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Global-loss-of-stability progressive collapse mechanisms of 3D steel frame buildings

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ABSTRACT

The progressive collapse phenomenon of buildings has gained great attention from the structural engineering community since the collapses of the World Trade Center in New York in 2011. The response of structural systems, to local damage is in the core of this attention. The available methods for the analysis of such a phenomenon are described in [1] which is the dominating code for progressive collapse today. The current work extends the analysis included in [4] by using a push-down analysis able to identify the global loss-of-stability phenomenon. As presented in recent publications [2]-[5], these loss-of-stability phenomena are usually more critical for the integrity of the structure than others (e.g. yielding-type of failures or local buckling of components). Through a 3D FEM analysis of steel buildings, the potential of global loss-of-stability is investigated, a collapse mode which is not usually considered by the codes or by structural engineers mainly due to its complexity.

1 INTRODUCTION

Many design codes mention progressive collapse resistance as a desired property of a structure, stating that local damage is acceptable in a structural system as long as it will not affect the overall structural integrity. Essentially, the codes most often provide a provision on structural capacity reserves so that the structure will keep standing while suffering local damage. Two publications provide methods for progressive collapse analysis and design: the General Services Administration, [6] and the Department of Defense Unified Facilities Criteria, [1]. Their design philosophy is based on the fact that progressive collapse is triggered by a local damage event and the way to simulate this event is to remove one key element from the structure. For steel framed buildings, these key elements are the columns. In this framework, most research work on progressive collapse has focused on the response and design of individual components of a structure instead of regarding the structure as an entirety. Recent work presented in [2], [3] and [5] has shown that progressive collapse could happen due to local instability in adjacent-to-the-removal columns. Although the importance of studying the capacity of individual components is obvious after the removal of a column element, the global stability capacity of structures should also be studied in detail during progressive collapse analysis. The purpose of this paper is to identify new progressive collapse modes which can potentially be induced due to global loss-of-stability.

2 THE PROBLEM OF GLOBAL STABILITY

2.1 General theoretical background

The problem of global loss of stability in a structural system is not new in the structural engineering community. Under seismic excitations, large horizontal displacements combined with vertical loading can lead to a loss of global stability and eventually result in global collapse. In the field of progressive collapse, there are so far two different modes of collapse. The first type involving beam/connections failure, occurs at the beams above the column removal. The two beams above removal lose their support and begin to behave like one beam. Plastic hinges form at the ends and the middle of the newly formed long beam. The consequent event is large vertical inelastic deformations eventually leading to the complete collapse of the system. The second type is local buckling and instability of individual columns. The loads that were supposed to be carried by the damaged column are transferred to columns adjacent to the damaged column, leading to their instability and buckling. The propagation of buckling in the system results to the complete collapse, [5]. Another unexplored so far type, which is most difficult to be observed, is direct loss of global stability. In this circumstance, neither beam failure nor local buckling will occur. The global stability capacity of the structure is degraded by limited yielding/plastification of beams or columns leading to the change of end restraints of components in the system which could activate a global collapse mode. The present work primarily focuses on the third type of progressive collapse mode and identifies it through FEM models of steel moment frame buildings.

2.2 Appropriate method of analysis

As presented in [5], the only appropriate method to capture a potential loss of stability phenomenon in a structural system is one including both material and geometric nonlinearities along with advanced computational techniques such as fiber simulation of element sections. The present work applies such methods.

3 NUMERICAL APPLICATION

3.1 Description of 2D analysis

Three steel moment resisting frames (as shown in figure 1) are selected for the 2D analysis. They all have 40 stories but have different number of bays:

(1) Frame [2D, A] has two bays;
(2) Frame [2D, B] has three bays;
(3) Frame [2D, C] has four bays.

