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Progressive collapse analysis of buildings with concentric and eccentric braced frames

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Abstract. In this study, the susceptibility of different symmetric steel buildings with dual frame system to Progressive Collapse (PC) was assessed. Some ten-story dual frame systems with different type of braced frames (concentrically and eccentrically braced frames) were considered. In addition, numbers and locations of braced bays were investigated (two and three braced bays in exterior frames) to quantitatively find out its effect on PC resistance. An Alternate Path Method (APM) with a linear static analysis was carried out based on General Services Administration (GSA 2003) guidelines. Maximum Demand Capacity Ratio (DCR) for the elements (beams and columns) with highest DCRs (DCR_{moment} and DCR_{shear}) is given in tables. The results showed that the three braced bays with concentric braced frames especially *X*-braced and inverted *V*-braced frame systems had a lower susceptibility and greater resistance to PC. Also, the results represented that the beams were more critical than columns against PC after the removal of column.

Keywords: Alternate Path Method (APM); Concentrically and Eccentrically Braced Frames (CBFs and EBFs); two and three braced bays; Demand Capacity Ratio (DCR); Progressive Collapse (PC)

1. Introduction

One of the factors that lead to potential Progressive Collapse (PC) of structures is removal of a load-bearing element such as a column. The PC of structures commences when a primary component or components, usually one or more columns, is eliminated.

Coda and Paccola (2014) developed a method for the geometrical and physical nonlinear analysis of plane structures and mechanisms. The semi-rigid connections were considered for connecting the frame elements. The total lagrangian formulation was applied based on positions. They used various examples to represent the efficiency of their formulation, comprising of a PC analysis of a high-rise building subjected to a real seismic load. Gerasimidis (2013) conducted a study using an analytical method to indicate the collapse mechanism of a steel frame regarding the corner column elimination. Furthermore, the author studied the impact of various elements in building such as; the column removal location, the number of floors, the vertical irregularity and the design of the frames.

Tohidi *et al*. (2014) studied a numerical estimation of PC for precast concrete cross wall buildings. They developed three-dimensional finite element models of the pullout behavior of strands in the keyway of precast concrete blocks. They also investigated the ductility behavior of floor joints subjected to line and uniform loads applied from upper walls. Furthermore, through a calibration procedure for a series of laboratory pullout tests conducted by the Portland Cement Association (PCA), the interfacial bond properties were set up applying numerical modeling. Also, the similar modeling method was then exerted in the subsequent three dimensional non-linear numerical analyses to simulate the ductility behavior of precast concrete floor joints in the absence of underlying wall supports. They found a significant agreement between finite elements and experimental results.

Le and Xue (2014) conducted a study including two-scale numerical model to evaluate the PC risk of reinforced concrete frame structures. They represented the potential damage zones of beams, columns and joint panels. The probabilistic collapse behavior of a two-dimensional 30-story reinforced concrete structural frame under different column removal scenarios was investigated by using the model, where the occurrence probabilities of various possible collapse extents were calculated. Overall, they compared present probabilistic approach with the prevalent deterministic analysis method.

Elsanadedy *et al*. (2014) conducted a study to assess the vulnerability of a typical multi-story steel framed building in Riyadh when subjected to blast scenarios against PC. They applied a commercial finite element package (LS-DYNA) to simulate the building response under blast generated waves. Finally, they made their recommendations to mitigate (or control) the PC potential based on the finite element analysis results.

Keyvani *et al*. (2013) developed a finite element modeling method to simulate punching and post punching behavior of flat plates. They investigated the PC potential of a sixteen-column flat slab numerically. Analysis showed that the lateral restraint in flat slab structures was provided

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by the slab itself and it is not necessary to restrain the slab edges. Moreover, the results of this study proved that contribution of the non-continuous tensile reinforcing bars at the location of the columns to the post-punching strength were insignificant compared to the integrity bars.

Elkoly and El-Ariss (2014) proposed a numerical procedure and a technique to mitigate PC of reinforced concrete continuous beams after elimination of interior columns. They used finite-element modeling and a push down analysis to simulate column removal. The proposed technique included external unbounded fiber reinforced plastic cables attached to the beam at anchorage locations and deviators/ saddle point(s) only, without being posttensioned. They concluded that the beams' strength was mitigated by the proposed technique.

Jiang *et al*. (2014) conducted a study using OpenSees software to find the fire-induced PC mechanisms of steel structures applying dynamic analysis method. They also considered the influences of the load ratios, beam sizes and fire scenarios on the collapse behavior of frames. Thus, they applied single-compartment fire scenarios in the edge bay and central bay, respectively. They concluded that the central bay fire was less prone to induce the collapse of structures than the edge bay fire.

Tsai (2012) developed a comparison study between analytical load increase factor (LIF) and dynamic increase factor (DIF) expressions with the empirical formulae recommended in the UFC guidelines using static analyses. By applying static and dynamic analyses for eight moment resisting frames subjected to column loss Tsai found that the proposed analytical expressions may be an alternative in estimating the LIF and DIF for PC analysis.

Zoghi and Mirtaheri (2016) conducted a study including an existing seismically code-designed steel building using Alternate Path Method (APM) to assess its resistance against PC. Finally, they proposed a formula to determine potential of collapse of the structure based on the quality and quantity of the produced plastic hinges in the connections.

