Evaluation of progressive collapse of special steel moment frames
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Abstract. The point of this study was to assess the progressive collapse resisting capacity of special steel moment frame structures and the behaviour of buildings which have different height when they are losing one of their exterior columns. Two buildings were considered for this research, 7-storied and 12-storied buildings. Corner column as well as one of the middle columns was removed to evaluate the importance and the effect of the location of removed column in structural response. General Services Administration (GSA) and Department of Defence (DoD) guidelines are considered for choosing the method of analysis. Nonlinear dynamic analysis procedures were carried out to investigate the behavior of structures. Thus, maximum vertical displacement in the point of column removal for each structure was measured. In addition, both buildings have cover plate connections which are considered to be rigid in modelling.

Introduction
Progressive collapse occurs when local damage of structural elements or failure of some structural components hastens collapse of all or a large part of structure. This complex phenomenon is always along with nonlinear behaviour and large deformation, therefore, the damaged system must find a new load path to transfer the forces from failed region to the stable one. Catenary action which is defined as an ability of beams to resist the vertical displacement could reduce the structural damage and distribute the forces from damaged region to the other part of structure to avoid or delay progressive collapse phenomenon. Catenary action also decreases the bending moment of the beams in the damaged part of structure.

The GSA [1] and the DoD [2] recommend alternate load path method for analyzing the structures potential of progressive collapse and each method has its specified load combination for dynamic analysis. In this methodology, the impact of unorthodox loading is presented by column removal. The GSA offers different places for column removal as a consideration for this type of loads (middle and the corner column of building). Thereafter, the structure is analyzed in order to find out whether the initiated damage could extend to the other part of structure or not. There are four types of analysis procedure in the guidelines for progressive collapse, being, linear elastic static (LS), linear dynamic (LD), nonlinear static (NS), and nonlinear dynamic (ND).

Marjanishvili [3] investigated merits and demerits of each type of analysis in detail. Kim and Kim [4] compared the response of various buildings that were subjected to the different analysis procedures to determine the differences of analysis methods. Powell [5] drew a comparison between different analysis methods and depicted that nonlinear analysis had more realistic result than linear analysis. Moreover, it was concluded that the impact factor that is represented in the GSA and considered as dynamic effects could be decreased.

In this study, nonlinear dynamic analyses have been used to appraise the progressive collapse potential of steel moment frames designed in accordance with AISC [6]. The results of nonlinear analysis procedure recommended by the GSA and DoD guidelines were compared. Furthermore, the influence of different parameters that could contribute to the performance of buildings during progressive collapse such as the location of column removal and the number of story were studied.
Analytical model

Acceptance criteria and load combination for nonlinear dynamic analysis. For nonlinear dynamic analysis procedures, the GSA and the DoD consider maximum plastic hinge rotation as well as ductility as criteria for progressive collapse. Table 1 depicts the acceptance criteria of steel beam and column ductility and rotation for progressive collapse in the GSA guideline [1].

<table>
<thead>
<tr>
<th>Component</th>
<th>Ductility</th>
<th>Rotation(rad)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel beams</td>
<td>20</td>
<td>0.21</td>
</tr>
<tr>
<td>Steel columns (tension controls)</td>
<td>20</td>
<td>0.21</td>
</tr>
<tr>
<td>Steel columns (compression controls)</td>
<td>1</td>
<td>-</td>
</tr>
</tbody>
</table>

The definition of ductility here is the ratio of ultimate deflection at the column removal point to the yield deflection.

For dynamic analysis the GSA [1] recommends the following load combination in every bay.

\[ DL + 0.25LL \]  

Where \( DL \) represents dead load and \( LL \) is live load.

In the DoD guideline [2] load combination is different from the GSA and the DoD proposes bigger load factor for \( DL \) and \( LL \) in comparison with the GSA and the wind load is also involved in load combination as it is shown in below:

\[ (1.2DL + 0.5LL) + 0.2WL \]  

Where \( WL \) corresponds to wind load.

