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Progressive collapse analysis of two existing steel buildings using a linear static procedure

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Abstract. In this study, the vulnerability of two existing asymmetric steel building frames to Progressive Collapse (PC) is assessed. The buildings have different frame systems, steel sections and number of stories (nine and six). An alternate path method (APM) with a linear static analysis (LS) is carried out according to General Services Administration (GSA) 2003 guidelines. The Demand Capacity Ratio (DCR) of each primary element (beams and columns) is given with its specific details for all frames. The results show that the nine-story building with a dual frame system (moment frame with bracing system) has a lower susceptibility and greater resistance to PC than the six-story building with a simple building frame system (gravity system with bracing system). Implementing built-up box-shaped sections for columns is a better choice than using built-up I-shaped sections because there is no weak axis for the box section.

Keywords: Alternate Path Method (APM); deflection; Demand Capacity Ratio (DCR); Linear Static Analysis (LS); Progressive Collapse (PC)

1. Introduction

The Progressive Collapse (PC) of structures commences when a primary component or components, usually one or more columns, is eliminated. When a column is suddenly removed as a result of a vehicle collision, explosion, terrorist attack, earthquake or other natural or artificial hazard, gravity loads (both dead loads and live loads) are transmitted to adjoining columns in the structure. If these primary elements are not appropriately designed to bear and redistribute the overloading, that portion of the structure or even the entire structure may collapse. The columns of a building persist in failing until the extra loading on the columns becomes steady. Consequently, a significant portion of the building may fall down because of the greater and more fundamental damage to the building beyond that caused by the preliminary impact. The progress of consecutive damage during the PC that occurred in the Alfred P Murrah building in Oklahoma City in 1995 resulted in 168 fatalities. The loss of life that has resulted from increasingly frequent terrorist attacks, such as the nearly 3000 lives lost in the September 2001 World Trade Center case, has led

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to the development of new guidelines for assessing and preventing PC. These include GSA (2003) and UFC (2010).

To decrease destructive events in buildings, the National Institute of Standards and Technology (NIST) (2007) has published the following list of potential load hazards that can trigger PC: accidental events (airplane crashes, car crashes, etc.), errors in the design and/or construction process, fire accidents, violent and harsh changes in air pressure (explosions), accidental overloads, explosions caused by bombs, vehicular collisions, and hazardous materials.

This study aims to investigate the PC potential of two existing asymmetric steel frame systems by considering different numbers of stories and various steel sections. The results are obtained from the point of structure vulnerability to PC using the Alternate Path Method (APM) and analyzed by a linear static procedure based on GSA 2003 guidelines. In cases of the buildings failing due to PC, they are rehabilitated and the appropriate recommendations for preventing PC are presented.

2. Related works

While many investigations have been carried out on reinforced concrete structures, fewer have been performed on steel structures, especially on dual frame systems (moment frame with bracing system).The researches on the PC resistance of steel framed buildings are gradually increasing with the improvements on steel material, technology and method particularly in the developed countries. Song *et al*. (2010) investigated PC experimentally and through computational analysis of two buildings, the Ohio union building and the Bankers Life and Casualty Company (BLCC) building, based on linear static analyses of both buildings. The results showed that the columns in the top story were under pressure from their own weight more than the other columns as a result of a loss of columns. That problem was associated with a smaller cross section and lower moment of inertia. Song *et al*. (2010) concluded that the Ohio union state building could satisfy the GSA PC criteria for all frame members. Only five columns failed in the Ohio union building; in contrast, the BLCC building may not be able to satisfy guidelines proposed by the GSA criterion after removal of even the first columns. Calculation of the Demand Capacity Ratio (DCR) and the maximum displacement showed that the buildings were most susceptible to PC after the removal of the last columns and also showed that the beams were more critical in withstanding impact loads than columns. Kim and Kim (2009) conducted research focused on the analysis of the collapse process in buildings constructed of steel moment frames through a scientific consideration of seismic connections. The particular variables in this study included resisting capacities against PC, such as RBS (reduced beam section), WUF-W (welded unreal forced flange–welded web connection) and WCPF (welded cover plated flange). The authors compared two types of buildings constructed using steel moment frames. One building type was for high seismic load, while the other was for medium-level seismic load. The study led to the conclusion that the most effective element in counteracting PC was the cover plate connection, especially for medium level seismic sites. Khandelwal *et al*. (2009) performed research evaluating the PC of steel braced fames using models based on validated computational simulation procedures applying the APM. They concluded that a frame that was braced eccentrically was more resistant to PC than one that was braced concentrically in a ten-story building. Sadek *et al*. (2009) studied the behavior of steel beam column structures based on two types of moment-resistant connections. They applied a significant amount of load under displacement control up to the level that led to connection failure.

