# Investigation of Collapse-Resisting Capacity of Braced Steel Moment Frames

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## ABSTRACT

In this paper, the probability of the occurrence of progressive collapse in three-story, six-story and nine-story steel buildings is evaluated. These three structures are designed based on common codes. According to the obtained results, it can be observed that the six-story and nine-story buildings cannot fulfill the acceptance criteria of UFC4-023-03 guideline. To enhance the collapse-resisting capacity of these buildings, steel braces are deployed. By performing non-linear static and dynamic analysis, it is induced that the presented retrofitting methods decrease the probability of the occurrence of progressive collapse. In addition, the displacements of the structures with a removed column are considerably reduced. In other words, the aforesaid buildings are able to satisfy the acceptance criteria of UFC guideline by utilizing the suggested retrofitting method.

**KEYWORDS:** Progressive collapse, Steel frames, Steel braces, Non-linear analysis.

## INTRODUCTION

Due to events like explosions, fire, impact by vehicles and human errors, local damage occurs in structures. The spread of local damage leads to the collapse of an entire structure or a disproportionately large part of it. The term “progressive collapse” has been utilized to describe this phenomenon. Recently, analytical tools for investigating the progressive collapse potential of new or existing buildings have drawn considerable attention of researchers.

It should be mentioned that UFC 4-023-03 (UFC, 2010) is one of the well-known standards widely employed to assess buildings' resistance to progressive collapse. This standard proposes some methods for analyzing structures for progressive collapse. One of them extensively utilized is named the alternate path method. In this approach, the ability of structures to resist omission of a specific vertical element is checked without regard to threat causing loss of the element. This technique ensures that alternate load paths are available when a vertical structural element fails. As a consequence, the structure's collapse is prevented. In a case that the designed structure does not satisfy the acceptance criteria, it should be redesigned or retrofitted to avoid progressive collapse. To analyze structures for progressive collapse, four methods are utilized; namely, linear static, linear dynamic, non-linear static and non-linear dynamic methods. These schemes have their own advantages and disadvantages (Marjanishvili, 2004; Marjanishvili and Agnew, 2006).

Recently, extensive studies have been conducted on progressive collapse of structures. Kim et al. (2014) explored the resistance capacity of a steel moment frame with an MR damper against progressive collapse and proposed a preliminary design procedure for the dampers to prevent progressive collapse. Jalali Larijani

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et al. (2013) studied the exposure of two existing asymmetric steel building frames to progressive collapse. Due to that there is no weak axis for the box section, using built-up box-shaped sections for columns is a better option than implementing built-up I-shaped sections. They showed that the resistance of the structure with dual-frame system against progressive collapse is comparatively much greater than that of a simple building frame system. Hashemi Rezvani and Asgarian (2014) investigated the effect of seismic design level for progressive collapse mitigation in seismically designed concentric braced frame buildings. Fu (2009) stated that the dynamic response of the structure is mainly related to the affected loading area after the column removal, which also determines the amount of energy needed to be absorbed by the building. Also, he mentioned that under the same general conditions, a column removal at a higher level will induce larger vertical displacement than a column removal at ground level. Khandelwal et al. (2009) studied previously designed 10-story prototype buildings by applying the alternate path method. They concluded that an eccentrically braced frame is less vulnerable to progressive collapse than a special concentrically braced frame. Kim and Kim (2009) assessed the progressive collapse of steel moment frames designed based on LRFD approach. In this work, both linear and non-linear analyses were performed and the results obtained were compared. Moreover, the effects of column removal and number of stories on the responses were evaluated. In this study, the two-dimensional models of structures were applied. Purasinghe et al. (2012) took advantage of SAP2000 commercial software for three-dimensional analysis of buildings. In this research, linear static, non-linear static, linear dynamic and non-linear dynamic analyses were conducted. Liu (2011) used the alternate path technique with each of the three analysis procedures; i.e., linear static, non-linear static and non-linear dynamic methods, for analysis and optimum design of steel moment frames to resist progressive collapse. Accordingly, he concluded that the usage of linear static method led to the most conservative and heaviest design against progressive collapse. In contrast, more economical designs were achieved when non-linear static and dynamic tactics were employed. Recall that non-linear schemes were more time-consuming than linear ones. It should be added that two-dimensional models were deployed in this work. In the design process of structures, the collapse-resisting capacity of structures can be increased by taking specific precautions, such as the selection of appropriate sections and beam spans. But, retrofitting is required for existing structures. Kim and Shin (2013) assessed the effect of prestressing tendons on the progressive collapse performance of buildings. In this study, they observed that the usage of tendons in buildings vulnerable to progressive collapse induced by sudden removal of a first story column caused the structure to behave in a more stable manner. Orton et al. (2009) experimentally evaluated the influences of CFRP plates on the resistance and continuity of existing reinforced concrete buildings.

