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Effects of Structural Bracing on the Progressive Collapse Occurrence

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Abstract

Statistics of human losses and financial casualties induced progressive collapse, as one of the new and modern concepts in the field of civil engineering, have doubled the importance of having knowledge about this phenomenon and strategies to reduce its effect. Progressive collapse starts with a local failure with loss of local load-carrying capacity of a small portion of the structure and spreads throughout the structure from element to element. These consecutive failures may cause the collapse of either the entire structure or a major part of it. This paper studies the effect of adding a bracing system to the steel moment frames designed for seismic loads through a nonlinear dynamic method according to GSA-2003 and UFC-4-023-03 criteria. The study was conducted using computational simulation of building models with two different elevations of three and six floors located in a moderate seismicity region. The simulation results showed higher resistance against the progressive collapse of the structure in the braced steel moment frames and less sensitivity to the removal of the column in the braced spans in comparison to the spans without bracing. The prediction of possible progressive collapse in the UFC-4-023-03 criterion is more conservative than the GSA-2003 criterion. Although generally there is no significant difference between the analysis results of these two criteria.

Keywords

Progressive Collapse, Braced Steel Moment Frames, Steel Moment Frames, UFC-4-023-03, GSA-2003

1. Introduction

Engineering structures could be exposed to natural disasters such as earthquakes, hurricanes, storms, floodwater, fire, and man-made extreme hazardous events including explosions and clashes during operation time. Structures are conventionally designed for possible incidents that may occur during their lifetime. Nevertheless, severe events that are not considered in the design of a structure may lead to catastrophic failures such as progressive collapse (Ellingwood et al., 2007, Kim et al., 2009, Panahi et al., 2021). Nowadays, progressive collapse occurrence in structures during an earthquake or even an explosion near the structure is considered the main challenge. The progressive collapse has primarily been studied after the Ronan Point Apartment Tower collapse in 1968 in London and then, the World Trade Center collapse in 2001. The apprehensive statistics of human losses and financial casualties associated with the progressive collapse of the aforementioned structures made this issue an immensely colossal challenge in the structural engineering field (Griffiths et al., 1968, Gross and McGuire, 1983, Pearson and Delatte, 2003, Bažant and Verdure, 2007, Starossek, 2009).

The definition of progressive collapse is developed over time and by many different regulations such as the US General Services Administration (GSA, 2003), and the Department of Defense (UFC-3-340-02, 2008). It is defined in UFC-4-01-01 (2003) that "the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it". In simpler words, the progressive collapse commences with the removal of the local carrying loads of one load element or more of the structure and continues with the incidence of some failures in other elements of the structure continually that are not directly influenced by the primary local failure. Despite the comprehensive studies on the linear and nonlinear behavior of the steel moment resisting frames (MRF), concentrically braced frames (CBF), and eccentrically braced frames (EBF), as well as their connections, limited studies have been accomplished on the effects of progressive collapse on the structural responses of these systems and their corresponding connections (Starossek, 2007, Khandelwal et al., 2009, Azad and Topkaya, 2017).

In GSA (2003), the demand capacity ratios (DCR) for connections were defined to quantify the robustness of a structure. Powell (2005) conducted numerical analyses for two case studies including a steel-moment resisting frame and concrete slab building frame based on principles of progressive collapse in association with static and dynamic analysis methods. The results of this study showed that the dynamic multiplier =2 applied in the linear static analysis could efficiently simulate the actual nonlinear dynamic response of structures. Ruth et al. (2006) found that dynamic multiplier=1.5 shows the dynamic effect more accurately, especially in the case of steel moment frames. Kim and an (2009) studied the strength capacity against the progressive failure of steel moment frames through the suggested alternate path method (APM) in GSA and UFC criteria and observed that when a nonlinear dynamic analysis was chosen; it led to larger structural responses. Fu (2009) concluded that under the same general conditions, a column removal at an upper story would induce larger vertical displacement than a column loss at ground level. Kim and Kim (2009) investigated the progressive collapse-resisting capacity of steel moment frames using an alternate path method and compared nonlinear dynamic and linear static analysis. The results of this study demonstrated that the linear static analysis yields the more conservative estimation for progressive collapse. To quantitively assess the susceptibility of the structure to the incidence of progressive collapse, another robustness measure was introduced by Starossek and Haberland (2011) based on the static stiffness of the structure, damage multiplier, and energy-based measures. Due to the bearing capacity, Liang and Long (2012) defined the importance of coefficients of components to evaluate the structural robustness quantitatively against progressive collapse. Their new indicator captured the effects of different load types, span uniformity, and the number of spans in a concentrically braced system. Huo et al. (2012) adopted ductility indices for connections as the rates of ultimate rotation capacity under an impact to the rotation that the catenary action began to form. These ductility indices were implemented to assess the robustness of the proposed method. Asgarian and Rezvani (2012) introduced a new algorithm which can inspect the progressive collapse potential of buildings. This algorithm was utilized for progressive collapse analysis of two designed concentrically braced frames with different numbers and locations of braced bays. Kazemzadeh Azad et al. (2018) conducted research on eccentrically braced systems and examined the results of experimental and numerical studies on the strength, rotation capacity, and strength of link beams to propose reasonable response factors for practical design procedure of these systems. Tavakoli and Afrapoli (2018) illustrated that the robustness of steel braced structures with various lateral load resisting systems under seismic progressive collapse is more than steel moment frames.