The main goal of this study was to define the behavior of the connections, including the ability of the connections to resist the tensile forces occurring in beams. They found a significant agreement between their experimental and simulation-based research methodologies. Sadek *et al*. (2010) conducted a study comprising two experimental and computational methodologies relating to two steel framed structures that included three columns and two beams. This study was performed on two ten-story buildings that were designed to eliminate the probability of PC. They eliminated the beam–column assemblies from the exterior frames. The first test specimen consisted of connections with welded, unreinforced flange–bolted web while the second specimen consisted of connections equipped with reduced beam sections. The results of this study showed that the rotational capacities for both connections were twice as large as the values achieved from the seismic test. Khandelwal *et al*. (2008) developed some scientific models for evaluating the resistance efficiency of steel framed buildings against PC. They found a higher level of resistance among frames specified for high seismic loads than among those designed for moderate seismic loads by evaluation with the APM.

Olamti *et al*. (2013) focused on evaluating buildings subjected to explosions. They performed robustness evaluation of steel frame structures provided for computing robustness curves. Analyses showed that the obtained robustness curves provided a suitable tool that can be applied for risk management and assessment purposes. Hoffman *et al*. (2011) applied the computational study of column loss situations for typical multi-story steel buildings with perimeter moment frames and composite steel-concrete floors. Overall, the steel buildings that were assessed in this study showed the appreciable robustness.

3. Methodology

3.1 Flowchart approach to assessing the PC potential

Fig. 1 describes the procedure for assessing PC using an APM based on linear static analysis.

3.2 Methods for preventing PC

Researchers have proposed three methods for reducing the probability of disproportionate collapse in buildings: an APM, improved local resistance for critical components, 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 a suitable means for evaluating and preventing the process of PC in buildings of up to ten stories (low to medium rise). Thus, an 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.

3.3 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.

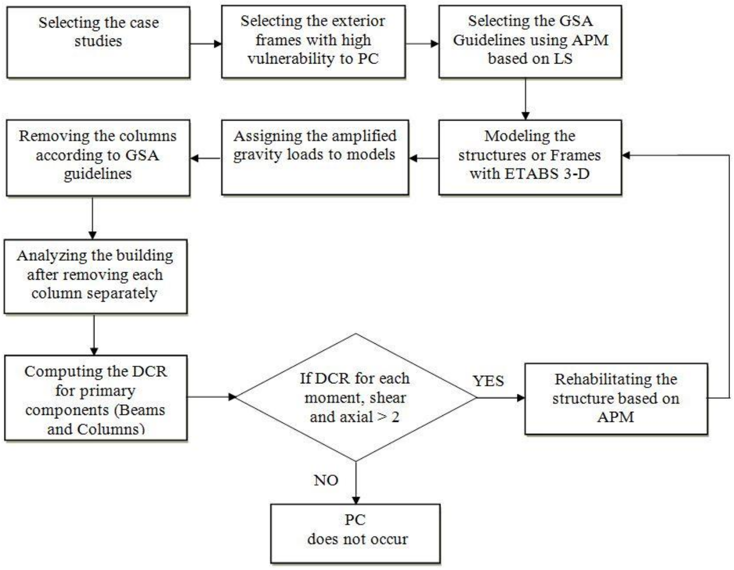


Fig. 1 Flowchart for processes carried out in assessing PC

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

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

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

3.4 Calculation of the DCR

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

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

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:
DCR < 2.0: for a typical structural configuration
DCR < 1.5: for an atypical structural configuration
Cases that have been chosen for this study have a typical structural configuration.

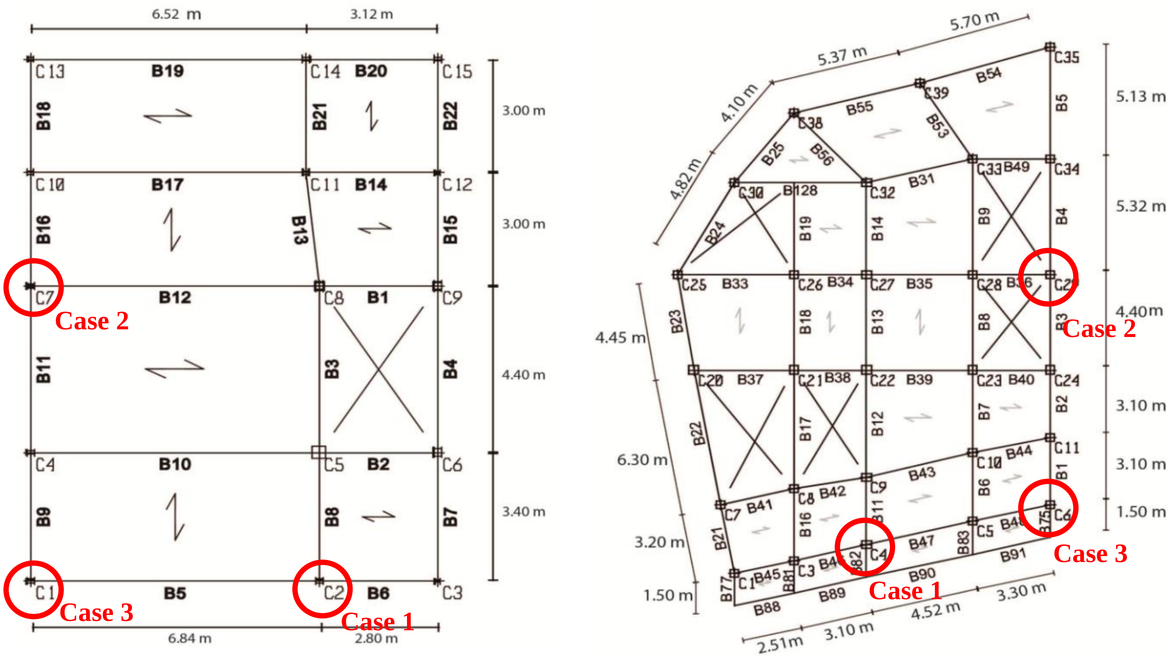
It should be mentioned that the loading pattern used in this study was based on gravity alone (amplified dead and live loads), so computation of the DCR values for braces was omitted.

