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# Progressive collapse scenario in steel structures with irregularity in height

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Research Paper

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Sudden removal of load-bearing elements such as columns in engineering structures, and lack of sufficient capacity to withstand the overload caused by removal of these elements can cause damage and Progressive Collapse (PC) in structures. Therefore, the effect of sudden column removal and structural capacity against PC scenarios in medium and high-rise buildings is investigated in this study. The irregularity in height has a great influence on lateral behaviour of structures and it affects the design of cross-sections. Various sudden column removal scenarios are investigated in this research for steel structures with and without irregularity in height. To assess the effects of sudden column removal, the Alternate load Path Method (APM) and Nonlinear Dynamic Analysis (NDA) are utilized. In addition, a Nonlinear Static Analysis (NSA) is performed to investigate the capacity of structures against the PC phenomenon. Using OpenSees software, 10-, 15- and 20-storey structures with three distinct irregularity types are analysed during four different column removal scenarios. The results are presented in the form of static and dynamic nonlinear curves. The results indicate that making geometric irregularity in height in the sudden column removal scenario can cause the reduction of capacity and growth of the structural response in comparison to the structure with regularity in height. Moreover, the capacity of structures increases and the dynamic response declines by increasing the number of elements in the structures.

**Key words:**
**progressive collapse, steel structures, irregularity in height, nonlinear static analysis, nonlinear dynamic analysis**

Prethodno priopćenje

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## Scenarij postupnog rušenja čeličnih konstrukcija s nepravilnostima po visini

Naglo uklanjanje nosivih elemenata kao što su stupovi s inženjerskih konstrukcija, te nedovoljna sposobnost preuzimanja dodatnih opterećenja zabilježenih nakon uklanjanja takvih elemenata, može nanijeti oštećenje i uzrokovati postupno rušenje takvih konstrukcija. U ovom radu istražen je učinak naglog uklanjanja stupova te konstrukcijski kapacitet srednje visokih i visokih građevina u različitim situacijama postupnog rušenja. Nepravilnosti po visini uvelike utječu na bočno ponašanje konstrukcija i na projektiranje poprečnih presjeka. Istraživani su različiti scenariji naglog uklanjanja stupova kod čeličnih konstrukcija s nepravilnostima po visini ili bez njih. Kako bi se procijenio utjecaj naglog uklanjanja stupa, primjenjene su metoda alternativnih putova prijenosa opterećenja (APM) i nelinearna dinamička analiza (NDA). Osim toga, provedena je i nelinearna statička analiza (NSA) kako bi se istražio kapacitet konstrukcija s obzirom na pojavu postupnog rušenja. Programom OpenSees analizirane su konstrukcije od 10, 15 i 20 katova s tri različite vrste nepravilnosti, a prema četiri različita scenarija uklanjanja stupova. Rezultati su prikazani u obliku statičkih i dinamičkih nelinearnih krivulja. Dobiveni rezultati pokazuju da geometrijske nepravilnosti po visini u slučaju scenarija naglog uklanjanja stupa mogu uzrokovati smanjenje kapaciteta i povećanje konstrukcijskog odziva u usporedbi s konstrukcijama bez nepravilnosti po visini. Uz to, kapacitet konstrukcija se povećava, a dinamički se odziv smanjuje s povećanjem broja elemenata u konstrukcijama.

**Ključne riječi:**
**postupno rušenje, čelične konstrukcije, nepravilnost po visini, nelinearna statička analiza, nelinearna dinamička analiza**

## 1. Introduction

Progressive Collapse (PC) is the chain spread of an initial local failure from element to element resulting in the collapse of the whole structure or a disproportionately large part of it [1-3]. Structural safety has always been the main preoccupation for the engineers and persons responsible for the design of civil engineering projects [4-7]. One of the mechanisms of structural failure which has drawn more attention over the past few decades, is referred to as PC (Progressive Collapse). In case of sudden failure of one or several structural members, any redistribution of load would cause the failure of other structural elements and finally the building would be progressively destroyed. In the USA, General Services Administration (GSA) [8] presented practical criteria to decrease the potential of PC in federal buildings. Unified Facilities Criteria, Department Of Defense of U.S (UFC–DoD) [9] provided criteria for the design of the existing buildings. Kim and Kim [10] investigated structural capacity against PC in steel moment frames. In this research, linear and nonlinear models of static and dynamic analyses are used based on GSA and UFC–DoD guidelines. The results showed that NDA (Nonlinear Dynamic Analysis) is an accurate method for evaluating PC potential of structures. Kim and Dawoon [11] examined the effect of chain performance on the potential of PC in the steel moment frame structures. NDA and NSA (Nonlinear Static Analysis) were performed in 3- and 6-storey models with and without braces, using the APM (Alternate load Path Method), proposed by the GSA guideline. The results show that the push down curve, obtained by considering the chain performance in NSA, was higher than the case of ignoring the chain performance. Usefi et al. [2] and Kim and Hong [12] assessed the performance of PC in irregular high-rise structures by means of NDA and NSA and found that the structures with more elements have higher resistance to PC. Mashhadiali and Kheyrodin [13] studied the collapse behaviour of 48-storey building models under sudden loss of corner load-bearing elements from the first storey, using NSA and NDA. Jalalilarijani [14] assessed the effects of column removal in structures with various bracing systems using the Linear Static Analysis (LSA). Mahmoud et al. [15] surveyed the potential of PC due to seismic loads in MRF and Braced Frames designed by Egyptian local standards. They used UFC–DoD guidelines (APM method) to simulate the PC potential. Rahnavard et al. [16] examined PC in tall steel composite buildings by nonlinear analysis. Based on the result of their study, side case and centrical case removal of braces in MRF structures proved to be more critical and destructive compared to corner case removal. Kordbagh and Mohammadi [17] studied the influence of seismicity level and height of building on the progressive collapse resistance of steel frames. Kiakojuri et al. [18] studied progressive collapse of steel moment-resisting frames using static and dynamic incremental analyses.

Although there have been numerous investigations regarding progressive collapse in various structures, some topics have remained overlooked, one of which is the effect of irregularity in height in assessing the progressive collapse potential in mid-rise and high-rise buildings. Accordingly, this paper presents a comprehensive investigation on the effects of sudden column removal based on nonlinear static and dynamic push down analyses. The investigated cases, analysis methods, and results, are provided in the following sections

## 2. Modelling and analysis

### 2.1. Modelling and designing

Since the aim of this research is to survey the effects of sudden column removal considering the irregularity effects in mid-rise and high-rise structures, three-dimensional 10-, 15- and 20-storey structures were first modelled and designed linearly. SAP2000 [19] software was used to design the mentioned structures. After linear modelling, structures were subjected to gravity load based on Iranian loading standards. The spectral dynamic analysis method was used for lateral loading and structural analysis in accordance with Buildings Standard - 2800 [20]. For lateral loading, it was assumed that the type of soil beneath the structures is "type 3 soil" and that the level of earthquake hazard is "high". Individual floors are considered to be 3 meters high. The structures in the X direction include six 4-meter spans, and structures in the Y direction include five 5 -meter spans, as shown in Figure 1.

