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*Research Paper*

# Proposal of an Energy Based Assessment of Robustness Index of Steel Moment Frames under the Seismic Progressive Collapse

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**ABSTRACT:** One of the aims of earthquake engineering is to build secure structures against random loads and also various damage types under lateral loads. Progressive collapse, a word that has attracted attention of many researches after the failure of the World Trade Center, can occur under abnormal loads such as explosion or natural causes like earthquakes. Resistance to progressive collapse is expressed by a parameter called Robustness. The purpose of this study is to survey various methods of calculating robustness index under lateral loads, especially seismic loads, in steel moment frames. So three steel structures with 4, 8 and 15-story and intermediate moment frames were designed and analyzed subsequently. Different methods of measuring the robustness indexes were compared and eventually presented a simple method to assess robustness index based on nonlinear dynamic analysis. Robustness index introduced using this method, which is based on the types of Pancake and Zipper collapses and energy parameters, tries to express an appropriate standard for structural strength against earthquakes.

**Keywords**: Earthquake, Pancake Collapse, Progressive Collapse Robustness Index, Zipper Collapse.

## INTRODUCTION

Nowadays, building errors or designing weak structural elements and subsequently vulnerability of the entire structure because of weakness has become an important challenge. These issues cause removing elements due to unexpected events, such as earthquake and seismic stimulations and eventually progressive collapse of structural elements. Failure of one or more structural bearing members because of unexpected loads which finally leads to collapse of the entire structure, is called progressive collapse (Kim et al., 2011). Progressive collapse is defined as the spread of an initial damage from one member to another, leading to extensive partial or total collapse of the structure (Rezaei et al., 2018). For the first time, after destruction of a part of Ronan Point building in London (1986), engineers paid attention to this topic. After the events of September 11, 2001, several standardization committees started to rethink and improve their standards pertaining to progressive collapse design procedures (Menchel, 2009). Observing structural damages in past earthquakes shows that seismic loads can

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cause damages leading to the loss of supports and the initial failure of structural elements can spread to other bearing components in different directions (Rashidi Alashti, 2012). After the failure of an element in a structure, its loads should be redistributed on the other elements and the structure must provide some new paths to carry the load. If such new load paths are not provided, collapse progression will begin in the structure (Kheyroddin et al., 2019).

Also, seismic progressive collapse can cause problems during intensive earthquakes in structures designed on the basis of current standards and even leads to destruction of the entire structure. In other words, any weakness in designing or structural elements implementation may cause progressive collapse phenomenon in structures during seismic loading. So it seems necessary to study the effect of this type of collapse in structures (Yu et al., 2010).

The General Services Administration (GSA) (2003) and the Department of Defense (DOD) (2005) in the United States of America provided instructions that include strengthening strategies in building structures towards progressive collapse. Among various methods of designing buildings against progressive collapse, regulations generally suggest Alternative Path Method (APM). In this approach, the structure is designed in the way that if one component fails, alternate paths will be available for the load and a general collapse will not occur. In fact, the damaged element in this method will be removed and it will be checked that if structural elements can bear additional loads caused by this removal or not. It should be noted that usually structural elements will not be removed with vibrations of severe earthquakes like their removal in explosion loading. In contrast, structural elements resistance or stiffness may reduce dramatically through seismic loading and cause elements to lose their performance. In other words, if the stiffness or element resistance reaches to 20% of its initial value, it will be considered as failure. Also, to evaluate the progressive collapse phenomenon in structures, regulations recommend these methods of analysis in order to find alternative load paths; Linear Static, Linear Dynamic, Nonlinear Static and Nonlinear Dynamic (Kim and Kim, 2009).