4. Numerical examples

4.1 Description of buildings

Two case studies were selected for this study. Building A had six stories while building B had nine stories. Both buildings were located in Iran. The first story plans of these buildings are depicted in Fig. 2. In addition, detailed information of these buildings is shown in Table 1.

The steel section designations for the short and long sides of the six- and nine-story steel buildings (exterior frame, beside the road) are shown in Figs. 3 and 4, respectively.



(a) The six-story building

(b) The nine-story building

Fig. 2 The first story plan of the buildings

Table 1 Detailed information for the nine and six-story buildings

Title	Nine-story building	Six-story building
Geometry type	Asymmetric	Asymmetric
Structural system	Dual frame system (Moment frame with bracing system)	Simple building frame system (Gravity frame with bracing system)
Structural system against earthquake	Bracing system100%+moment frame30%	Only bracing system
Code	AISC-ASD 89	AISC-ASD 89
No. of span in X and Y directions	4-bay & 6-bay	2-bay & 4-bay
Type of roof	In-situ concrete slab	In-situ concrete slab

Table 1 Continued

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	
Connection	Column to column	Continuous between the two story levels	Continuous between the two story levels
	Beam to column	Rigid	Pinned
	Braces	Pinned	Pinned
Loading	LL & DL for floors	500 Kg/m^2 and 300 Kg/m^2	200 Kg/m^2 and 370 Kg/m^2
	DL of surrounding wall	800 Kg/m^2	1420 Kg/m^2
	DL of stair box in X direction	2000 Kg/m^2	1420 Kg/m^2
	LL & DL for roof	150 Kg/m^2 and 300 Kg/m^2	350 Kg/m^2 and 320 Kg/m^2

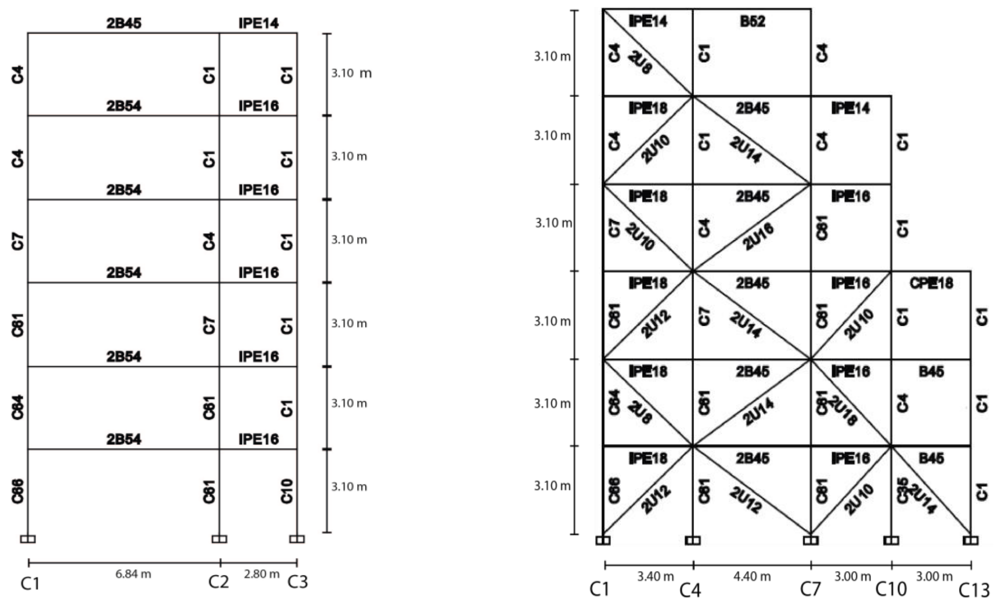


Fig. 3 Section labels for the two sides of the six-story building

The six-story building sections consisted of a built-up I-shaped section, IPE sections, a built-up I-shaped section with welded plates, a double I-shaped section, and a double IPE-section with welded plates. Braces made up double channel sections and were labeled with the letter "U".

The nine-story building sections consisted of a built-up box-shaped section for columns and the built-up I-shaped section designated as PG in Fig. 4. In addition, braces made up double channel sections and were labeled with the letter "U".

