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Evaluation of Progressive Collapse in Steel Structures Designed Based on Iranian Code of Practice for Seismic Resistant Design Buildings (Standard No. 2800), 4th Edition and Iranian National Building Code (INBC), Part 10

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ABSTRACT

This research takes a look at the gradual collapse of steel buildings with dual systems of moment resistant and braced frames on four, eight, and ten stories high. Errors in design or construction, fires, gas explosions, inadvertent overloading, vehicle accidents, bomb blasts, and other extreme events can all cause structures to gradually collapse. The dynamic study of these events appears important due to the action of these factors during a brief time period. The loss of a loved one unexpectedly is used to examine the impact of the aforementioned situations. Part 6 and Part 10 of the Iranian National Building Code, as well as the Iranian Code of Practice for Seismic Resistant Design Buildings (Standard No. 2800), Fourth Edition, were used to create the structures in the study. Abaqus finite element software is used to model the structural frames and evaluate the forces and displacements experienced by the members. Subsequently, the loads and their application to the structure, components of the analysis process model (APM), and the unexpected loss of members are used to establish the dynamic response of the structure. Analyses of the braced frames analyzed indicate that the loss of a center column is more detrimental than the loss of a corner column. To rephrase, the corner columns of a perimeter frame are more secure than the center ones.
Key words: Progressive collapse, steel structures, Iranian Code of Practice for Seismic Resistant Design Buildings (Standard No. 2800), progressive failure, Abaqus.

1. INTRODUCTION

Aprogressive collapse is the breakdown of an entire structure or a significant portion of it due to the spread of an initial local rupture from one component to another. Design or construction flaws, fires, gas explosions, inadvertent overloading, automobile accidents, bomb explosions, and so on are all potential threats and unexpected loads that might lead to gradual collapse. These threats are less likely to materialize, thus they are either ignored throughout the design process or dealt with through indirect methods. Most of them could trigger an event in a short amount of time and result in shifting reactions. Researchers in the 1970s detected progressive collapse when a tower near Ronan Point, England, collapsed in part. When the World Trade Center was attacked by terrorists on September 11, 2001, gradual collapse once again became a major topic of discussion. Designing for loads that may be imposed during the lifespan of a structure is permissible under current building rules. Most buildings aren't made to withstand the kinds of rare events that might bring about a worldwide collapse. Most building codes only offer broad suggestions for mitigating the impact of progressive failure on buildings subjected to loads in excess of their design capacities. For the prevention of gradual collapse, the three most commonly cited design approaches are. The first strategy involves minimizing vulnerability to losses, whereas strategies two and three are employed to offer escalating resistance to collapse (1, 2). Lew proved that the link in the chain

action can greatly lessen the flexure by bracing the beam axially (3). Using several different types of seismic connections, Park and Kim (4) investigated the possibility of progressive collapse in metal structures. The investigation revealed that the reduced beam section (RBS)'s extremely flexible behavior gives the highest load bearing capability against collapse. Kim demonstrated that the probability of a slow collapse diminishes with increasing height. The unconventionally braced frame, as proposed by Khandelwal et al. (6), is substantially more resistant to slow collapse than the special concentrically braced frame (SCBF). When compared to the two variables given by GSA and UFC (7), Kim et al. showed that the DAF is larger. Under typical and uniform conditions, Fu observed, the loss of a column at a higher level would result in a greater vertical displacement than the loss of a column at a lower floor (8). Kim et al. came to the conclusion that, among the several pre-fabricated frame types, the inverted V-type frame is the most adaptable to slow collapse. Asgarian and Hashemi Rezvani shown (in the number 9), the strength of a concentrically braced structure is affected by the number of braced spans. A two-story moment frame's resistance to slow collapse following a sudden loss of a ground-level column was tested experimentally by Chen et al. (10) with and without a concrete base. During the load redistribution process, the results demonstrated that a concrete foundation plays a significant role and reduces the risk for slow collapse. When Chen et al. (11) examined the effect of horizontal bracing on the resistance of a steel moment frame to progressive collapse, they found that the displacement and rotation angle of the model with horizontal bracing were much lower than those of the model without horizontal bracing. Gerasimidis determined the likelihood of a steel frame's slow collapse due to the loss of a corner column (12). To illustrate the process of a steel frame's collapse due to the loss of a corner column, he suggested an analytical approach based on the development of a critical ductility curve. Tavakoli and Rashidi (13) investigated the possible flexural strength of a seismically loaded multi-story steel structure with damaged columns in various locations. According to their research, if a building experiences internal column loss instead of corner column loss, the structure will be more stable. They also talked about how, as a building's story count grows, the building's resistance to progressive collapse under lateral pressures improves. This is because more structural parts are involved in preventing the transfer of extra loads. After studying the fragility of an 11-story steel moment frame, Hosseini et al. and Yousefi et al. came to the same conclusion: the loss of a corner column on the bottom level causes the failure of the neighboring building (14, 15). According to the literature, prior researchers have paid little attention to how span length affects the resistance of steel moment frames to slow collapse. The purpose of this investigation is to evaluate how seismically engineered steel moment frames behave and how much resistance they have to slow collapse as a function of span length. Thus, three structures are planned employing a moment resistant steel frame with varying span lengths and a fixed frame length. After that, we check each UFC's first-floor column loss by analyzing the perimeter frame. To further analyze the structure's response to a column loss on the ground level, we compute the dynamic amplification factor (DAF) and the demand/capacity ratio (DCR).