Based on the research conducted by Larijani *et al*. (2013), they arrived to this conclusion that applying moment frame system (especially dual frame system) in steel buildings was better choice than simple frame system. Also, implementing built-up box-shaped sections for columns had a lower risk of PC. Therefore, dual frame system and box section for columns were used in this study.

This research applied different ten-story dual frame systems with both types of braced frames (concentrically and eccentrically braced frames) by considering different numbers and locations of braced bays (two and three braced bays). Finally, their effects on PC resistance were assessed.

2. Methodology

2.1 Flowchart approach to assessing the PC potential

Fig. 1 describes the procedure for assessing PC in designed steel buildings with concentric and eccentric braced frames using an APM based on linear static analysis.

2.2 Methods for preventing PC

Researchers have proposed three methods for reducing the probability of disproportionate collapse in buildings: the APM, improved local resistance for critical elements and inter-connection or continuity. According to the U.S General Services Administration (GSA 2003) and the Interagency Security Committee (ISC 2001), an APM is an appropriate means for assessing and preventing the process of PC in buildings of up to ten stories (low to medium rise). Thus, the APM was used in this study. According to ASCE 7 (2005), buildings subjected to an alternate path analysis will be enhanced such that if a primary component faces damage or collapse, PC will not occur. The APM is used for analyzing and preventing the collapse. This method is based on redundancy improvement, ensuring that the loss of any single component would not eventually lead to PC (Larijani *et al*. 2013).

2.3 Choice of the method of analysis and guidelines

Different guidelines, such as GSA and UFC, are being used for assessing the process of PC. Among them, the GSA Guidelines, which consider structures under ten stories, are the most appropriate for this case study (Larijani *et al*. 2013).

According to the GSA Guidelines, linear static analysis is the preferred method for analyzing structures having the potential for PC. Therefore, in this study, an APM using linear static analysis was applied to evaluate and prevent the PC of the buildings.

According to the GSA guidelines, amplified vertical loads should be used for static analysis procedures in the chosen case studies as follows

$$\text{Load} = 2(\text{DL}+0.25\text{LL}) \tag{1}$$

where DL=the dead load and LL=the live load.

2.4 Calculation of the DCR

To determine the susceptibility of the building to PC, the DCR should be calculated using Eq. (2)

$$\text{DCR}=\text{QUD}/\text{QCE} \tag{2}$$

where

QUD=the acting force determined or computed in the element or connection/joint and

QCE=the probable ultimate capacity of the component and/or connection/joint.

Referring to DCR criteria defined through a linear static approach, different elements in the structures and connections with DCR value less than 1.5 or 2 are considered not to be collapsed as follows (GSA 2003 guidelines):

DCR<2.0: for a typical structural configuration

DCR<1.5: for an atypical structural configuration

Cases that have been chosen for this study had a typical structural configuration.

It should be mentioned that the loading pattern used in this study was based on gravity alone (amplified dead and