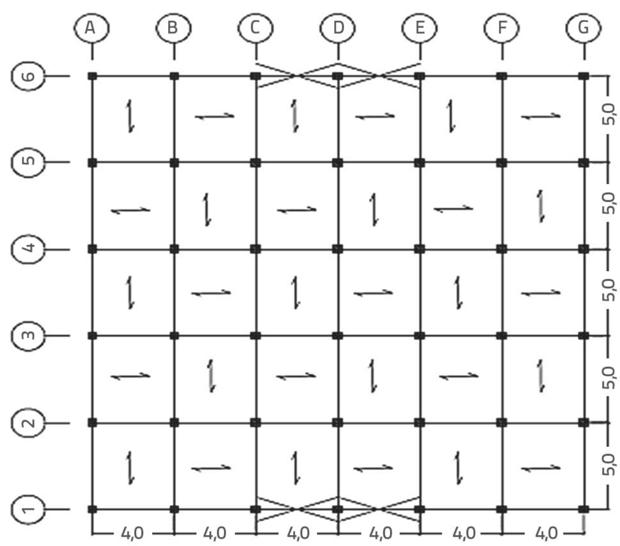


**Figure 1. Elementary plan of structural floors**

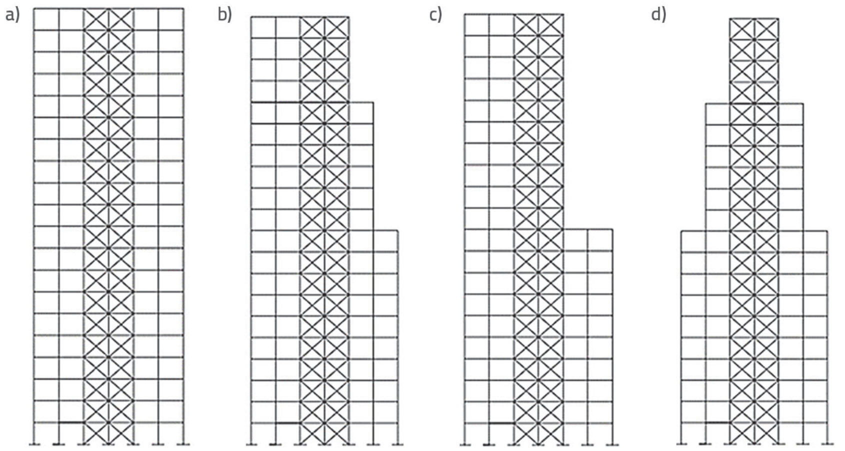


**Figure 2. View of regular and irregular models in height for a 20-storey model: a) Regular model; b) Irregular model for case 1; c) Irregular model for case 2; d) Irregular model for case 3**

10, 15 and 20 storey structures were designed after the modelling and analysis. Structures designed in the X direction have a dual concentrically braced system with a special ductility, while in the Y direction they have a moment resisting frame with a special ductility. In this paper, column sections are considered as square boxes, beams are symmetric I sections, and braces are standard UNP sections. The selected sections are shown in Table 2. In this table, hw-tw represents dimensions of the web and bf-tf shows dimensions of the flange in the I-shape sections of the beam. The dimensions of the sections are in millimetres (mm).

In order to consider the effects of geometric irregularities in height, it is assumed that the structures are recessed in height in three cases, and three irregular states are considered for the structure.

The intended models for irregular 20-storey structures are shown in Figure 2. In the selection of irregularity effects, an effort was made to consider irregular geometric effects in the structure simultaneously with decreasing mass in the upper floors. In all models of 10, 15, and 20-storey buildings, case 1 involves gradual removal of one span in the first step and, in the subsequent step, gradual removal of two spans on one side of the upper floors of structures. Case 2 involves sudden removal of two spans of the upper floors on one side, and case 3 shows gradual removal of one and two spans on both sides along the height of the structure.

Span reduction for the 15-storey building is illustrated in Figure 5. Although the main aim of this figure is to represent the finite element model in the OpenSees software, span reduction for making irregularity is obvious in this figure, and is applied to the model in 9th and 13th floors. For the 10-storey building, span reduction has occurred in the 6th and 8th floors.

The Load Resistance Factor Design (LRFD) method is used to design structures. ST37 steel is assumed to be used to design the structures (AISC360) [21]. Linear parameters and loading values required for modelling and designing the structures analysed in this research are shown in Table 1. In the spectral dynamic analysis, the design spectrum is selected based on the standard design spectrum specified in 2800 codes. In the nonlinear design and modelling of the structures, the amount of dead and live load is considered to be the same for all structural models. Dead and live load of the building roof are considered to be similar to those of other floors. Snow load with the value of 1.5 KN/m$^2$ is also considered in the design and modelling of the structure for the building roof.

**Table 1. Modelling and design parameters**

| Parameter | Value | Unit |
|---|---|---|
| Dead load | 6 | kN/m$^2$ |
| Live load | 2 | kN/m$^2$ |
| Yield stress | $2.4 \cdot 10^5$ | kN/m$^2$ |
| Expected yield stress | $2.88 \cdot 10^5$ | kN/m$^2$ |
| Modulus of elasticity, E | $2 \cdot 10^8$ | kN/m$^2$ |
| Poisson ratio | 0.3 | - |
| Mass per unit volume | 8 | kN/m$^3$ |

## 2.2. Sudden column removal scenario

PC modelling does not require complex cyclic behaviour [22]. There are two issues in discussing the sudden removal of a column and the PC scenario. The first matter is related to the location of column loss and the second one is the loading condition and how to suddenly remove the columns. There are various methods for structural analysis of the sudden removal of columns [23]. Nonlinear Static Pushdown Analysis (NSPA) is usually used in order to study the behaviour of structures in the face of the progressive collapse phenomenon. In this approach, the desired column will firstly be removed from the structure based on the APM, and then gravity load will be applied to the structure on the basis of the GSA standard loading [24]. The analysis continues in the displacement controlled conditions, and the vertical displacement of the damaged point will be considered as the push down curve based on the gravity load factor (the ratio of applied load to actual load of the structure), as shown in Figure 3. According to the nonlinear static push down analysis, the load factor in the static push down method would be obtained by dividing the required force of a structure to