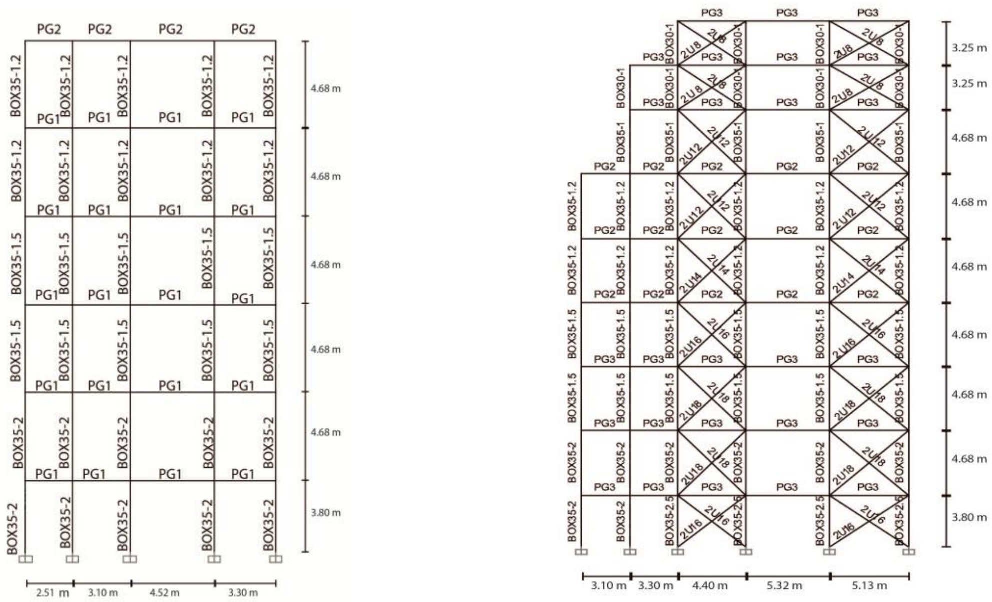


Fig. 4 Section labels for the two sides of the nine-story building

4.2 Selecting the columns for removal

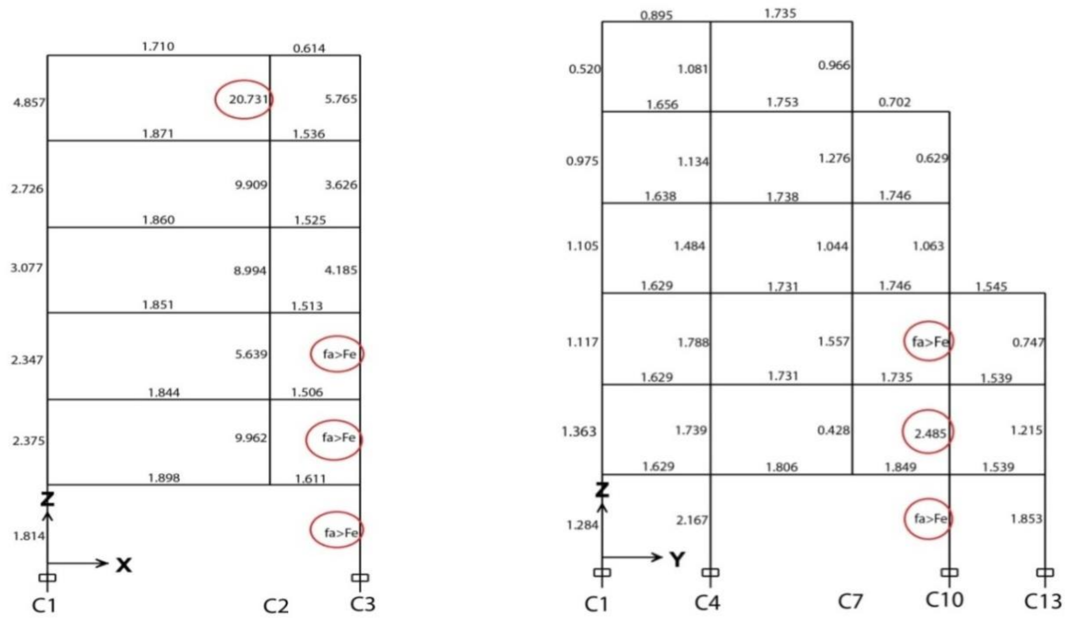
To calculate the DCR value according to the GSA guidelines, the first step is analyzing the sudden removal of a column one floor above the ground (1st story) located at or near the middle of the short side and long side of the building, respectively. This situation was assessed in case 1 and case 2, as shown in Fig. 2. The second step is analyzing the sudden removal of first story column located at the corner of the building. This situation was assessed in case 3, as shown in Fig. 2. The analysis results and the DCR values for beams and columns were calculated, and the vulnerability to PC of the two existing buildings with different frame systems was assessed.