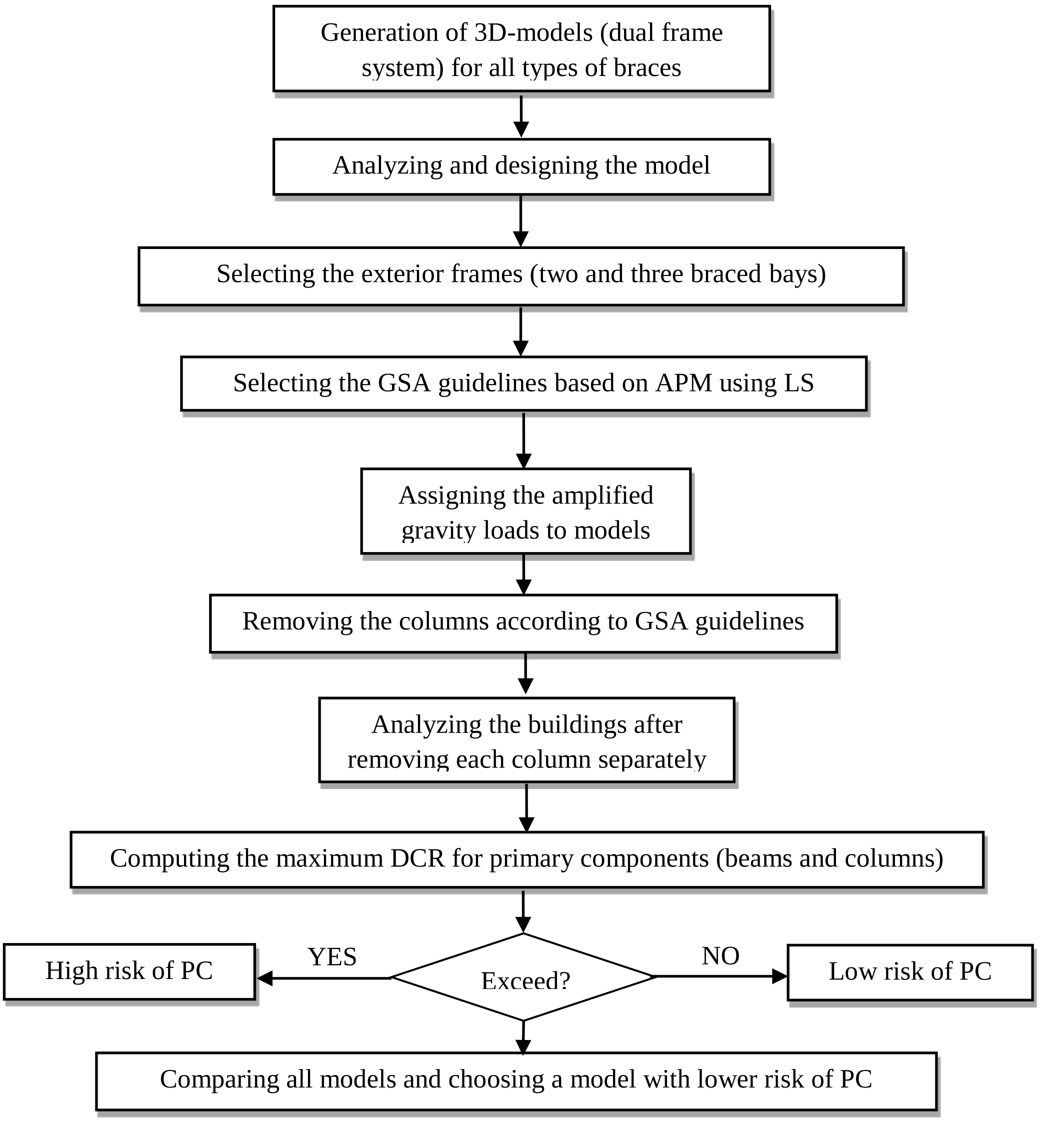


Fig. 1 Flowchart for PC analysis

live loads), so computation of the DCR values for braces was omitted.

2.5 Description of designed steel buildings

The considered structures were ten-story regular steel moment resisting framed structures with concentrically and eccentrically braced frames which were designed based on Iranian code of practice for seismic resistant design of buildings (Standard No. 2800 2004) using ETABS software. The braces systems used in this study included Concentrically Braced Frames (CBFs) and Eccentrically Braced Frames (EBFs). In order to investigate the impact of the number and location of braced bays, the exterior North-South frames were considered with two braced bays while the exterior East-West frames had three braced bays, as shown in Fig. 2. Concentrically and eccentrically braces in diagonal, *X*, *K*, *V* and Inverted *V* shapes in different numbers and locations (two and three braced bays) were used to quantitatively find the suitable braced frame system (s), numbers and locations of braced bays against PC, as

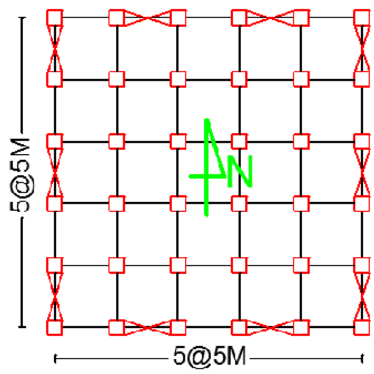


Fig. 2 Plan view of buildings

shown in Figs. 3 and 4. Table 1 represents cross sections for all members. Detailed information of these buildings is depicted in Table 2. The height of floors stands at 3 meters as shown in Figs. 3 and 4. As well as, the structures have no irregularity in elevation or plan. The steel section designations for the concentric braced frames are depicted in Fig. 5.

