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Evaluation of progressive collapse potential of multi-story moment resisting steel frame buildings under lateral loading

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KEYWORDS

Progressive collapse;
3-D push-over analysis;
Moment resistance frame;
Alternate load path.

Abstract The aim of this study is to investigate whether MRF steel structures that have been designed based on seismic codes, are able to resist progressive collapse with damaged columns in different locations under seismic loading.

For this purpose, 3-D and 2-D push-over analysis of structures is carried out. The progressive collapse potential has been assessed in connection with 5 and 15-story buildings with 4 and 6 bays by applying the alternate load path method recommended in UFC guidelines.

Member removal in this manner is intended to represent a situation where an extreme event, such as vehicle impact or past earthquake shock or construction error, may cause a critical column, as a result of local or global buckling, to lose a part or whole of its load bearing capacity.

In contrast with 3-D models, two-dimensional frames represent a higher sensitivity to base shear reduction and element removal. In the case of middle column removal, the structure is more robust than in a corner column removal situation. The influence of story number, redundancy and location of critical eliminated elements has been discussed.

1. Introduction

Structural safety has always been a key preoccupation for responsible for the design of civil engineering projects. One of the mechanisms of structural failure that has gathered increased attention over the past few decades is referred to as progressive collapse. One or several structural members suddenly fail, whatever the cause (accident, attack or earthquake), and the building then collapses progressively, every load redistribution causing, in turn, the failure of other structural elements, until the complete failure of the building or a major part of it [1].

Examples of structures having suffered a partial or full progressive collapse are actually quite few and far between. This phenomenon is now gradually taken into account in design standards because of the catastrophic nature of its consequences, rather than for its high probability of occurrence. The attention of the engineering community was first drawn to the issue of progressive collapse following the partial collapse of a building called 'Ronan Point', in London, in 1968. The field of structural response to abnormal events has even gathered further impetus following the unfortunate events of September 11, 2001. Several normalization committees started to rethink and improve their standards pertaining to progressive collapse design procedures. These include, for instance, the United States Department Of Defense (DOD) or UFC, General Services Administration (GSA), and Euro codes.

Various definitions may be found for the term 'progressive collapse'. NIST, the United States National Institute of Standards and Technology proposed that the professional community adopt the following definition: 'Progressive collapse is the spread of local damage, from an initiating event from element to element, resulting eventually in the collapse of an entire structure or a disproportionately large part of it, also known as disproportionate collapse' [2].

Progressive collapse design strategies consist of identifying three approaches for progressive collapse mitigation. This classification unfolds as follows:

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(a) Specific local resistance;
(b) Alternate load path method;
(c) Prescriptive design rules.

Approaches (a) and (b) are referred to as direct, while approach (c) is referred to as indirect. The Alternate load Path Method (APM) consists in designing the structure so that stresses can be redistributed following the loss of a vertical bearing element. Many degrees of idealization may be adopted for the design process, ensuring the existence of alternate load paths, ranging from static linear analysis through static nonlinear analysis, to dynamic linear or nonlinear analyses. This alternate load path approach was selected as the preferred one by several standards, such as GSA and UFC. Both organizations have issued guidelines that specify fully detailed computational procedures. The present work focuses on this approach.

Jinkoo Kim and Tawan Kim [3] investigated nonlinear dynamic and linear static procedures using two-dimensional frame analysis. They found that steel moment frames designed for lateral loads, as well as gravity loads, are less vulnerable to progressive collapse. It was observed that the potential for progressive collapse was highest when removing a corner column and the possibility of failure decreased as the number of stories increased. Hartato Wibowo [4] showed that progressive collapse phenomena can occur during earthquakes; therefore, it is not limited only to gravity and blast loads. Wibowo and Lau [5] also focused on the significance of seismic load effects in the progressive collapse behavior of structures. It is concluded that the seismic progressive collapse of structures can be analyzed by modifying the current analysis procedures. Kapil Khadelwal and Sherif El-Tawill [6] investigated the progressive collapse resistance of seismically designed braced frames, namely; Special Concentrically Braced Frames (SCBF) and Eccentrically Braced Frames (EBF). The simulation results showed that the EBF system is less vulnerable to progressive collapse than the SCBF system. Min Liu [7] used structural optimization techniques for design of seismic steel moment frames with enhanced resistance to progressive collapse. The potential of progressive collapse is assessed using the alternate path method with each of three analysis procedures (i.e. linear static, nonlinear static and nonlinear dynamic), as provided by UFC. The 2-D numerical example showed that the linear static procedure has the most conservative result. In contrast, nonlinear static and dynamic procedures lead to a more economical design. Jinkoo Kim and Jun-Hee Park [8] studied the sensitivity of the design parameters of steel buildings subjected to progressive collapse. The analysis result showed that among the design variables, the beam yield strength was ultimately the most important design parameter in moment resisting frame buildings, while the column yield strength was the most important design parameter in dual system buildings. Jinkoo Kim [9] investigated the progressive collapse performance of 3-D irregular tall buildings by nonlinear static and dynamic analyses, and found that the buildings with more structural elements have more resistance against progressive collapse. Feng Fu [10] used a 3-D finite element modeling method for the progressive collapse analysis of high-rise buildings and compared numerical results with experimental data. Z.X. Li [11] explored seismic damage and the progressive failure of steel structures via a numerical procedure.

As we know, progressive collapse is a phenomenon that can occur because of human-made or natural hazards. Research into the progressive collapse in structures generally focuses on gravity and blast loading, and intends to determine the capacity of a structure to resist abnormal loading. Nevertheless, earthquake load may also cause progressive collapse, due to partial or complete failure of critical elements. Consider a malfunctioned column in the first story of a building, due to design and construction error or vehicle impact. The mainshock of a repeated earthquake can cause the load bearing capacity of this element to be lost. When the structure reaches equilibrium condition, occurrence of strong after-shocks puts the structure with the damaged column in a lateral loading condition. In this work, the progressive collapse behavior of model structures under this scenario is assessed.