Furthermore, some experimental investigations were conducted into the progressive collapse of steel frames (Song et al., 2014, Dinu et al., 2016, Li et al., 2017, 2018). Li et al. (2018) tested the progressive collapse of three two-story, four-bay planar steel frames under a column removal scenario. The results of this study include the collapse failure patterns, presentation of alternate load transfer path, and measurements of deformation and strains of remaining structural elements.

The above-mentioned studies have corroborated the importance of having knowledge about the progressive collapse phenomenon to mitigate the risks and hazards, and augmentation of emergency responses to ensure the overall stability of the remaining sections of the structure. According to the above literature review, the problem of progressive collapse of the steel moment resisting frames, concentrically braced frames, and eccentrically braced frames with their connections detail has been given full attention via different numerical, and experimental studies for a long time. However, by scrutinizing the literature studies on the progressive collapse of steel structures and their sub-categories, it can be observed that there seem to be few/if any published documents on the occurrence of progressive collapse of steel moment frames with bracing. In other words, the researcher limited their investigations only either to moment-resisting frames or braced frames. With the interest of gaining insights into this issue, a set of computational simulations of building models with two different elevations of three- and six-story frames located in a moderate seismicity region have been conducted to model the incidence of progressive collapse of steel moment braced frames. The steel moment braced frames were analyzed through a nonlinear dynamic method and designed according to GSA (2003) and UFC-4-023-03 (2009) criteria.

2. Design Methods to Confront Against Progressive Collapse

The design methods to confront the progressive collapse are divided into three general categories (UFC-4-023-03, 2009).

- Event Control Method is applied to prevent an event that causes progressive collapse. It's worth noting that the event control method is mainly based on protective issues, and it is not considered in the field of civil engineering.
- Indirect Design Method is based on improving the continuity of the connections in the nodal points of the structure and increasing the ductility and uncertainties of the system. In this method, after the occurrence of failures in some parts of the structure, the rest parts of the system provide enough strength for stability and improve the continuity of the structure. However, this method is not suggested to be designed against progressive collapse since the removal of the components or any unusual specific loading is not considered in this method. Furthermore, the requirements for ductility of the buildings and improving the structure's performance for strength against progressive collapse are specifically considered in ASCE7-05 (2005) and ACI-318-08 (2008).
- Direct Design Method considers the strength against progressive collapse directly in the design process through two methods:
  - ➢ The alternate path method (APM) is looking to provide an alternate path for load-carrying after the occurrence of failure to prevent the development of local damage and consequently prevent the whole collapse. The structure is designed in a way that if any component becomes destroyed, an alternate path for carrying the load of that component is available (GSA, 2003). For building structures, the alternate path method incorporates the event of a vertical element failure tuning the structure such that it can bridge over the failed element through the redistribution of the load to the remaining structure (Kwasniewski, 2010, Skordeli and Bisbos, 2010). Therefore, the critical elements of such structures mainly include columns or load-bearing wall elements (Izzuddin et al., 2008). The method employs three analysis procedures: linear static, nonlinear static, and nonlinear dynamic (Ellingwood et al., 2007).
  - ➢ In Enhanced Load Resistance Method (ELR) a specific structure component is required for strength against an unusual loading. This method requires enough ductility and strength to be considered for a specific component. Hence, designing the critical components with the increasing effect of design loading coefficients is conducted in a way that some extra strength for carrying unusual loads is provided (UFC-4-023-03, 2009). This method is based on the strengthening of some parts of the structure in which analyzing the alternate path is not possible.

3. Progressive Collapse Analytical Methods

In this study, the nonlinear dynamic method was used to analyze three- and six-story steel moment-braced frames. The non-linear dynamic method is based on GSA (2003) and UFC-4-023-03 (2009) criteria.
The load combination on the spans that are not directly placed above the removed component is as follows (GSA, 2003):

$$G_{SD} = [D + (0.25L)] \tag{1}$$

where D and L are the dead and live loads on the structure, respectively. This loading is defined in UFC criterion as follows.

$$G_{ND} = (0.9 \text{ OR } 1.2)\, D + (0.5L \text{ OR } 0.2S) \tag{2}$$

where GND is the increased gravity loads in the non-linear dynamic analysis, and the term S refers to snow load. Besides, a lateral force (LLAT) is applied in the combinations of gravity loads to the structure in the following form of Eq. (3).

$$L_{LAT} = 0.002\Sigma P \tag{3}$$

Where ΣP is the total gravity loads in the considered floor.
The applied load initiated from zero and increased steadily and proportionally to the whole of the structure. When the structure reached the balanced condition, the intended component was removed to simulate the progressive collapse of the steel moment braced frames.