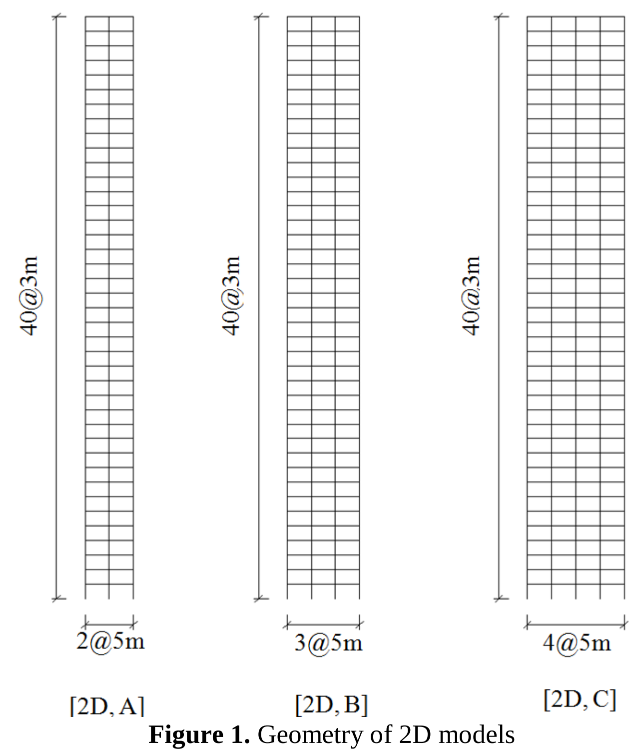

Figure 1. Geometry of 2D models

The height of each story is 3m and the width of each bay is 5m. The columns of the first floor of each frame are assumed to be fixed on ground. All beam - column connections are completely rigid moment connections. The frames are designed as office buildings and the dead, live, wind and earthquake loads follow provisions of the design code of New York City. The cross sections for these 2D models are shown in table 1.

Floor	External Columns	Internal Columns	Beams	Floor	External Columns	Internal Columns	Beams
38-40	W12x35	W12x40	W18x50	17-19	W21x223	W21x166	W24x104
35-37	W14x61	W14x61	W18x50	14-16	W21x248	W21x182	W24x117
32-34	W16x77	W18x76	W18x86	11-13	W24x279	W24x192	W24x117
29-31	W18x97	W21x101	W18x86	8-10	W27x307	W27x217	W24x117
26-28	W18x130	W21x111	W18x86	5-7	W27x368	W27x258	W24x117
23-25	W18x158	W21x147	W18x86	2-4	W30x433	W27x258	W24x117
20-22	W21x182	W21x147	W24x104	1	W30x477	W27x258	W24x117

Table 1. Cross section assignment of 2D models

For the purposes of progressive collapse analysis, all these three models lose one of their interior column at the 1st floor simulating initial damage and are analyzed in ABAQUS [7]. The material used in this analysis is ASTM A992 which has a young's modulus of 200GPa, tensile yield strength of 345MPa and an ultimate strength of 450MPa at a plastic strain of 0.2. The element type used for the FEM models is ABAQUS element B32 which has two Gauss points in each element. According to DOD, a dynamic increase factor of 1.26 is applied to the vertical load on all the beams above removal. Additionally, a horizontal load of 0.1kN is applied at each floor as an imperfection load to break the symmetry of the models. The imperfection load is small so that it does not affect the progressive collapse mechanism.

3.2 Results of 2D analysis

The deformed shapes of the three frames at collapse are shown in figure 2.