2. MATERIALS AND METHODS

2.1. Geometry of model

Three Iranian high-rises (Figures 2, 3, and 4) with 4, 8, and 10 stories are planned using ETABS software and traditional structural sections. Floor heights are considered to be 3 m throughout these steel structures, and there is a uniform layout throughout (see Figure 1). To withstand lateral loads, a modest moment frame in one direction is paired with bracing in the opposite direction. The beams to the columns and the columns to the foundation both employ pinned and permanent connections. The ST37 steel used in all parts has a 3700 kg/cm2 ultimate stress and a 2400 kg/cm2 yielding stress. The floor dead and live loads are 200 and 335 kg/m2 while the roof dead and live loads are 150 and 310 kg/m2. Assuming that the building is in Iran's Seismic Zone 4, the resulting seismic loads may be calculated. Tables 1, 2, and 3 display the final structural design findings.

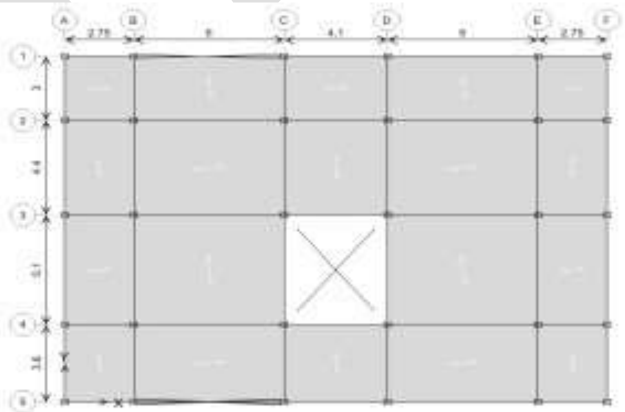

Figure 1

. Typical plan of studied buildings

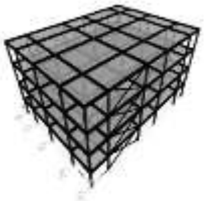

Figure 3. 3D view of 8-story building assessed via ETABS

Figure 4. 3D view of 10-story building assessed via ETABS

Table 1. Design results of members of 4-story steel structure

Floors	*Columns*	*Main beams*	*Braces*
Ground floor	BOX 25×25×1	2IPE270	2UNP100
First floor	BOX 25×25×1	2IPE270	2UNP100
Second floor	BOX 20×20×1	2IPE240	2UNP100
Third floor	BOX 20×20×1	2IPE240	2UNP80

Table 2. Design results of members of 8-story steel structure

Floors	*Columns*	*Main beams*	*Braces*
Ground floor	BOX 45×45×1.6	BOX 25×25×1	2UNP140
First floor	BOX 45×45×1.6	BOX 25×25×1	2UNP140
Second floor	BOX 40×40×1.6	BOX 25×25×1	2UNP140
Third floor	BOX 40×40×1.6	BOX 25×25×1	2UNP120
Fourth floor	BOX 40×40×1.6	BOX 25×25×1	2UNP120
Fifth floor	BOX 30×30×1.6	BOX 25×25×1	2UNP120
Sixth floor	BOX 30×30×1.6	BOX 25×25×1	2UNP100
Seventh floor	BOX 30×30×1.6	BOX 25×25×1	2UNP100