**Table 2. Characteristics of structural sections**

| 20 storey | | | | 20 storey, Case 1 | | | |
|---|---|---|---|---|---|---|---|
| Storey | Column | Brace | Beam ($h_w$-$t_w$ / $b_f$-$t_f$) | Storey | Column | Brace | Beam ($h_w$-$t_w$ / $b_f$-$t_f$) |
| 1-4 | box 500-20 | 2UNP200 | 400-12 / 250-20 | 1-2 | box 500-20 | 2UNP200 | 400 / 12-250-2 |
| 5-8 | box 450-15 | 2UNP160 | 400-12 / 250-20 | 3-8 | box 450-15 | 2UNP160 | 400 / 12-250-2 |
| 9-12 | box 400-15 | 2UNP100 | 400-12 / 250-20 | 9-12 | box 400-10 | 2UNP100 | 400 / 12-250-2 |
| 13-16 | box 400-10 | 2UNP100 | 350-8 / 250-15 | 13-16 | box 400-10 | 2UNP100 | 400-8 / 200-10 |
| 17-20 | box 250-10 | 2UNP100 | 300-8 / 200-12 | 17-20 | box 250-10 | 2UNP100 | 300-8 / 200-12 |
| 20 storey, Case 2 | | | | 20 storey, Case 3 | | | |
| Storey | Column | Brace | Beam ($h_w$-$t_w$ / $b_f$-$t_f$) | Storey | Column | Brace | Beam ($h_w$-$t_w$ / $b_f$-tf) |
| 1-2 | box 500-20 | 2UNP200 | 400-12 / 250-20 | 1-2 | box 450-20 | 2UNP200 | 400-10 / 250-1.5 |
| 3-8 | box 450-12 | 2UNP160 | 400-12 / 250-20 | 3-8 | box 400-15 | 2UNP160 | 400-10 / 250-1.5 |
| 9-12 | box 400-10 | 2UNP100 | 400-12 / 250-20 | 9-12 | box 400-10 | 2UNP100 | 400-8 / 200-10 |
| 13-16 | box 400-10 | 2UNP100 | 400-8 / 200-10 | 13-16 | box 350-10 | 2UNP100 | 400-8 / 200-10 |
| 17-20 | box 250-10 | 2UNP100 | 300-8 / 200-12 | 17-20 | box 250-8 | 2UNP100 | 300-8 / 200-12 |
| 15 storey | | | | 15 storey, Case 1 | | | |
| Storey | Column | Brace | Beam (hw-tw / bf-tf) | Storey | Column | Brace | Beam (hw-tw / bf-tf) |
| 1-2 | box 450-20 | 2UNP200 | 400-8 / 200-20 | 1-2 | box 450-20 | 2UNP200 | 400-8 / 200-20 |
| 2-4 | box 400-15 | 2UNP160 | 400-8 / 200-20 | 2-4 | box 400-15 | 2UNP160 | 400-8 / 200-20 |
| 5-8 | box 400-10 | 2UNP100 | 400-8 / 200-20 | 5-8 | box 400-10 | 2UNP100 | 400-8 / 200-20 |
| 8-10 | box 350-10 | 2UNP100 | 350-8 / 200-15 | 8-10 | box 350-10 | 2UNP100 | 350-8 / 200-15 |
| 11-12 | box 250-10 | 2UNP100 | 300-8 / 200-12 | 11-12 | box 250-10 | 2UNP100 | 300-8 / 200-12 |
| 13-15 | box 200-10 | 2UNP100 | 250-8 / 150-12 | 13-15 | box 200-10 | 2UNP100 | 250-8 / 150-12 |
| 15 storey, Case 2 | | | | 15 storey, Case 3 | | | |
| Storey | Column | Brace | Beam (hw-tw / bf-tf) | Storey | Column | Brace | Beam (hw-tw / bf-tf) |
| 1-2 | box 450-20 | 2UNP200 | 400-8 / 200-20 | 1-2 | box 450-15 | 2UNP200 | 400-8 / 150-15 |
| 2-4 | box 400-15 | 2UNP160 | 400-8 / 200-20 | 2-4 | box 400-15 | 2UNP160 | 400-8 / 150-15 |
| 5-8 | box 400-10 | 2UNP100 | 400-8 / 200-20 | 5-8 | box 400-10 | 2UNP100 | 400-8 / 150-15 |
| 8-10 | box 350-10 | 2UNP100 | 350-8 / 200-15 | 8-10 | box 350-10 | 2UNP100 | 350-8 / 200-15 |
| 11-12 | box 250-10 | 2UNP100 | 300-8 / 200-12 | 11-12 | box 250-10 | 2UNP100 | 300-8 / 200-12 |
| 13-15 | box 200-10 | | 250-8 / 150-12 | 13-15 | box 200-10 | 2UNP100 | 250-8 / 150-12 |
| 10 storey | | | | 10 storey, Case 1 | | | |
| Storey | Column | Brace | Beam (hw-tw / bf-tf) | Storey | Column | Brace | Beam (hw-tw / bf-tf) |
| 1 | box 400-15 | 2UNP120 | 300-8 / 200-20 | 1-2 | box 400-10 | 2UNP120 | 300-8 / 200-20 |
| 2-4 | box 400-10 | 2UNP120 | 300-8 / 200-10 | 2-4 | box 400-10 | 2UNP120 | 300-8 / 200-10 |
| 5-8 | box 250-12 | 2UNP100 | 200-8 / 200-10 | 5-8 | box 250-12 | 2UNP100 | 200-8 / 200-10 |
| 8-10 | box 200-10 | 2UNP100 | 200-8 / 150-10 | 8-10 | box 200-10 | 2UNP100 | 200-8 / 150-10 |
| 10 storey, Case 2 | | | | 10 storey, Case 3 | | | |
| Storey | Column | Brace | Beam (hw-tw / bf-tf) | Storey | Column | Brace | Beam (hw-tw / bf-tf) |
| 1-2 | box 400-10 | 2UNP120 | 300-8 / 200-2 | 1-2 | box 350-12 | 2UNP120 | 300-8 / 200-10 |
| 2-4 | box 350-12 | 2UNP120 | 300-8 / 200-10 | 2-4 | box 350-12 | 2UNP120 | 300-8 / 200-10 |
| 5-8 | box 250-12 | 2UNP100 | 200-8 / 150-10 | 5-8 | box 200-10 | 2UNP100 | 200-8 / 150-10 |
| 8-10 | box 200-10 | 2UNP100 | 200-8 / 100-10 | 8-10 | box 200-10 | 2UNP100 | 200-8 / 100-10 |

reach a specific displacement based on the gravity load patterns by the actual force of the structure. In fact, the load factor of 1 is related to the condition in which the structure is exposed to its actual loading. Therefore, this factor is an indicator of the capacity the structure shows against the instability resulting from the sudden column removal. The NDA is used to evaluate the response of the structure to the sudden column removal [24].

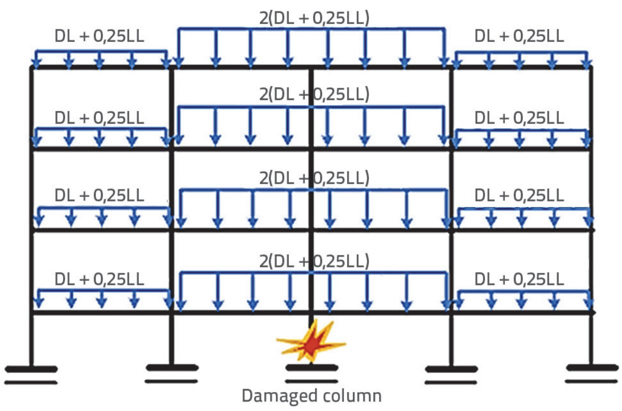


**Figure 3. Method of applying local failure and load combination related to NSA**

According to the GSA standard, APM is one of the most useful methods for evaluating the PC potential through simulation of sudden column removal. In this method, the structure is firstly analysed under gravity load. Subsequently, the load on the desired element in the structure is determined and then the load is applied in the direction opposite to the upper node after removing the element. In order to perform an NDA, the gravity load with a load combination of DL + 0.25LL reaches its real value linearly within five seconds. To eliminate vibrations, the load remains constant for two seconds and then, suddenly, the reaction of the deleted column will be removed from the structure as shown in Figure 4 (a) and (b), and the resulting vibrations are eliminated as a vertical displacement of the column and the time is presented as an NDA [8]. This curve illustrates dynamic response of the structure to sudden column removal.

There are numerous software applications for NDA nowadays. OpenSees is one of the finite element software programs that is available to users in the open-source mode [25]. This software has many capabilities for nonlinear modelling and dynamic analysis. In this software, it is possible to delete any load or delete any element during the analysis. Due to the capabilities of this software, an attempt has been made to model the sudden column removal scenario in a realistic way by combining the proposed method of APM, suggested in the GSA standard, and the ability to remove elements during analysis. It is supposed that gravity load is applied linearly with a combination of DL + 0.25LL within five seconds, as shown in Figure 4 (without the R curve). Then continue as a constant and as a function of time like the curve "W", in Figure 4.

To model the sudden removal of a column, it is assumed that in the seventh second of the analysis the desired columns are removed from the structure and that the structure begins to vibrate. As stated in this study, OpenSees software is used for the NDA. The nonlinear Beam-Column element is used for the nonlinear modelling of beams and columns. P-Delta effects are also considered in the elements modelling by using the Corotational command. In OpenSees, there are three methods for assigning local axes: linear, P-Δ, and Corotational. The Corotational is a comprehensive method that takes into account both the effects of P-Δ and the effects of large deformations. Therefore, since the effects of P-Δ are important in progressive collapse, the Corotational method, in which both the effects of P-Δ and the effect of large deformations are considered, is used. Steel01 material is utilized from the library of OpenSees software for modelling steel elements considered in the current study. Material specifications mentioned in Table 1 are used for the definition of this material. The finite element model with distributed plasticity in the form of fibre with 10 integration points is used for modelling the elements. The secondary stiffness coefficient is assumed to be equal to 0.02. Therefore, the model used for the elements is based on distributed plasticity with 10 integration points in which the behaviour of each fibre is based on the stress-strain relationship, and is considered according to Steel01 material.