5. Results and analysis

5.1 DCR for moment in a six-story building

In case 1 as shown in Fig. 2, the DCR_{moment} when the middle column C2 was eliminated was greater than 2, with the maximum DCR_{moment}=20.731. This meant that the structure had a high PC potential as depicted in Fig. 5(a). It should be mentioned that f_a (the computed axial stress) is greater than F_e (the allowable Euler stress), which shows that the structure is not able to tolerate the additional axial force that could be created as a result of an accidental overload.

In case 2 as shown in Fig. 2, the DCR_{moment} was less than 2 for all elements, although two columns could not resist the existing axial force, which meant that $f_a>F_e$. The overall behavior of this case was better than that of case 1 while there was no bracing system, as shown in Fig. 5(b).



(a) Case 1 (b) Case 2

Fig. 5 DCR_{moment} in six-story building

In case 3, as shown in Fig. 2, when column C1 from the short side was eliminated, the DCR's flexure was less than 2, showing that PC could not happen in this case, as shown in Fig. 6(a).

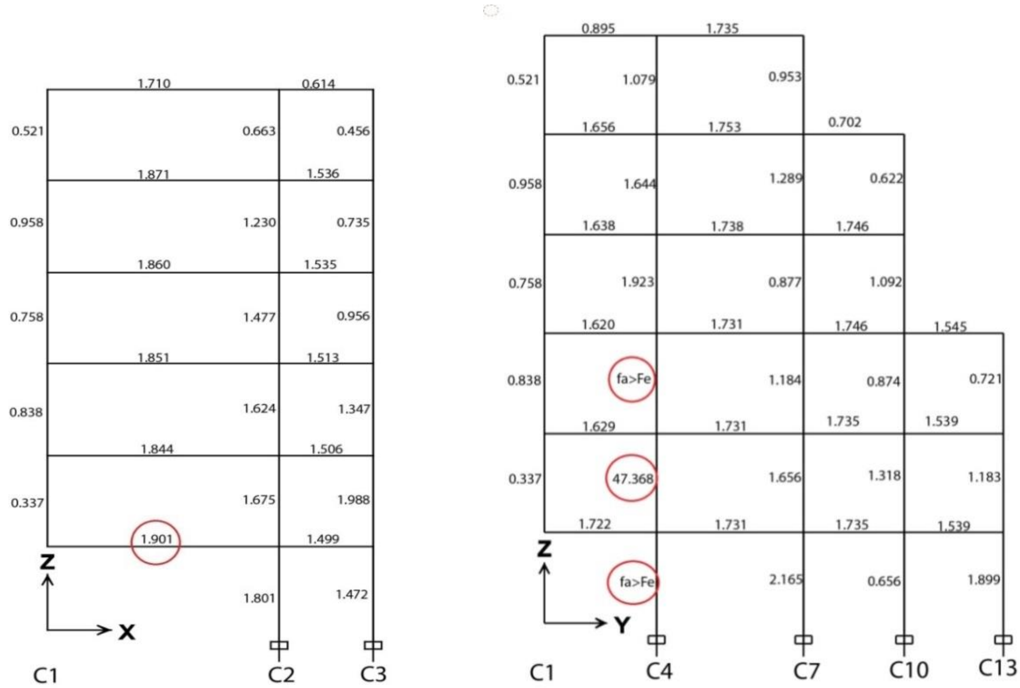
Fig. 6(b) shows that the DCR's flexure for the long side when column C1 was eliminated reached the very high value of 47.368, which was well above 2. After assessing DCR_{moment} in this frame, it was realized that this frame had the worst behavior compared to the other frames and also had a very high susceptibility to PC in the case of the sudden removal of a column.

5.2 DCR for shear in six-story building

Calculation of the DCR_{shear} after removal of column C2 in case 1, as shown in Fig. 2, indicated that PC would not occur, as shown in Fig. 7(a). In case 2, as shown in Fig. 2, the DCR_{shear} when column C7 was eliminated was less than 1.330, which indicated that the building had sufficient resistance against PC, as shown in Fig. 7(b). In case 3, as shown in Fig. 2, the DCR_{shear} for the short side when column C1 eliminated was less than 2 for all members as shown in Fig. 8a. In this case, the DCR_{shear} for the long side is represented in Fig. 8(b).