Table 3. Design results of members of 10-story steel structure

Floors	*Columns*	*Main beams*	*Braces*
Ground floor	BOX 50×50×1.6	BOX 35×35×1.6	2UNP140
First floor	BOX 50×50×1.6	BOX 35×35×1.6	2UNP140
Second floor	BOX 45×45×1.6	BOX 35×35×1.6	2UNP140
Third floor	BOX 45×45×1.6	BOX 35×35×1.6	2UNP120
Fourth floor	BOX 45×45×1.6	BOX 35×35×1.6	2UNP120
Fifth floor	BOX 35×35×1.6	BOX 30×30×1.6	2UNP120
Sixth floor	BOX 35×35×1.6	BOX 30×30×1.6	2UNP100
Seventh floor	BOX 35×35×1.6	BOX 30×30×1.6	2UNP100
Eighth floor	BOX 30×30×1.6	BOX 30×30×1.6	2UNP80
Ninth floor	BOX 30×30×1.6	BOX 30×30×1.6	2UNP80

The examples are created using the same criteria as the actual buildings. There are three operational buildings under consideration. The floor height is 3.2 m and the spread is 5 m in all three examples. According to the Iranian National Building Code, parts 6 and 10, which cover the loads applied to buildings (10) and the design and construction of steel buildings (11), respectively, three buildings with 4-, 8-, and 10-story dual systems of moment resistant and concentrically braced frames are designed to investigate progressive collapse in braced steel structures. Design (12) takes into account the seismic standards of the Iranian Code of Practice for Seismic Resistant Design Buildings (Standard No. 2800). ETABS was used for the building's design. The skeleton is virtualized.

with Abaqus' nonlinear time history analysis, we can look into the forces in the frame members and the displacements at the nodes based on the steel sections for the beams, columns, and bracing; from there, we can figure out the system's dynamic response to the loads, the manner in which the loads are applied, the APM components, and any sudden column losses.

2.2. Modeling hypotheses

- Connections between beams and columns, frames and steel plates, and columns and supports are all fastened.
- The modeling is executed in N and mm units.
- Temporal comparisons of the samples are made

In addition to its application in historical research, the von Mises yield criteria provides a standard for determining material yield and assessing the quality of steel components.

Proposed structural models are assessed for their vulnerability to gradual collapse over 5 distinct states. First, the finite element models are examined in an external frame (no column loss). The second version involves the removal of a corner column on the first level. In the final configuration, the exterior frame is missing a central column. The fourth condition involves an internal frame analysis of the finite element models with no column loss. In addition, the fifth configuration hides the internal frame's central column. Table 4 and Figure 5 detail the various states.

Table 4. Introduction of states without column loss

State	*Frame position*	*Column position*
First	External	Without column loss
Second	External	A1 (ground floor)
Third	External	D1 (ground floor)
Fourth	Internal	Without column loss
Fifth	Internal	D2 (Ground floor)

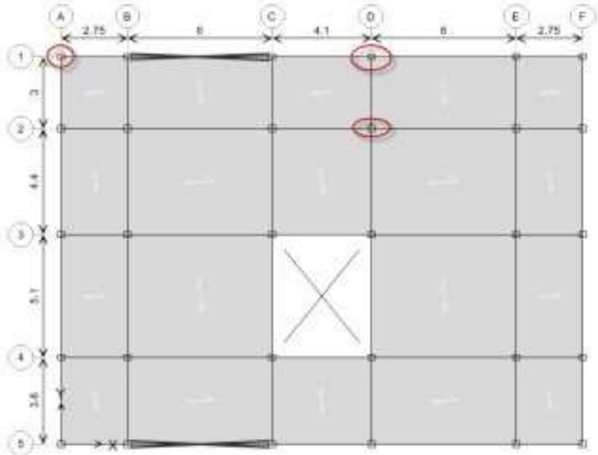

Figure 5. Position of column losses

2.3. Loading

Loading on entire spans is according to the equation below.(1)

$$G_N = \Omega(1.2DL + o,5LL) + 0.002\sum DL + LL$$

where LL and DL are the live and dead loads, respectively, $0.002\sum DL + LL$ is the lateral load assumed for each story and separately applied to 4 sides of the building, G_n

is the gravity load for the whole structure and Ω_N is the column loss factor calculated according to the UFC 4-23- 03 using Equations 2-4.