In this paper, the roles of steel braces in improving the performance of structures against sudden loss of columns are investigated. In this process, the alternate path approch is employed. To achieve this goal, three different steel intermediate moment frames are designed. Then, their collapse-resisting capacity is evaluated by performing non-linear static analysis (NSA) and non-linear dynamic analysis (NDA). Finally, braces are deployed for retrofitting these structures against progressive collapse.

**Properties of Sample Structures**

To evaluate the aforementioned method of retrofitting, three steel structures with 3, 6 and 9 stories with intermediate resisting frame are considered. Seismic loads are applied based on ASCE 7-05(ASCE, 2005). It should be added that UBC97 (UBC, 1997) code is used to design the structures. The dead load exerted on the stories and the roof is equal to 6kN/m$^2$. Additionally, the live loads applied to the stories and the roof are 2kN/m$^2$ and 1.5 kN/m$^2$, respectively. The perimeter wall loads of the stories and the roof are

9.5kN/m$^2$ and 2.5kN/m$^2$, correspondingly. It should be mentioned that the yield strength of steel equals 240 MPa. In Figure 1, a plan view of the sample buildings is demonstrated. Also, Figure 1 specifies the locations of the removed columns for alternate path analysis based on UFC guideline.

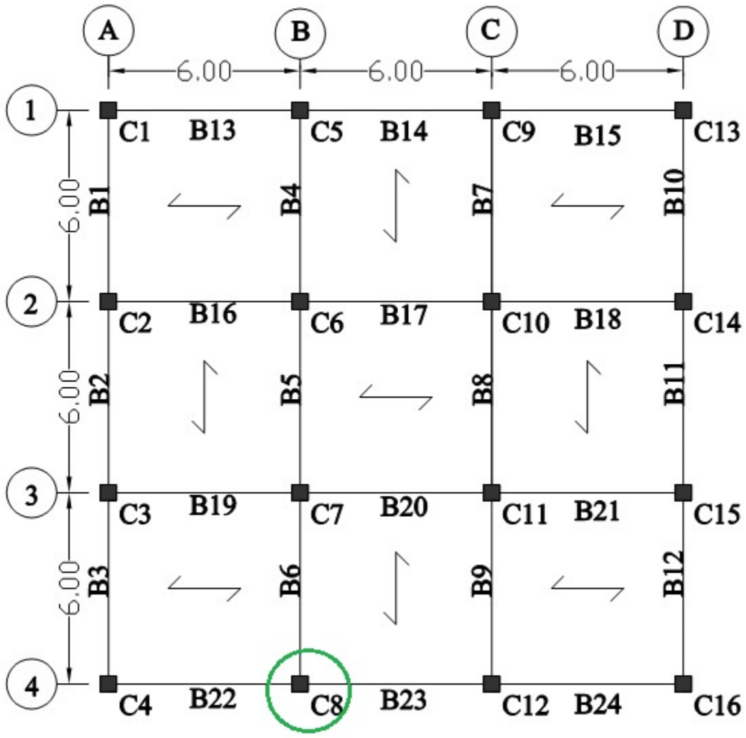



**Figure (1): Plan view of sample buildings and locations of removed columns**

Box columns and IPE beams are deployed in all stories.

**Modeling Procedure of Progressive Collapse**

Based on UFC guideline, the locations of columns for performing the alternate path method are specified in Figure 1. In this paper, various scenarios of corner column removal are studied. In these scenarios, column C8 is omitted in different stories. In each case, the structural behavior is assessed after the column removal. It is worth emphasizing that retrofitting is required if the UFC acceptance criteria are not satisfied. To perform alternate path analysis, SAP2000 (CSI, 2009) software is utilized. Note that the UFC guideline proved the capability of this commercial software for analyzing progressive collapse. In this work, NSA and NDA are deployed for evaluating the structure resistance to progressive collapse.

**Load Combination**

According to UFC guideline, in NSA, the subsequent increased gravity load combination should be applied to those bays immediately adjacent to the omitted element and at all floors above the removed element:

$$G_N = \Omega_N[(0.9 or 1.2)D + (0.5 L or 0.2 S)] \quad (1)$$

In this equation, dead load, live load and snow load are denoted by D, L and S, respectively. Moreover, $\Omega_N$

is the dynamic increase factor used in NSA. According to UFC guideline, this factor has the coming appearance for the steel moment frame:

$$\Omega_N = 1.04 + 0.45/(\frac{\theta_{pra}}{\theta_y} + 0.48) \quad (2)$$

$\theta_y$ *and* $\theta_{pra}$ are yield rotation angle and plastic rotation angle, correspondingly. $\theta_{pra}$ is determined based on ASCE 41 (ASCE, 2006) and UFC guideline. For steel elements, yield rotation angle is specified according to Equation 5-1 proposed in ASCE 41.

In other bays of the structure, the gravity load should be computed as:

$$G = (0.9\ or 1.2)D + (0.5L\ or\ 0.2S) \quad (3)$$

In NDA, the load combination G is exerted on all the bays. In progressive collapse analysis, a notional lateral load should be applied to the building in combination with gravity loads. This notional lateral load can be computed as:

$$L_{LAT} = 0.002 \sum P \quad (4)$$

in which the sum of dead loads and live loads acting on a specific floor is shown by $\sum P$. Based on the aforesaid loads, four separate analyses should be performed for each main direction of the building. In each analysis, both lateral and gravity loads are included.