There are several methods for modelling plastic behaviour of materials and elements. Two common methods for such

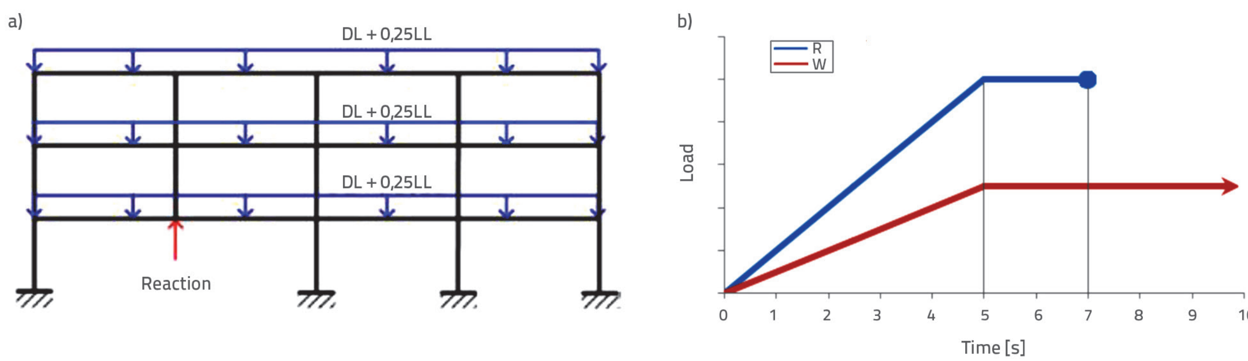


**Figure 4. NDA according to GSA Standard (GSA): a) Load pattern; b) Time history of gravity loads**

modelling involve the use of concentrated and distributed plasticity model. In the concentrated plasticity method, the total plastic behaviour of the materials is considered in the form of moment-rotation curves in the beams and moment-rotation along with the interaction of axial force in the columns. In the distributed plasticity method, the properties of the plastic joint are considered as fibres. These fibres can be present along the element or can be considered as dots along the element. These points are defined as integration points. As stated in the OpenSees software manual, the strain hardening coefficient is expressed by a coefficient b. This coefficient expresses the ratio of hardening in the plastic area to the initial hardening and is usually considered in various sources as ranging from 0.01 to 0.05.

There are two comprehensive methods in plasticity modelling, the concentrated plastic joint method and the distributed plasticity method [26]. In the concentrated plasticity method, which itself is divided into two methods of force-control and displacement-control, it is assumed that the integration points are located at critical points of the member. In columns, for example, concentrated plastic joints are placed at both ends of the column where the shear is maximum. These points are considered as two points of integration. In distributed plasticity, which involves neither deformation-control nor force-control, but stress-strain relationships are used, the integration points other than the critical points are used in order to reduce the volume of the equations and also to maintain accuracy. The number of these points in the OpenSees software is maximum ten points, which can be used in the equations in a number of ways. The following is a description of these methods.

The Nonlinear Beam-Column element is one of the most useful and applicable elements for modelling nonlinear behaviour of elements that is available in the library of the OpenSees software. Properties and formulation of this element is explained in the manual of the OpenSees software. In this paper, ten points are considered as integration points. Integration points can be used in two places: along the element and at the height of the section (the height of the section is used in cases where the fibre section is used, which is not desirable for this research). If the fibre command is used in section modelling, the user must first assume that according to the section conditions, the section must first be divided into several parts and then its properties will be assigned to each section. In this case, the integration points will be assigned automatically. But in this research, the patch quad command is used to make sections, which is a more accurate method. In this method, first the shape of the section is created by giving the coordinates of different parts of the section. Then, by giving the division points, each part of the section in any direction is meshed in accordance with the required number. Therefore, the division of the cross section can be considered not as points of integration but as meshing and this is the difference between the points of integration and the meshing that must be specified.

Tolerance for the satisfaction of element compatibility is equal to $10^{-16}$ and the maximum number of iterations to undertake to satisfy element compatibility is assumed as 1. Additionally, the Corotational Formulation is used for the definition of local axes of this element. Furthermore, sections of this element are defined as fibre type with nonlinear stress-strain relationship according to Steel01 material. An example of 3D modelling of structures analysed in this study is shown in Figure 5.

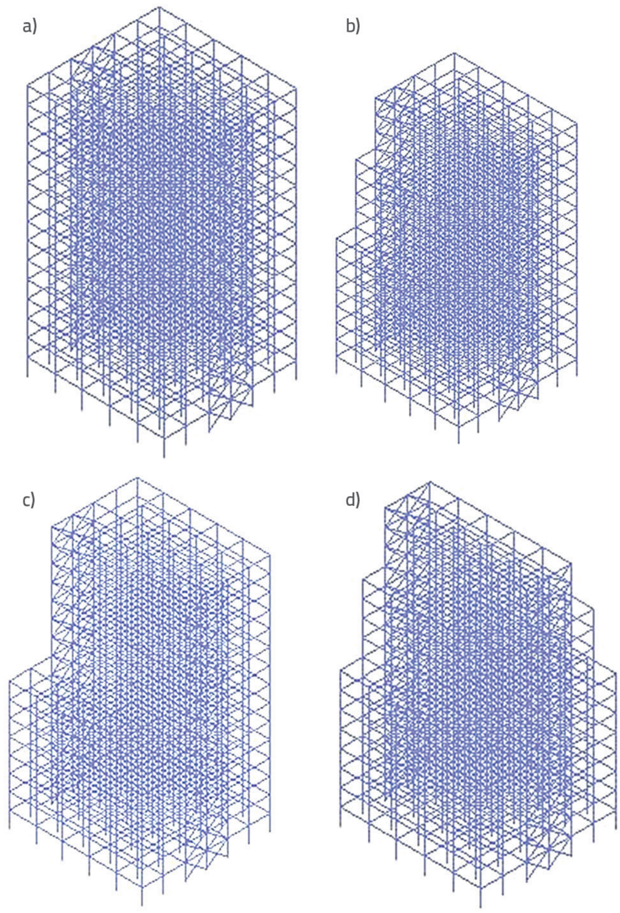


**Figure 5. 15-storey structures models in the OpenSees software: a) 15 storey; b) 15 storey – Case 1; c) 15 storey – Case 2; d) 15 storey – Case 3**

Various modes were considered when discussing in this study the location of local failure and sudden removal of columns. Four column removal scenarios were considered for each structural type.

Thus various situations were considered regarding the local failure position and the sudden column removal scenario. The sudden column removal scenarios are; columns D1 and D3 and G3 on the 6th floor, and column D3 on the 8th floor of the 10-storey structure, as well as other scenarios, involving columns D1 and D3 and G3 on the 8th floor and D3 on the 12th floor of the 15-storey structure, and columns D1 and D3 and