4. Specifications of Structural Models

Two different structural systems including the steel moment-resisting frame and the dual system of the moment-resisting frame along with concentrically braced frame were utilized as structural systems of at hand problem. It is hypothesized that the buildings were located on soil type II according to UBC-97 (1997). The strain hardening and damping ratio of the buildings were considered as 2% and 5%, respectively. Furthermore, the buildings were designed with moderate ductility. The floor elevation in all the buildings was assumed the same at 3.2m and the structures' spans are 5m, 6m, and 7m as shown in Figure 1. After designing the structure according to the LRFD method, the cross-sections were grouped in defining the specifications of the structures. The BOX sections, IPE, and double-angle sections were applied for columns, beams, and bracing elements, respectively.

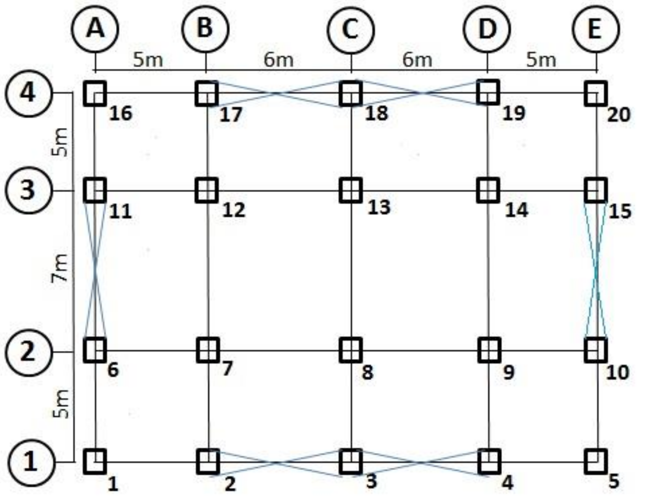


Figure 1. Plan of building with moment-resisting frames and moment-resisting braced frames.

The modeled buildings were analyzed and designed via SAP-2000 software. The non-linear dynamic analysis process was performed as follows: at first, the structure was analyzed without the removal of any component and with different loading patterns based on each regulation. Then, the target component was removed and internal forces including axial, lateral, and moment corresponding to the removed element were applied as external forces in opposite directions with respect to the original ones on the node placed above the target component. Thereafter, the time-history procedure of progressive collapse was implemented as schematically depicted in Figure 2. For more detailed descriptions of the calculation procedures of the progressive collapse simulation procedure, readers can be referred to the seminal studies of (Kim and Kim, 2009, Kim et al., 2009, 2011).

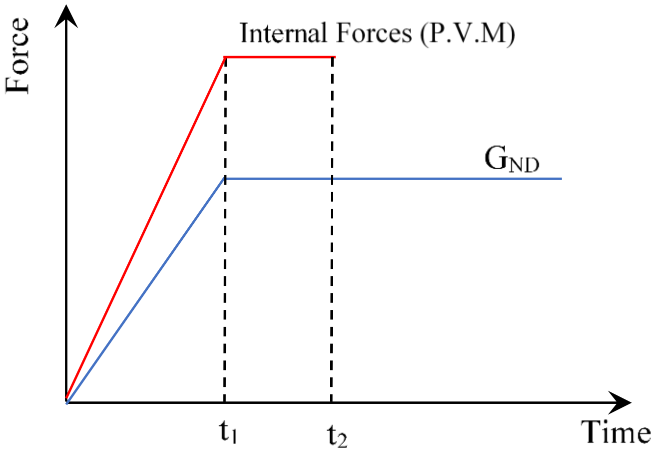


Figure 2. Schematic illustration of the applied load method in nonlinear dynamic analysis (Kim and Kim, 2009).

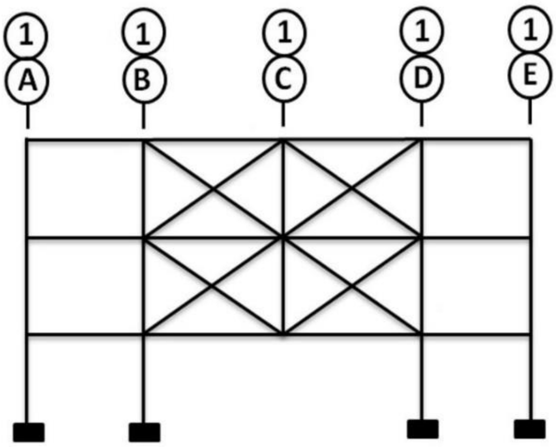
5. Progressive Collapse of Steel Moment Braced Frames

In this section, the effects of adding braced systems to the conventional steel moment-resisting frames were investigated. The effect of bracings on the potential of progressive collapse occurrence could be classified into two categories. The first category is related to the effect of the location of the bracing systems on the progressive collapse to find the optimal configuration, while the second category is attributed to the effect of adding the bracings despite their place of them on the progressive collapse potential. In this study, only the effects of adding the bracing systems regardless of their optimal locations were addressed. Through comparative analyses, the displacement responses as well as the formation of hinges in the steel moment-braced frames were compared with those of conventional steel-moment frames.