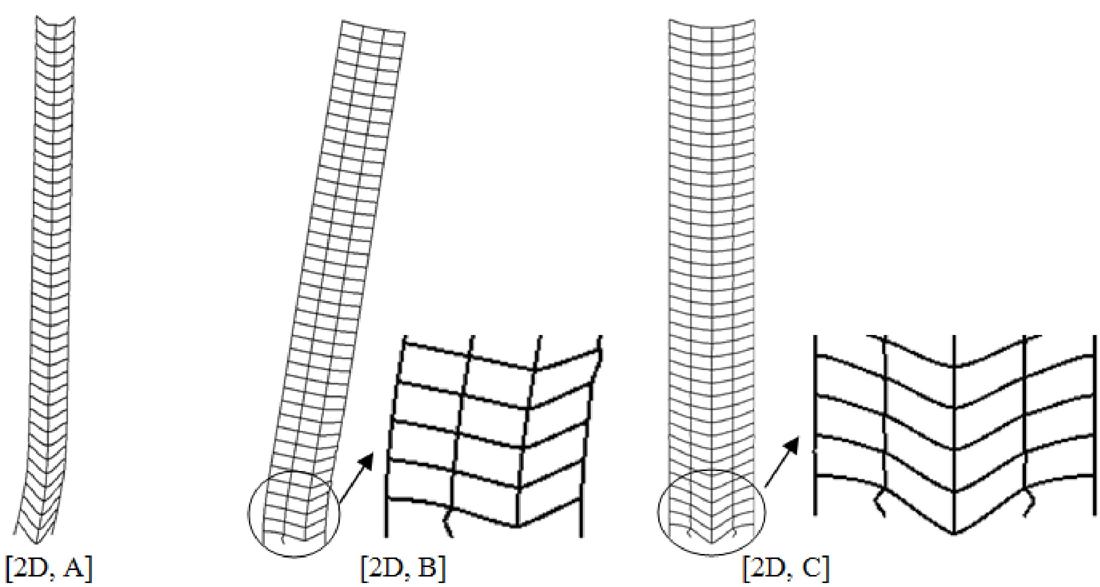

Figure 2. Collapse mechanisms of 2D models

The collapse of frame [2D, C] happens at a vertical load of 91.6kN/m and is initiated by the nonlinear buckling of the two adjacent to the removal columns. Since the imperfection is so small and the model is symmetric, the collapse mechanism is completely vertical. For frame [2D,B], the collapse happens at a load of 102kN/m and again is initiated by the nonlinear buckling of an adjacent column. Due to lack of symmetry, frame [2D,B] collapses laterally. Figure 4 shows the axial force in adjacent columns as a function of the applied load on the structure. For frames [2D, B] and [2D, C] the adjacent to the removal columns, which undergo inelastic buckling, reach an axial force after which they cannot take any more load. This point is combined with the dramatic increase of the horizontal displacements at the middle of their height indicating the appearance of inelastic buckling failure. This phenomenon combined with the deformed shape as shown in figure 2 shows that a nonlinear, inelastic buckling failure initiates the collapse of frames [2D, B] and [2D, C].

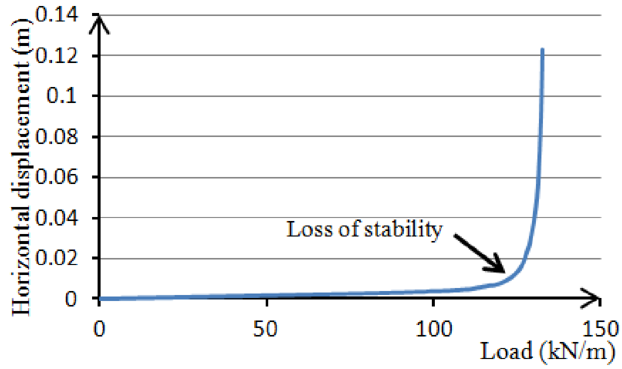

Figure 3. Horizontal displacement at the top of frame [2D, A]

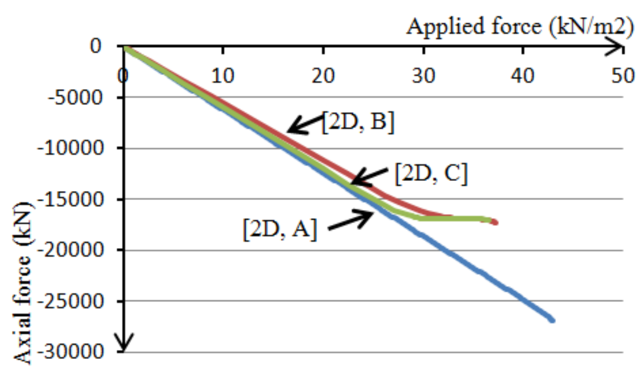

Figure 4. Acting axial force in adjacent columns as a function of applied force for the 2D models