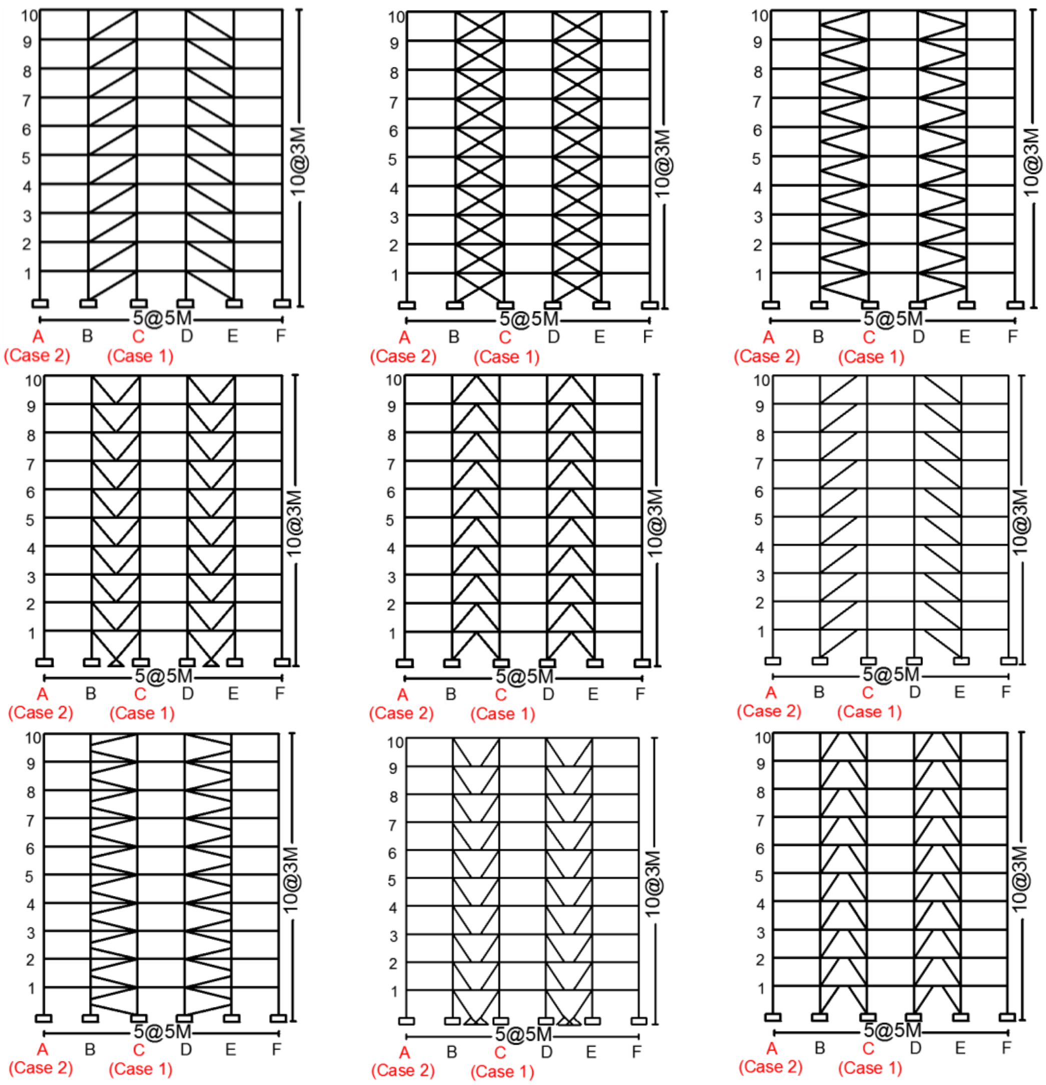


Fig. 3 Concentric and eccentric braced frames (North-South elevation)

Table 1 Cross sections for all members

Columns	Beams	Braces
B175×175×15	IPE160	B125×125×10
B200×200×15	IPE180	B125×125×12
B200×200×20	IPE200	B175×175×15
B225×225×20	IPE220	B200×200×15
B250×250×20	IPE270	B200×200×20
B275×275×20	IPE300	B225×225×20
B300×300×25	IPE330	B225×225×25
B325×325×30	IPE360	B250×250×20
B350×350×30	IPE400	B250×250×25
B375×375×30		
B400×400×40		

2.6 Selecting the columns for removal

To calculate the DCR value according to the GSA guidelines, the first step is analyzing the sudden removal of a first story column located at or near the middle of the building. This situation was assessed in case 1 labeled with *C*, as shown in Figs. 3 and 4, as well as mentioned in Tables 4 and 5. The second step is analyzing the sudden removal of a column one floor above the ground located at corner of the building. This situation was assessed in case 2 labeled with A, as shown in Figs. 3 and 4, as well as mentioned in Tables 4 and 5. The analysis results and the maximum DCR values for beams and columns were computed, and the susceptibility to PC was assessed by considering different braced frame systems as well as different numbers and locations of braced bays.

2.7 Design details of buildings

According to the Iranian Seismic Code No. 2800 (Standard No. 2800 2004), different seismic regions are classified based on related seismic vulnerability analyses.

Each region has special earthquake acceleration, as depicted in Table 3. The applied frames were placed in the high seismic zone with a design earthquake acceleration of 0.30 g. Based on mentioned code, the equivalent static analysis method is only allowed for symmetric structures which are not taller than 50 m or asymmetric structures which are not taller than 18 m. Since the considered

