# Sensitivity of Progressive Collapse of MRF to Redundancy Factor

**Amin GHANNADIASL**
*Department of Civil Engineering, Faculty of Engineering, University of Mohaghegh Ardabili, Iran, Email: aghannadiasl@uma.ac.ir*

**Saeed MORTAZAVI**
*Department of Civil Engineering, Faculty of Engineering, University of Mohaghegh Ardabili, Iran, Email: m.saeedmortazavi@gmail.com*

**Hassan REZAEI DOLAGH**
*Department of Civil Engineering, Faculty of Engineering, University of Mohaghegh Ardabili, Iran, Email: hassan7205@yahoo.com*

***Abstract:*** Progressive collapse mainly refers to a chain failure in a structure. If one of the structural members gets a local collapse, it will lead to the other member's failure and eventually an additional collapse in the building. Over the past few decades, the assessment problems of the potential for progressive collapse, as well as improving the performance of structures against this type of failure, are widely discussed. In this study, the effects of redundancy factor on improving the performance of the structure with moment frame system against progressive failure are investigated. Also, the adequacy of the numerical value of the recommended redundancy factor of the design codes is worked out. In the selected models, the effects of redundancy are studied by increasing the number of spans and floors. Furthermore, the redundancy factor with both regular and irregular plans is modeled. The selected structures are studied with and without considering the redundancy factor, as well as by removing the columns which are more susceptible to progressive collapse. Finally, the results of the nonlinear static analysis of the models were obtained.

*Keywords:* Progressive collapse, Redundancy factor, Seismic performance, Moment frame system, Nonlinear static analysis.

## 1. INTRODUCTION

Progressive collapse refers to the expansion of a primary local failure within the structure. Due to the removal of one or more bearing members, the collapse develops such as chain in the structure and leads to the failure of the entire or part of the structure. Actually, with local failure in a structural member, its adjacent members also collapse and deteriorate. Kim used static linear and nonlinear dynamic methods for two-dimensional frame structures. They found that moment frames designed for lateral loads were less vulnerable to progressive collapse [1]. Wibowo and Lau showed that progressive collapse could also occur during an earthquake [2]. Therefore, progressive collapse is not only limited to explosive and gravity loads, it can even occur under the influence of earthquake force. Kapil and El-Tawil presented progressive collapse in converging and diverging braced frames. They concluded that structures with diverging braces have less vulnerability to progressive collapse than converging bracing structures [3]. Mashhadiali and Kheyroddin studied the progressive collapse of high-rise steel buildings with a hexagonal bracing system [4]. Kim and Hong investigated the performance of progressive collapse in irregular tall buildings using nonlinear dynamic and static analysis [5]. Astaneh-Asl et al. studied the strength of an ordinary steel structure and its floor system against a progressive collapse due to the removal of a column. [6]. Fragopol and Curley, with other researchers, introduced the index of indeterminacy and key element [7].

Ang and Tang defined structures that are not indeterminate. By eliminating an element from the structure, the destruction of the whole system can be occurred [8]. Also, the redundancy factor for structures was provided by Cornell [9]. Curley and Pundy concluded that structures could have different redundancy degrees over time [10]. De et al. showed that the addition of elements, even with less resistance, could increase the indeterminate of the structure [11]. Bertero estimated the effects of the earthquake by using nonlinear analysis [12]. Song and Wen examined the indeterminate effect on a special moment frame system, and the studied models were constructed on 3 and 9 floors with different spans [13]. Kuo-Wei and Yi-Kwei investigated the indeterminate effect on moment frame in steel structures under seismic loads and provided that the NEHRP 97 code considered the effect of the building’s area on redundancy factor [14]. Dashti investigated that applying a coefficient of 1.2 in a concentrically braced frame is necessary, and for regular structures like irregular structures coefficient of 1.2 has to be considered [15]. Mir Shahi, in his studies of the indeterminate degree and the increase in the coefficient of the behavior of the frames, concluded that increasing the indeterminacy in the structures used in the concentrically braced frame did not lead to an increase in the coefficient of behavior [16]. Mohammadi et al. investigated indeterminacy effects on the seismic behavior of reinforced concrete moment frames [17]. Cortez Benitez and Tona Colonga utilized the redundancy factor in the seismic design of structures [18]. Also, Godins-Domingos and Tona-Colonga performed a study on the redundancy factor on seismic performance of concrete structures with Sharon bracing [19]. Ghannadiasl and Moratazavi utilized redundancy factor 1.2, to achieve optimal performance in structures that are not indeterminately sufficient. Also, in regular structures that are not indeterminately sufficient, they should be designed using a redundancy factor of 1.2 like irregular structures. Therefore, the regularity or irregularity of the structures does not affect the application of this coefficient. From the results, it was found that the redundancy factor provided in Standard No. 2800, which is 1.2, for structures that are not indeterminately sufficient is appropriate, and the lower values lead to poor designing, and higher amounts lead to non-economic designing. A relationship between the floor area and the redundancy factor should also be provided [20].