Table 1: Progressive collapse acceptance criteria.

Component	Rotation (rad)-(GSA2003)	Rotation (rad)-performance level (UFC2009)
Steel beams	0.21	CP
Steel columns	0.21	LS

Table 2: Progressive collapse acceptance criteria for ductility (GSA2003).

Component	Ductility
Steel beams	20
Steel columns	20

Moment resistance frames are the most popular structural systems commonly used in regions of high seismic risk. However, the progressive collapse behavior of MRF steel structures, when critical members are lost, is yet to be investigated and is the focus of this paper.

The Unified Facilities Criteria (UFC) guideline [12] employs the Alternate load Path Method (APM) to ensure that structural systems have adequate resistance to progressive collapse under abnormal or blast loading. Nonetheless, the present work applied this method to seismic loading. This methodology generally follows a "missing column" scenario to assess the potential of progressive collapse, using a push-over analysis. It is checked whether or not a building can successfully absorb the loss of a critical column. Naturally, all procedures are based on several assumptions and levels of idealization, but the absence of computational methodology in standards, and the lack of validation in others, justifies the need for a research program focusing on alternate load path analysis for structures under earthquake load.

The objective of this paper is to study the progressive collapse potential of steel moment frames under lateral loading designed per the Iranian building code [13]. The results of pushover analysis in target displacement were compared with UFC and GSA acceptance criteria for nonlinear analysis.

The results of these analyses consist of the capacity curve and the quality of plastic hinge generation in elements. These results are compared with acceptance criteria and, then, the residual shear capacity index (R) of the damaged structure is evaluated. The results vary more significantly depending on variables such as the number of spans, the location of column removal and the number of building stories.

2. Analysis methodology

2.1. Progressive collapse acceptance criteria

For the nonlinear analysis procedures, UFC and GSA guidelines specify maximum plastic hinge rotation as the acceptance criteria for progressive collapse. Table 1 shows these acceptance criteria.

GSA (2003) guideline [14] also specifies maximum acceptable ductility for members as shown in Table 2.

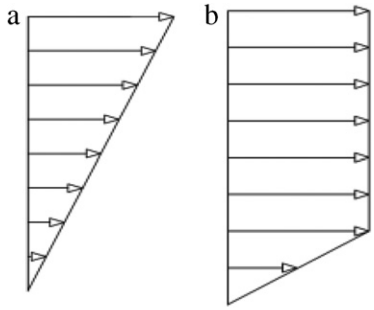

Figure 1: Applied lateral loading distribution patterns. (a) Triangular pattern; (b) uniform pattern.

2.2. Analysis method for progressive collapse under lateral loading

Progressive collapse refers to the phenomenon whereby the local damage of structural elements caused by abnormal loads results in the global collapse of the structure. Real life structures are obviously three-dimensional. The 3-D simulations can include 3-D effects for the flexural behavior of the members. The push-over terminology is very common in seismic design, where practitioners often use such quasi-static equivalent analyses.

To carry out nonlinear static analysis for progressive collapse under seismic loading, 3-D and 2-D models of a multi-story MRF steel structure were considered and the push-over analysis, in the presence of partially or completely damaged columns, was used.

In this study, it is supposed that a critical column has lost 40%, 70% and 100% of its effective area, respectively, due to past events. When push-over analysis was performed on these models, with different locations of column elimination, the hinge rotation in beams and columns was checked and compared with progressive collapse acceptance criteria.

2.3. Lateral load patterns introduction and applied load

There are probabilities for several various modes of damage. It is necessary that selected lateral loading distributions produce the critical damage mode. According to FEMA356 [15], at least two types of load distribution pattern must be used. In this work, these patterns are selected as follows and shown in Figure 1:

I. Distribution, which is proportional to lateral loading distribution in structural height, according to the equivalent static method (triangular distribution).

II. Uniform distribution in which the lateral load is distributed proportionally with each story weight.

The gravity load on the members is a combination of Dead Load (DL) and Live Load (LL). The load combination was assumed to be DL+0.25LL, and, because the vertical element lost its load bearing capacity during past events and the lateral load is imposed in the other time, the dynamic inertial effect is neglected. In other words, it is supposed that column loss occurred because of a main-shock, and when the damaged structure reached equilibrium condition, the after-shock occurred. So, there is no need to consider the dynamic multiplier of gravity loads generated from the sudden removal of