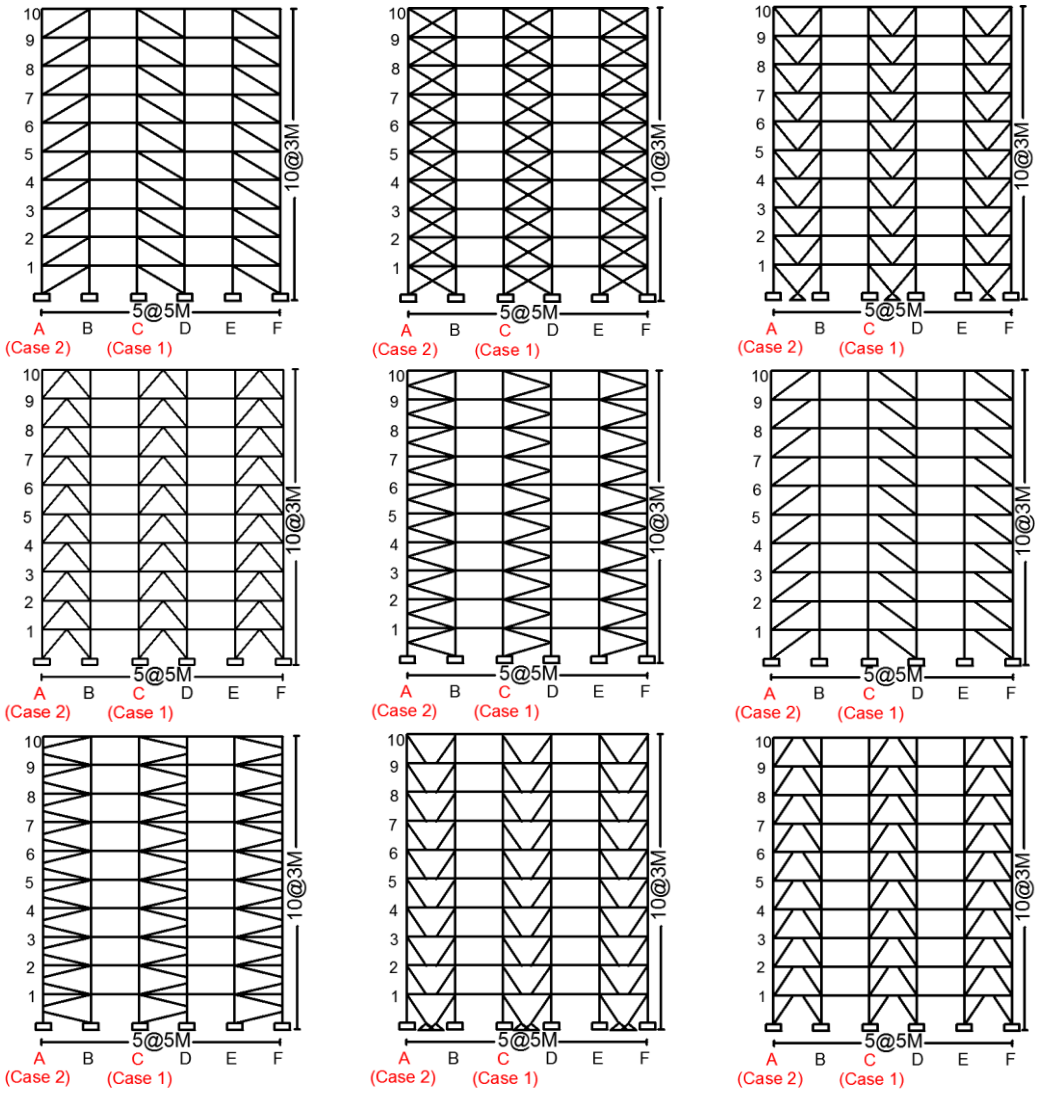


Fig. 4 Concentric and eccentric braced frames (East-West elevation)

structures were symmetric and were not taller than 50 m. The static analysis method was considered for their design.

The base shear force should be calculated according to Eq. (3) in the equivalent static method, as in UBC 94 (Standard No. 2800 2004).

$$V=C\times W \qquad (3)$$

where V is the base shear of the structure, C is the base shear coefficient and W is the equivalent weight of the structure calculated in accordance with Eq. (4).

$$W=\text{total dead load}+\beta\times(\text{live load})\quad 0\leq\beta\leq 1 \qquad (4)$$

For the residential buildings β is equal to 0.2. The base shear coefficient is calculated from Eq. (5) as follows

$$C=\frac{ABI}{R} \qquad (5)$$

where $A\times B$ is the design spectral acceleration, expressed as the ratio of gravitational acceleration for the basic period of buildings T and soil type which is calculated according to Eq. (6) and Table 3, I is the importance factor and R is the response modification factor.

$$B=1+S\left(\frac{T}{T_0}\right) \qquad 0\leq T\leq T_0$$

$$B=S+1 \qquad T_0\leq T\leq T_S$$

$$B=(S+1)\left(\frac{T_S}{T}\right)^{\frac{2}{3}} \qquad T\geq T_S \qquad (6)$$

The basic period T is calculated in accordance with Eq. (7) for steel braced frames and can be increased up to the rate of 25% when the analytical period of the buildings resulted from the dynamic analysis is more than its experimental one. T_0, T_S and S are factors describing soil effects.

$$T=0.05H^{\frac{3}{4}} \qquad (7)$$

The considered buildings located on soil type C (average shear wave velocity would be 175-375 m/s). According to (Standard No. 2800 2004), for this type of soil, T_0=0.15, T_S=0.7 and S=1.75. For the ten-story buildings investigated in this research T=0.64 and the third formula in Eq. (6) was used to calculate B. An importance factor of I=1, response modification of R=7 and seismic zone factor of A=0.30 were considered in the design of

Table 2 Detailed information of buildings

Title		Ten-story buildings
Geometry type		Symmetric
Structural system		Dual frame system (moment frame with bracing system)
Structural system against earthquake		Bracing system 100% + moment frame 30%
Code		Iranian code of practice for seismic resistant design of buildings
No. of span in *X* and *Y* directions		Five-bay with two braced bays & five-bay with three braced bays
Type of roof		In-situ concrete slab
Material properties		Modulus of Elasticity (E)=2.039E+10 Kg/m^2, Poisson's Ratio (ν) = 0.3, Weight per Unit Volume (W)=7833 Kg/m^3, Mass per Unit Volume (M)=798.1 Kg/m^3, Minimum Yield Stress (F_Y)=2.4E+7 Kg/m^2 and Effective Tensile Stress(F_U)=3.7E+7 Kg/m^2
Geometry type		Symmetric
Connection	Column to column	Continuous between the two story levels
	Beam to column	Rigid
	Braces	Pinned
Loading	LL & DL for floors	200 Kg/m^2 and 600 Kg/m^2
	DL of surrounding wall	800 Kg/m^2
	DL of stair box in X direction	2000 Kg/m^2
	LL & DL for roof	150 Kg/m^2 and 500 Kg/m^2