For the first time, the redundancy factor has been provided in the NEHRP 97, UBC 1997, and IBC 2000 codes. The parameter ρ (redundancy factor) is calculated in the NEHRP 97 and UBC1997 codes by using the equation (1):

$$\rho = 2 - \frac{6.1}{r_{\max}\sqrt{A_B}} \tag{1}$$

where $A_B$ is the floor area and $r_{\max}$ stands for the shear ratio of the element to the floor shear that occurs at 2/3 height of the floor. ASCE 7, NEHRP divides the structural system into two indeterminacy and non-indeterminacy categories, which considers the redundancy factor of the range of 1 and 1.3. As a result, for structures that are not indeterminately sufficient, the earthquake lateral force is considered to be 1.3 times the normal load. In Standard No. 2800 (4$^{th}$ Edition), the range of this coefficient is 1 and 1.2; for buildings that are not indeterminately sufficient, the base shear is increased by 20% [21]. Using the nonlinear static method, the strength of dual steel moment frames with various eccentric bracings against progressive collapse was evaluated by Salmasi and Sheidaii [22]. Naji provided an efficient method called tie force by selecting a limit state of collapse according to section properties [23]. Adam et al. presented an intent review that defines all the main advances that have taken place since the beginning of the 21st century in the field of progressive collapse and robustness of buildings [24]. A practical method for the progressive collapse analysis of reinforced concrete framed buildings was derived by Al-Sallouma et al. [25]. Dramatically, an experimental investigation on the collapse behavior of a frame exposed to natural fires by Guobiao et al. [26]. Yang et al. deduced a probabilistic analysis of steel-concrete composite floor against progressive collapse based on the ductility of steel connections using a component-based connection model [27]. The influence of plan irregularities on the progressive collapse of steel structures with various seismic activity was examined by Ebrahimi et al. [28]. Nonlinear dynamic analyses for a structure that lost a column during a seismic activity based on Applied Element Method performed by Elshaer et al. [29]. A static study and a vibration analysis of a platform were performed by Calin et al. The finite element method is used to determine the deformability of the concrete platform for different loads [30]. A study of a six-degree-of-freedom system based on a Stewart platform mechanism was provided by Geng and Haynes [31].

Due to the positive effect of redundancy degree on the load redistribution of the collapsed member to other structural members, if the structure is not sufficient in terms of redundancy degree, the designing problem can be acceptably solved by considering the redundancy factor. In this paper, some structures with a moment steel frame system are studied in two groups with various spans.

## 2. NONLINEAR STATIC ANALYSIS

To evaluate the performance of the structures under earthquake loads, especially when the structure enters the plastic region, the linear analysis is not helpful to investigate and model the actual behavior of the structure. Because in the linear analysis, it is not possible to obtain accurate information on the response of the structure under various loads, when the structure enters a nonlinear state, the nonlinear analyses must be used instead. The nonlinearity of the structure can be geometric or a type of material. Among nonlinear analyses, time history analysis is a complicated and time-consuming method. Therefore, in this paper, the nonlinear static analysis method is used to investigate the seismic performance of the studied models. The considered analysis is simpler than nonlinear static analysis of time history but has got its weaknesses. Overall, the displacement-force diagram, the rotation of the plastic hinges in the elements, and the structure response to the earth's motion can be derived using a nonlinear static analysis method.