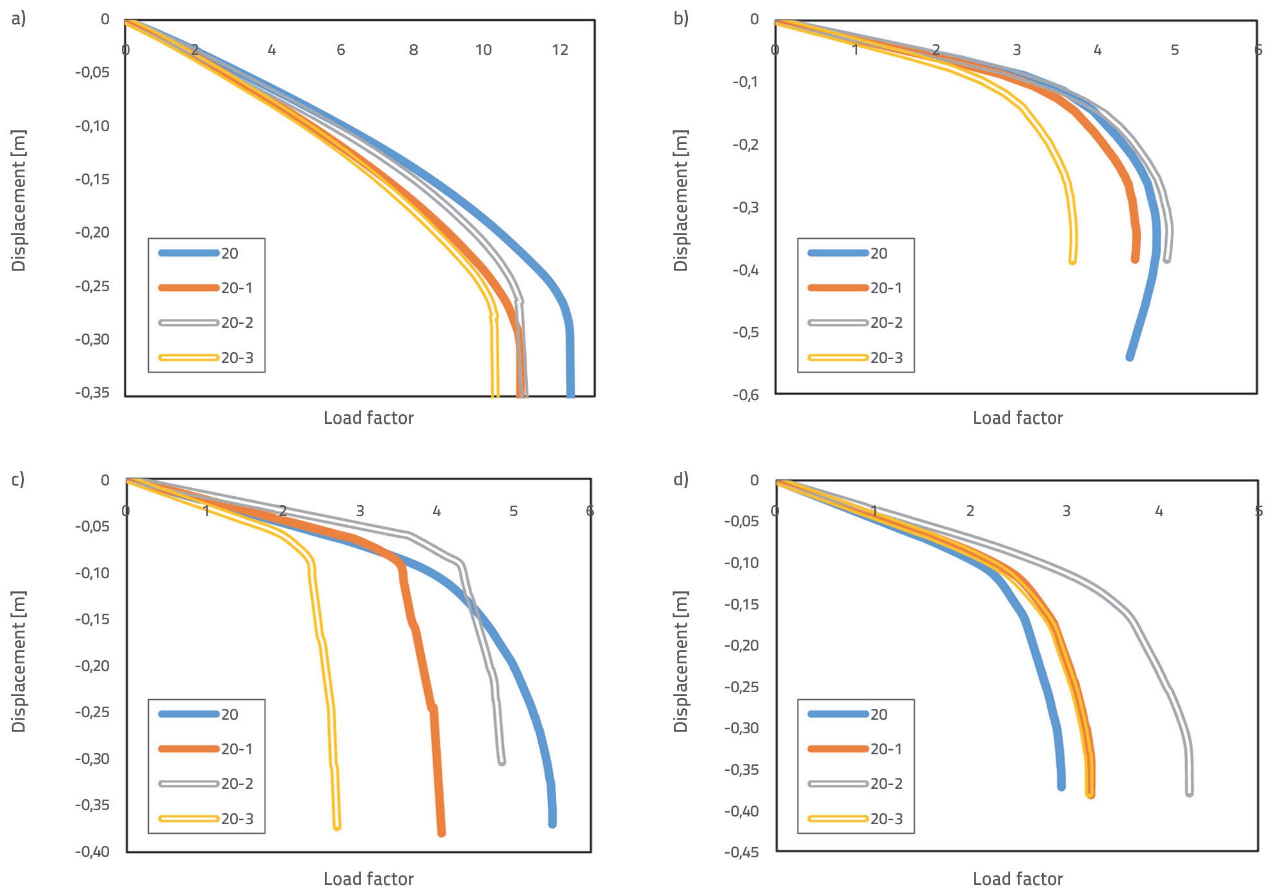


**Figure 6. Static pushdown curves for 20-storey structures, a) D1-10th floor; b) D3-10th floor; c) G3-10th floor; d) D3-16th floor**

G3 on the 10th floor and D3 on the 16th floor of the 20-storey structure, as shown in Figure 1.

## 3. Study of the analysis results

After nonlinear modelling of structures, the two main objectives of this research are to investigate the effects of sudden column removal and to study the capacity of the structures in this research against the phenomenon of progressive collapse according to GSA standard by NSA and NDA.

In this paper, the phrase "response of the structure" relates to the vertical displacement of the removed column's top point in the dynamic push down analysis, and the phrase "capacity of the structure" relates to the values of yield and ultimate load factors in the static push down curve of the structure. Initially, the capacity of structures used in this research was studied for the case of the sudden column removal in the above-mentioned scenarios. Figure 6 shows static pushdown curves for 20-storey structures. Static push down curves are usually presented so that the negative displacement of the damaged point is on the vertical axis, and the calculated load factor is on the horizontal axis. These curves are generally compared based on the yield and ultimate displacements, and are substantially presented for comparing the capacity of the structure in the case of force redistribution induced by sudden column removal. Hence, interpretation of these curves is usually qualitative and phrases such as "higher capacity" or "lower capacity" are used.

As can be seen in static pushdown curves given in Figure 6, the load factor of yielding is higher than one in all structural models and in all column removal scenarios. This means that these structures have a sufficient capacity against the PC caused by column removal scenarios.

Based on the GSA standard, a structure has sufficient capacity to resist force redistribution if the yield displacement occurs in load factors of more than 1. If the ultimate displacement occurs in load factors lower than 1, the structure does not have sufficient capacity against force redistribution and would surely reach the global failure point. Therefore, if the yield displacement occurs in load factors higher than 1, it can be concluded that the structure is resistant against sudden column removal and that no progressive collapse will take place. According to the above explanations, the structure will have sufficient capacity against progressive collapse in this investigation, since the load factor corresponding to the yield displacement is higher than 1 in all scenarios.

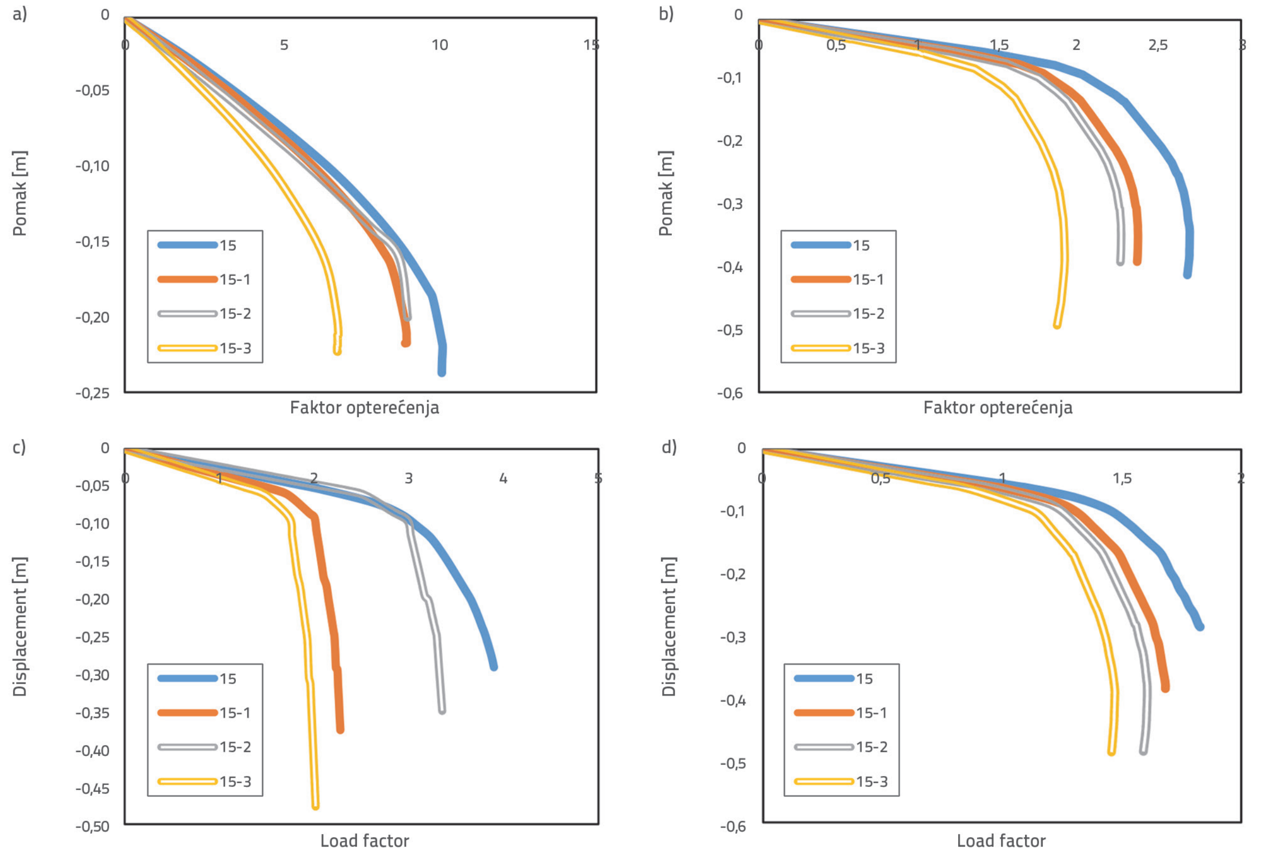


**Figure 7. Static pushdown curves in 15-storey structures: a) D1 - 8th floor; b) D3 - 8th floor; c) G3 - 8th floor; d) D3 - 12th floor**