Table 3 Earthquake acceleration for different seismic regions (Standard No. 2800 2004)

Zone	Seismic hazard	Earthquake acceleration (g)
1	Very high	0.35
2	High	0.30
3	Medium	0.25
4	Low	0.20

these buildings. Therefore, the base shear coefficient for buildings was calculated as C=0.118.

3. Results and discussion

Tables 4 and 5 represent the maximum DCR_{moment} and the maximum DCR_{shear} for beams and columns under amplified load after the removal of column (case 1 and case 2), as shown in Figs. 3 and 4. In addition, concentrically and eccentrically braces in diagonal, *X*, *K*, *V* and Inverted *V* shapes were implemented in different numbers and locations.

Calculation of the maximum DCR_{moment} and the maximum DCR_{shear} of beams and columns, after the removal of column C in case 1 are shown in Table 4. The obtained results of CBF buildings showed that most of

Table 4 Maximum DCR_{moment} and Maximum DCR_{shear} for concentric and eccentric bracing in two bays (exterior North-South frame)

Frame type	Braced type	RC	2D+0.5L (beam)				2D+0.5L (column)			
			DCR$_m$	S/S	DCR$_s$	S/S	DCR$_m$	C/S	DCR$_s$	C/S
CBF	Diagonal	A	3.185	A-B/2	0.742	A-B/2	2.152	B/4	0.087	B/10
		C	2.530	C-D/1	0.399	C-D/2	0.935	E/1	0.043	F/10
	X Brace	A	3.157	A-B/2	0.738	A-B/2	1.824	B/5	0.109	B/10
		C	1.241	B-C/1	0.398	C-D/2	0.956	B/2	0.043	F/10
	K-Brace	A	3.147	A-B/2	0.736	A-B/2	1.553	B/5	0.109	B/10
		C	1.449	C-D/1	0.449	C-D/2	1.211	B/1	0.125	F/10
	V-Brace	A	3.205	A-B/2	0.748	A-B/2	2.193	B/5	0.114	B/10
		C	2.129	C-D/1	0.378	C-D/2	0.935	B/2	0.035	F/10
	Inverted *V*-Brace	A	3.182	A-B/2	0.746	A-B/2	2.471	B/4	0.115	B/10
		C	1.706	B-C/1	0.475	C-D/2	1.493	B/2	0.048	F/10
EBF	Diagonal	A	3.160	A-B/2	0.738	A-B/2	2.031	B/4	0.111	B/10
		C	1.371	C-D/1	0.827	C-D/2	1.080	B/4	0.059	F/10
	K-Brace	A	3.099	A-B/2	0.727	A-B/2	1.202	A/2	0.127	B/10
		C	1.485	C-D/1	0.459	C-D/2	1.012	B/2	0.231	F/10
	V-Brace	A	3.216	A-B/2	0.747	A-B/2	1.772	B/5	0.113	B/10
		C	1.536	C-D/1	0.371	C-D/2	0.798	B/5	0.038	F/10
	Inverted V-Brace	A	3.209	A-B/2	0.739	A-B/2	2.136	B/4	0.114	B/10
		C	1.870	C-D/1	0.739	C-D/2	1.578	B/2	0.049	F/10

In all Tables, S/S and C/S stand for span/story and column/story, respectively. RC refers to removed column.

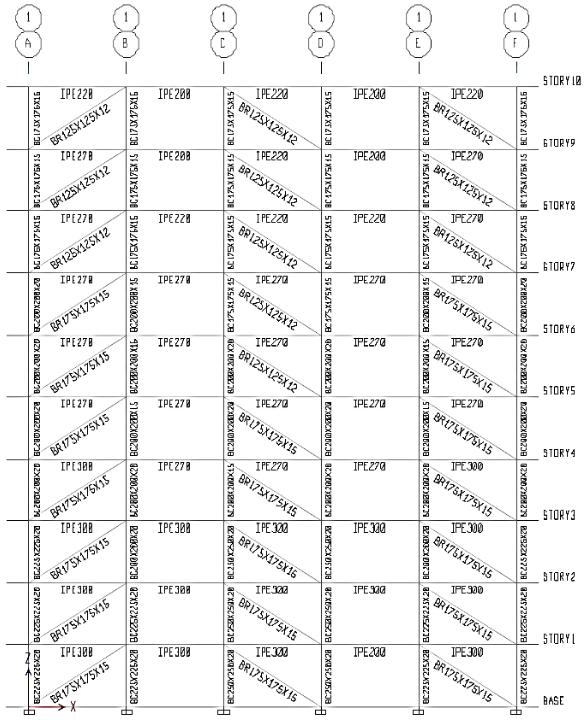
DCR_{moment} of beams exceeded the limitation (DCR>2) except for *X*, *K* and inverted *V*-braced systems. It should be mentioned that concentric braces were located in the exterior North-South frames of buildings, as depicted in Figs. 2 and 3. In this situation, diagonal braced system with DCR_{moment}=2.530 in case 1 labeled with *C* had highest risk against PC. Thus, the performance of CBF buildings wasn't effective in resisting PC except for *X*, *K* and inverted *V*-braced system based on the DCR_{moment} values. According to the maximum DCR_{moment} obtained from removing column *C*, all columns could satisfy GSA guidelines. After removal of column *A* in case 2, *V*-braced system with DCR_{moment}=3.205 had highest risk against PC in CBF buildings. In this situation, the DCR_{moment} values of columns for diagonal, *V* and inverted *V* braced systems were 2.152, 2.193 and 2.471, respectively. Moreover, Table 4 shows that there was no high risk of PC occurrence related to existing shear force after the removal of column (case 1 and case 2).