## 3. INVESTIGATED STRUCTURES

In this paper, the selection of structures in the three-dimensional is considered in terms of floor number. These structures consisting of 4, 5, 6, and 10 floors have been evaluated and analyzed on soil type III and relative seismic zone (the peak ground acceleration (PGA) of 0.30g). The height of the floors is chosen 3 meters in each structure, which is common in buildings in Iran. The number of spans is considered 3 and 5 for each group of floors. Due to the considered height for floors, the 4, 5, 6, and 10-story structures have got a height of 12, 15, 18, and 30 meters from the foundation level, respectively. The geometric plan of the buildings with 3 and 5 spans is shown in Figure 1. Also, 3D views of the studied buildings are shown in Figure 2. In this study, all the joints of the beam to the column and the supports are assumed as clamped. The box and the girder sections were used for the cross-sections of beams and columns, respectively. On the other hand, the type of steel is St37 with yield stress $F_y = 2400\ kg/cm^2$ and ultimate stress $F_u = 3700\ kg/cm^2$. The yield stress coefficient to the expected stress is considered 1.1. Consumed concrete has compressive strength $f_c = 250\ kg/cm^2$ and all reinforcement used in type AIII with yield stress $F_y = 4000\ kg/cm^2$ and ultimate stress $F_u = 6000\ kg/cm^2$. The elasticity modulus of steel is $E_{st} = 2E6\ kg/cm^2$. Also, Poisson's ratios of steel and concrete are $\upsilon_s = 0.3$ and $\upsilon_c = 0.2$, respectively.

## 4. METHODOLOGY

Generally, progressive collapse-resistant design methods are divided into two parts:

1) Indirect method:
   In this method, it is possible to design the structure against progressive collapse, taking into account the minimum requirements of indeterminate, continuity, and flexibility in the structure.
2) Direct method:
   In this method, the structure is directly analyzed, which is the most common technique is the Alternate Path method (AP). In this approach, the most critical states for continuity of the beams are created by removing the external (peripheral) or internal columns, in which the ability to re-distribution of the system is evaluated.