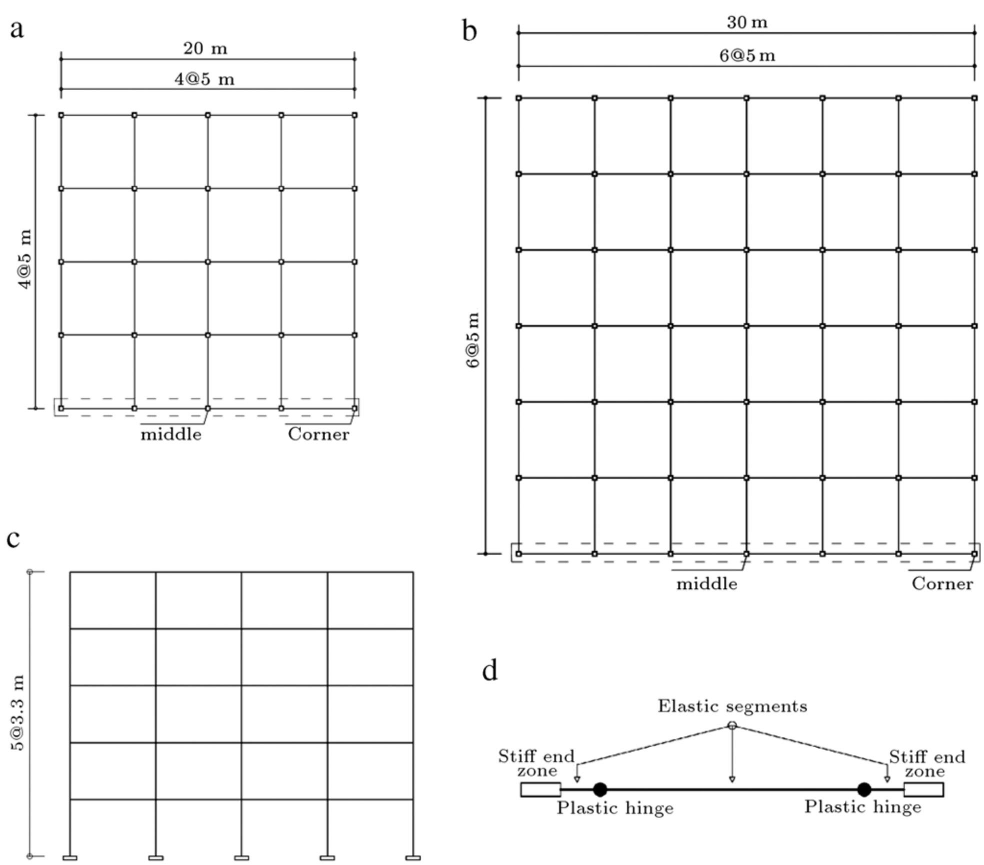

Figure 2: Model structures. (a) Structural plan of 4-bay structure; (b) structural plan of 6-bay structure; (c) elevation of 5-story model structure; (d) beam element in model structure.

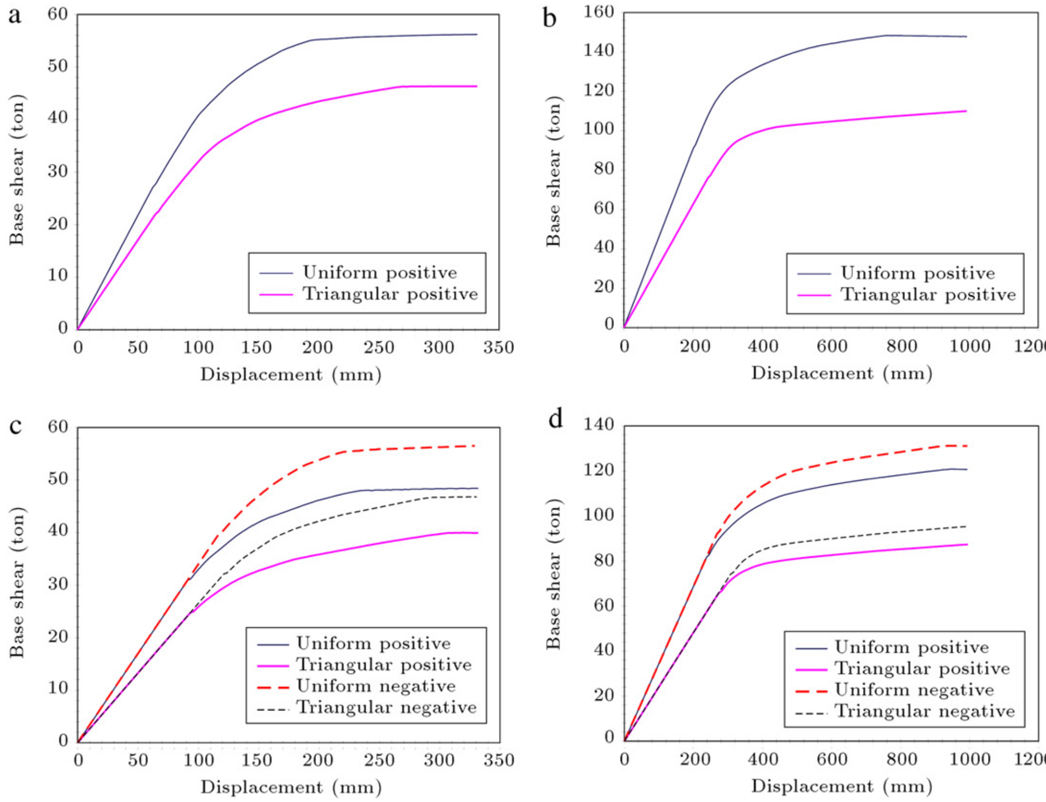

Figure 3: Comparison of capacity curves in different lateral load patterns in two-dimensional structures. (a) 5-story 4-bay middle column removed; (b) 15-story 4-bay middle column removed; (c) 5-story 4-bay corner column removed; (d) 15-story 4-bay corner column removed.

a column, while the dynamic effect of column elimination is merged.

3. Computational modeling

3.1. Model structures

5-story and 15-story MRF buildings with 4 and 6 bays were prepared to assess progressive collapse, as shown in Figure 2. The steel moment frames are designed to resist both gravity and lateral loads, in accordance with the Iranian code, and have passed all seismic criterion regarding strength and drift limits. The structure is considered to be in a high risk seismic zone and designed with special steel moment frames. The columns and girders were designed with $F_y = 240$ MPa (Tables 3 and 4 show the member size of the analysis model structures). For numerical analysis, 3-D and 2-D models were used, and, for the material model, an elastic-perfectly plastic model was used. In column removal, for the purpose of APM analysis, beam to beam continuity is assumed to be maintained across a removed column.

In the beams, the axial load effect is negligible, but, for the columns, the ultimate bending moment is substantially affected by the presence of the design axial load. In all analyses performed, resistance criteria relative to shear effort are neglected, despite the GSA and UFC documents prescribing some limitations. This implies that only the ability to take into account stress redistribution for bending efforts is investigated.

Many different models are available in literature for steel element simulation, ranging from micro models to macro models. Lumped plastic hinges coupled with a linear elastic section were used for the models, so that these moment hinges in the beams are placed at the member end with half of the beam depth distance from the adjacent column face, as shown in Figure 2(d). The advantage of macro models lies in their moderate computational cost, as well as their ease of implementation.

Table 3: Member size of 5-story analysis model structures.

