names.

Designed dead and live loads have been considered, respectively as 5.9 and 1.5 kN/m$^2$ on roof floor and, respectively as 5.9 and 1.5 kN/m$^2$ on other floors. Design seismic loads were calculated on the basis of ASCE 7-10 guidelines; the design spectral acceleration parameters $S_s$ and S1 are 0.804 and 0.388, respectively, in the IBC (2012) format; and the site coefficients $F_a$ and $F_v$ are 1.2 and 1.6, respectively. Dual systems response modification



(a) Inverted V Brace with Alternate Configuration (IVA) (b) Inverted V Brace with Neighbor Configuration (IVN)

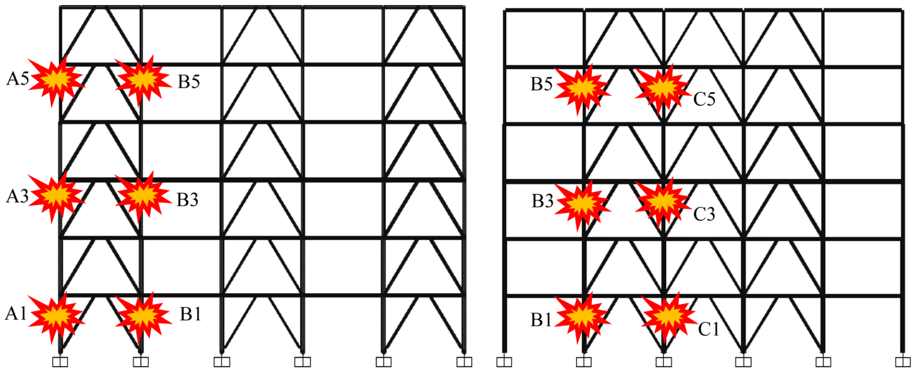
(c) V Brace with Alternate Configuration (VA) (d) V Brace with Neighbor Configuration (VN)

(e) X Brace with Alternate Configuration (XA) (f) X Brace with Neighbor Configuration (XN)

**Figure 3.** Congurations of various braced frames.

**Table 2.** Frame Sections of IVA Model

| | | 1st-2nd storeys | 3rd-4th storeys | 5th-6th storeys |
|---|---|---|---|---|
| Columns | Exterior | W12×152 | W10×100 | W8×48 |
| | Interior | W10×77 | W8×58 | W8×40 |
| Beams | Exterior | W8×48 | W8×48 | W8×48 |
| | Interior | W10×100 | W10×100 | W10×100 |
| Braces | | HSS6×6×5/8 | HSS6×6×5/8 | HSS6×6×1/2 |



(a) Neighbor Configuration (b) Alternate Configuration

**Figure 4.** Location of column removal for Structural models for two different types of arrangement of braces.

coefficient (steel moment frame with eccentrically steel bracing) was considered to be 8. Bracing sections on all structural models were selected in form of square hollow steel sections and seismic compactness of the sections were controlled. ASTM A500 grade C steel ($F_y$=322 MPa) was used for braces and ASTM A992 steel ($F_y$=345 MPa) was used for all the beams and columns. Table 2 shows the sections of IVA structural model members as a sample.

Location of column removal for structural models for two different types of arrangement of braces (at alternate and neighbour spans) has been shown in Fig. 4.

For comprehensive assessment of progressive collapse of eccentrically braced frames, removal of columns in any of the first, third and fifth floors of studied structural models were separately done. Table 3 shows different cases of alternate path analyzed models with members removed in each case, with a total of 36 analytical models

**Table 3.** Alternate path method (APM) analysis cases

| APM case | Models | Column removed |
|---|---|---|
| 1 | IVA-VA-XA | A1 |
| 2 | IVA-IVN-VA-VN-XA-XN | B1 |
| 3 | IVN-VN-XN | C1 |
| 4 | IVA-VA-XA | A3 |
| 5 | IVA-IVN-VA-VN-XA-XN | B3 |
| 6 | IVN-VN-XN | C3 |
| 7 | IVA-VA-XA | A5 |
| 8 | IVA-IVN-VA-VN-XA-XN | B5 |
| 9 | IVN-VN-XN | C5 |

studied. For executing the analysis, intended column was removed from the analytical model from the very beginning and the structure under load as shown in Fig. 1 has been analyzed in nonlinear static form. According to equation (1) and using specifications of sections, the load increase factor equals 1.166.