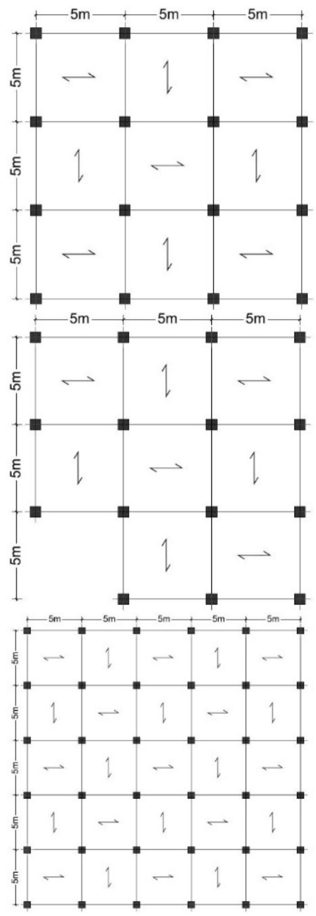



**Figure 1.** Plans of the studied structures

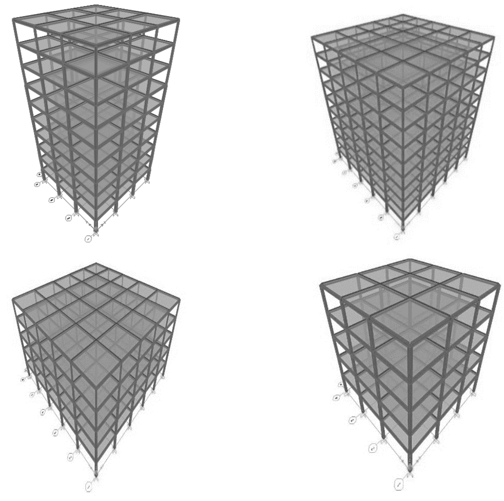
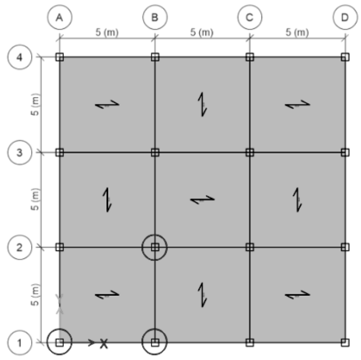
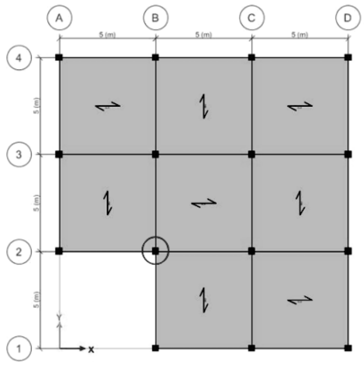


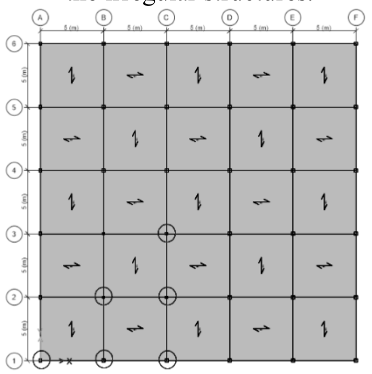
**Figure 2.** The studied structures




a - The location of the removed columns in the plan of the 3 span structures.




b - The location of the removed columns in the plan of the irregular structures.




c - The location of the removed columns in the 5 span structures.

**Figure 3.** The location of the removed columns in structures

The first important and fundamental instruction for progressive collapse analysis and design was published by the U.S. General Services Administration (GSA) in 2003, also known as GSA2003, and is used in many designs and articles. The U.S. Department of Defense (DoD) has also published several guides that get back to 2009, which the latest version was published in 2013, which eliminates all indirect methods, and only the indirect methods are declared. In this study, in 5-story frames, the columns on the first floor, the third floor, and the roof, and also in the 10-story frames, the columns on the first floor, the fifth floor, and the roof are removed. The location of the removed columns in the plan of 3 and 5 spans structure are shown in Figs. 3a to 3c.

## 5. ANALYSIS RESULTS

In this investigation, the effects of redundancy factor on improving the structural performance with moment frame system against progressive failure are studied under the scenarios set out in the previous section. In each of these scenarios, the structural response is examined via nonlinear static analysis. In Figures 4 to 7, the nonlinear static structural diagrams are depicted in terms of the displacement of the floor and the base shear for 5 and 10-story buildings. Figure 8 presents the comparison of the pushover graph of the studied structures. The most important point for assessing the performance of the studied structures against the scenarios mentioned above is the functional level of the plastic hinges of structural elements. Therefore, the consequences of the analyzed models are presented below.