**Properties of Plastic Hinges**

Plastic hinges of columns and beams are defined based on FEMA356 (2000).

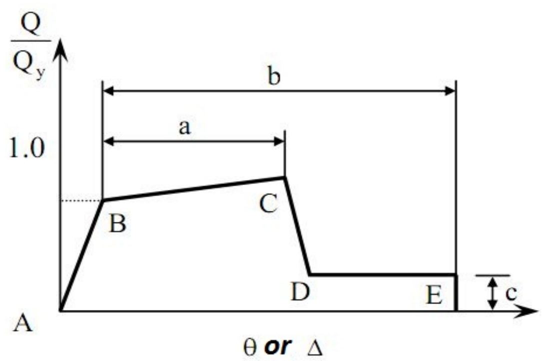



**Figure (2): Load-displacement relation of the plastic hinges (FEMA, 2000)**

The plastic hinges are placed at the ends of the columns, whilst the plastic hinges of beams are located at their ends and mid-points.

**Analysis Procedure**

Herein, the steps of progressive collapse analysis are briefly introduced. Firstly, the initial structure is analyzed under the load combinations. Then, one of the columns is removed. Afterwards, the internal forces of the deleted column are calculated and applied to the structure as a load case to the joints at each column end. Note that forces are gradually exerted on the structure in static analysis, while loads are instantaneously applied to the buildings in dynamic analysis. After performing the analysis, the acceptance criteria of the guideline used are checked.

**Structure Acceptance Criteria**

Acceptance criteria of elements include deformation and resistance requirements. Accordingly, stresses induced in elements should be less than their design material strength. To control the deformation limits, the computed deformations are compared with the allowable values presented in the UFC guideline. If the primary and secondary structural elements do not fail in NSA and NDA, the aforesaid structure satisfies the acceptance criteria. According to UFC, for beams and columns, if the plastic hinge rotation exceeds the Life Safety (LS) limit, then the element will fail. In case of element failure, the structure should be redesigned or retrofitted.

**Progressive Collapse Analysis of the Assumed Structure**

For assessing the resistance of the assumed structure to progressive collapse, several corner column removal scenarios are considered. In this study, removal scenarios are analyzed by NSA and NDA. It should be mentioned that, in this study, C8 is the only removed column in stories, within separate analysis methods.

**Non-linear Static Analysis**

In the three-story building, removal of column C8 from the first story leads to the appearance of hinges in some of the beams. Note that their corresponding rotations do not exceed the allowable limit (LS). But, loss of columns in the 6-story building results in plastic hinges the rotations of which exceed the allowable limit. For instance, omission of column C8 from the first story causes the plastic hinges to rotate more than the LS limit. Consequently, the acceptance criteria of the code are not satisfied and it is required to redesign the structure or retrofit it. In the 9-story building, the rotations of the plastic hinges are less than the LS limit, when a column is removed from the first story and second story. Nevertheless, loss of column from other stories leads to the emergence of impermissible rotations in plastic hinges. In other words, the acceptance criteria of the code are not satisfied anymore. In Table 1, the results of NSA are summarized. Based on this table, it can be observed that loss of column C8 from the 6-story and 9-story buildings results in rotations greater than the LS limit. Hence, these structures are not accepted by the UFC guideline and are required to be redesigned or retrofitted.

**Table 1. The produced plastic hinges of structures in the NSA process**

| Story from which a column is removed | 9-story building | | 6-story building | | 3-story building | |
|---|---|---|---|---|---|---|
| | Total number | Rotations which exceed LS limit | Total number | Rotations which exceed LS limit | Total number | Rotations which exceed LS limit |
| St1 | 33 | - | 30 | 4 | 12 | - |
| St2 | 36 | - | 25 | 4 | 7 | - |
| St3 | 30 | 1 | 20 | 3 | 2 | - |
| St4 | 27 | 3 | 15 | 2 | | |
| St5 | 26 | 6 | 9 | - | | |
| St6 | 20 | 6 | 5 | 3 | | |
| St7 | 16 | 3 | | | | |
| St8 | 10 | 3 | | | | |
| St9 | 4 | 2 | | | | |

**Non-linear Dynamic Analysis**

In NDA, the rotations of all plastic hinges of the 3-story and 6-story buildings do not exceed the allowable limit when a column is removed from any of the stories. In the 9-story building, omission of a column from stories 1 to 8 does not result in unallowable rotations in plastic hinges. But, loss of column C8 of the last story causes the plastic hinges of beam B22 to exceed the safety limit. Therefore, retrofitting is required for this structure. In Table 2, the results of NDA are listed. Accordingly, it is observed that removal of a column from the last story of the 9-story building causes one of the hinges to rotate more than the allowable limit.