Bay	Story	External beams	Internal beams	Columns (unit: cm)
4	1–4	W12 × 19	W14 × 26	Box35 × 35 × 1.0
	5	W10 × 17	W12 × 19	Box25 × 25 × 1.0
6	1–2	W12 × 22	W14 × 26	Box35 × 35 × 1.2
	3	W10 × 17	W14 × 26	Box35 × 35 × 1.2
	4	W10 × 17	W12 × 22	Box35 × 35 × 1.2
	5	W10 × 15	W12 × 22	Box25 × 25 × 1.0

Table 4: Member size of 15-story analysis model structures.

Bay	Story	External beams	Internal beams	Columns (unit: cm)
4	1–5	W18 × 35	W21 × 44	Box55 × 55 × 2.0
	6–10	W18 × 35	W21 × 44	Box45 × 45 × 1.5
	11–13	W12 × 19	W16 × 31	Box40 × 40 × 1.5
	14	W10 × 17	W14 × 22	Box35 × 35 × 1.0
	15	W10 × 17	W10 × 17	Box30 × 30 × 1.0
6	1–3	W18 × 35	W21 × 44	Box55 × 55 × 2.0
	4–8	W18 × 35	W21 × 44	Box45 × 45 × 1.5
	9–10	W16 × 31	W18 × 35	Box45 × 45 × 1.5
	11	W16 × 31	W18 × 35	Box40 × 40 × 1.5
	12	W14 × 22	W16 × 31	Box40 × 40 × 1.5
	13–14	W12 × 19	W14 × 22	Box35 × 35 × 1.5
	15	W8 × 10	W10 × 17	Box25 × 25 × 1.0

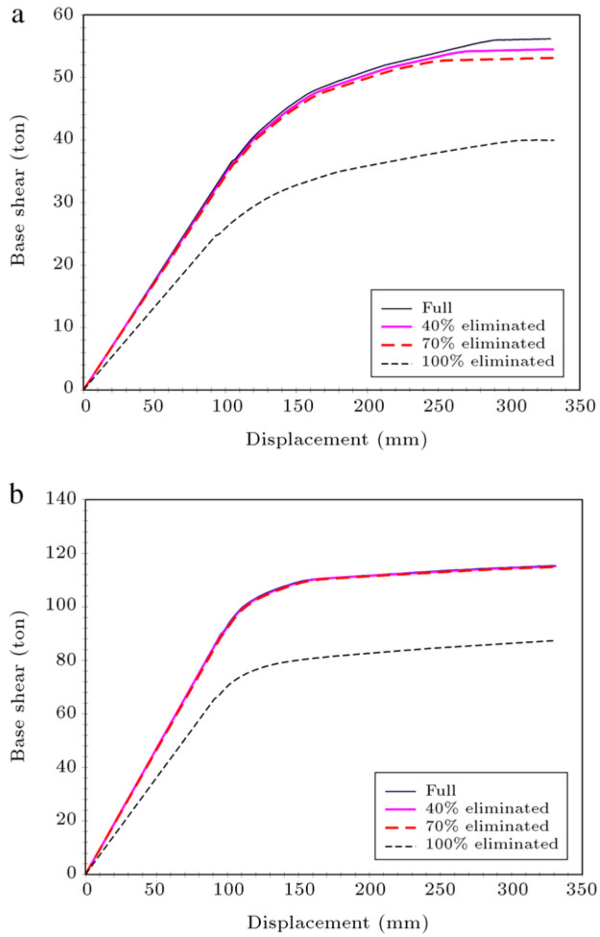

Figure 4: Comparison of capacity curves in different percents of column elimination in two-dimensional structures. (a) 5-story 4-bay corner column removed; (b) 15-story 4-bay corner column removed.

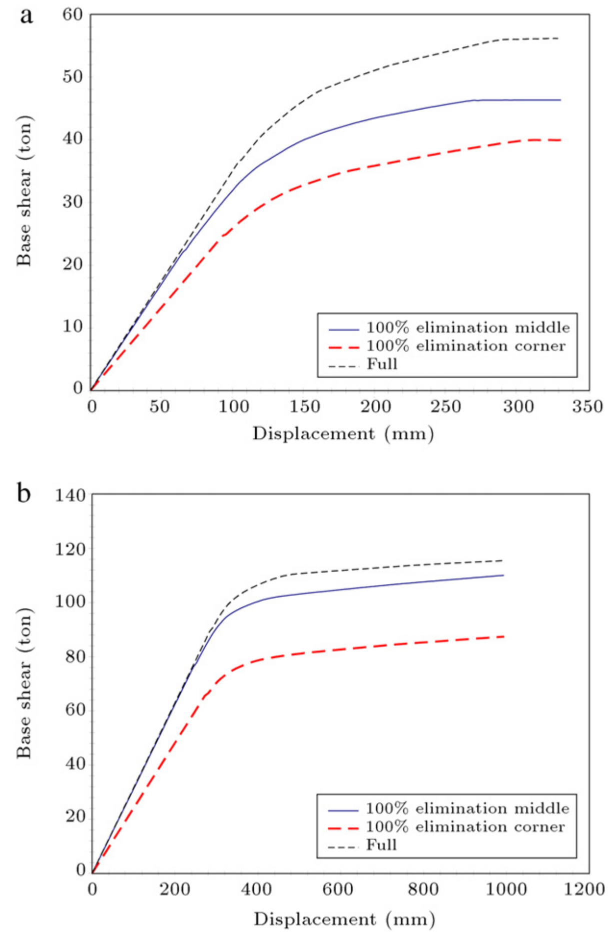

Figure 5: Comparison of capacity curves in different locations of column removal in two-dimensional structures. (a) 5-story 4-bay structure; (b) 15-story 4-bay structure.

4. Progressive collapse analysis of structures

4.1. Capacity curves in 2-D push-over analysis

As we know, it is necessary that selected lateral loading patterns produce the critical damage mode. It is obvious that the curves of various lateral distributions are significantly different. For decreasing computational time, 2-D push-over analysis was performed on the exterior frame of the building, and the difference between capacity curves in triangular and uniform patterns, in positive and negative directions of the load, was assessed. Figure 3 illustrates the analysis result.