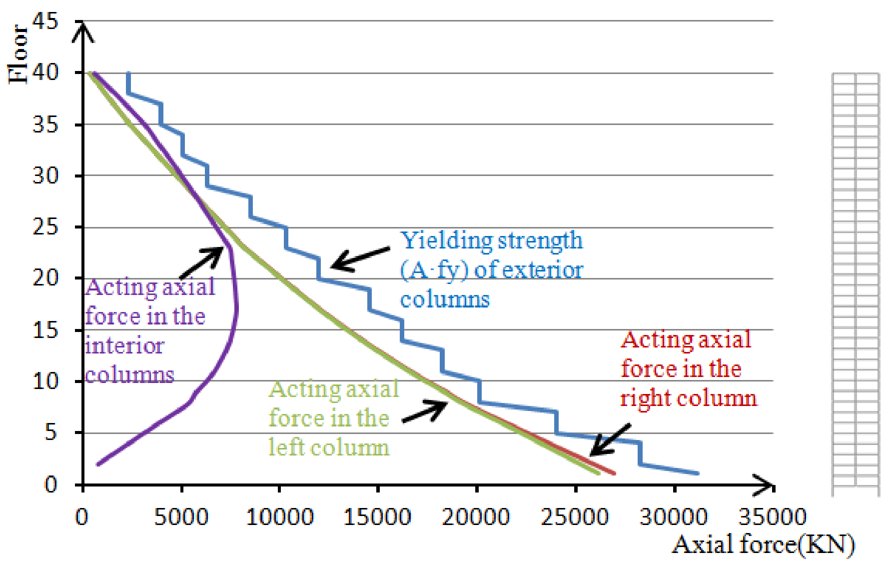

Figure 5. Axial force in columns at all floors of frame A

However, the collapse mechanism of frame [2D, A] is completely different, as shown in figure 2. For this case, the collapse mechanism involves a lateral collapse while the FEM model is completely symmetric. The analysis of frame [2D, A] is terminated due to the existence of negative eigenvalues at a vertical load of 133.1kN/m. As shown in Figure 3, the horizontal displacement at the top of frame [2D, A] is almost zero at the beginning but starts to increase rapidly at an applied load of 120kN/m. This phenomenon again indicates the occurrence of loss of stability, without the appearance of any local buckling failures in the adjacent columns. The axial forces in the two remaining columns keep increasing without reaching their yield strength when the collapse happens. This also can be seen in figure 5 which shows that the axial forces in all the columns of frame [2D, A] are below yielding strength. As a result, all the columns of frame [2D, A] remain elastic and do not experience any inelastic failure when collapse happens.

3.3 Description of 3D analysis

According to the 2D analysis, the slender frame (frame with 2 bays) is vulnerable to global loss of stability. To further investigate the potential global loss of stability of a slender steel building, a 3D analysis is performed. For the 3D analysis, the frame selected is again a steel moment resisting frame with exactly the same material and design provisions as for the 2D analysis. To identify global loss of stability, the frame has forty stories and two bays in both directions. The boundary conditions of the columns at the 1st floor are assumed to be pinned. The height of each story is 3m, while the width of each bay in the x direction is 6m and in the y direction is 5m so that the frame is no longer symmetric. The orientation of the columns is shown in figure 6, while the cross sections are listed in table 2. For the FEM analysis, the B32OS element has been assigned and no imperfection loads are applied since the frame is already non-symmetric. The following paragraphs present the results of eigenvalue analysis (denoted as E) as well as analysis with both material and geometric nonlinearities (denoted as M+G) in ABAQUS [7].

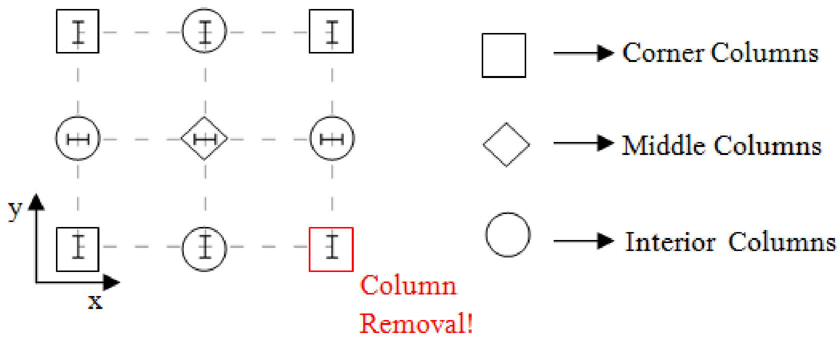

Figure 6. Beam orientation assignment for the 3D model

Floor	Corner Columns	Middle Columns	Interior Columns	Exterior Beams	Interior Beams
38-40	W14x43	W14x43	W14x43	W12x45	W12x45
35-37	W14x43	W14x53	W16x77	W12x45	W12x45
32-34	W14x53	W16x77	W18x97	W12x45	W14x61
29-31	W16x67	W18x97	W21x122	W12x45	W14x61
26-28	W18x86	W21x111	W21x147	W14x53	W16x77
23-25	W21x101	W21x132	W21x182	W14x53	W16x77
20-22	W21x111	W21x147	W24x192	W16x67	W18x86
17-19	W21x132	W21x166	W24x229	W16x67	W18x86
14-16	W21x147	W21x182	W24x250	W18x71	W21x93
11-13	W21x166	W24x207	W24x279	W18x71	W21x93
8-10	W21x182	W24x250	W27x281	W18x76	W21x101
5-7	W24x207	W24x279	W27x307	W18x76	W21x101
4	W24x207	W24x279	W27x307	W18x86	W21x111
2-3	W24x250	W30x357	W30x391	W18x86	W21x111
1	W24x250	W30x357	W30x391	W21x101	W24x131