Starossek (2007) has classified progressive collapse into six types. This classification is based on the structural behavior and how the collapse distributes and it does not depend on primary causes of the building damage. 1) Pancake-type collapse: In this type of collapse, the directions of elements failure and redistribution of forces caused by failure are the same like the failure of WTC buildings. 2) Zipper-type collapse: In this type of collapse, the directions of elements failure and redistribution of forces are perpendicular to each other such as cable-stayed bridges. 3) Domino-type collapse: This type of collapse occurs because of elements or structures overturning and their collision to adjacent elements or structures. 4) Section-type collapse: This type of collapse occurs because of yielding in sections and distribution of forces in adjacent elements. 5) Instability-type collapse: This type of collapse happens because of the elimination of support constraints and loss of stability in structures. In static manner if elements work as pressure-bearing, the instability type will be called buckling. 6- Mixed-type collapse: Many of occurred collapses cannot be placed in one of the mentioned types and have the features of several collapses together. These types of collapses are called mixed-type collapses.

Khandelwal et al. (2009) investigated the progressive collapse of steel braced frames designed on the basis of seismic standards. They compared resistance against progressive collapse of frames using APM method. Simulations showed that EBF braced frames are less vulnerable during progressive collapse compared to SCBF frames due to gravity loads. Tavakoli and Kiakojurri (2013) studied the progressive collapse of steel structures based on APM method under blast loading. They assessed various scenarios caused by the sudden removal of column. The results showed that the removal of column affects the total response of structure under blast loading.

Wibowo and Lau (2009) presented a brief overview of the progressive collapse phenomenon in structures. Methods and requirements of several standards to prevent progressive collapse were discussed. Significance of seismic loads influence on behavior of structures was considered under this type of collapse. It was concluded that the seismic progressive collapse of structures can be analyzed by modifying the current methods. Kim and Kim (2009) have investigated resistant capacity against progressive collapse in steel moment frames. Comparing the results of analysis, they concluded nonlinear dynamic analysis is an exact method to assess the potential of progressive collapse in buildings. SzynisZewski and Krauthammer (2012) studied the progressive collapse of steel structures according to energy flow. In this method, if the kinetic energy caused by the sudden removal of a column counterbalances with the plastic and elastic strain energy and structure's damping , the structure will remain safe, otherwise it will suffer from damaging.

Janssens (2012) in her assessments about progressive collapse modeling scenario, concluded that linear static analysis method is conservative compared to nonlinear analysis. Also dynamic effects play a pivotal role in the failure of structures. So it should be examined in studies. Her studies also indicated that applying different damping values in structures does not have a remarkable effect on progressive collapse scenarios and this parameter can be ignored.

Tavakoli and Naghavi (2015) evaluated the potential of progressive collapse in concrete structures under gravity and lateral loads. Results showed that concrete structures under gravity loads have good resistance during the progressive collapse scenario. In their investigations they also discussed about the role of seismic isolation in progressive collapse. Results indicated that seismic isolation has no effect on the scenario of progressive collapse under gravity loads. But under lateral loads, structures with seismic isolation showed high resistance. Choubey and Goel (2016) studied progressive collapse behaviour of RCC building under extreme loading events such as gas explosion in kitchen, terroristic attack, vehicular collisions and accidental overloads. Al-Salloum et al. (2017) present an advanced numerical analysis procedure to predict the progressive collapse potential of RC buildings exposed to blast generated waves. Tian et al. (2017) present an evaluation method of important members and a novel dynamic analysis method for simulating the progressive collapse of long-span spatial grid structures. Mashhadi and Saffari (2016) investigated the effect of damping ratio on nonlinear dynamic analysis response and dynamic increase factor in nonlinear static analysis of structures against column removal. Chen et al. (2016) analyze structural member`s sensitivity to abrupt removal of a column to determine a sub-structure resisting progressive collapse. Wilkes et al. (2019) presented a new method for assessment of progressive collapse mechanism based on energy flow. This method is based balanced between elastic and plastic energy with kinetic energy. Tavakoli and Moradi (2019) assessed failure time of steel frame subjected to fire load under progressive collapse scenario. Shen et al. (2019) studied on critical member for progressive collapse analysis. Jia et al. (2019) studied progressive collapse and robustness of steel frame buildings. Shan et al. (2019)

studied on robustness of RC buildings to progressive collapse. They presented a comparison between robustness and resilience of RC building.