Investigation of capacity curves in studied frames showed that a uniform load pattern produces a larger base shear capacity than a triangular pattern in all locations of column removal. It is obtained that in the case of middle column removal, because of symmetrical conditions, load patterns in positive and negative directions generate the same shear capacity curve, but, when the corner column is removed, positive and negative results are different. As a result, a triangular pattern in a positive direction shows the least base shear capacity and can be used as a critical case. In Figure 4, the effect of different percentages of column elimination is illustrated.

It is shown that column elimination in a complete manner has a stronger effect in base shear capacity reduction. The effect of elimination percent is more explicit in the 5-story structure. Investigation into the critical location of column removal, based on two-dimensional model structures with triangular patterns in a positive direction, is shown in Figure 5.

It is concluded that by removing the corner column, base shear capacity reduces significantly, but that middle column elimination has less effect on the capacity curve, as shown in Figure 5. For further accuracy, three-dimensional modeling was also investigated.

4.2. Capacity curves in 3-D push-over analysis

After complete elimination of columns in the exterior frame of the model, as shown in the dotted rectangle in Figure 2, 3-D push-over analysis in a triangular load pattern and in a positive direction was carried out, and the capacity curves in different locations and stories were compared.

Figure 6 shows the capacity curve of the model structures in different locations of column removal under complete elimination conditions. It can be seen that when the number of bays increases, the sensitivity of the structure against column removal decreases, and, with building elongation, the difference between the capacity curves is going to be negligible. So, the number of stories and the redundancy of the structures have a large influence on progressive collapse potential and the robustness of the structures, because the number of participating members and the load redistribution path greatly increases

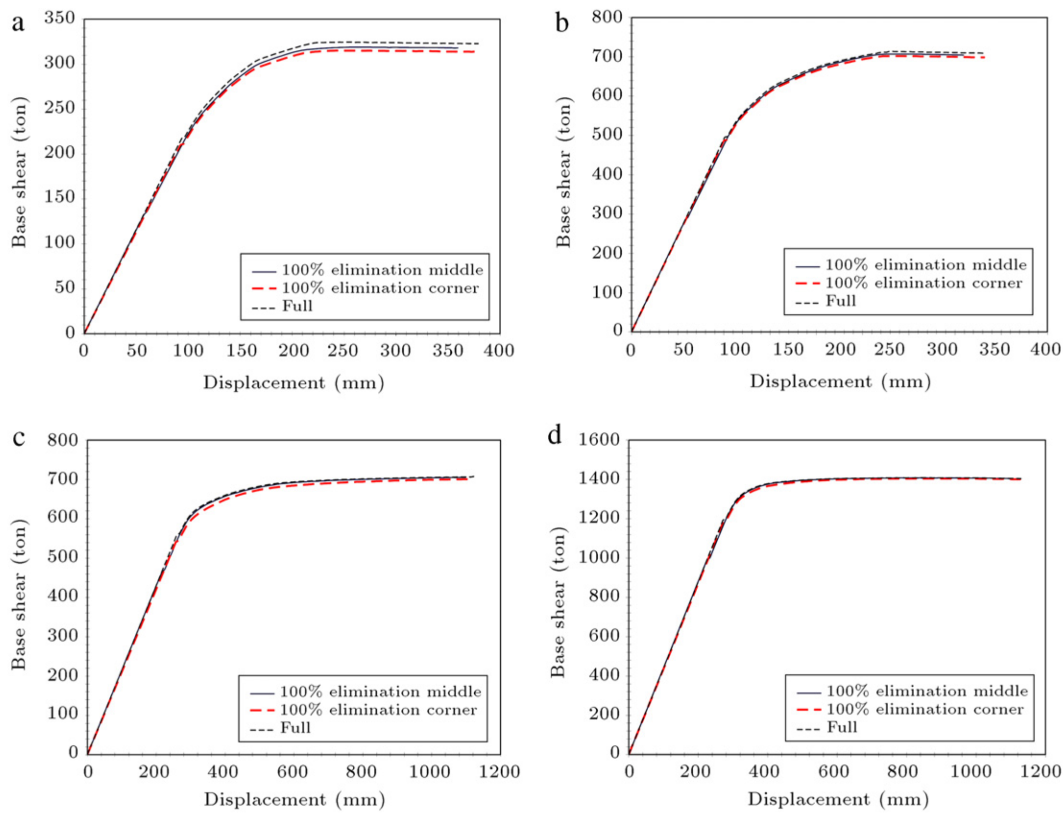

Figure 6: Comparison of capacity curves in different locations of column removal in three-dimensional structures. (a) 5-story 4-bay structure; (b) 5-story 6-bay structure; (c) 15-story 4-bay structure; (d) 15-story 6-bay structure.

Table 5: Structural robustness assessment for 2-D models under two lateral load patterns.

Story	Bay	Removed column	V intact (ton) uniform pattern	V damaged (ton) uniform pattern	V intact (ton) triangular pattern	V damaged (ton) triangular pattern	Robustness uniform pattern	Robustness triangular pattern
5	4	Middle Corner	64.3	54.9 45.5	50.3	43 35.5	0.85 0.71	0.85 0.71
15	4	Middle Corner	137.3	130.9 102.9	104.3	98.8 77.5	0.95 0.75	0.94 0.74

with increasing bay and story numbers. What is understood is that the difference between curves in 3-D analysis is less than in 2-D analysis.

4.3. Robustness indicator

Robustness is defined as insensitivity to local failure. In other words, robustness describes the structural ability to survive the event of local failure. A robust structure can withstand loading, so that it will not cause any disproportionate damage.

In order to better classify the results, let us define an indicator of robustness. If the design load is equal for the intact and damaged structure, R can be written to [16]:

$$R = \frac{V_{(damaged)}}{V_{(intact)}}, \tag{1}$$

where V is the base shear capacity and R is the residual reserve strength ratio. If the intact and damaged structures have the same capacity, the value is one. If the damaged structure has no capacity, it is zero. For a lateral load, the base shear capacities can be found by performing a static push-over analysis [17].