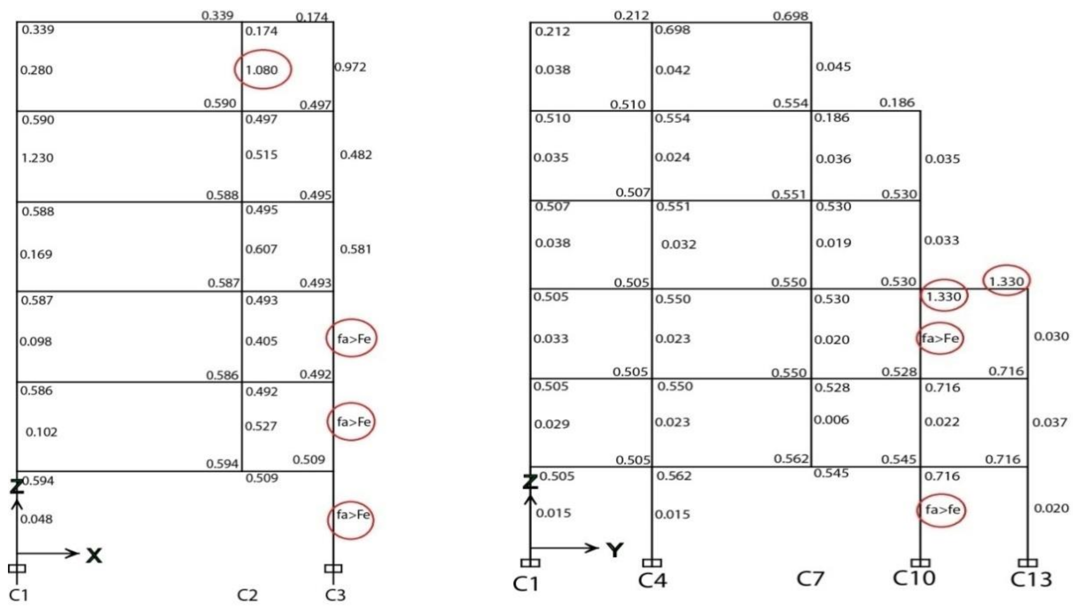
5.3 Maximum DCR and maximum deflection in six and nine-story buildings

The maximum DCR and maximum deflection of six and nine-story buildings, after the removal of columns are shown in Tables 2 and 3, respectively.



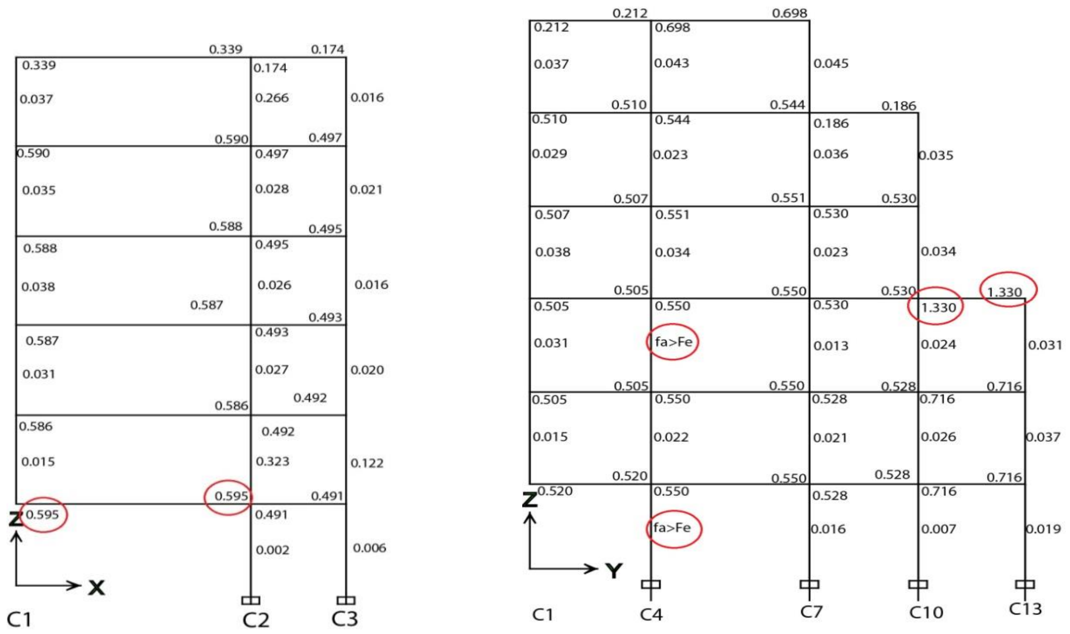
(a) Short side (b) Long side

Fig. 6 DCR_{moment} in six-story building (Case 3)



(a) Case 1 (b) Case 2

Fig. 7 DCR_{shear} in six-story building



(a) Short side (b) Long side

Fig. 8 DCR_{shear} in six-story building (Case 3)

Table 2 Maximum DCR and maximum deflection for six-story building after the removal of columns, based on GSA guidelines