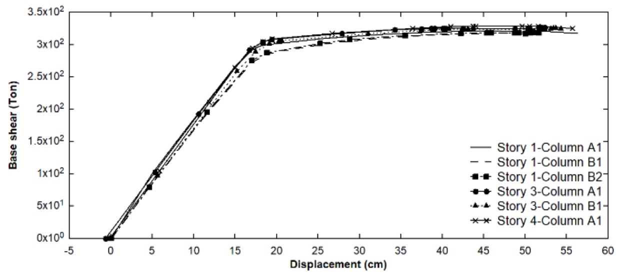



**Figure 4**. Pushover graph of 5-story structure (3 spans)

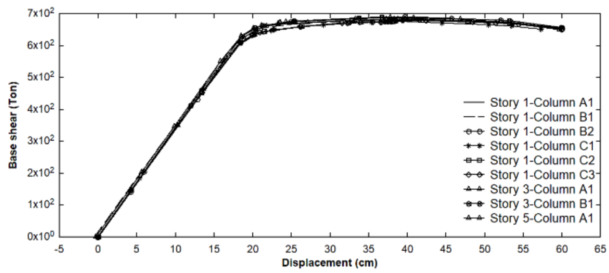



**Figure 5.** Pushover graph of 5-story structure (5 spans)

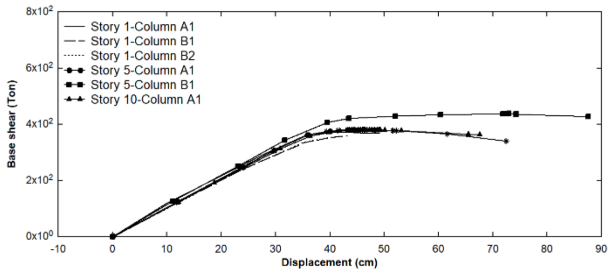



**Figure 6.** Pushover graph of 10-story structure (3 spans)

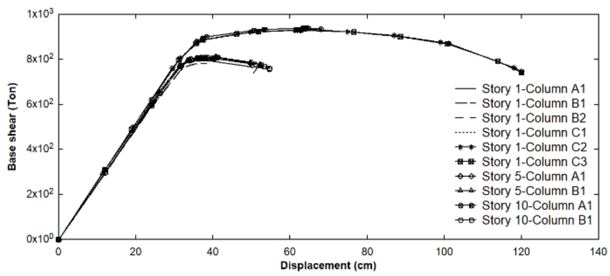



**Figure 7.** Pushover graph of 10-story structure (5 spans)

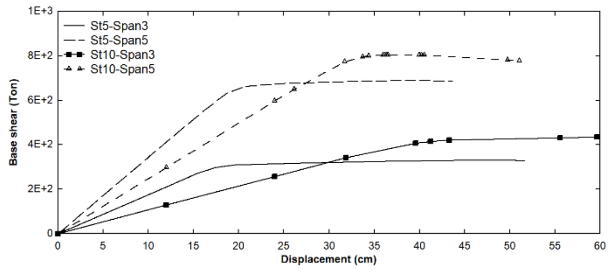



**Figure 8.** Comparison of the pushover graph of the studied structures

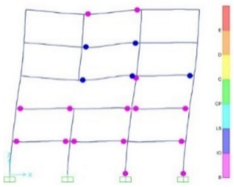
According to the UFC code, the allowable performance level for plastic hinges in the beams and columns is collapsing and lateral safety, respectively. Due to the different kinds of collapsing, it is not possible to provide all the scenarios which lead to plastic hinges in the structures and the behavior of the structure against progressive collapse. Furthermore, the over-strength and flexibility factors are calculated for each model with the redundancy factor of 1 and presented in Table 1. As shown in Table 1, the over-strength and flexibility factors increase by increasing the number of floors and spans. Also, results of plastic hinge formation in the frame with the redundancy factor of 1 that exposed to column removal are shown in Figure 9. In Table 2, the numbers of plastic hinges that pass through certain static nonlinear analytical levels are presented for structures with the redundancy factor of 1. The performance level and the number of plastic hinges for 4 and 6-story structures with the regular plan that is exposed to column removal in story 1 are presented in Tables 3 and 4. Also, these results for 4 and 6-story structures with the irregular plan are shown in Tables 5 and 6. In this study, the lateral load pattern is applied as uniform (UEX) and triangular (TEX) lateral load patterns [20]. Also, it can be seen from Tables 3 to 6 that the improved performance of models due to the application of the redundancy factor of 1.2 is expected and evident. However, the performance improvement is different and has a specific pattern for the number of floors and spans because of the structural resistant system of the structures. The earthquake-resistant system is the moment-resisting frame. Therefore, the redundancy factor can change the dimensions of the beams and columns, and the difference between cross-sections may vary with the different redundancy factors. Also, in all models, applying a redundancy factor of 1.2 leads to decreasing displacement, i.e., structures show non-elastic behavior in much displacement, while increasing the basic shear in models with a redundancy factor of 1.2 is obvious. One of the most important consequences that can be seen is that, some of the structures that passed the life safety performance level or even collapsed. It can achieve the accepted performance level by applying the redundancy factor of 1.2. Therefore, the redundancy factor slightly increases the seismicity weight of the structures, but in contrast, the seismic performance of the structure is improved under a different lateral load pattern.