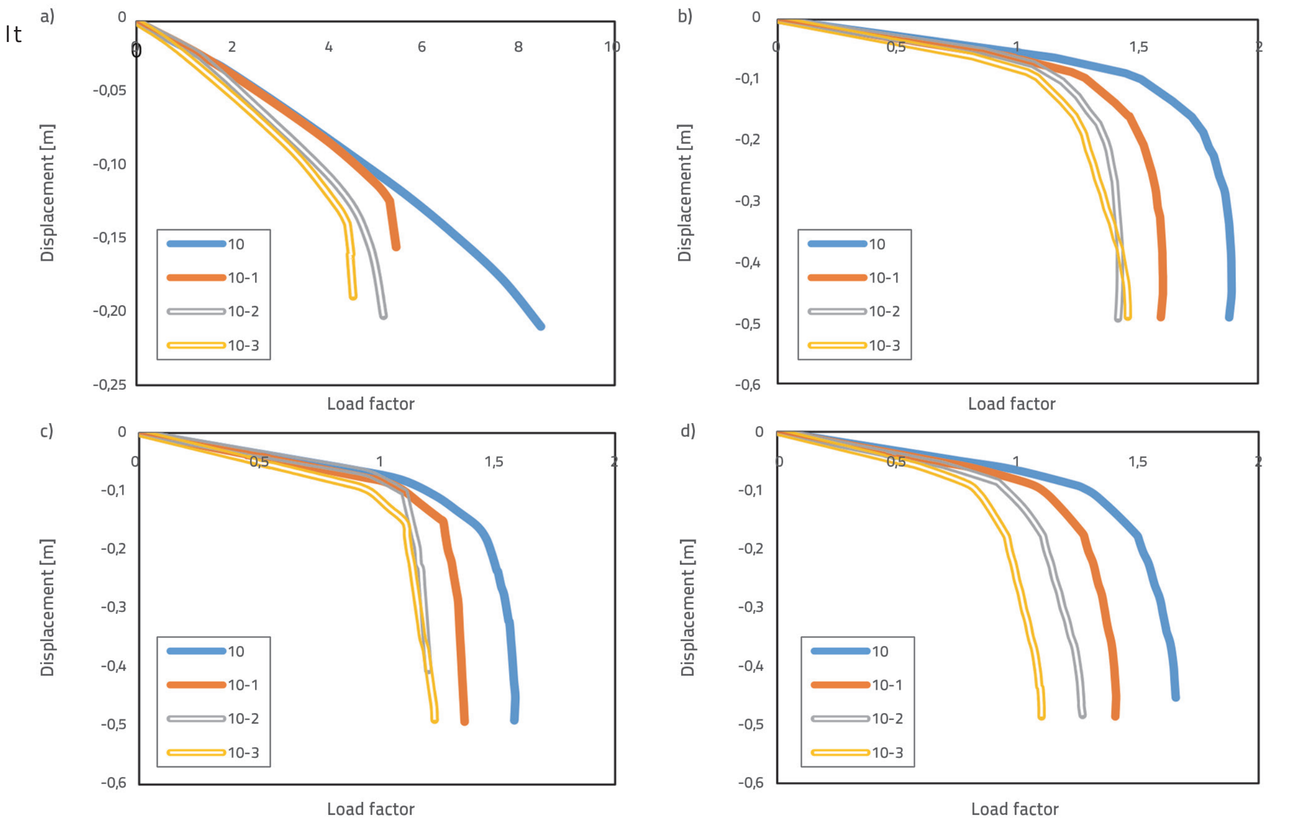


**Figure 8. Static Pushdown curves in 10-storey structures: a) D1 - 6th floor; b) D3 - 6th floor; c) G3 - 6th floor; d) D3 - 8th floor**

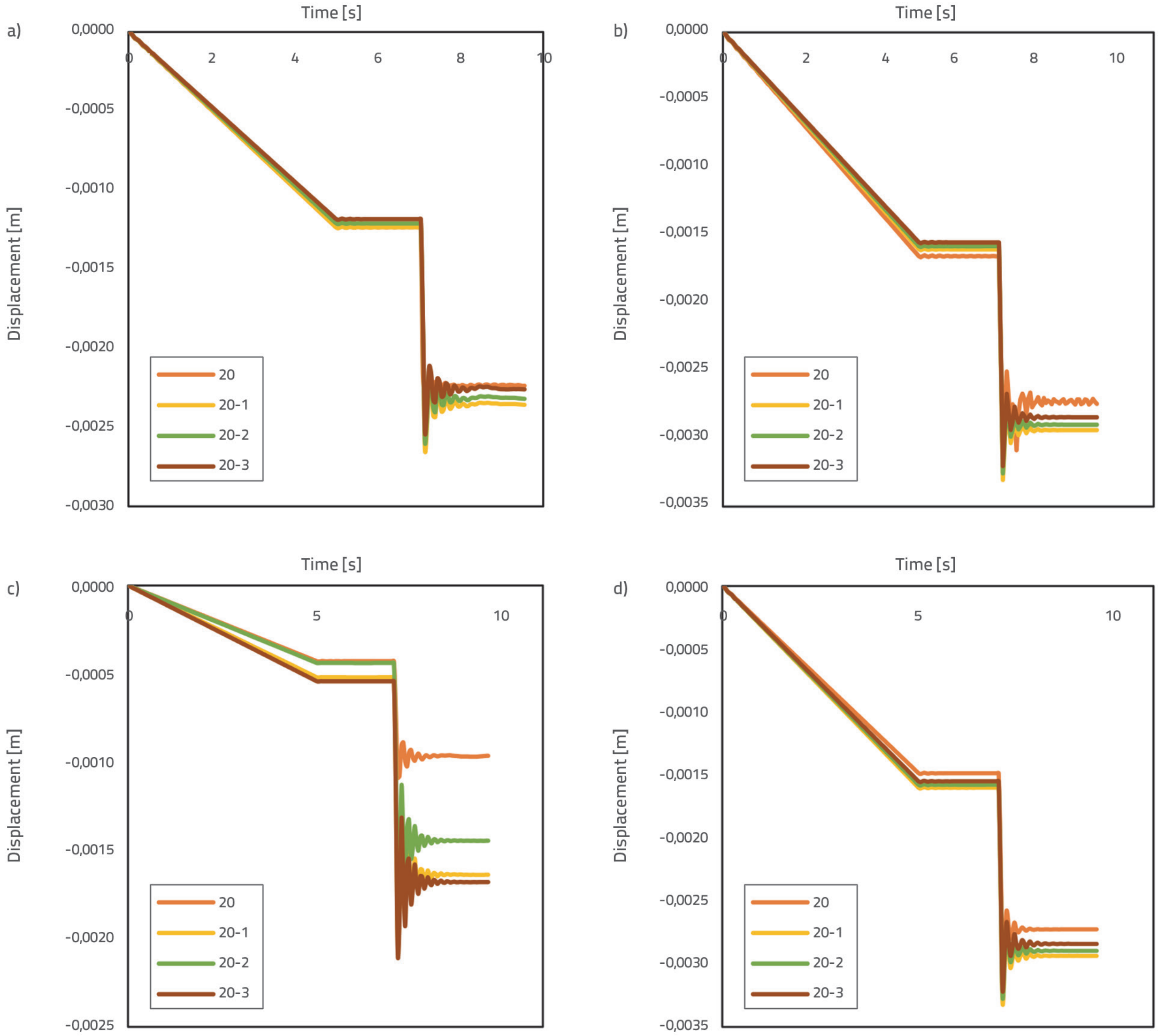


**Figure 9. Nonlinear dynamic curves of 20-storey structures: a) D1 - 10th floor; b) D3 - 10th floor; c) G3 - 10th floor; d) D3 - 16th floor**

can be seen in Figure 6 (a) that the capacity of the structure to deal with the PC phenomenon is higher in the case of local failure in braced span, than in other situations. In this situation, the non-irregular structure has higher capacity against local failure compared to irregular structures.