## 4. Results and Discussion

Nonlinear static alternate path analysis has been executed by removing intended column and the gradual increase in the gravitational load according to the suggested pattern of UFC guidelines. At every step during the push-down analysis, i.e., at each level of the vertical displacement, the amount of equivalent load corresponding to the displacement level was determined. The amount of the load was referred to as the “load factor,” which represents the ratio of the equivalent load to the full gravity load. The maximum load factor exceeding 1.0 implied that the imposed load specified in the UFC guideline could be supported by the structure after a column was suddenly removed (Kim and Hee-Park, 2011). In other words, if the alternate path analysis results to a maximum load factor smaller than unity, this implied that the structure was nonresistant against progressive collapse under the load combination of UFC guidelines. However, if load factor reaches the unity and also the acceptance criteria mentioned in Table 1 is satisfied, structure will have sufficient strength against progressive collapse.

In this section, to provide results of the alternate path analysis, an abbreviated three-part naming, including the type of model, bracing arrangement and the number of removed column was utilized. As an example, IVA-B3 implied structure with inverted V eccentric braces, IV, with the alternating arrangement type of braced spans, A, such that B3 column in its third story was removed.

In the following, initially the results of the progressive collapse analysis for B3 column removal from IVN sample structure was provided with full details, then the results from the elimination of this column was addressed in other samples and then for briefness, only the comparative graphs of columns removal results were shown and discussed.

### 4.1. Influence of removing B3 center column

Results of the nonlinear static alternate path analysis related to B3 column removal from IVN model in Fig. 5 has been shown in form of a graph of load factor-upper node displacement of removed column. According to the results of the analysis, until the load factor of 1 structure has not reached the nonlinear zone, reaching the nonlinear behavior boundary occurred by the formation of the first plastic hinge of structure in load factor of 1.21. According to Fig. 5, the structure was able to tolerate load factor of 1.48 without overall collapse. Displacement of upper point of the removed column for maximum load factor of 1.48 was equal to 4.51 cm. According to acceptance criteria of UFC guidelines, in a load factor of 1.33, link beams of fourth, fifth and sixth stories passed from the collapse prevention performance level (CP) and practically

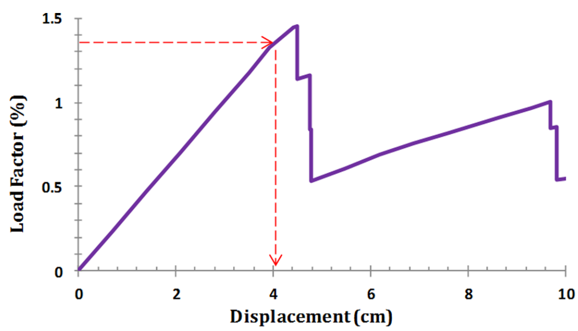



**Figure 5.** graph of load factor-displacements of IVN-B3 sample.

had lost its ability of load bearing and suffered significant damages. Figure 6 shows the stepwise process of plastic hinge formation and collapse behavior of this structure for different load factors. One of the fortes of collapse behavior of this structure is that there is no plastic hinge formation in columns. On the contrary, the weakness of this structure is its brittle and non-ductile behavior and destruction of structure for very little displacement in the event of progressive collapse. Table 4 shows a summary of results of progressive collapse analysis on this structural model.

Table 5 shows maximum reliable load factor according to acceptance criteria of UFC guidelines and corresponding maximum displacement of upper node for removing B3 column from IVA, VA, VN, XA and XN models. As the simulation results in Table 5 shows, all systems can be able to absorb redistributed forces caused by sudden removal of this column so well that its signs are lack of plastic hinge formation in columns and bracings, with the main focus of hinges being on link beam elements and having acceptable progressive collapse load factor (bigger than one). Another result obtained according to the amount of displacement and maximum load factor mentioned in Table 5 was that models with eccentrically inverted V braces possessed relatively better performance.