**Table 2. The hinges formed in the buildings when NDA is performed**

| Story from which a column is removed | 9-story building | | 6-story building | | 3-story building | |
|---|---|---|---|---|---|---|
| | Total number | Rotations which exceed LS limit | Total number | Rotations which exceed LS limit | Total number | Rotations which exceed LS limit |
| St1 | 46 | - | 33 | - | 15 | - |
| St2 | 44 | - | 26 | - | 10 | - |
| St3 | 38 | - | 21 | - | 5 | - |
| St4 | 32 | - | 16 | - | | |
| St5 | 28 | - | 10 | - | | |
| St6 | 21 | - | 5 | - | | |
| St7 | 16 | - | | | | |
| St8 | 11 | - | | | | |
| St9 | 4 | 1 | | | | |

The results obtained from NSA and NDA show that column loss causes 6-story and 9-story buildings to be vulnerable to progressive collapse. Due to this fact, these structures are required to be retrofitted. For this purpose, the cross braces are installed in the exterior frames of these buildings. Based on trial and error, the sections of these braces are selected to avoid the appearance of plastic hinges with impermissible rotations.

**Investigation of Retrofitted Structure**

In Figure 3, the retrofitted structure is shown. The spans of the last story of the exterior frames are retrofitted by cross braces.

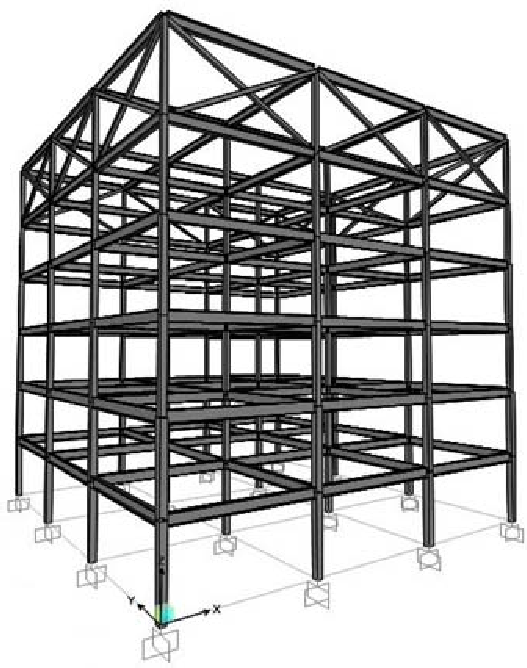
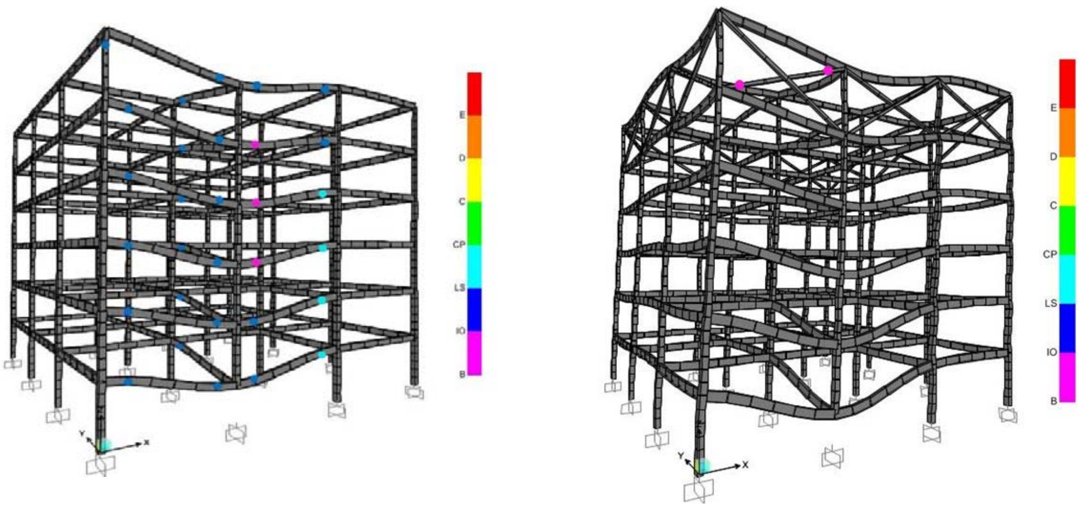


**Figure (3): The retrofitted building**

**Non-linear Static Analysis of the Retrofitted Structure**

By performing progressive collapse analysis, it is observed that some of the plastic hinges fail. The sections of the retrofitted structure are selected based on trial and error. In this way, box section (140mm*140mm*12.5mm) is chosen for retrofitting the 6th story of the building. Afterwards, the analysis procedure is repeated. According to the findings, no hinges fail. In fact, the usage of braces provides new paths for transferring loads. As a result, the bending moment of the beams and their rotations are reduced.

The deformation of the initial building in which the column is removed from the first story is demonstrated in Figure 4a and the corresponding deformation of the retrofitted structure is illustrated in Figure 4b.