5.1 Critical Failure Pattern of Steel Moment Braced Frames against Progressive Collapse

As illustrated in Figure 1, the concentrically braced system was added to A and E frames between 2-3 spans and 1 and 4 frames between B-C and C-D spans. The bracing elements were modeled with double angle cross sections by application of automatic optimal search manner. The compound systems of concentrically braced moment frames were designed in two balance levels for three-, and six-story building frames. Thereafter, the models were evaluated against progressive collapse after the removal of target elements. These two balanced levels for the simulation of the progressive collapse of steel-braced frames were defined in two different states. In state 1 (depicted in Figure 3.a), the target column and its surrounding bracings were removed simultaneously, while in state 2 (Figure 3.b), only the sole action of the target column was removed. The occurrences of these two probable states were mainly reported in the literature (Kim and An, 2009, Kim and Kim, 2009). To find the critical failure pattern among these two states, a comparison was made between the results of nodal displacements-time diagrams. Then the critical state was selected and utilized for further analyses.

Figure 4 demonstrates the results of nodal displacement-time diagrams of the node placed above columns 3 and 6 on the first floor for both removal and failure states. These diagrams were reported for the case of 6-story building frames which were designed based on UFC-4-023-03 (2009). Similar observations can be made by the application of GSA (2003), different story levels, and building frames in different directions. It was observed that state 1 in which the bracings are removed with the target column renders more displacement in the nodes above the removed column, meaning that state 1 is more critical. For the sake of brevity, the further analyses in the following sections were reported only for the case of state 1.



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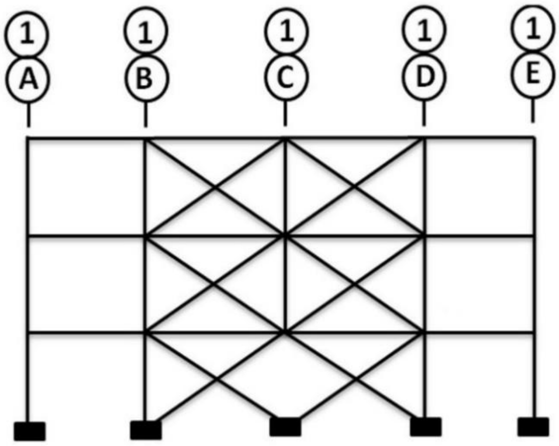
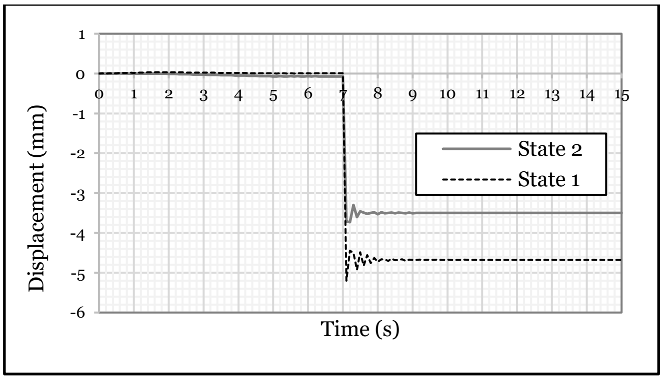


(b)

Figure 3. Schematic illustration of the removal methods; a) state 1, b) state 2.



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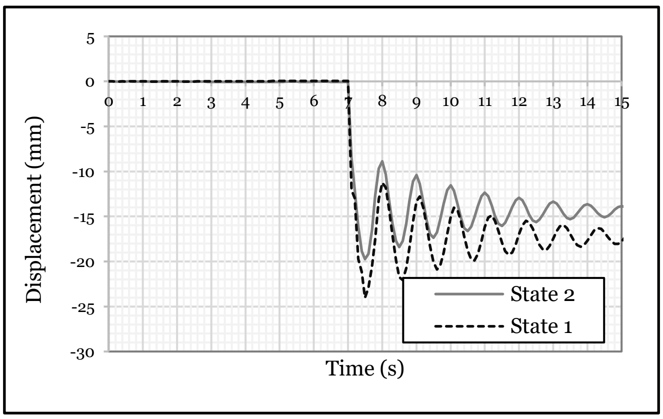


(b)

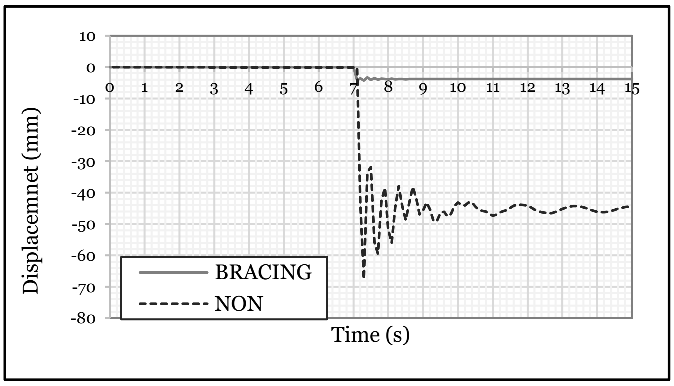
Figure 4. Displacement-time diagram of six-story building frames with the removal of a) Column 3, b) Column 6, and two different failure states according to the UFC-4-023-03 (2009) design criterion.