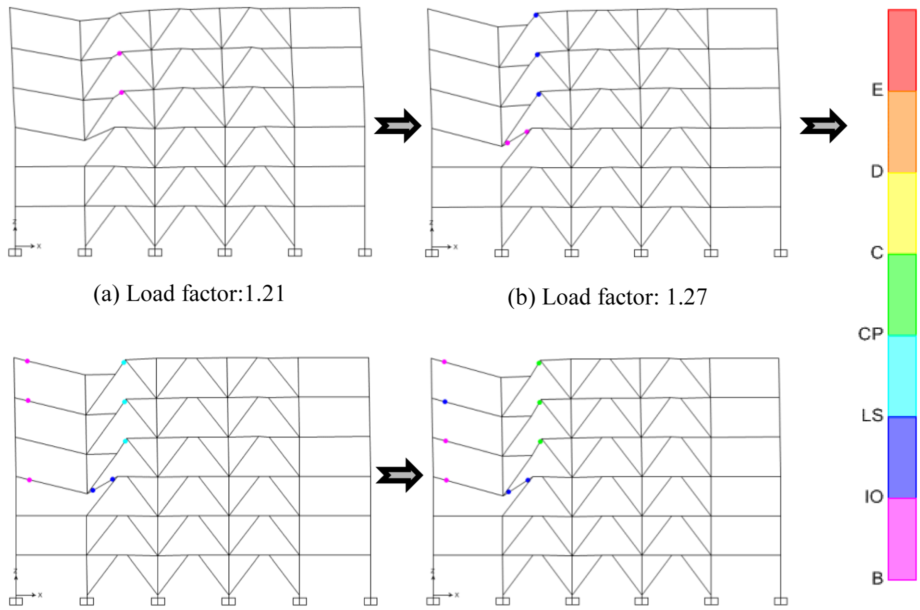
Figure 7 shows the graph of load factor-removed column upper node displacement of all models for removal of B3 column. As observed in the Fig. 7 relating to the removal of the B3 column in six sample buildings, a change in the type of bracings was effective in a change of progressive collapse behavior but a change in the arrangement of bracings from neighbor mode to alternate mode demonstrated no important effect on the results. That was due to the specific location of intended column that was always neighbored with a braced span. According to Fig. 7, all structures demonstrated a non-ductile failure behavior.

### 4.2. Comparative Study of progressive collapse analysis results

In order to review the effect of column removal in the strength of structures against progressive collapse and also choosing the type of eccentrically bracing system

**Table 4.** summary of IVN-B3 sample analyze results

| Resistance level | Plastic Hinges | | | | Maximum vertical displacement in location of the removed column(cm) | Maximum load factor |
|---|---|---|---|---|---|---|
| | Beam Elements | | Link Beam Elements | | | |
| | Performance level | The number of plastic hinges | Performance level | The number of plastic hinges | | |
| Progressive collapse | B-IO<br>IO-LS<br>LS-CP | 3<br>2<br>0 | B-IO<br>IO-LS<br>LS-CP | 0<br>0<br>3 | 4.03 | 1.33 |
| Overall collapse | B-IO<br>IO-LS<br>CP-C | 3<br>3<br>0 | B-IO<br>IO-LS<br>CP-C | 0<br>0<br>3 | 4.51 | 1.48 |




(a) Load factor:1.21

(b) Load factor: 1.27

(c) Load factor: 1.32

(d) Load factor: 1.48

**Figure 6.** distribution of plastic hinges of IVN-B3 sample at various loading levels.

**Table 5.** summary of results of nonlinear static analysis for B3 column removal

| Model | Maximum vertical displacement in location of the removed column (cm) | Maximum load factor | The number of plastic hinges |
|---|---|---|---|
| IVA-B3 | 4.43 | 1.31 | 13 |
| VA-B3 | 4.85 | 1.19 | 15 |
| VN-B3 | 4.81 | 1.21 | 15 |
| XA-B3 | 2.31 | 1.23 | 14 |
| XN-B3 | 2.29 | 1.24 | 14 |