(2)

3. RESULTS AND DISCUSSION

3.1 Axial force criteria for columns

When columns are removed at various stages, the load is transferred to neighboring members, who must be able to support the increased pressures that result. When a column is removed, the distribution of forces in the remaining members may be observed by measuring the axial forces in the columns on each side of the empty space. It is important to note that other columns can still hold the capacity to carry the imposed load even in case of removal of some primary load bearing components thanks to the seismic design of all sections and the lack of interference of seismic loadings during progressive collapse.

$$\Omega_N = 1.08 + 0.76\left(\theta_{pra}\ \theta_y + 0.83\right)$$

Subsequently, the percentage of axial force changes in columns in the frames of 4-, 8- and 10-story buildings are listed in Figure 6 and Figure 8.

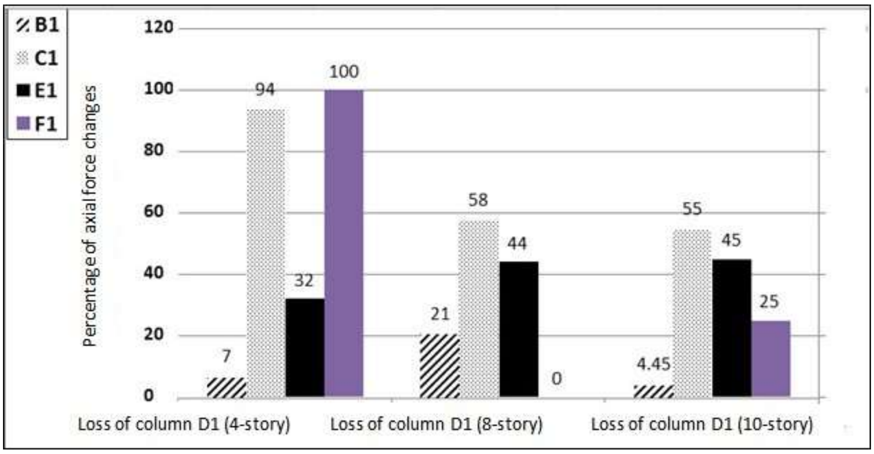

Figure 6. Axial force changes for loss of column D1

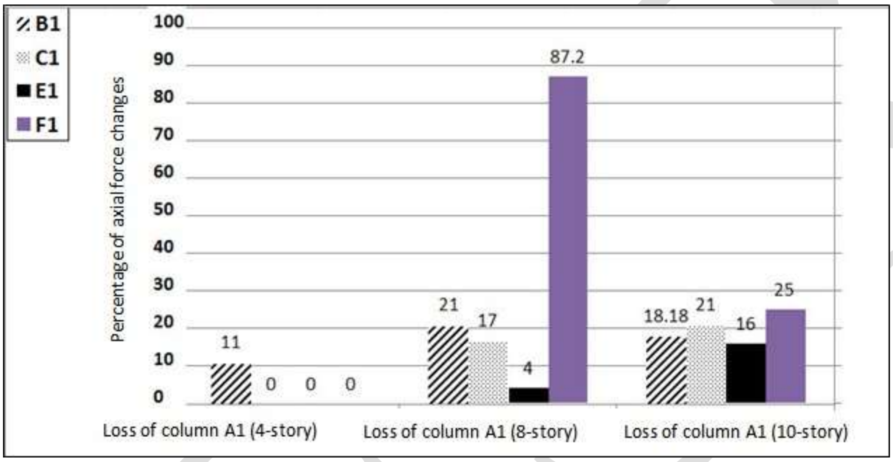

Figure 7. Axial force changes for loss of column A1

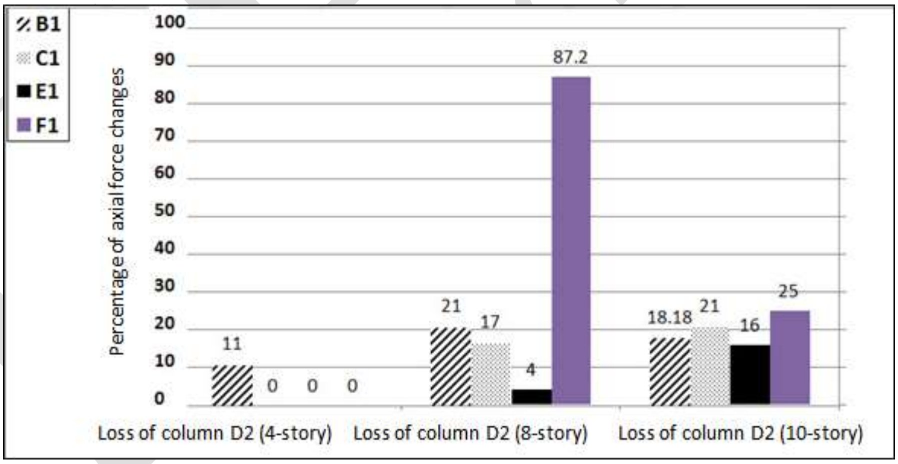

Figure 8. Axial force changes for loss of column D2

Figure 6 shows that when the third state column D1 is eliminated, the corner column F1 experiences the greatest axial force changes. Since only the axial force of column B1 changes when corner column A1 is removed, column B1 aids in preventing the gradual collapse of the structure when column A1 is removed. However, the existence of cross bracing in span BC is a major factor in why the axial force changes of columns C1, E1, and F1 are all equal to zero. This is because bracing can aid in the redistribution of forces in the members next to the position of the deleted column.

Column D2 is lost, and the axial force changes as a percentage, as seen in Figure 8. In the actual framework, D2 is a central support column. When a column is removed from the internal frame of a structure, the columns around it undergo far more changes than they did before (column loss in the external frame), with the axial force reaching 2.6 times the initial value in some columns (such as column C1). Therefore, it can be inferred that the column loss in interior frames demonstrates a more significant condition of structural behavior than the loss of external columns when evaluating the possibility for progressive collapse in the structure. In addition, the bearing capacity of the columns of the structure can be substantially helped by the bracing of the exterior frame after the process of column loss, while the internal frame under the structure has no bracing.

3.1.1 Brace axial force criteria

The effect of bracing on the redistribution of axial forces following the processes of column loss is examined in Figures 9, 10, and 11 by looking at the differences between state 2 and state 3. When column A1 close to the braced span is removed, the axial force exerted by the brace is plainly much lower than when column D1 is removed. When a central column is removed, more axial force is created in the braces than when a corner column is removed. Therefore, it is crucial to give the loss of the center column more weight than the loss of the corner columns when planning for progressive collapse in a structure's bracing. This is correct regardless of the number of stories in each of the three buildings (4, 8, or 10).