5.2 Effect of Bracings on the Progressive Collapse in Braced Spans

In this section, the progressive collapse potential was studied for the removal of columns 3 and 6 once in a case where the bracings were arranged in the spans in which the considered column was removed and once in a case where no bracing systems were considered in the intended spans. By comparing these two cases, the importance of adding bracing to the conventional steel moment-resisting frames was determined.

In the case of the removal of column 3 along with the corresponding braces on the first floor, the convergent braces on the other floors were placed on both sides of the removed column on the upper floors. In the case in which column 6 and its braces were removed, the convergent braces on the other floors were placed only on one side of the columns. Generally, the results of three-, and six-story building frames with respect to the GSA (2003), and UFC-4-023-03 (2009) criteria showed that the arrangement of bracings in a span that was directly influenced by the removal of the column could be effective in decreasing the potential of progressive collapse and increase of building capacity and strength against this failure.

The displacement-time diagrams obtained from non-linear dynamic analysis by GSA (2003) and UFC-4-023-03 (2009) criteria are plotted in Figures 5, and 6, respectively. These diagrams are depicted for the removal of columns 3 and 6 in six-story building frames, once with bracing systems (compound frames) and once without bracing (moment-resisting frames). It could be deduced from the displacement-time diagrams that by adding the bracings to the spans in which the columns were removed, the displacements caused by the removal of the columns were decreased. Consequently, the load-carrying capacity against the progressive collapse was increased. By comparing different parts of Figures 5, and 6, it can be inferred that the displacement-time diagrams of six-story building frames in accordance with GSA (2003), and UFC-4-023-03 (2009) criteria are in good agreement with each other, and negligible discrepancies between the extreme values of displacement can be observed. However, these negligible deviations could be attributed to the inherent differences between the methods of analysis and design adopted in each regulation. It is worth mentioning that the terms "BRACING" and "NON" referred to the moment-resisting frame along with bracing elements, and moment-resisting frame without bracing elements, respectively.



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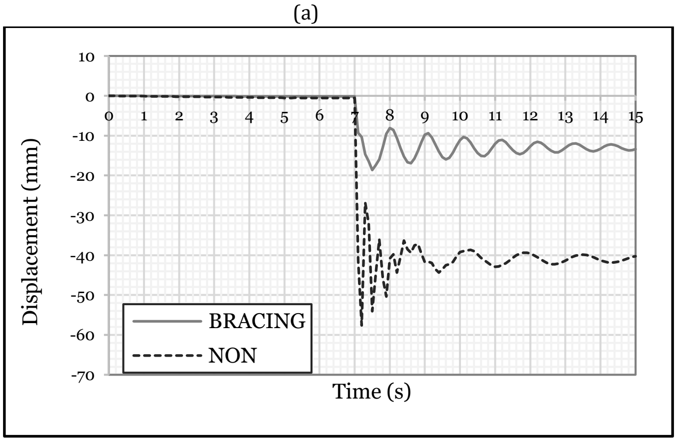
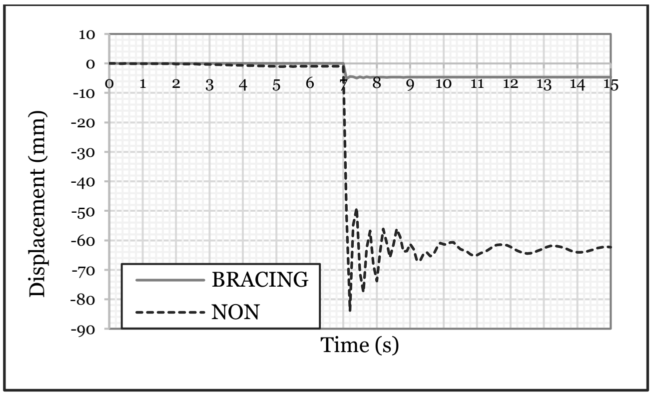


(b)

Figure 5. Displacement-time diagram of six-story building frames with the removal of a) Column 3, b) Column 6, by the GSA (2003) design criterion.