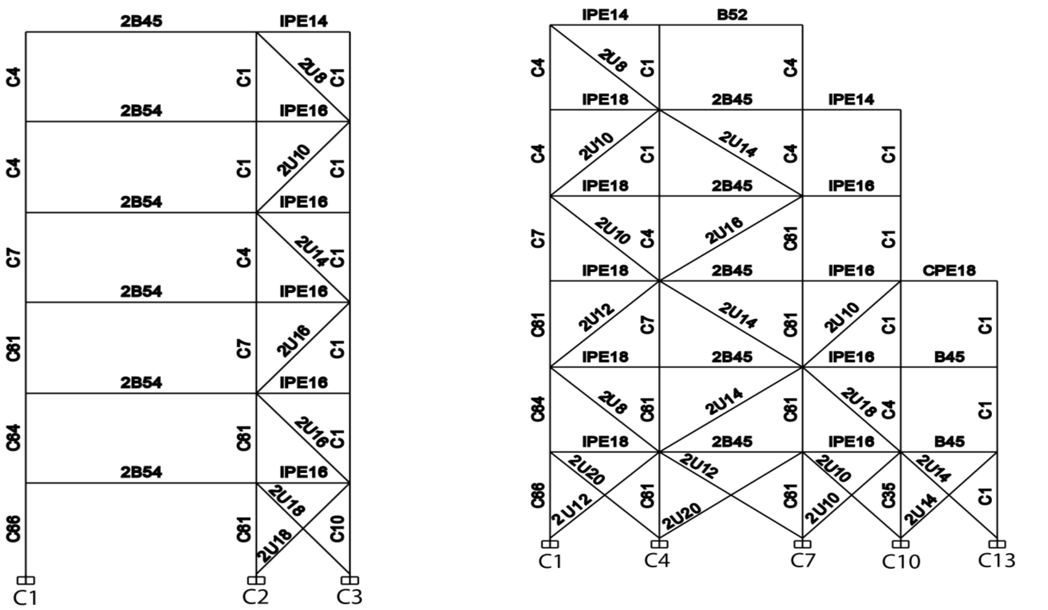
Title	DCR_{moment}	DCR_{shear}	Deflection for Beam (m)	Deflection for Column (m)
Middle of Short Side	20.731	1.080	0.045	0.1144
Middle of Long Side	2.485	1.330	0.018	0.0173
Corner of Short Side	1.988	0.595	0.045	0.0366
Corner of Long Side	47.368	1.330	0.017	0.0366

Table 3 Maximum DCR and maximum deflection for the nine-story building after the removal of columns, based on GSA guidelines

Title	DCR_{moment}	DCR_{shear}	Deflection for Beam (m)	Deflection for Column (m)
Middle of Short Side	1.087	0.511	0.008	0.0364
Middle of Long Side	0.903	0.566	0.005	0.0366
Corner of Short Side	0.790	0.382	0.007	0.0366
Corner of Long Side	1.058	0.567	0.007	0.0366

In Table 3, all DCR values for moment and shear were less than 2, which meant that this building had very low vulnerability to PC and a lower deflection than the six-story building.

The DCR values of the six-story building that were greater than 2 were related to the small cross-sectional areas of the columns; thus, the building should be rehabilitated to fix that problem.



(a) (b)

Fig. 9 Short side (X direction) and long side (Y direction) elevation of the six-story building after rehabilitation

Table 4 Maximum DCR and deflection for a six-story building after rehabilitation and removal of the columns, based on GSA guidelines

Title	DCR_{moment}	DCR_{shear}	Deflection (Beam-m)	Deflection (Column-m)
Middle of Short Side	1.954	0.594	0.045	0.0240
Middle of Long Side	1.883	1.330	0.018	0.0169
Corner of Short Side	1.901	0.595	0. 045	0.0153
Corner of Long Side	1.967	1.330	0.017	0.0153

One of the methods for rehabilitating structures with slender columns against PC is to add braces to the frames. The braces cause the forces to be redistributed among the elements when forces are produced by accidental overload.

5.4 Rehabilitation process for a six-story building

In this study, the building was rehabilitated as follows:

The exterior frame had a gravity frame without a bracing system in its short side only. By adding X braces on the first floor and diagonal braces on the other floors, the exterior frame was rehabilitated. In this way, abnormal forces in some of the members were transferred to other members. This action also solved the “slender column” problem. The steel sections were double channel, as detailed in Fig. 9(a). It can be observed from Fig. 9(b) that diagonal braces in the first floor were reinforced by introducing additional diagonal braces to each of the existing diagonal braces, thereby forming a cross-bracing system. The steel cross sections were double channel, as shown in Fig. 9(b).

All calculated DCR values were less than 2. It was thus concluded that the structure had a low potential for PC. It should be mentioned that by rehabilitating the frame by adding braces, there was no effect on any beams' DCR or deflections, but the DCRs and deflections for the columns were lower, leading to lower computed axial stresses (f_a was lower than F_e). This means PC was prevented in this model. The maximum DCR and maximum deflection are shown in Table 4.

5.5 Recommendation for minimizing the vulnerability of the six–story building to PC

The vulnerability of the six-story building to PC was minimized by doing the column rotation and by implementation of V-braces in the first floor of the exterior frame along the road.

The columns which were built-up double I-shaped section with plates, and double IPE sections were bent around their weak axes (in this case, the long side). Later the columns of the exterior frame of the long side were rotated 90 degrees, and the bending of the column happened around the strong axes (flange). Thus, the DCR values have been calculated and minimized particularly along the long side. The maximum DCR and maximum deflection are shown in Table 5.