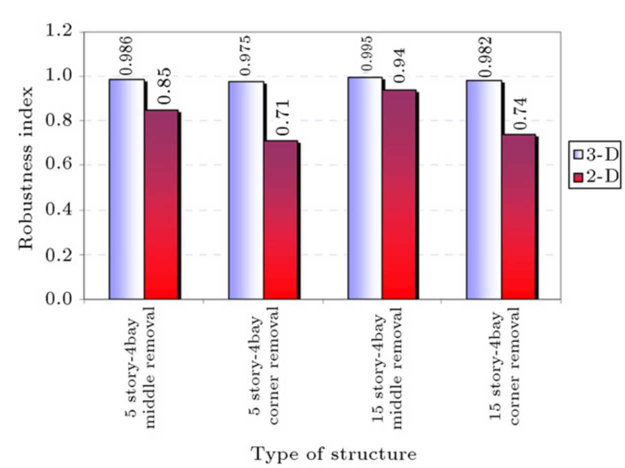

Figure 7: Comparison of robustness index in 2-D and 3-D models.

Robustness index values, with different lateral load distributions, in 5- and 15-story two-dimensional models, and with different locations of column elimination, were indicated in

Table 6: Structural robustness assessment for 3-D models.

Story	Bay	Removed column	V intact (ton) triangular pattern	V damaged (ton) triangular pattern	Robustness triangular pattern
5	4	Middle	319.5	315	0.986
		Corner		311.5	0.975
5	6	Middle	683.8	680.2	0.995
		Corner		674.8	0.987
15	4	Middle	650	646.5	0.995
		Corner		638	0.982
15	6	Middle	1374	1371	0.998
		Corner		1361	0.991

Table 3. Study on these quantities show that in the case of middle column removal, the strength reduction is less than in a corner column removal scenario. So, it can be concluded that the location of the damaged element has a great effect on structural robustness quantity. It is also noticed that lateral load patterns have no significant effect on the robustness index, and the result is almost the same. Table 4 shows the 3-D model results. Comparison of Tables 5 and 6 shows an egregious difference between two- and three-dimensional results in the robustness index. 2-D frames are more sensitive to base shear reduction and element removal, whereas 3-D models have less sensitivity to abnormal events. As well as 2-D models, in the case of middle column removal, the structure is more robust than in the corner column removal situation. It is also obtained that the higher number of bays and stories induces a higher level of robustness index.

Figure 7 illustrates a comparison of the two- and three-dimensional robustness index. It is noticed that two-dimensional models are more affected by the local failure of a vertical element than three-dimensional structures. The difference between results reduced while ascending floor numbers.

So, for many cases, it is better to ensure robustness by choosing a redundant structure, which is characterized by being statically indeterminate. Therefore, the structure is able to provide an alternative load path if the structure is damaged.

4.4. Assessment of the ductility ratio requirements

Assessment of ductility requirements is a very important aspect of design against progressive collapse. Whether a construction standard allows for the failure of structural elements or not, it is important to make sure that members and their connections maintain their strength through large deformations and local redistributions, associated with the loss of key structural elements [18–20].

The ductility ratio limit state that is mentioned in the GSA guideline is defined as the ratio of maximum vertical displacement and yield vertical displacement. It is a common practice

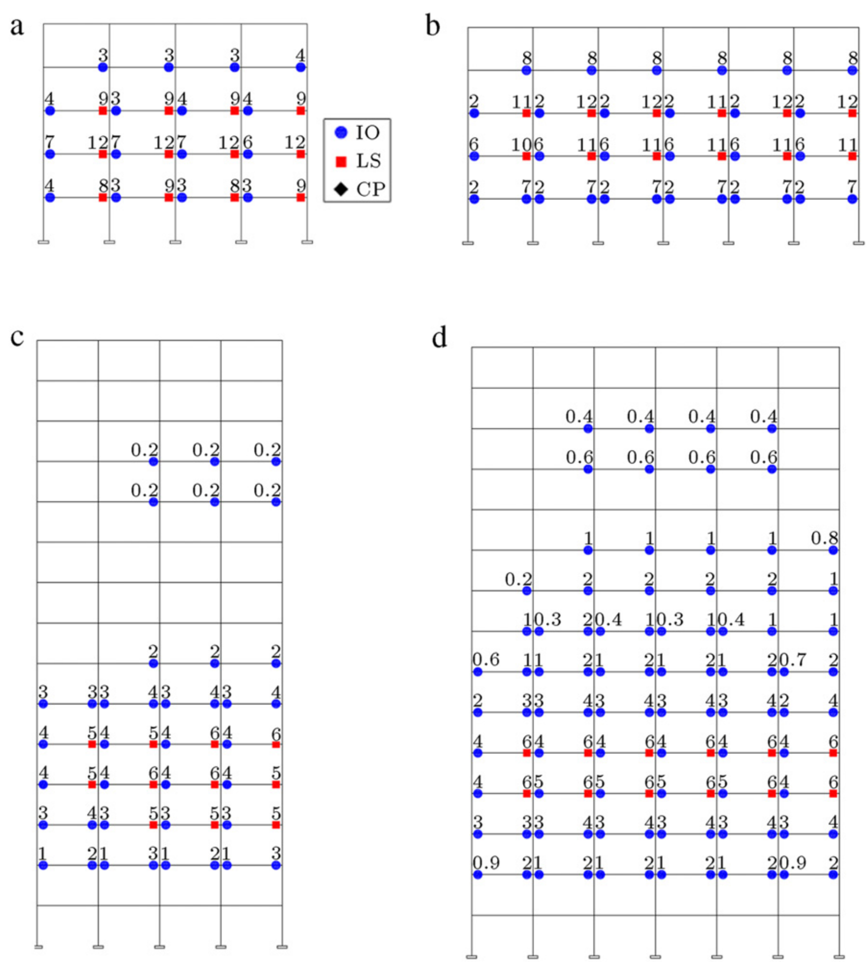

Figure 8: Formation of hinges in intact structure under triangular positive load pattern (rotation in thousandth of radian). (a) 5-story 4-bay structure; (b) 5-story 6-bay structure; (c) 15-story 4-bay structure; (d) 15-story 6-bay structure.