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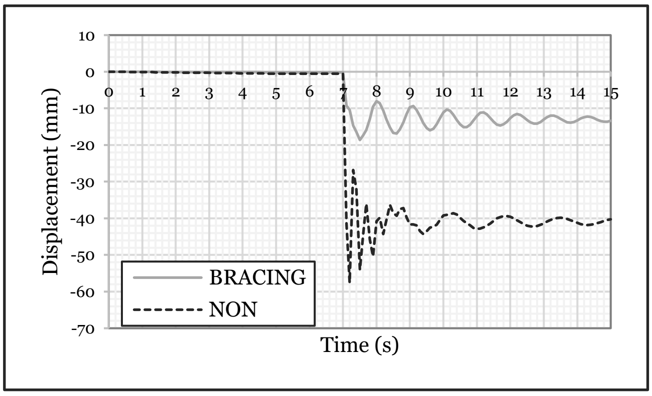
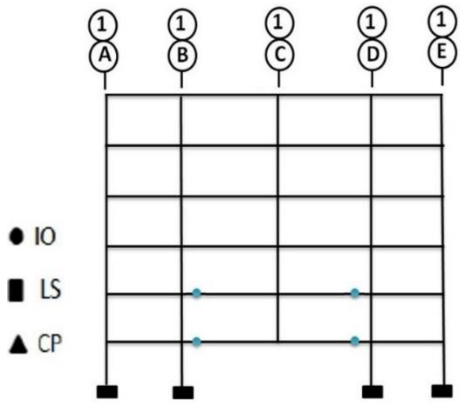


(b)

Figure 6. Displacement-time diagram of six-story building frames with the removal of a) Column 3, b) Column 6, by UFC-4-023-03 (2009) design criterion.

Figures 7, and 8 demonstrate the schematic illustration of the formation of the plastic hinges for two cases of compound steel braced moment frames and moment-resisting frames in the non-linear dynamic analysis in conjunction with the removal of columns 3 and 6, respectively. According to FEMA-350, four building performance levels are defined including Operational (OP), Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) performance levels. These performance levels may be correlated with minor or no damage, moderate damage, severe damage, and near collapse damage states, respectively. Note that, Figures 7, and 8 are presented only for the UFC-4-023-03 (2009) criterion. It can be seen from Figure 7 that in the case of compound steel braced moment frames, by removing column 3, the plastic hinges formed only in the bracing's elements, and no hinges formed in the beams and columns of the frames. On the other hand, plastic hinges formed in the beams of steel moment-resisting frames without a bracing system. Therefore, the compound building frames showed higher resistance and strength against the progressive collapse occurrence in comparison to the moment-resisting frames without bracings elements. In the case of the removal of column 6, some hinges formed in beams of two spans above the removed column in steel moment-resisting frames without a bracing system. In the case of compound steel braced moment frames, plastic hinges are formed only in beams of one span above the removed column, and bracing elements. However, it can be concluded that using bracing elements increases the overall stability of the structure against progressive collapse by the transmission of plastic hinges from the main structural elements, i.e., beams to the bracing elements. Another justification for the formation of some plastic hinges in beams of the compound system can be attributed to the automatic selection of the optimal cross sections of the elements with demand/capacity ratio of around 1. As the bracing elements sustain the dominant values of lateral forces in the compound system, the section moduli of beams are lower than those of moment-resisting frames.



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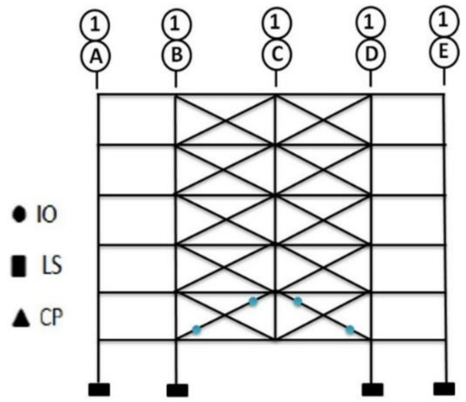
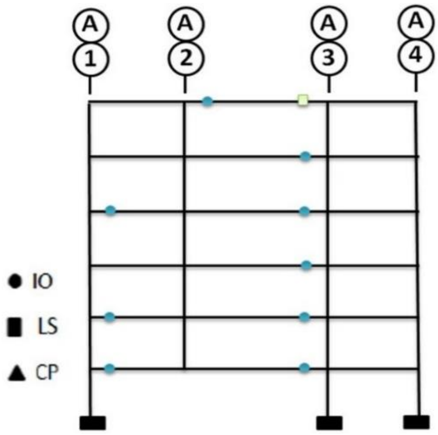


(b)

Figure 7. Plastic hinge formation with the removal of column 3, a) moment-resisting frame without bracing elements, b) compound steel braced moment frame, in accordance with UFC-4-023-03 (2009) design criterion.



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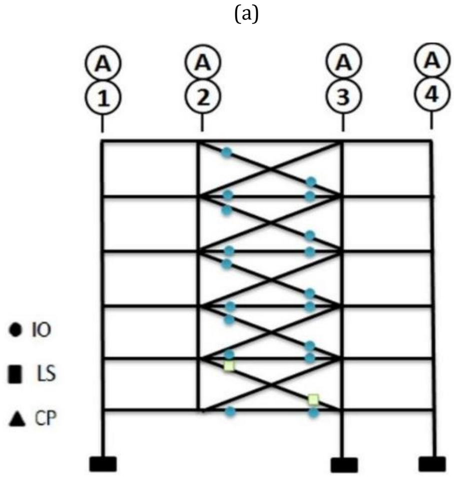


(b)

Figure 8. Plastic hinge formation with the removal of column 6, a) moment-resisting frame without bracing elements, b) compound steel braced moment frame, in accordance with UFC-4-023-03 (2009) design criterion.