**Table 1.** Over strength and flexibility coefficients of the studied structures.

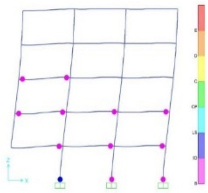
| Structure | Spans | Over strength coefficient | Flexibility coefficient |
|---|---|---|---|
| 5-Story | 3 | 1.12 | 3 |
| | 5 | 1.14 | 3.15 |
| 10-Story | 3 | 1.16 | 3.42 |
| | 5 | 1.25 | 3.63 |

**Table 2.** Plastic hinges in the studied structures.

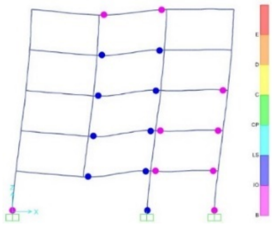
| Model | Span | Removed column-Story | IO-LS | LS-CP | >CP |
|---|---|---|---|---|---|
| 5-Story | 3 | A1-1 | 89 | 2 | 20 |
| | | B1-1 | 93 | 1 | 16 |
| | | B2-1 | 93 | 2 | 19 |
| | | A1-3 | 90 | 4 | 20 |
| | | B1-3 | 95 | 3 | 23 |
| | | A1-5 | 93 | 0 | 22 |
| | 5 | A1-1 | 210 | 19 | 96 |
| | | B1-1 | 232 | 27 | 94 |
| | | B2-1 | 241 | 22 | 94 |
| | | C1-1 | 219 | 38 | 78 |
| | | C2-1 | 226 | 23 | 95 |
| | | C3-1 | 228 | 32 | 94 |
| | | A1-3 | 198 | 39 | 117 |
| | | B1-3 | 217 | 0 | 17 |
| | | A1-5 | 208 | 28 | 104 |
| 10-Story | 3 | A1-1 | 91 | 0 | 12 |
| | | B1-1 | 58 | 0 | 10 |
| | | B2-1 | 56 | 0 | 0 |
| | | A1-5 | 131 | 5 | 16 |
| | | B1-5 | 124 | 4 | 22 |
| | | A1-10 | 111 | 2 | 35 |
| | 5 | A1-1 | 189 | 3 | 36 |
| | | B1-1 | 65 | 0 | 21 |
| | | B2-1 | 39 | 0 | 18 |
| | | C1-5 | 244 | 9 | 35 |
| | | C2-1 | 260 | 10 | 182 |
| | | C3-1 | 260 | 10 | 181 |
| | | A1-5 | 195 | 4 | 38 |
| | | B1-5 | 241 | 11 | 36 |
| | | A1-10 | 204 | 5 | 38 |



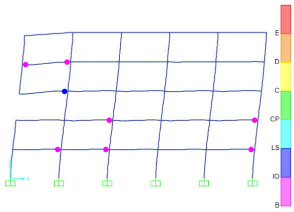
Plastic hinges formed by removing column B1 on the third floor of 5-story structure with 3 spans.



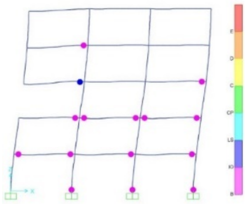
Plastic hinges formed by removing column A1 on the first floor of 5-story structure with 3 spans.



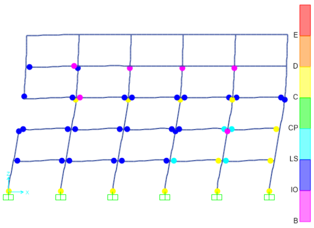
Plastic hinges formed by removing column B1 on the first floor of 5-story structure with 3 spans.