Table 5 Maximum DCR and maximum deflection for a six-story building after rotating the columns by 90 degrees

Title	DCR_{moment}	DCR_{shear}	Deflection for Beam (m)	Deflection for Column (m)
Middle of Short Side	20.731	1.080	0.045	0.1144
Middle of Long Side	2.643	1.330	0.018	0.0173
Corner of Short Side	1.986	0.595	0.045	0.0366
Corner of Long Side	10.335	1.330	0.017	0.0366

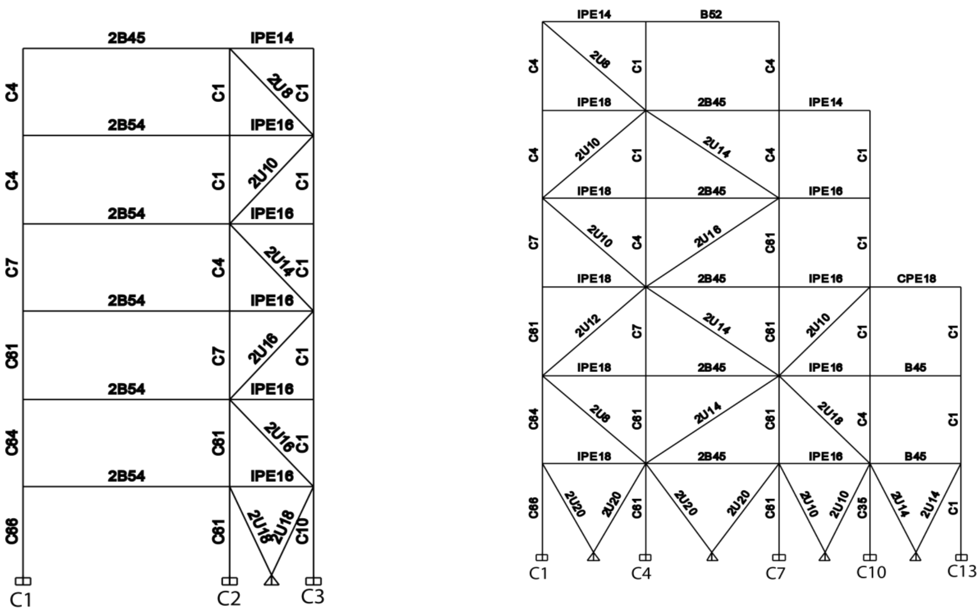


Fig. 10 V-Braced frames implemented on the first floor of the short and long sides

Table 6 Maximum DCR and maximum deflection for a six-story building after implementing the V-bracing system on the first floor based on GSA guidelines

Title	DCR_{moment}	DCR_{shear}	Deflection for Beam (m)	Deflection for Column (m)
Middle of Short Side	1.911	0.594	0.045	0.0216
Middle of Long Side	1.849	1.330	0.018	0.0168
Corner of Short Side	1.901	0.595	0. 045	0.0151
Corner of Long Side	1.753	1.330	0.017	0.0151

Secondly, by implementing V-braced frame on the first floor on both sides of the building as depicted in Fig. 10, the resistance of the structure to PC was increased in comparison with the X-braced system first floor rehabilitation model because it had support against the ground.

The calculation of the DCR for moment and shear after the removal of columns was conducted for the case in which V-braces were implemented on both sides of the first floor. All DCR values were less than 2 and f_a was lower than F_e; in addition, the DCR values of moment in this case were less than the DCR values of the first rehabilitated model represented in Table 4. Maximum DCR and maximum deflection are shown in Table 6. As a result, a smaller cross-sectional area was used in this case, which indicated that an applied V-brace system is more economical than an X-brace system for first floor.

6. Conclusions

By considering the structural system, it appeared that a dual frame system had a lower vulnerability to PC than a simple building frame system. This also means that using rigid beam-column connections or a moment frame system in a steel frame was better for the resistance of buildings to PC. The six-story building was faced with possible PC; the problem was solved by inserting braces into the framing system. The behavior of a dual frame system in terms of PC was much better than that of a simple building frame system even after rehabilitation. The better performance of the nine-story structure was due to the difference in the structural system, the type of column sections and the differences in the bracing systems which were installed on the selected exterior frames. Meanwhile, it should be indicated that the bracing system of the nine-story building was an X bracing system, while in the six-story building a diagonal bracing system was used. The prominent outcomes of this study were:

- In the dual-frame system, the resistance of the structure to PC was comparatively much greater and more effective than a simple building frame system.
- In the columns, usage and implementation of built-up, box-shaped sections (square boxes), especially for the frames that are exposed to exterior or interior damage, resulted in greater resistance to PC when compared with the built-up I-shaped sections, IPE section and its combinations and derivatives.
- The built-up I-shaped columns that were positioned on the periphery of the structure should be disposed and located in the direction that the bending of the column occurred around the strong axis (flange); in other words, the moment pivoted around strong axis.

When PC or similar subjects are considered it is better not to use gravity frames with bracing systems. In other words, if it is used in structures it is recommended not to implement diagonal bracing systems for the first floor above the ground. The X-bracing system or inverted V-bracing systems are more appropriate to be used in these locations, and a V-bracing system could be significantly better than an X-bracing system because it has support against the ground.

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