Table 5 Maximum DCR_{moment} and Maximum DCR_{shear} for concentric and eccentric bracing in three bays (exterior East-West frame)

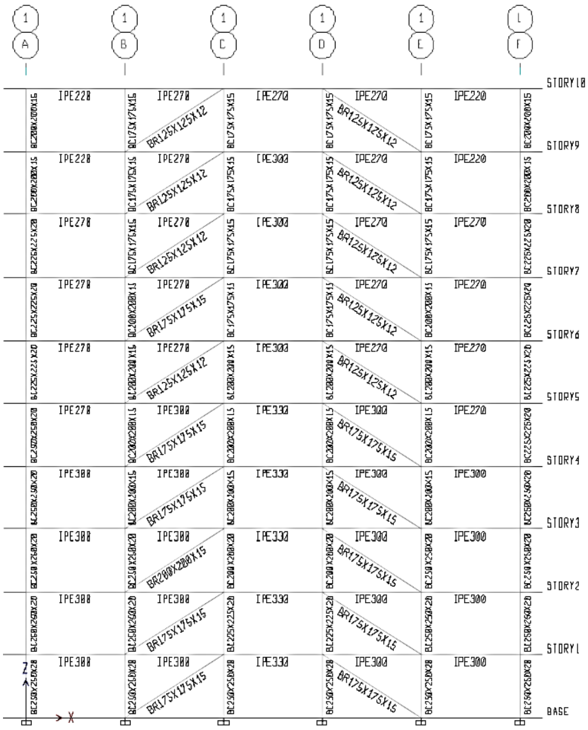
Frame type	Braced type	R.C	2D+0.5L (beam)				2D+0.5L (column)			
			DCR_m	S/S	DCRs	S/S	DCR_m	S/S	DCRs	S/S
CBF	Diagonal	A	2.169	A-B/1	0.741	A-B/2	1.371	B/3	0.033	B/10
		C	1.771	B-C/1	0.424	C-D/2	1.063	D/1	0.040	F/10
	X Brace	A	1.144	A-B/1	0.366	A-B/1	0.996	B/2	0.032	B/10
		C	1.179	D-E/1	0.384	C-D/2	1.129	D/1	0.032	F/10
	K-Brace	A	1.858	A-B/1	0.383	A-B/2	1.211	B/2	0.028	B/10
		C	1.619	B-C/1	0.437	C-D/2	1.247	D/1	0.039	F/10
	V-Brace	A	1.468	A-B/1	0.375	A-B/2	1.021	B/3	0.024	B/10
		C	1.950	B-C/1	0.380	C-D/2	1.079	D/4	0.031	F/10
	Inverted *V*-Brace	A	1.349	A-B/1	0.423	A-B/2	1.402	B/2	0.032	B/10
		C	1.386	C-D/1	0.459	C-D/2	1.351	D/2	0.046	F/10
EBF	Diagonal	A	2.029	A-B/1	0.987	A-B/2	1.570	B/3	0.056	B/10
		C	1.839	B-C/1	0.887	C-D/1	1.074	D/3	0.066	F/10
	K-Brace	A	1.620	A-B/1	0.390	A-B/1	1.257	B/2	0.232	B/10
		C	1.597	B-C/1	0.483	C-D/2	1.251	D/2	0.306	F/10
	V-Brace	A	0.852	A-B/1	0.347	A-B/2	0.893	B/3	0.024	B/10
		C	1.894	B-C/1	0.398	C-D/2	0.936	D/4	0.024	F/10
	Inverted *V*-Brace	A	1.583	A-B/1	0.634	A-B/1	1.264	B/2	0.028	B/10
		C	1.626	B-C/1	0.960	C-D/2	1.449	D/3	0.051	F/10

Overall, all beams and columns could satisfy GSA guidelines.

By considering the eccentric braces located in the exterior North-South frames of buildings, the results showed that all the DCR_{moment} values of beams in case 2 exceed the limitation (DCR>2) where column *A* located in corner of buildings was removed. In this situation, *V*-braced system with DCR_{moment}=3.216 in case 2 had highest risk against PC. Meanwhile, all DCR_{moment} values of beams were less than 2 after the removal of the column *C* or Case 1. In addition, after the removal of column *A* in case 2, the DCR_{moment} values of columns showed the EBF buildings could resist PC except for diagonal and inverted *V*-braced system. The maximum DCR_{moment} values for columns of diagonal and inverted *V*-braced systems were DCR_{moment}=2.031 and 2.136, respectively. Whereas, the buildings were able to resist PC after the removal of column *C*. Table 4 shows there was no high risk of PC occurrence related to existing shear force after the removal of column (case 1 and case 2).