Table 2. Cross section assignment of the 3D model

3.4 Results of 3D analysis

3.4.1 Eigenvalue analysis

Figure 6 shows the deformed shape of the eigenvalue analysis. The first eigenvalue is 38.9kN/m2, showing the weak axis buckling failure of part of the frame in the x direction.

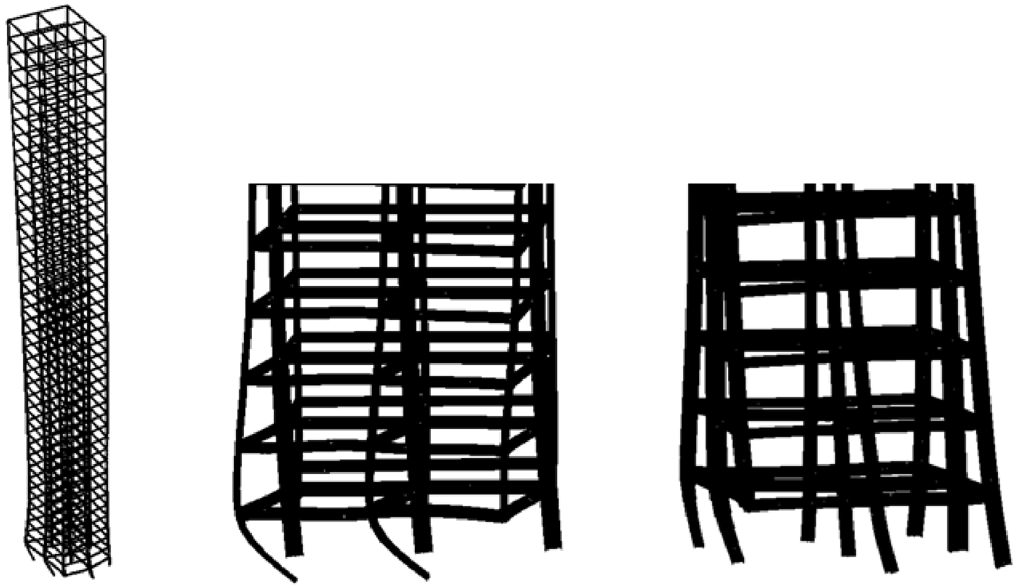

Figure 7. Collapse mechanism under eigenvalue analysis for the 3D model

3.4.2 M+G analysis

The M+G analysis is terminated due to the occurrence of negative eigenvalues at a load of 15.9kN/m2 which is much smaller than that of the eigenvalue analysis. During the eigenvalue analysis, all elements remain pure elastic while plastification and inelastic deformation are allowed to form during M+G analysis. This causes the significant drop of the collapse load from E to M+G. The deformed shape of the frame at collapse is shown in figure 8 and coincides with the one provided by the eigenmode analysis. The color contour on the elements denotes von Mises stresses. The black color is intentionally selected to show von Mises stresses which have exceeded the yield strength and the red color Von Mises stresses close to but not exceeding the yield strength.