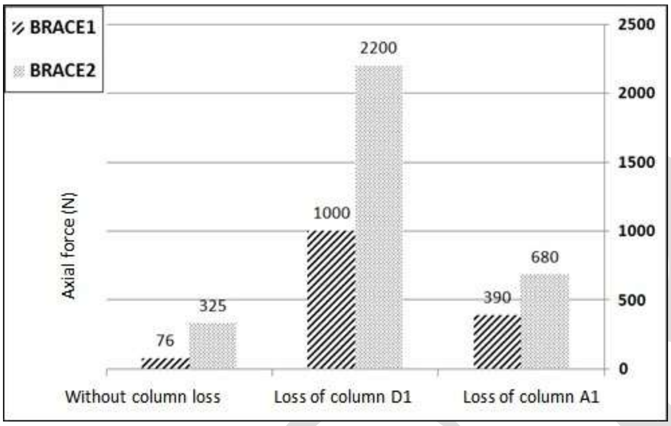

Figure 9. Comparison of forces produced in braces for 4-story building

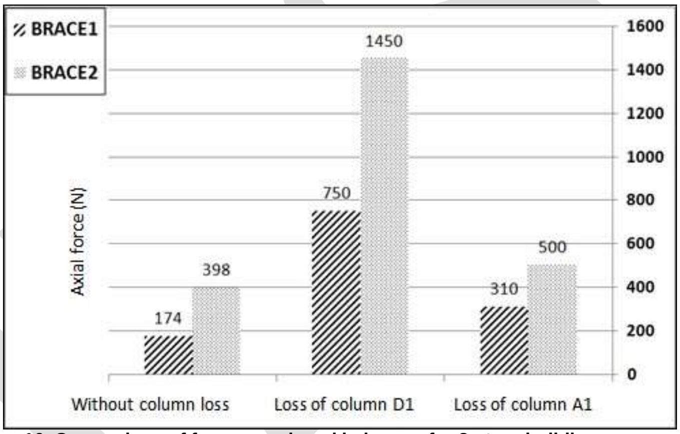

Figure 10. Comparison of forces produced in braces for 8-story building

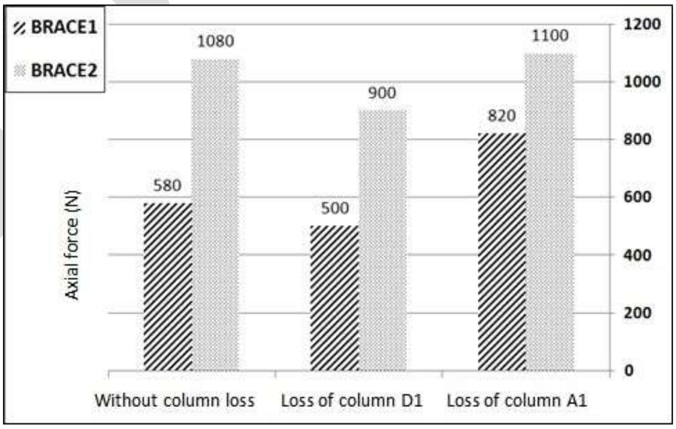

Figure 11. Comparison of forces produced in braces for 10-story building

3.1. Resistance criterion

An acceptance criterion for the alternative load path is the demand/capacity ratio (DCR) described in form of Equation 3.

(3)

$$DCR = \frac{Q_{UD}}{Q_{UC}}$$

Q_{UD} is the force calculated by the analysis of a member or connection and Q_{UC} is the expected capacity for a member or connection.

DCR values are calculated for the critical beams of studied models and shown in form of a chart in Figure 12. Evidently, the DCR values are less than 2 for all beams, so all of them are within rang

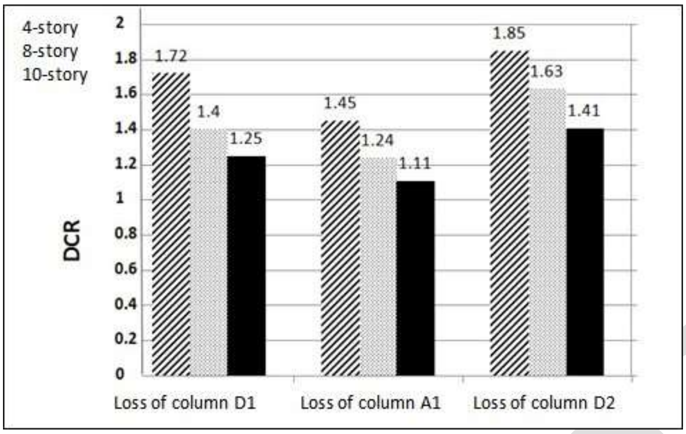

Figure 12. Comparison of DCR values for beams in second, third and fifth states

The fifth state (loss of column D2) appears to be a more essential condition for evaluating the potential of progressive collapse in the structure when compared to the loss of an external column when evaluating DCR values for the beams in the second and third states. DCR values for the third state are greater than those for the second state when comparing states with the loss of column A1 and those with the loss of column D1. It follows that the progressive collapse resistance of the building is less when an interior column is removed from the exterior frame than when a corner column is removed. Another takeaway from Figure 7 is that DCR values fall with rising floor height, which indicates that there is less of a risk of increasing collapse as the building gets taller.

3.5. Displacement criterion

Figure 13 presents the maximum displacements at the position of column loss in the second, third and fifth states for the 4-, 8- and 10-story buildings. As observed in most states, the displacements at the position of column loss for the 4-story building are more than those for the 8- and 10- story buildings. This means that as the height increases, thedisplacement at the position of column loss decreases. Therefore, it can be concluded that as the height increases, the potential of progressive collapse declines in the structure