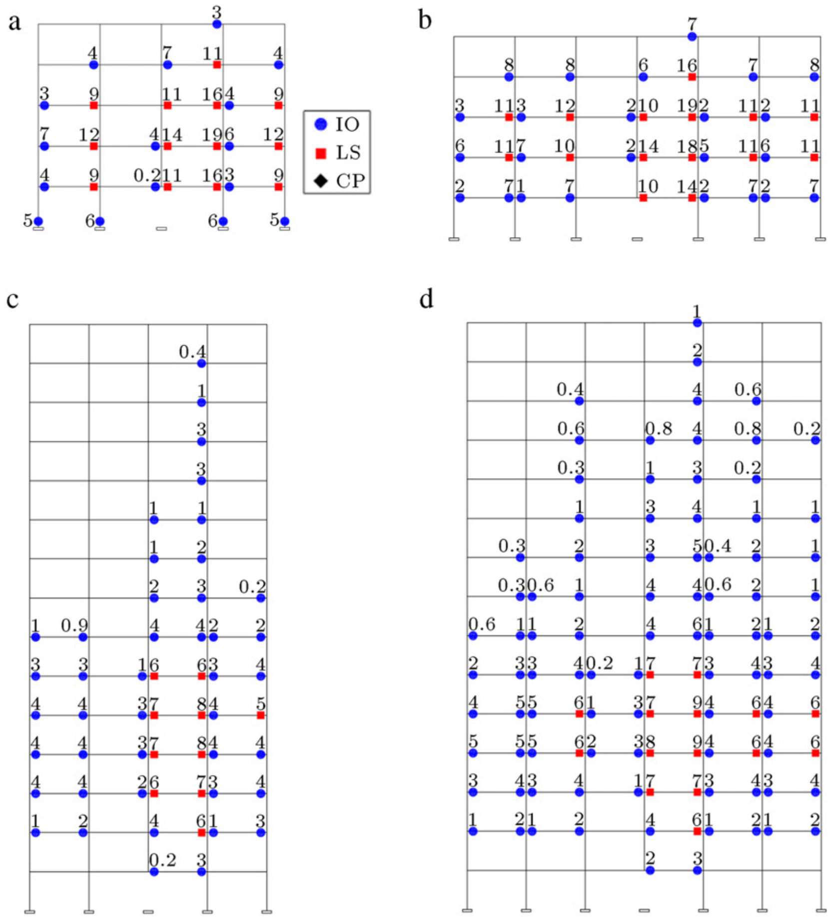

Figure 9: Formation of hinges when middle column is completely removed under triangular positive load pattern (rotation in thousandth of radian). (a) 5-story 4-bay structure; (b) 5-story 6-bay structure; (c) 15-story 4-bay structure. (d) 15-story 6-bay structure.

in progressive collapse analysis for the yield displacement to be obtained by nonlinear static push down analysis and for maximum displacement to be computed from nonlinear dynamic analysis. However, in this work, with a little change, the yield displacement is obtained by nonlinear push-over analysis when the hinge on the beams in the bay in which the vertical element is removed, becomes yield, and maximum displacement is computed from push-over analysis in target displacement [21,22]. Table 7 indicates this ratio in a variation of stories and bays using a triangular load pattern, when the middle column is removed.

It is observed that the ductility ratio in all cases is much less than the acceptance criterion of 20, so, in all situations, this criterion was satisfied. As a result, it is obtained that there is no potential of progressive collapse in these structures following the GSA2003 ductility requirements.

4.5. Plastic hinge rotation comparison

The hinge rotations requirement is the most important section of the guidelines to assess progressive collapse potential. As mentioned in Table 1, the UFC2009 guideline uses a collapse prevention performance level, and the GSA2003 proposes a limitation of 0.21 rad for the beams in the bays in which the vertical element is lost as the limit state of hinge rotation. If the mentioned elements exceed these limits and configure a failure mechanism on this bay, the structure has a high potential of progressive collapse. To evaluate the potential of failure, the performance of the hinges is investigated in target displacement. The target displacement is calculated by the displacement coefficient method, in accordance with FEMA273 [23]. Table 8 illustrates the target displacement obtained from a triangular load pattern capacity curve. Hinge rotations in the target displacement in the exterior frame of the models were compared with acceptance criteria for progressive collapse. Table 9 compared the hinge number in 2-D and 3-D models with uniform and triangular lateral load patterns, when the middle column was removed.

It is observed that there is no significant difference between the numbers of hinges in different lateral load patterns. So, as concluded later from capacity curves and the robustness index, from 3-D analysis, the triangular load pattern, as a critical case, is used. Assessment of hinge generation in the model structures is discussed as follows.

Figure 8 shows the location of plastic hinges and their performance level in intact structures. All columns in the target displacement remain elastic, and the beams not only did not exceed the collapse prevention performance level, but also did not meet this level in any hinge. It is also noticed that all structures meet the life safety performance level according to the Iranian seismic code.