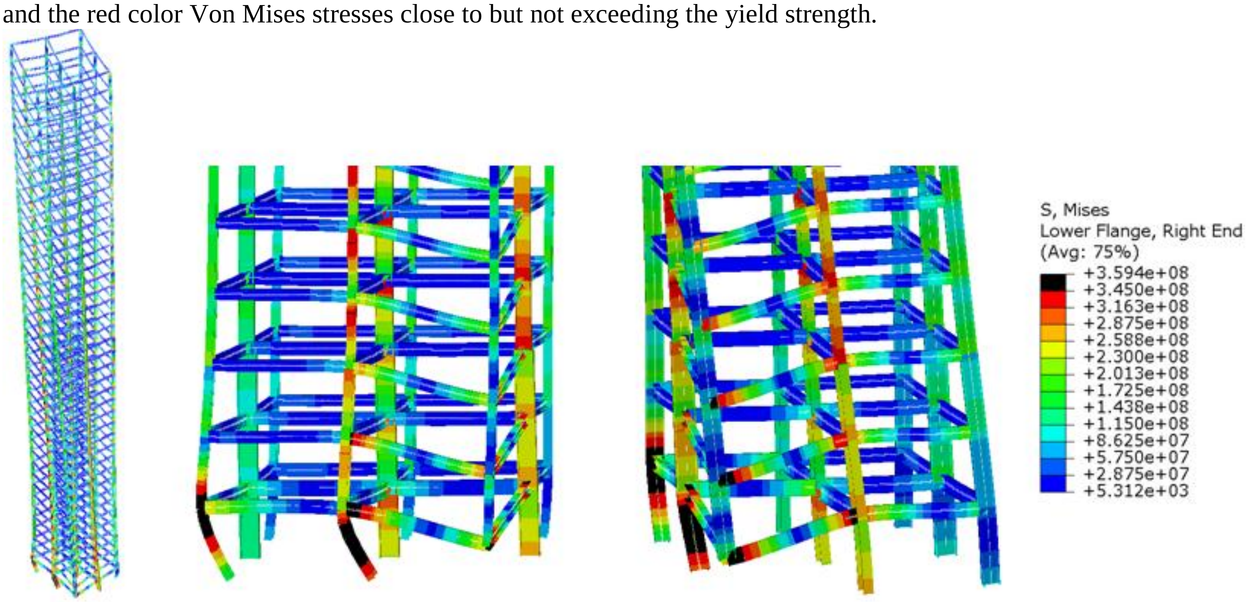

Figure 8. Collapse mechanism under M+G analysis for the 3D model

As mentioned before, the 3D M+G analysis is terminated due to negative eigenvalues which is a sign of losing stability. The loss of stability phenomenon can be identified through the development of horizontal displacements of the adjacent columns as shown in figure 9. The horizontal displacement in x direction increases significantly after a part of steady increase which clearly describes the occurrence of instability. This displacement figure also proves the out-of-plane buckling of the adjacent column which is shown in the collapse mechanism. The first graph in figure 10 shows the von Mises stress of the adjacent-to-the-removal column in the x direction and the second graph shows the von Mises stresses at the corner column next to that, again in the x direction. Obviously, the von Mises stresses at points 2, 4 have reached yield strength. Since plastification forms at the same side of two flanges, the out-of-plane buckling is again being proved to happen at both columns. Although two adjacent columns suffer out-of-plane buckling, they remain elastic as shown in figure 11. This clearly indicates that elastic out-of-buckling happens to these two adjacent columns.

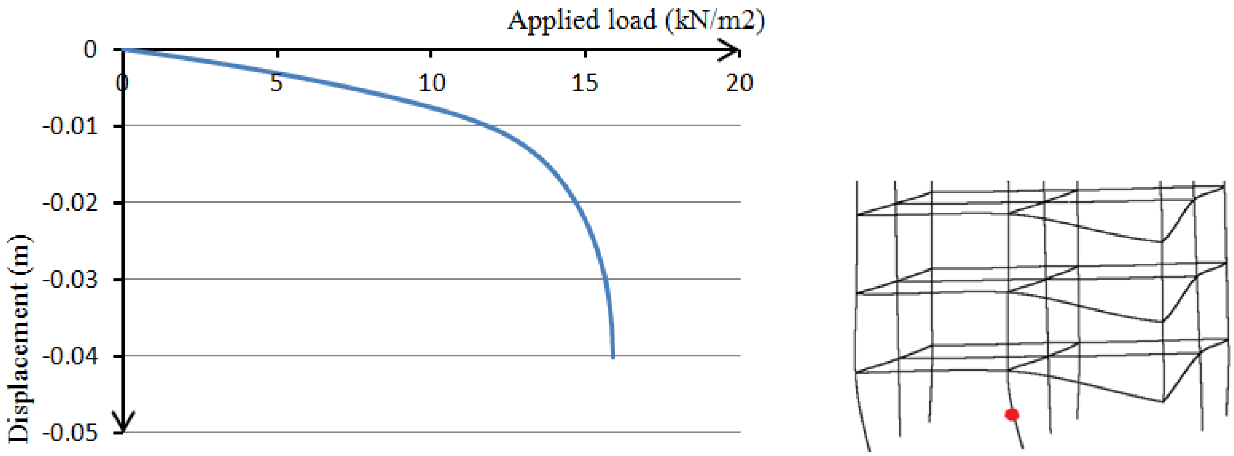

Figure 9. Horizontal displacement in x direction at middle of adjacent column for the 3D model