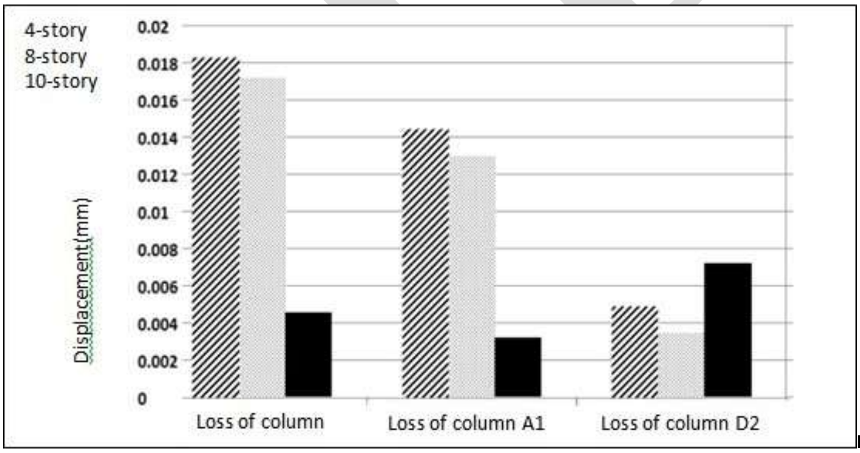

Figure 13. Comparison of displacements at position of column loss in different states

4. Control of criteria for displacement of members

The hinge rotation (θ) is another criterion of the member's response, which makes the maximum displacementresponse a function of the length of member's span and indicates the percentage of instability in critical regions of the member. In this study, the steel buildings are conventional structures that their performance levels are at the collapse threshold. The allowable values of ductility and hinge rotation are 2 and 0.035, respectively (16). The calculated ductility and maximum plastic hinge rotation of members are listed in Table 5. Obviously, all models do not exceed the allowable values specified by the codes.

Table 5. Ductility and plastic hinge rotation values for members

Third state		*Second state*		*First state*		*Stories*
μ	**θ_p**	**μ**	**θ_p**	**μ**	**θ_p**	
1.102	0.0113	1.48	0.012	1.906	0.0117	4
0.731	0.0211	1.006	0.0193	1.504	0.0155	8
1.237	0.0294	1.697	0.0143	1.697	0.0346	10

4. CONCLUSION

After the analysis process has converged, the following findings are derived from the deformed system model.

• According to the findings of research, the loss of center columns in braced frames is more detrimental than the loss of corner columns. In a perimeter frame, the middle columns are more at risk than the outside ones.The beam-connected columns nearest the deleted column are the ones that do the most to redistribute the load. It appears that the increased capacity of columns close to the deleted column plays a vital role in preventing progressive collapse, and the effect of nearby columns is greater than that of other columns.

• Due to the comparatively robust seismic design of members and short spans in typical steel structures in Iran, it is discovered that the computed DCR values are significantly less than the permissible range of GSA.

• In spite of the fact that the building in question satisfies all GSA criteria and the Iranian National Building Code, Part 6 and Part 10, as well as the Iranian Code of Practice for Seismic Resistant Design Buildings (Standard No. 2800), are successful in preventing progressive collapse, there is still room for improvement in the evaluation of the effect of irregularities across the plan and height, longer beam spans, and a greater number of stories.

• When considering the combined effects of lateral and gravity stresses, braced steel frames seem to be more stable than the examined moment frames.

• In a 4-story structure, the displacement at the location of column losses is larger than in an 8-story building, and in a 10-story building, it is larger than the value for the 8-story building. This suggests that the amount of movement at the location of column loss decreases with increasing height. It follows that the probability of a gradual collapse reduces with increasing height.

• When evaluating the possibility for progressive collapse, the fifth state (loss of column D2) appears to be more crucial than the loss of an exterior column, as evidenced by a comparison of DCR values of the beams in states 2, 3, and 5. DCR values in the third state are greater than those in the second state (loss of column A1) and the first state (loss of column D1). The progressive collapse resistance of the building is therefore concluded to be less for a loss of a central column in the exterior frame than for a loss of a corner column.

• As the building rises in height, the DCR value drops, meaning that there is less of a chance of a gradual collapse.

• The influence of bracing on the redistribution of axial forces following the processes of column loss is shown by comparing the second state (loss of column A1) with the third state (loss of column D1). It has been seen that the axial force produced in braces is significantly lower when the column A1 adjacent to the braced span is removed as compared to the loss of column D1. To rephrase, the loss of a center column causes a greater axial force to be created in braces than the loss of a corner column. This shows that the loss of a central column leads to a more serious state in braces than the loss of a corner column, hence braces should be designed with this possibility of gradual collapse in mind. These facts apply to all three buildings, whether they have 4, 8, or 10 stories.

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