**(a)** **(b)**

**Figure (4): Structure deformation after omission of column C8 from the first story according to NSA; a) initial structure and b) retrofitted structure**

To retrofit the 9-story building against progressive collapse, braces with dimensions of (160mm*160mm*12.5mm) are deployed. After adding the braces, progressive collapse analysis is performed. Column removal from the first story leads to the formation of a hinge in a brace. Axial deformation of this hinge does not exceed the allowable limit. No plastic hinges formed in the structure after column omission from other stories. In other words, the structure behaved in an elastic manner. It should be remarked that the selection of the appropriate sections of braces is based on trial and error.

**Non-linear Dynamic Analysis of Retrofitted Structure**

By investigating the initial 9-story building, it is demonstrated that one of the plastic hinges exceeds the LS limit after column removal form the last story. Based on NSA, the 6-story building is required to be retrofitted. In the previous section, the appropriate cross-sections of the braces of this structure were selected. Herein, the aforementioned braces are utilized in the last story.

After performing NDA on the retrofitted buildings, it is observed that no hinges are formed in the buildings after column loss and that the buildings behave in an elastic manner. In other words, the proposed retrofitting method is successful in increasing the collapse-resisting capacity of the buildings.

**Vertical Displacement of the Structure**

In Figure 5, vertical displacement of the structure after column loss is depicted. The maximum displacement occurs in the 6-story building. According to Table 2, no plastic hinges exceed the LS limit.

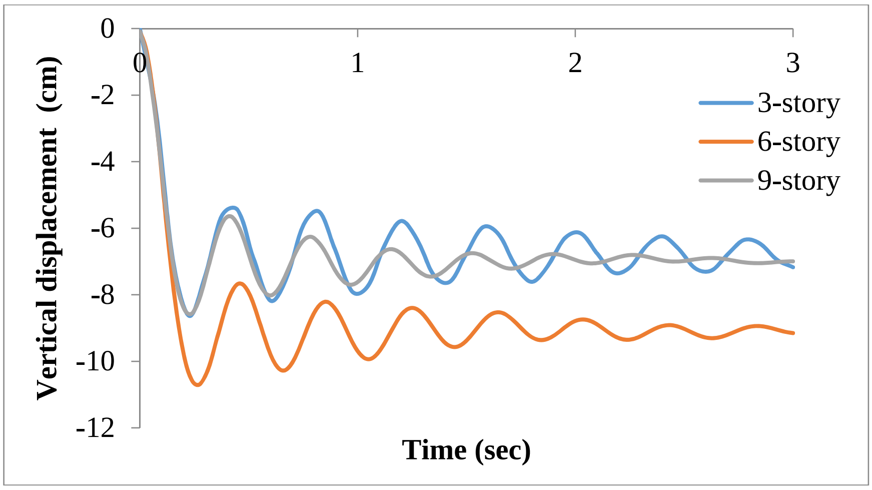



**Figure (5): Vertical displacements of the structures after removing a column from their first stories obtained from NDA**

After the loss of a column, its internal forces should be tolerated by other columns which are next to it. The beams, located in stories above this column, transfer the loads to the adjacent columns. In fact, these beams should be able to resist the extra load induced by column removal. The vertical displacement of the structure depends on the stiffness of the structural elements of the stories above the removed column and the magnitude of its internal forces. Based on Figure 5, it is not possible to propose a specific pattern for the number of stories and the magnitude of vertical displacement after the loss of a column. To prove the robustness of the proposed retrofitting technique, the vertical displacements of the initial structures and those of the retrofitted ones are compared in Figures 6 and 7. Accordingly, it is obvious that the suggested scheme is successful in reducing the possibility of progressive collapse in structures. In the 6-story building, the maximum displacement is 10.66 cm. After installation of braces, this value reduces to 2.18 cm. After the removal of column C8 from the first story of the 9-story building, the maximum observed displacement is 8.56 cm. This displacement is reduced to 3.12 cm by retrofitting the structure.

In NSA, the maximum displacement of the 6-story structure is 11.26 cm. After installation of the braces, this value is changed into 2.14 cm. The maximum displacement of the initial and retrofitted 9-story buildings is 8.42cm and 2.68cm, correspondingly.

**Internal Forces of the Members**

The elemental force distribution is changed by retrofitting structures. This is due to the fact that new paths are provided for transferring loads. In what follows, the internal forces of the 6-story building are assessed before and after retrofitting. After installation of the braces, the internal forces of the 9- story building change in a similar manner to those of the 6-story building. As a result, the internal forces of this structure are not evaluated to be brief. In Figures 8 and 9, the magnitudes of shear forces induced in the beams connected to the upper node of the removed column (C8) are shown. It should be added that these results are obtained by performing NDA. It is obvious that shear forces of beams B6, B22 and B23 are considerably decreased after retrofitting.