Push down curves have two important components indicating the capacity of the structure in a sudden column removal scenario. These two components are yield and ultimate load factors. The higher these two components are, the more capacity the structure has to tolerate force redistribution in progressive collapse induced by sudden removal of a column. Values of yield and ultimate load factors given in Figure 6(b) are lower than those given in Figure6(a) implying that the structure has much more strength in damage conditions in braced frames compared to unbraced ones. Moreover, by comparing Figures 6(b) and 6(d), it can be concluded that, under similar damage conditions, the capacity of the structure for bearing force redistribution would be lower if the damage occurs in higher spans.

The yield load factor in a regular structure is equal to 12, while this factor in the first, second and third cases of irregular structures equals to 10.2, 10.9, and 10.07, respectively. Therefore, the regular structure has a higher capacity in height in comparison to irregular structures. Figure 7 shows this process for 15-storey structures.

Curves given in Figure 7 show that the load factor decreases by declining the number of floors from 20 to 15. Moreover, the smallest load factor is related to the damage case on the 12th floor. In the models of the 15-storey structure, the highest load factors belong to non-irregular structures and the lowest load factors are related to structural models of case 3 which shows the minimum capacity against PC. Figure 8 shows the capacity of the structures against the column removal scenario during PC in 10-storey structures.

Like the 15-storey models, behaviour of non-irregular structures in height in the 10-storey models shows that the capacity of these structures against sudden column removal and their ability to redistribute excess forces is greater compared to other structures. Load factors in 10-storey structures are lower compared to other structures. In addition, these structures exhibit the highest potential against PC when local failure occurs in the braced span. The effects and responses to sudden column removal are also investigated in different scenarios. Figure 9 shows curves indicating response of structures to sudden column removal scenario in NDA for 20-storey structures.

The analysis of NDA curves in the sudden column removal scenarios indicates that in 20-storey structures, the vertical displacement of the top point of the removed column amount to a few millimetres. This shows that the column removal in high-rise structures leads to a negligible response. In fact, a sudden column removal from the structure will not cause PC in these structures and they will not have the potential of total failure in the event of this sudden removal. Furthermore, the study of the irregularity effect in the structure shows that irregularity in 20-storey models does not have significant effects on structural response, because the maximum displacement and final displacement are close to each other in structural models. However, 20-storey regular structures have the smallest response in all column removal scenarios. This structure has both the smallest maximum displacement and a sustained response. Also, the column D1, 10$^{th}$ floor removal scenario, shows that the sudden column removal in the adjacent braces creates the smallest vibration in the structure. Moreover, the results show that the presence of the braces in the area of column removal causes that the structure to be controlled by slower vibrations and smaller vibration domains, thus regaining its balance.

The NDA curves are shown in Figure 10 in the desired scenarios for the 15-storey structures. NDA curves of 15-storey structures show that in all models, as in 20-storey structures, a non-irregular structure has the smallest response compared to other structures. The sudden column removal scenario in a regular 15-storey structure has both the smallest maximum response and sustained response in

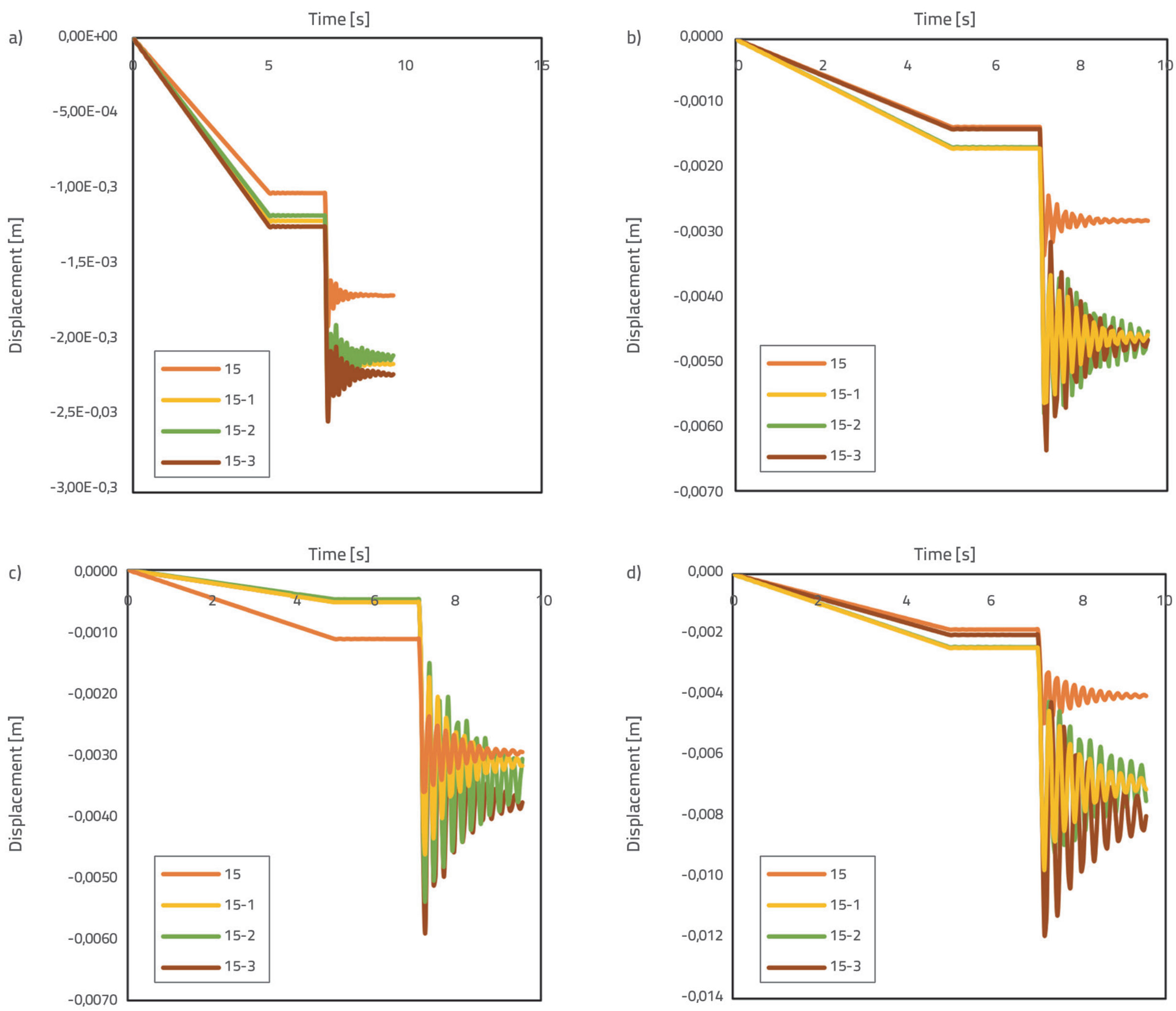


**Figure 10. Nonlinear dynamic curves of 15-storey structures, a) D1 - 8$^{th}$ floor; b) D3 - 8$^{th}$ floor; c) G3 - 8$^{th}$ floor; d) D3 - 12$^{th}$ floor**