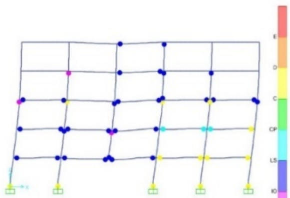
Plastic hinges formed by removing column A1 on the third floor of 5-story structure with 5 spans.



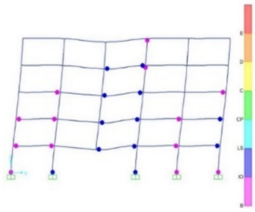
Plastic hinges formed by removing column A1 on the third floor of 5-story structure with 3 spans.



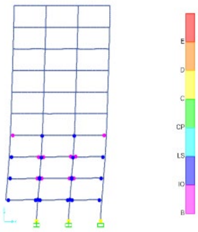
Plastic hinges formed by removing column A1 on the third floor of 5-story structure with 5 spans.



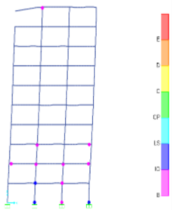
Plastic hinges formed by removing column C3 on the first floor of 5-story structure with 5 spans.



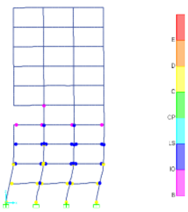
Plastic hinges formed by removing column C1 on the first floor of 5-story structure with 5 spans.



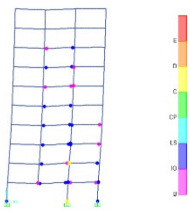
Plastic hinges formed by removing column A1 on the first floor of 10-story structure with 3 spans.



Plastic hinges formed by removing column A1 on the tenth floor of 10-story structure with 3 spans.



Plastic hinges formed by removing column A1 on the fifth floor of 10-story structure with 3 spans.



Plastic hinges formed by removing column B1 on the first floor of 10-story structure with 3 spans.

**Figure 9.** Results of pushover analysis, plastic hinge formation in the frame that exposed to column removal.

**Table 3.** Performance level and number of plastic hinges in 4-story structures (3 spans) with regular plan.

| Removed column | Lateral load pattern | Redundancy factor | IO-LS | LS-CP | >CP |
|---|---|---|---|---|---|
| A1 | TEX | 1 | 26 | 0 | 3 |
| | | 1.2 | 22 | 0 | 2 |
| | UEX | 1 | 5 | 0 | 1 |
| | | 1.2 | 0 | 0 | 2 |
| B1 | TEX | 1 | 32 | 0 | 2 |
| | | 1.2 | 28 | 0 | 1 |
| | UEX | 1 | 19 | 0 | 1 |
| | | 1.2 | 9 | 0 | 1 |
| B2 | TEX | 1 | 12 | 3 | 2 |
| | | 1.2 | 10 | 3 | 3 |
| | UEX | 1 | 12 | 3 | 0 |
| | | 1.2 | 7 | 2 | 1 |

**Table 4.** Performance level and number of plastic hinges in 6-story structures (3 spans) with regular plan.

| Removed column | Lateral load pattern | Redundancy factor | IO-LS | LS-CP | >CP |
|---|---|---|---|---|---|
| A1 | TEX | 1 | 63 | 1 | 3 |
| | | 1.2 | 36 | 14 | 2 |
| | UEX | 1 | 12 | 0 | 2 |
| | | 1.2 | 26 | 0 | 0 |
| B1 | TEX | 1 | 53 | 7 | 4 |
| | | 1.2 | 34 | 8 | 5 |
| | UEX | 1 | 13 | 0 | 2 |
| | | 1.2 | 31 | 0 | 2 |
| B2 | TEX | 1 | 42 | 3 | 3 |
| | | 1.2 | 46 | 2 | 1 |
| | UEX | 1 | 16 | 5 | 3 |
| | | 1.2 | 12 | 0 | 0 |

According to the tables, the structures are acceptable in the scenarios for removing the columns in the first and middle floors in the middle and lateral spans, and the performance level of the plastic hinges is in the life safety zone. For the middle floor column in the corner of the 5 and 3 spans, the life safety performance and collapsing are shown, respectively. It is seen that by increasing the redundancy factor in the structure resulting from the increase in the number of spans, the structure's performance against the scenarios of the corner column improves. However, this conclusion does not include scenarios related to the collapse of the roof column.