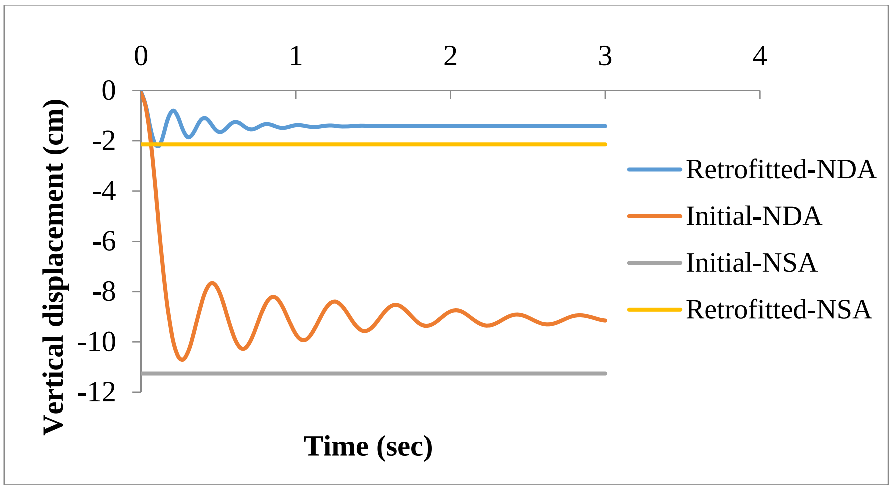



**Figure (6): Comparison of vertical displacements of the 6-story building before and after retrofitting in the case that a column is removed from the first story**

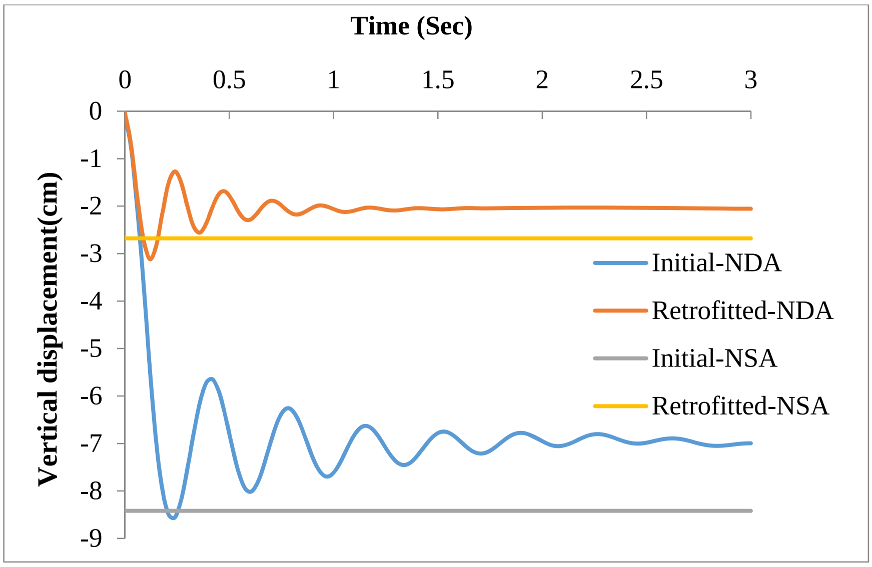
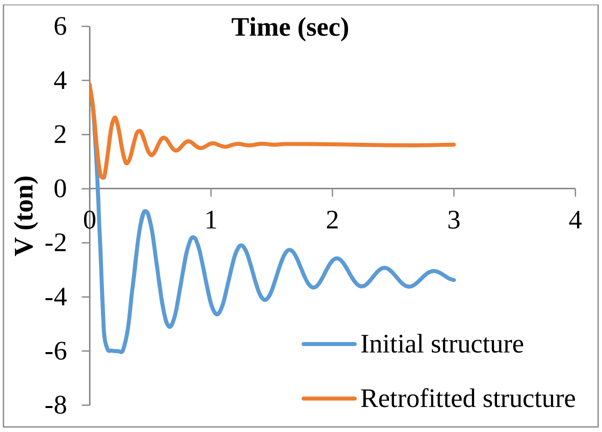
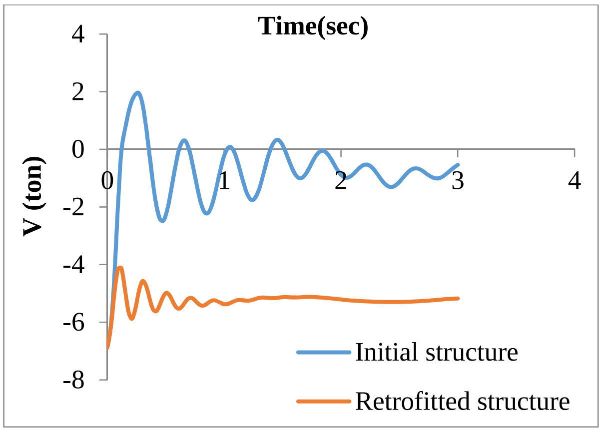