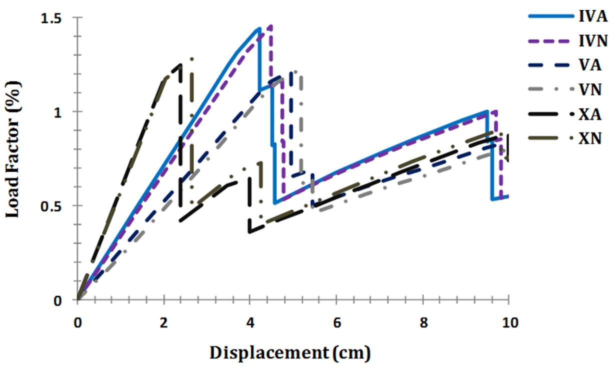


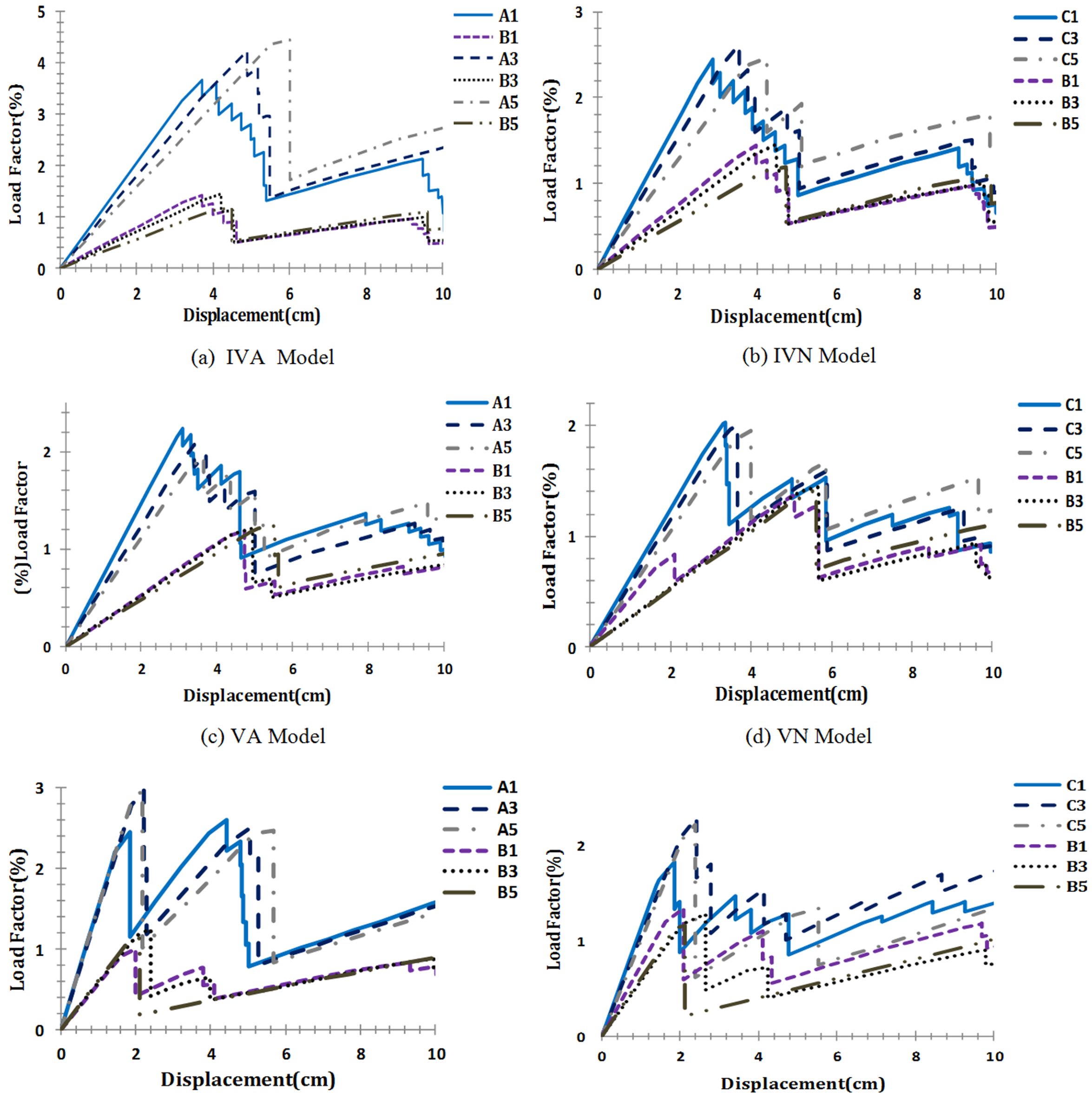

**Figure 7.** The graph of load factor-displacement of 6 sample buildings for removal of B3 column.

and an arrangement of it that has the best performance against progressive collapse, the graph of load factor-displacement of all sample structures for removal of different stories columns has been separately shown in Fig. 8.

According to graphs drawn in Fig. 8, capacity of structure for removal of columns with two neighbor braced spans are of average of 2 to 3 times higher than capacity of the same structure for the removal of columns with one neighbor braced span. This is because of high grade of structural indeterminacy and existence of more alternative paths for redistribution of forces. Therefore in these structural systems, the removal of a column with one neighbor braced span (column B) in different stories results in a more critical situation.

According to Fig. 8, IVA and IVN structures with inverted V bracing, respectively in alternate and neighbor span, have higher load factor and displacement compared to VA, VN, XA and XN structures with V-shaped and X-shaped bracings. The main reason for this is the better distribution of plastic hinges and existence of more alternative paths for distribution of forces caused by column removal.

On the other hand, VA and VN structures, with V-shaped bracing, respectively with alternate and neighbor span, although possessed enough capacity for resisting progressive collapse load, were weaker compared to two other systems and collapsed in lower load factor. The structure of V-shaped bracing is in such a way that in the event of column removal, a rigid triangular-shaped zone is formed in the middle of the span and a large shear force is created in link beams of related spans such that this




(a) IVA Model (b) IVN Model

(c) VA Model (d) VN Model

(e) XA Model (f) XN Model

**Figure 8.** Graph of load factor-displacements of all sample structures for removal of different floors columns.

matter results in immediate collapse and extreme brittle behavior of this kind of bracing (Fig. 6).