When the middle column was removed, the rotation and number of hinges with the life safety performance level in the beams above the lost element, were increased, but no plastic hinge rotation exceeded the given acceptance criterion. Figure 9 plotted the formation of these hinges. It is noticed that

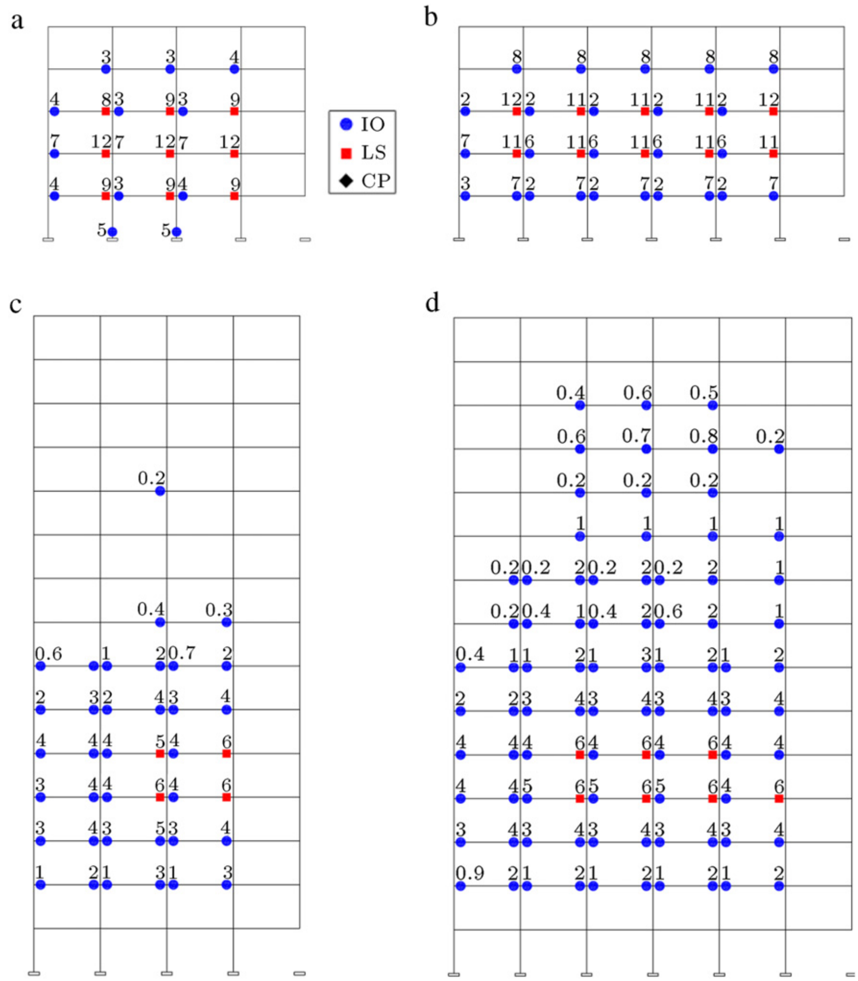

Figure 10: Formation of hinges when corner column is completely removed under triangular positive load pattern (rotation in thousandth of radian). (a) 5-story 4-bay structure; (b) 5-story 6-bay structure; (c) 15-story 4-bay structure; (d) 15-story 6-bay structure.

Table 7: Element ductility ratio assessment for 3-D models in middle column removal.

Story	Bay	Vertical displacement yield (mm) triangular pattern	Vertical displacement in target displacement (mm) triangular pattern	Ductility ratio triangular pattern	Satisfying GSA(2003) acceptance criteria
5	4	20.9	32	1.53	Yes
5	6	22.2	34.3	1.55	Yes
15	4	10.6	12.9	1.22	Yes
15	6	9.6	13.7	1.43	Yes

Table 8: Target displacement obtained from triangular load pattern capacity curve.

Model structure	Target displacement (mm)
5-story 4-bay	207
5-story 6-bay	215
15-story 4-bay	371
15-story 6-bay	389

because of carrying out the pushover analysis in a positive direction, hinge rotation in the right hand of the removed column increased and, in the left hand, decreased. Therefore, there is no potential for progressive collapse in the middle column loss situation, based on UFC2009 and GSA2003 guidelines.

It can be observed that in Figure 10, similar to the middle column removal case, no progressive collapse potential is probable, and the remaining structures survive under lateral loading in target displacement. So, we encounter similar situations and all hinges pass the acceptance criteria for progressive collapse.

It is noticed from hinge formation in local damaged structures, that safety margins for progressive collapse potential is sufficient enough, and there is no need to design structures with extra requirements.

5. Conclusions

3-D and 2-D models of a popular type of lateral loading resisting system, namely; spatial steel frame MRF, were developed in this work. The studied structures consisted of 5 and 15 floors with 4- and 6-bay buildings. All details and dimensions were obtained according to Iranian codes.

Table 9: Comparison of hinge number in 2-D and 3-D models.

Story	Bay	Removed column	IO performance level uniform pattern (2-D)	LS performance level uniform pattern (2-D)	IO performance level triangular pattern (2-D)	LS performance level triangular pattern (2-D)	IO performance level triangular pattern (3-D)	LS performance level triangular pattern (3-D)
5	4	Middle	12	11	13	11	11	13
15	4	Middle	46	8	52	5	47	10

It was observed in buildings designed according to seismic design specifications that when a column in the first story, for any reason, did not play its load bearing role properly, they were strengthened enough to resist progressive collapse, and no plastic rotations exceeded the given acceptance criterion. Since the rotation of plastic hinges is relatively small compared to acceptance criteria, despite having limited simulation, we can be sure that, in this situation, no progressive collapse potential is probable.

A comparison of lateral loading patterns showed that a triangular pattern induced the least capacity curves for intact and damaged structures, but the robustness index in uniform and triangular patterns is almost the same.

It was determined that, as the number of stories and bays increased, the capacity of the structure to resist progressive collapse under lateral loading also increased, because additional elements participated in resisting progressive collapse. It is also determined that increasing the number of bays, as well as stories, induces a higher level of robustness index.

This conclusion is reached without taking into consideration the beneficial effect of slab action. The panel zone in girder to column joints was assumed to be rigid, and connection properties were not considered. Although lumped plastic hinges can appear to provide a good solution to the modeling problem, they actually only shift the difficulty elsewhere, by raising the question relative to the numerical length of the hinge.

Nevertheless, it seems there is no concern about the occurrence of progressive collapse under seismic loading in a one column loss scenario for steel, special moment, resisting systems.

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