(a) East-West elevation



(b) North-South elevation

Fig. 5 Section labels for the concentric braced frames

Table 5 shows the maximum DCR_{moment} and the maximum DCR_{shear} of beams and columns where concentric and eccentric braces were implemented in the exterior East-West frames of buildings as depicted in Figs. 2 and 4. After the removal of column *A* in case 2, the DCR_{moment} values of beams showed CBF buildings with diagonal braced frame system cannot resist PC. As well as, the DCR_{moment} values of beams were less than 2 after the removal of the column *C* in

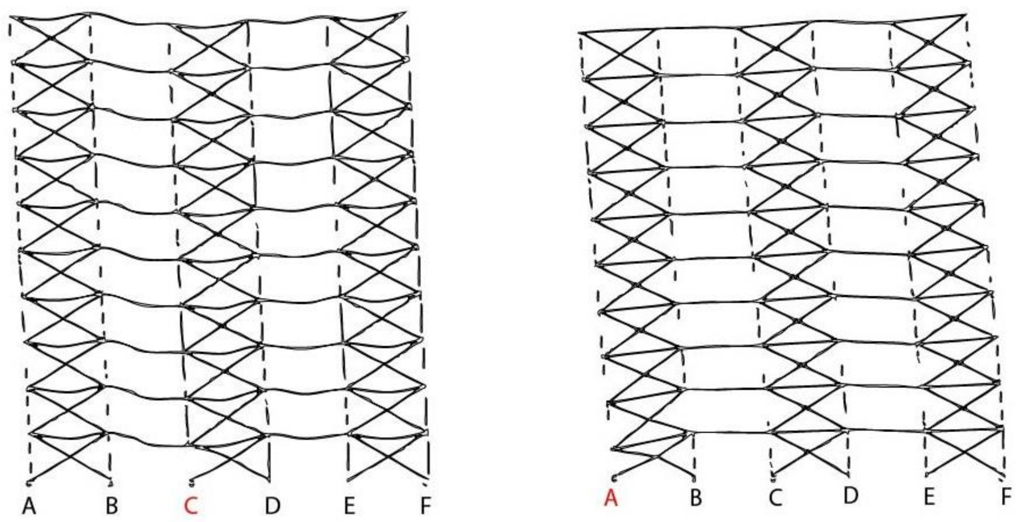


Fig. 6 Deformed shapes for *X*-braced frame system after the removal of column (case *C* and case *A*)

case 1. Thus, it led to resisting the CBF buildings against PC. In *X*-brace system, the maximum DCR_{moment} values of beams were 1.179 and 1.144 related to case 1 and 2 respectively, as shown in Fig. 5. It meant that *X*-brace system had lowest risk against PC. According to the calculated DCR_{moment} values, the behavior of columns against PC occurrence was better than beams. Also, there was no high risk of PC occurrence related to existing shear force after the removal of column (case 1 and case 2). After the removal of column *A* in case 2, the DCR_{moment} values of beams showed EBF buildings with diagonal-braced frame system cannot resist. In addition to, the DCR_{moment} values of beams were less than 2 after the removal of the column *C* in case 1. Overall, *K* and inverted *V*-braced frame system had lowest risk against PC in both cases. According to the calculated DCR_{moment} values, the behavior of columns against PC occurrence was better than beams. Also, there was no high risk of PC occurrence related to existing shear force after the removal of column (case 1 and case 2).

Fig. 6 shows the deformed shapes for *X*-braced frame system located in exterior East-West frames applying APM (cases 1 and 2).

4. Conclusions

The aim of this study was to investigate the performance of some steel structures with dual frame systems against PC. For this purpose, the CBF and EBF buildings were designed to investigate the influence of the different types of braces, numbers and locations of braced bays against PC. The exterior North-South frames were considered with two braced bays while the exterior East-West frames had three braced bays. In this study, Iranian code of practice for seismic resistant design of buildings was used for designing buildings. Also, the vulnerability of buildings to PC according to GSA guidelines was assessed.

The general conclusions of this study are as follows:

1. Buildings with two braced bays had higher vulnerability to PC than the buildings with three braced bays especially in the situation that the corner column was removed (most beams exceed the allowable limitation).
2. Comparison between the different bracing systems showed that the CBF buildings with three braced bays, *X*-braced and inverted V-braced frames had lower DCR values (DCR_{shear} and DCR_{moment}) led to higher resistance against PC.
3. The beams were more critical than columns in case of PC occurrence. It means that, when the building was designed to resist PC, it was better to use stronger beams.
4. In the dual frame system where the concentric braced were designed for lateral load, the resistance of structure against PC was comparatively greater than eccentric braced.
5. By increasing the number of stories (from 1st to 10th story) of the buildings, the capacity of the structures to resist PC increased or the DCR_{moment} values of top stories decreased.

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