5.3 Effect of Bracings on the Progressive Collapse in Unbraced Spans

In this section, the effects of removing columns of unbraced spans were investigated to study the progressive collapse occurrence on three- and six-story building frames. Analogous to the previous section, the results were reported for steel moment-resisting frames and compound steel-braced moment frames. Figure 9 demonstrates the nodal displacement-time diagram of six-story building frames including steel moment-resisting frame and steel braced moment frame with the removal of column 1. It can be seen from Figure 9 that the compound system experienced a higher level of nodal displacement over time. This phenomenon arises from the fact that adding the bracings elements to the moment frame system leads to the design of beams and columns with lower section moduli. Therefore, the impact of the progressive collapse by removing a column in an unbraced span is more pronounced in the compound system in comparison to the moment-resisting frame. This fact can be easily confirmed by making a comparison between the plastic hinge formation pattern of these frames. Figure 10 illustrates the formation of plastic hinges in the “A” axis in the state of removal of column 1 for both conditions of the compound system and the moment-resisting one. The results demonstrated that some plastic hinges are formed in the moment-resisting frame building after non-linear dynamic analysis with lower rotation values in the structural components of the frames. Whereas, plastic hinges were formed in the moment frame building with convergent braces, on both sides of the beams in 1-2 and 2-3 spans, as well as in the column of the first floor above the target element.

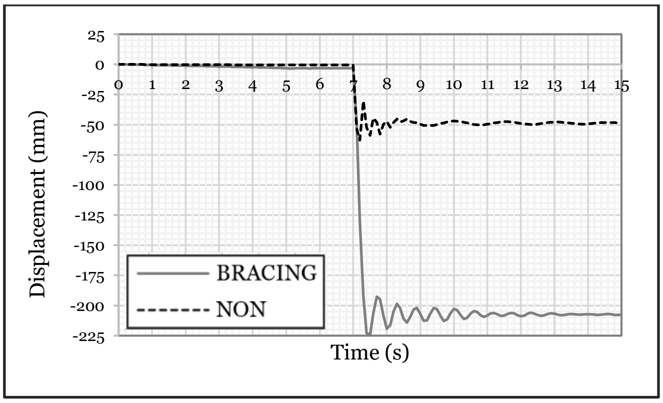
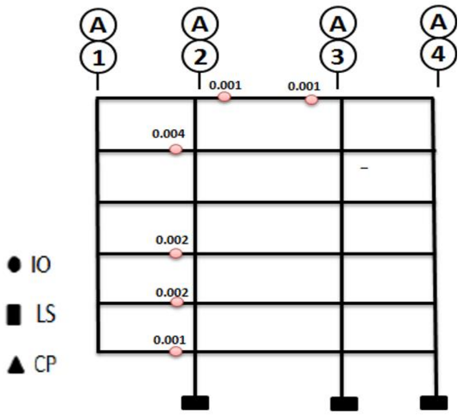


Figure 9. Displacement-time diagram of six-story building frames with the removal of column 1 by UFC-4-023-03 (2009) design criterion.

It can be seen from Figure 10 that although the rotation values of forming plastic hinges in the beams and columns of the compound system do not exceed the permitted limits reported in the UFC-4-023-03 (2009), the overall stability and resistance against the progressive collapse of the compound system with removal of a column on unbraced spans are lower than those of moment-resisting frames. Therefore, adding bracing elements to the moment-resisting frames and designing structural elements with the automatic selection of the optimal cross sections with demand/capacity ratio of around 1 lead to the low-resistance design against the occurrence of the progressive collapse. This observation is exactly opposite to the effect of bracing elements with the removal of the column on the braced spans.

Similar observation can be made for the nodal displacement-time diagram of three-story building frames including steel moment-resisting frame and steel braced moment frame with the removal of column 1. For the sake of brevity, the displacement-time diagram of three-story building frames was not reported, however, the compound system experienced a higher level of nodal displacement over time. Figure 11 shows the formation of plastic hinges in the “A” axis in the state of removal of column 1 for both conditions of the compound system and the moment-resisting one. The results of the three-story building frames were relatively similar to the results obtained for six-story building frames. It can be seen that some plastic hinges with lower rotation values formed in the moment-resisting frame in comparison to the compound frame. Therefore, the moment-resisting frame has higher resistance against the removal of the column in unbraced spans. Another observation that can be made by comparing different parts of Figures 10, and 11 is that the rotation values of plastic hinges in three-story building frames are higher than those of six-story building frames. Therefore, it can be concluded that by increasing the number of stories (i.e., height of the structure), the resistance against progressive collapse increases accordingly.