comparison with other structures. Examination of the place of sudden removal of columns also shows that in the scenario of sudden removal of column D1 - 8th floor, the structure has the lowest response compared to other scenarios and, in addition, the vibrations caused by the sudden removal of column in this case are lower compared to other scenarios. The column removal response in scenario D3 - 12th floor - shows that the removal of the column in the upper floors increases the response of the structure. In fact, the reduction of the number of load-bearing elements increased the response of the structure. In structural models related to the 15-storey structure, the highest response relates to the response caused by removing the column D3 -12th floor - in an irregular structure of case 15-3. In the study of irregularity effects, it can be concluded that the presence of irregularities in height can affect lateral loading, i.e. modal deformations and shear of the floors. But, in the sudden column removal scenario, lateral load and also lateral behaviour of irregular structures are not significant, and the important issue is the ability of the structure to withstand the overload caused by sudden column removal. The structure with more elements and structural sections has a greater ability to withstand excessive loads. Creating the irregularities discussed in this study leads to weight loss in the structure. Decreasing the number of elements in irregular structures, firstly reduces the number of structural elements that can be effective in bearing overload caused by the column removal and, secondly, as the weight of the structure decreases, the structural sections also become weaker and the load-bearing capacity decreases.

According to the Iranian code of practice for seismic design of structures - Buildings standard - 2800, the design earthquake load for seismic resistant structures is calculated based on the earthquake coefficient multiplied by the structure's weight. Since the removal of a portion of structural elements in height would lead to a total weight reduction, it is obvious that the quantity of earthquake force would decrease and the structure would be designed for lower levels of seismic loads. Therefore, it is plausible to achieve weaker structural elements in structures with irregularities in height rather than structures without any irregularity. However, this is not a general rule and can be merely a reason for the reduction

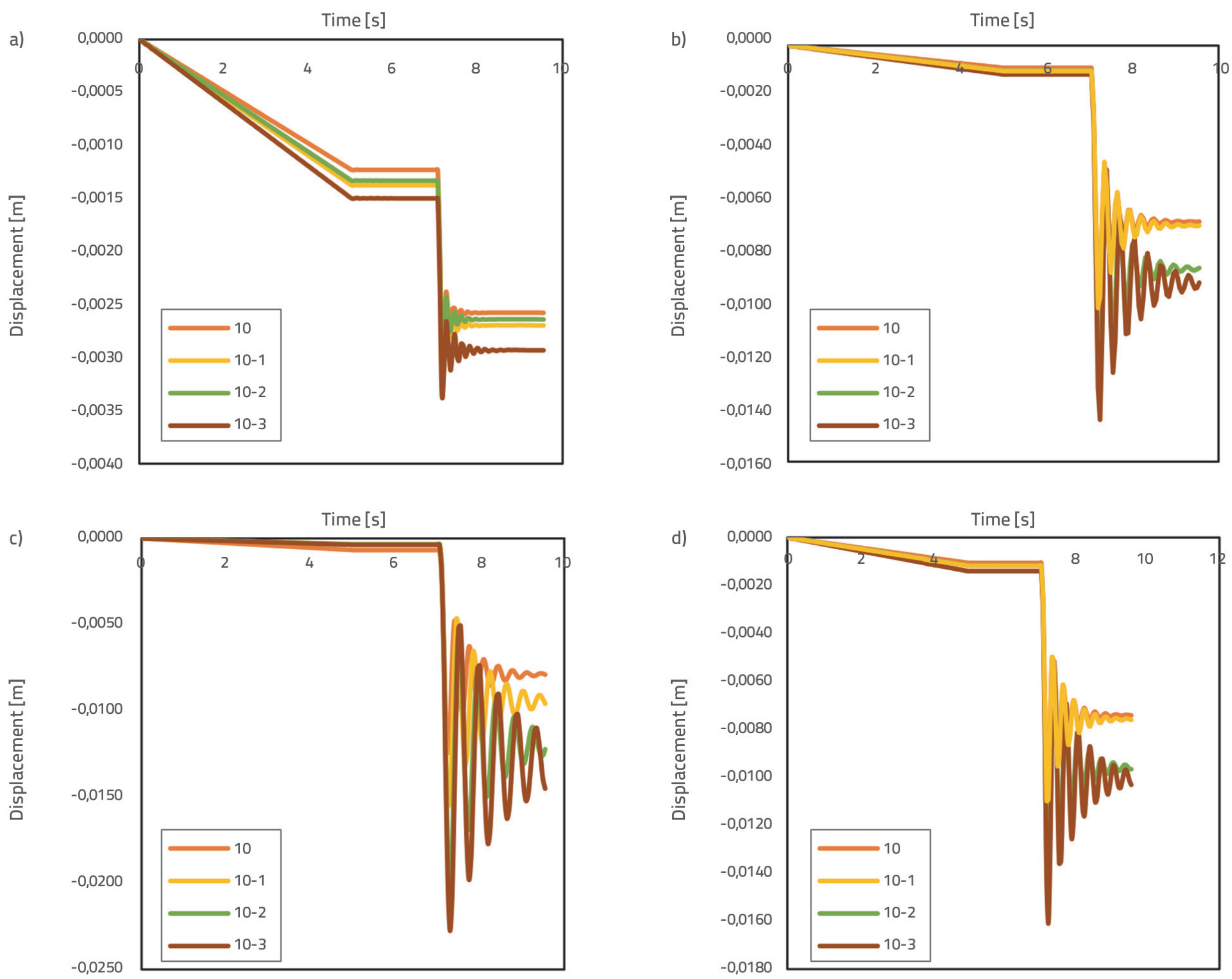


**Figure 11. Nonlinear dynamic curves of 10-storey structures: a) D1 - 6th floor; b) D3 - 6th floor; c) G3 - 6th floor; d) D3 - 8th floor**

of capacity and growth of response in structures with irregularities.

Therefore, it can be concluded that the increase of response in irregular structures in height is not due to lateral behaviour, but rather to the decrease in the number of elements and weakening of structural sections. Figure 11 shows this process for 10-storey structures.

The analysis of curves given in Figure 11 shows that the responses of 10-storey structures compared to the 15 and 20-storey structures have increased. Therefore, it can be concluded that the response of the structure to the sudden column removal increases by reducing the number of floors. In the 10-storey models, the highest response of structures relates to the case G3-6. Similar to the 15 and 20-storey models, irregular structures have the smallest response and the lowest potential in the case of sudden removal of a column. Generally, case 10-3 irregular structures have the highest response to sudden column removal. When considering all models, the smallest response involves the case of removing the column from the braced span. The existence of braces has decreased the highest response and vibrations because of sudden removal. Increasing the number of spans in the structure has increased the response to sudden column removal.

## 4. Conclusion

The purpose of this study was to investigate the behaviour of irregular structures in height in various PC scenarios after sudden column removal. In this research, 10-, 15- and 20-storey structures were subjected to sudden column removal after nonlinear modelling using the APM. Four-column removal scenarios were defined in three irregular structures and a non-irregular structure. Two issues involving the capacity of structures against sudden column removal, and their structural response, were evaluated by the NSA and NDA, respectively.

The results show that structural capacity increases with an increase in the number of floors in both regular and irregular structures. This issue caused a decrease in the dynamic response of the structure. Increasing the irregularity in height in the structure is inversely related to decreasing its potential in the progressive collapse scenario. The results also show that the removal of the column in the braced span can have the least response to the sudden removal of the column. As the number of floors decreases, the irregular effect of height on decreasing the capacity of structures against PC increases, and irregular structures with lower height have a greater response due to sudden removal of the column. However, the removal of one column in all the structures discussed in this study did not lead to PC and the structures reached the force equilibrium again.

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