As shown in Figure 1, for dynamic analysis, the load pattern which was recommended by Kim and Kim [4] is employed. Hence the amount of column’s axial force, bending moment, and shear force must be calculated before the column removal. Afterward, the column is replaced by equivalent concentrated loads in order to avoid undesirable vertical displacement in the column removal point. In this load pattern, the gravity loads and wind load are increased linearly and reached their full intensity in 5\(^{th}\) second and remained unchanged until the end of analysis procedures. The computed loads are also applied linearly and gradually in first five seconds and then remained constant for two seconds to let the systems gain a stable condition. Then these forces are abruptly removed at 7\(^{th}\) second to simulate the progressive collapse phenomenon and apply the dynamic effect of column loss to buildings.

![Fig.1 The method of inserting loads](image)

Structures characteristics. The structural plan of 7-story and 12-story buildings have been shown in Figure 2. Stories height and spans length are considered as 3m and 5m respectively. Each building has 5 spans in X direction and 3 spans in Y direction. The structures were assumed to be located at Tehran, Iran, which is regarded as a high seismic zone. Special moment resisting frame
system was used as a lateral and gravity load resisting system. The columns and beams were designed with ST37 \((F_y = 2400 \text{ kg/cm}^2)\). Structures’ beams were designed in I shape and the columns were designed in box shape.

![Fig.2 Structural plan for 7-story and 12-story models](image)

**Analytical modelling.** For dynamic analyses, the OpenSees program was used and exterior frame as it is shown in Figure 2 was modelled in the program. For modelling the plastic behaviour, the `beamWithHinges` element was employed [7]. For this type of element, the plastic hinge is defined at the beginning and the end part of each column and beam. In `beamWithHinges` element the nonlinear behaviour of members must be defined at plastic hinges. As a definition of `beamWithHinges` element the length between two hinges is considered to remain elastic during analysis. The post-yield stiffness of the structural members presumed to be 2% of the initial stiffness and as a routine consideration for large deformation analyses, 5% damping ratio was assumed.

**Nonlinear dynamic analyzes of structures for progressive collapse**

Nonlinear time history analyses were conducted with different conditions by removing the center and corner column to simulate the progressive collapse. The vertical displacements due to the time history for models with the GSA and the DoD load pattern are depicted in Figure 3. There are two graphs in each diagram, the simple solid line shows the results based on the DoD guideline and the double line type shows the results based on the GSA guideline.

![Graphs showing vertical displacements](image)

a) center column removal of 7-story frame  
b) corner column removal of 7-story frame
Figure 3 shows the vertical displacement for 7-story and 12-story frames.

Figure 4 and 5 show the rotation amount of plastic hinges in radian for 7-story model, which was analyzed with the GSA and DoD load pattern respectively.

Figure 4: Rotation of beams in radian for 7-story building subjected to GSA load pattern.

Figure 5: Rotation of beams in radian for 7-story building subjected to DoD load pattern.
When the 7-story structure was exposed to the DoD load pattern, as it is shown in Figure 5, the beams rotation increased considerably in comparison with the state that 7-story structure was analyzed with the GSA load pattern, as it is shown in Figure 4. The maximum beam’s rotation due to the DoD is about 15%, while this number is approximately 10% for the GSA load pattern. In both states the maximum beam’s rotation is less than acceptance criterion of 21%.

Conclusion

This study has investigated the progressive collapse potential of special moment resisting framed structures and the effect of story number and the location of removed column. The results have shown that the maximum of vertical displacement is reduced when the number of story is increased. Therefore, the results remarkably depend on the number of stories and when the story number is increased, the potential of progressive collapse is decreased which shows lower beams rotation for 7-story building. Moreover, the vertical displacements of models which have lost their corner columns is higher in comparison with the models where center column has removed. Thus, the structure which loses its corner column is more vulnerable compared to the one losing its center column. As it is shown in Figure 4 and 5, the beams rotation in the column removal span (when the center column was removed) has not exceeded the maximum specified rotation set by the GSA and the DoD guidelines. For the corner column removal state, the amount of plastic hinges rotation is increased, while the acceptance criteria like the center column removal is still satisfied.

References

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