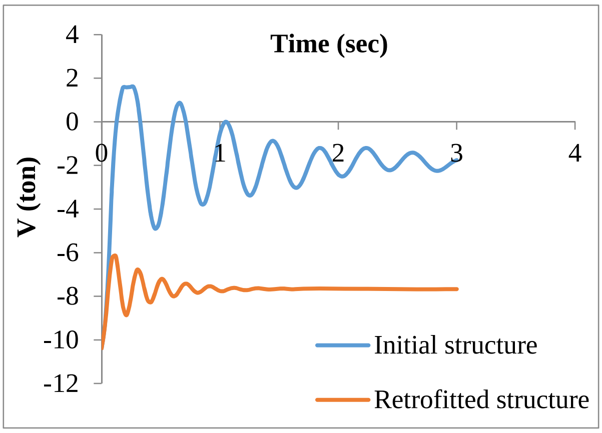

**Figure (7): Comparison of vertical displacements of the 9-story building before and after retrofitting in the case that a column is removed from the first story**




B22




B6




B23

**Figure (8): Shear forces induced in the beams connected to the upper node of column C8, the near end of beams**

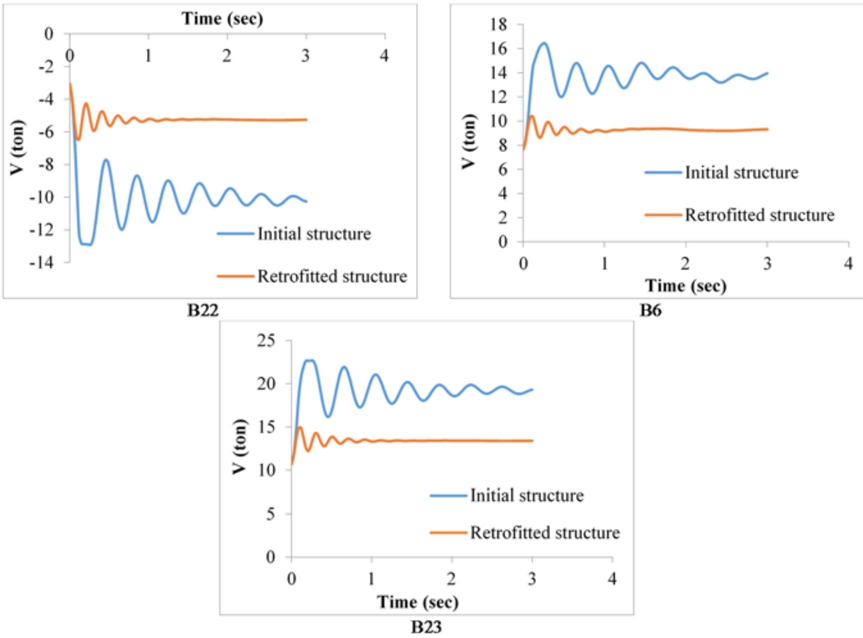



**Figure (9): Shear forces induced in the beams connected to the upper node of column C8, the far end of beams**

Figures 10 and 11 illustrate the bending moment of the beams connected to the upper node of column C8 located in the first story. It is clear that the usage of braces reduces the bending moment of the beams, especially the ones connected to column C8.