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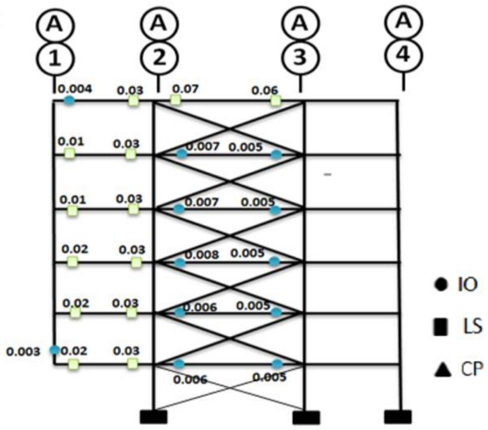


(b)

Figure 10. Plastic hinge formation and corresponding rotation values with the removal of column 1, a) moment-resisting frame without bracing elements, b) compound steel braced moment frame, in accordance with UFC-4-023-03 (2009) design criterion.

To double-check the impact of the number of stories in the progressive collapse of building frames, a twelve-story steel braced moment-resisting frame was designed against progressive collapse with the removal of column 1 in the "A" axis. Figure 12 shows the formation of plastic hinges in the "A" axis frame in the state of removal of column 1 in the compound system. It can be seen that some hinges with lower rotation values than three- and six-story building frames generated in the beams of the first to the fifth floor. It is worth mentioning that no plastic hinges were formed in the twelve-story steel moment-resisting frame. Therefore, it can be deduced that the impacts of progressive collapse on the low-rise building are more highlighted.

6. Summery and Conclusion

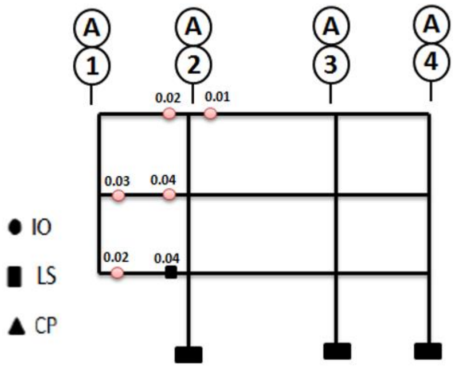
This paper aims to study the effect of adding bracing elements to the conventional steel moment-resisting frames for seismic loads through a nonlinear dynamic method according to GSA-2003 and UFC-4-023-03 criteria. The study was conducted using computational simulation of building models with two different elevations of three and six floors located in a moderate seismicity region. The results of this study have been compared with different numbers of stories and different lateral load-carrying systems, and the following results and conclusions were made through the comparison of different conditions.

The buildings that are designed according to the seismic-resistant design of codes and related regulations were not necessarily resistant to progressive collapse.

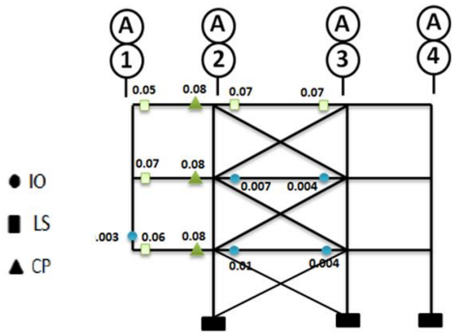
The remarking point in the assessment of progressive collapse in moment frame buildings with concentrically braced is the location of the bracing elements. The location of bracing elements and configuration of building frames have considerable effects on the resistance against the progressive collapse in different spans of the building and cause the prediction of progressive collapse occurrence to become more complex in braced moment frame structures than in the moment frame structures without the bracing.

The results showed that the bracing elements increase the resistance of building frames against progressive collapse with the removal of the braced column, while the opposite trend can be seen with the removal of the unbraced column.

By increasing the number of stories of building frames, the resistance of both compound building frames and moment-resisting frames increases. This fact can be easily confirmed by observation of the rotations of plastic hinges formed in building frames.



(a)



(b)

Figure 11. Plastic hinge formation and corresponding rotation values with the removal of column 1, a) moment-resisting frame without bracing elements, b) compound steel braced moment frame, in accordance with UFC-4-023-03 (2009) design criterion.

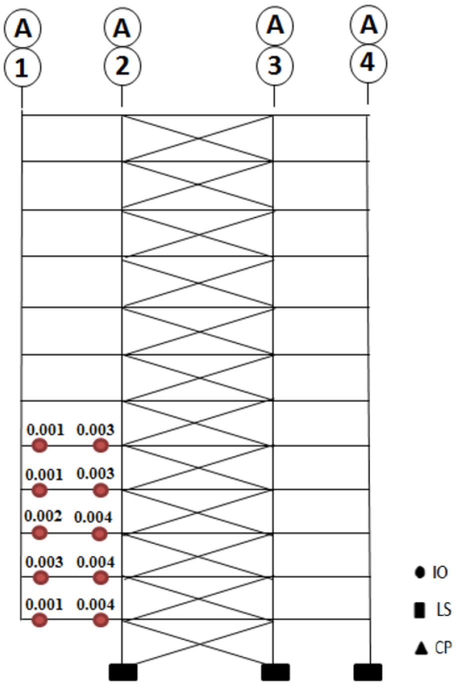


Figure 12. Plastic hinge formation and corresponding rotation values with the removal of column 1 in compound steel braced moment frame, in accordance with UFC-4-023-03 (2009) design criterion.

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