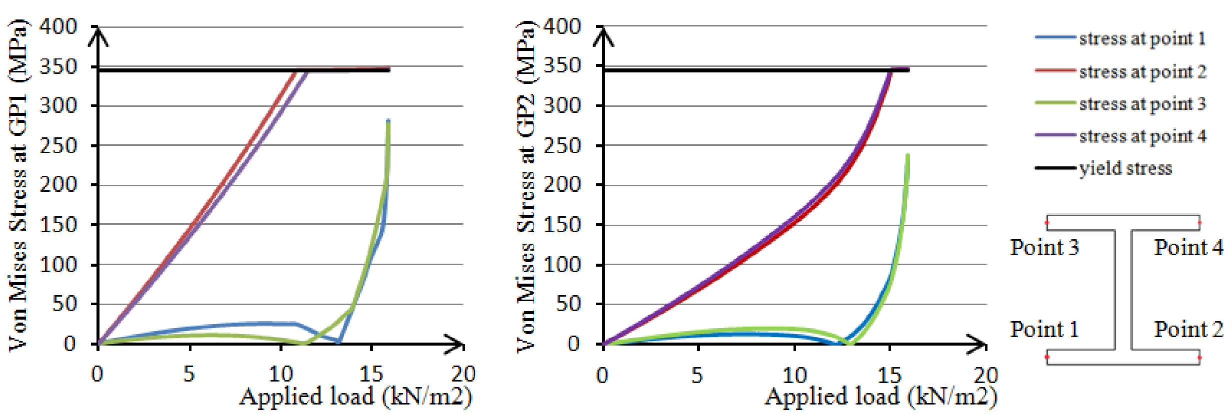

Figure 10. Von Mises stress at two adjacent columns in the x direction for the 3D model

4 CONCLUSION

This paper has shown that in 2D analysis, progressive collapse could happen while all elements remain in the elastic zone and are safe. Thus, it has identified that progressive collapse will happen due to global loss of stability without any local buckling. The appearance of limited yieldings/plastifications at the ends of the beams above the column removal, gradually degrade the global stability capacity of the whole structure and eventually lead to global collapse. This is very important in progressive collapse resistant design since it is very difficult to be observed but is a potential collapse mode for any slender building. Compared to 2D analysis, the respective global collapse mode load for the 3D analysis is significantly lower, identifying a weak axis global collapse mode. In addition, as shown in the 3D analysis, the acting axial forces of all the columns are much lower than their yield strength when the global collapse happens.

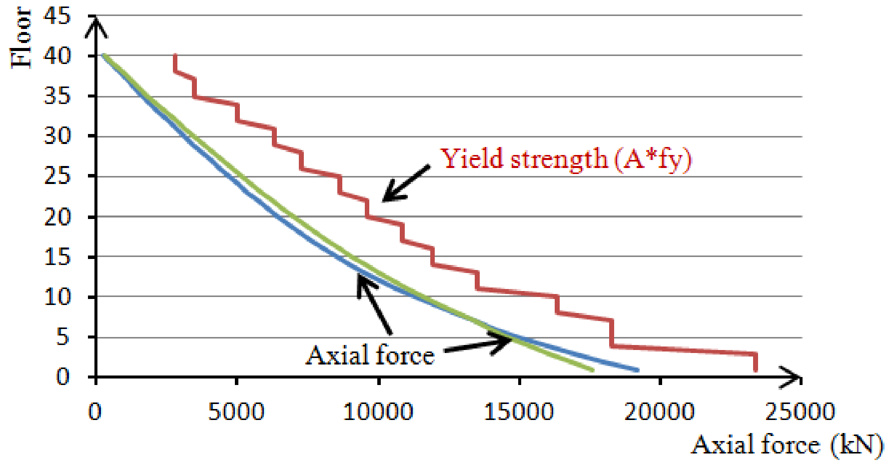

Figure 11. Axial force at adjacent columns

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