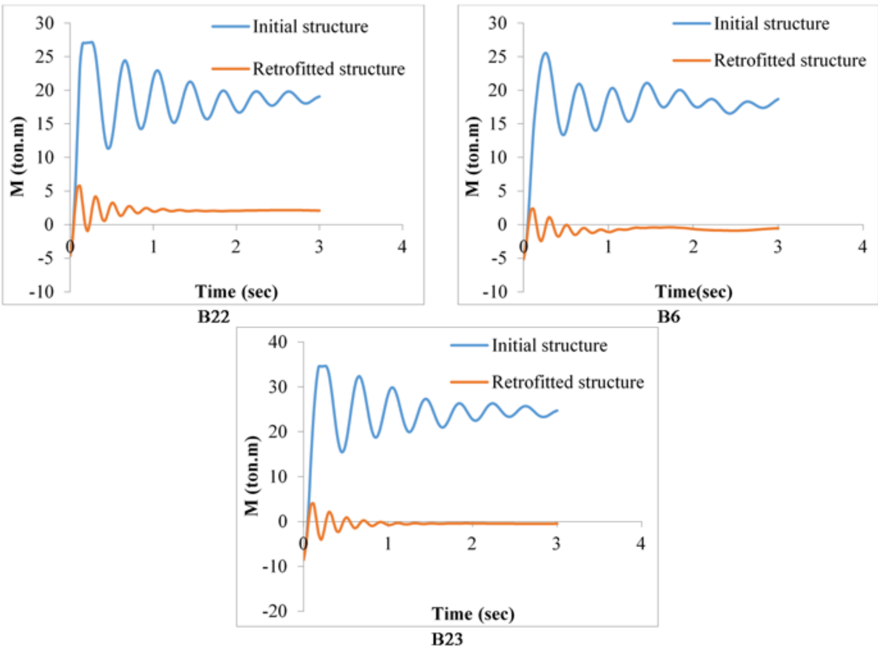



**Figure (10): Bending moments induced in the beams connected to the upper node of column C8, the near end of beams**

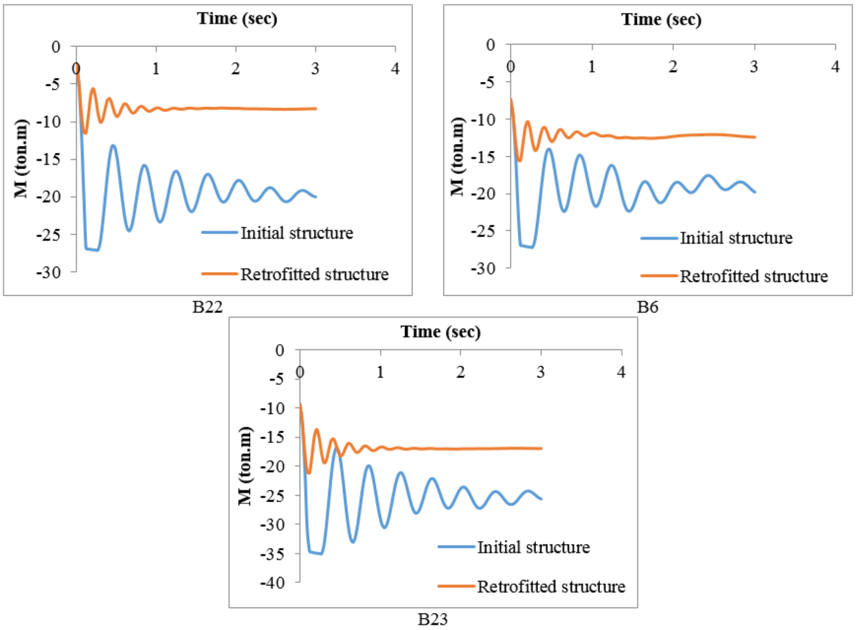



**Figure (11): Bending moments induced in the beams connected to the upper node of column C8, the far end of beams**

In addition to the beams, the internal forces of the columns adjacent to the removed column are changed. Figure 12 shows the internal axial forces of these members before and after retrofitting. Recall that column C8 is removed from the first story.

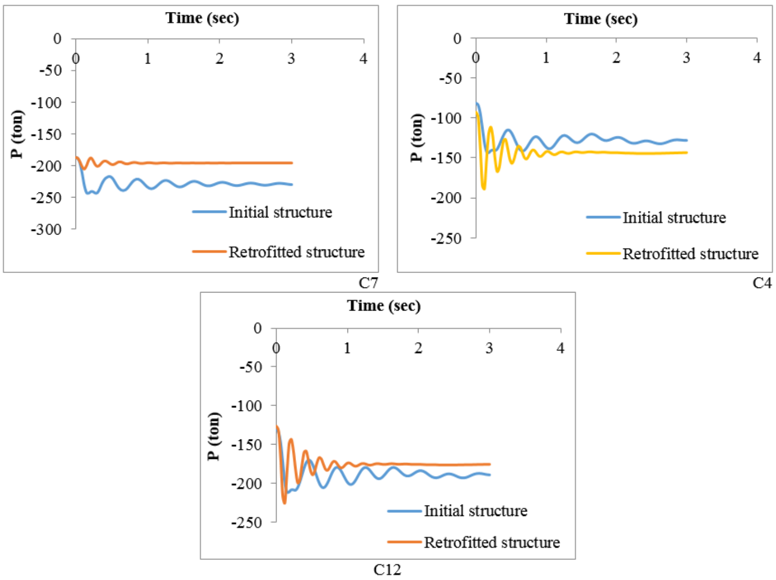



**Figure (12): Variation of axial internal forces of the columns adjacent to column C8 of the first story before and after retrofitting**

## CONCLUSIONS

In this work, the collapse-resisting capacity of 3-story, 6-story and 9-story steel buildings were evaluated. By investigating the initial 3-story structure, it is observed that the deformation of members does not exceed the allowable limits. But, by performing both NSA and NDA analysis on the other two structures, it is observed that the rotations of the plastic hinges formed in beams exceed the permissible limit after the loss of a column. Hence, these structures are required to be retrofitted for satisfying the acceptance criteria of the UFC code. To achieve this goal, braces are deployed in this study. By conducting analysis on the retrofitted structures, the following conclusions are made:

- It is not possible to relate number of stories to vertical displacement of the structure after column removal. Vertical displacement is dependent on the ratio of the elements' stiffness to the internal force of the omitted column.
- In NSA, more plastic hinges with impermissible rotations are formed in comparison to NDA. This issue highlights that NSA analysis is more conservative than NDA.
- For increasing the collapse-resisting capacity of the structures, it is required to provide more paths for transferring loads from the removed column to the other elements of the structure. This aim can be achieved by installing braces. In this way, the structure behaves in an elastic manner. Besides, this retrofitting method considerably reduces vertical displacement.
- Adding braces to the last story reduces shear forces and bending moments of the beams connected to the removed column (C8).

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