**Table 5.** Performance level and number of plastic hinges in 4-story structures with irregular plan

| Model | Lateral load pattern | Redundancy factor | IO-LS | *LS-CP* | >CP |
|---|---|---|---|---|---|
| 3 | TEX | 1 | 24 | 0 | 4 |
| | | 1.2 | 17 | 0 | 6 |
| | | 1.3 | 31 | 0 | 0 |
| | UEX | 1 | 3 | 0 | 5 |
| | | 1.2 | 0 | 9 | 3 |
| | | 1.3 | 16 | 0 | 3 |
| 5 | TEX | 1 | 26 | 2 | 5 |
| | | 1.2 | 25 | 1 | 4 |
| | | 1.3 | 20 | 0 | 8 |
| | UEX | 1 | 16 | 3 | 4 |
| | | 1.2 | 9 | 1 | 4 |
| | | 1.3 | 14 | 0 | 11 |

**Table 6.** Performance level and number of plastic hinges in 6-story structures with irregular plan

| Span | Lateral load pattern | Redundancy factor | IO-LS | LS-CP | >CP |
|---|---|---|---|---|---|
| 3 | TEX | 1 | 50 | 8 | 5 |
| | | 1.2 | 36 | 11 | 6 |
| | | 1.3 | 30 | 11 | 6 |
| | UEX | 1 | 13 | 0 | 7 |
| | | 1.2 | 8 | 0 | 2 |
| | | 1.3 | 10 | 0 | 5 |
| 5 | TEX | 1 | 49 | 6 | 5 |
| | | 1.2 | 32 | 12 | 2 |
| | | 1.3 | 24 | 12 | 3 |
| | UEX | 1 | 19 | 0 | 5 |
| | | 1.2 | 21 | 0 | 1 |
| | | 1.3 | 20 | 0 | 4 |

## 6. CONCLUSION

In this paper, the 4, 5, 6, and 10-story buildings with the height of 3 meters/story in 3 and 5 spans are investigated. Then, by applying the redundancy factor 1 and 1.2, the performance of the structures is evaluated. According to the results of the nonlinear static analysis, it is seen that in buildings with a regular plan, by applying a coefficient of 1.2, the structure's performance after the failure of the member, leads to safety regulations. On the other hand, for buildings with irregular plans, the coefficient of 1.3 improves the performance of the structure. As seen in the tables, with an increase of 20% to 30% of the base shear, the number of plastic hinges created in the structure has decreased considerably. Increasing the indeterminacy in the structures by increasing the number of floors and spans leads to improving the performance of structures under the removal of columns in the first and middle floors. However, increasing the redundancy degree can not be a reliable way to stabilize the structure against progressive collapse. Scenarios related to the removal of columns on the roof floor are independent of the redundancy degree of the structure. Therefore, increasing the number of floors and spans does not affect the failure of the roof floor. Thus, the roof floor is at the highest level of the structure, and the redundancy degree of the elements in this category is independent of the increasing number of the floors. Scenarios related to the corner columns are worse than the lateral columns, and the lateral columns are worse than the middle ones. In the case of scenarios related to middle floor columns in the corner of the 5 and 3 spans, the life safety performance and collapsing occur, respectively. It means, by increasing the redundancy factor in the structure resulting from the increase in the number of spans, the structure's performance against the scenarios of the corner column improves. It can be seen that if the structure is not sufficient in terms of redundancy degree, the designing problem can be considered solved by the redundancy factor.

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