Among two forms of arrangements of neighbor and alternate, the removal of the same column in both models with one neighbor braced span (e.g., B1, B3 and B5 columns) shows similar results (according to Fig. 8). But for the removal of column with neighbor braced spans, in arrangement of alternate bracing because of placement of bracings in two perpendicular planes, the gravitational forces exerted exhibited less influence with a higher progressive collapse capacity (according to Fig. 7(a)-7(c)-7(e)) and greater compliance. However, in the arrangement of neighbor bracing for removal of the column with two neighbor braced spans, bracings were located in one plane and the effect of gravitational forces for removal of the intended column was higher (according to Fig. 7(b)-7(d)-7(f)) and as the result capacity of the structure was lower. On the other hand, the arrangement of alternate bracing supports the removal of more columns of existing spans than the arrangement of neighbor bracing against progressive collapse and this is one of the most important advantages of this type of arrangement.

## 5. Conclusion

In structures with eccentrically lateral seismic bracing system, by removing each of the critical elements determined by UFC progressive collapse instruction and by executing progressive collapse analysis via alternate path method on the reviewed samples, the following results have been obtained:

(1) Eccentrically inverted V type bracing system provided more ductile behavior and better performance compared to other systems against progressive collapse because of providing more suitable alternative paths and the ability of better distribution of plastic hinges in the structure. By changing the type of bracing system from inverted V-shape to V-shaped or X-shaped bracing, significant decline was observed in structures progressive collapse-resisting capacity.

(2) Among the type of arrangement of bracing, arrangement of alternate bracing supported more column removal compared to the arrangement of neighbor bracing against progressive collapse and demonstrated better performance. As a result, among these six types of analytical model structures, the building with dual special moment frame with inverted V type eccentrically bracing, with alternate arrangement of bracings demonstrated the best performance compared to the other structures.

(3) It was obvious that the removal of column with one braced span compared to its removal in situations with two neighbor braced spans because of the providing less alternative paths can lead to more critical situations in terms of the dangers of progressive collapse.

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