## ROBUSTNESS INDEX

One of the common expressions in the literature of progressive collapse is robustness index. Various definitions have been provided for robustness. One of them is the ability of structures to confront accidents such as fire, explosion, impact and human errors without any particular damage to the main structure. Another definition is the ability of structures or the elements to resist against damage causing no fast or brittle failure in accidents such as explosion, impact, fire or human errors.

Robustness also is defined as insensitivity of the structure to a localized failure. In other words, robustness is resistance of structures during localized failure. A robust structure can resist during loading without any disproportionate damage. Robustness can be assessed using three different ways: Non-probabilistic, Probabilistic and Risk-based approaches. In this paper, estimation of robustness index has only discussed Non-probabilistic method. Several terms have been suggested to assess the robustness. The most famous ones which can be mentioned are the methods based on stiffness, damage, energy and the base shear. The simplest method to estimate the robustness is stiffness method which is calculated using Eq. (1):

$$R_z = min \frac{\det K_j}{\det K_0} \quad (1)$$

where $R_z$: is robustness based on stiffness, $K_j$: is stiffness of damaged structure and $K_0$: is the stiffness of undamaged structure. In static pushover analysis, the roof of the structure will be pushed under lateral loads patterns. The pushdown curve actually indicates the capacity of a single degree of freedom structure. Hence, in this investigation it is assumed that whole the structure is single degree of freedom which can be resulted in the fact that the stiffness matrix in this research is equivalent to a single degree of freedom structure. In order to calculate the robustness index, the stiffness of the system is considered equivalent to a single degree of freedom structure in both damaged and not damaged structures.

Another method for assessment of robustness is damage method. This method is based on damage estimation in the structure due to initial failure. According to this approach, robustness is achieved in a dimensionless form based on damage using Eq. (2).

$$R_d = 1 - \frac{p}{p_{lim}} \quad (2)$$

where, $R_d$: is robustness based on damage method, $P$: is the maximum total damage due to localized failure, and $P_{lim}$: is the limit of accepted damage. It should be noted that $P$ and $P_{lim}$ refer to the damage in addition to the initial damage. Numerical stimulation of these two quantities can be done for mass, volume, surface or even financial loss. It is noteworthy that the robustness index derived from this method relates to targets of designing directly. Using damage method to estimate robustness can be too complicated or even unusable. Another assessment method of robustness index in structures is the energy method. According to this method, robustness index will be calculated based on the Eq. (3).

$$R_e = 1 - max \frac{E_{r,j}}{E_{f,k}} \quad (3)$$

where $R_e$: is robustness index based on energy, $E_{r,j}$: is the released energy during localized failure of the structural member j which damages the member k, and $E_{f,k}$: is the required energy for failure of the damaged element k (Smith, 2006). If loadings of intact and damaged structure are the same, robustness index can be defined using Eq. (4):

$$R = \frac{V_{(damaged)}}{V_{(intact)}} \tag{4}$$

in which $R$: is the robustness index, $V_{(damaged)}$: is the base shear in the damaged structure and $V_{(intact)}$: is the base shear in the intact structure (Straub and Faber, 2005).

In all of these methods, when robustness index is equal to 1, the localized failure will not have any effect on the structural strength and structure will be safe. When this parameter is zero, the structure will fail totally. Indexes which are calculated based on equation 3 will be used on the basis of released energy during the sudden removal of columns and will not be useful for progressive collapse under lateral loading. So in progressive collapse under lateral loading, Eqs. (1) and (4) must be used inevitably.

**Table 1.** Beam and column size in structures (4, 8 and 15-story buildings)

| | **4-story** | |
|---|---|---|
| **Story** | **Column** | **Beam** |
| 1-2 | Box 25x1 | W 12x30 |
| 3-4 | Box 20x1 | W 12x30 |
| | **8-story** | |
| **Story** | **Column** | **Beam** |
| 1-2 | Box 35x1.5 | W 12x55 |
| 3-4 | Box 35x1 | W 12x40 |
| 5-6 | Box 20x1 | W 12x40 |
| 7-8 | Box 20x1 | W 12x30 |
| | **15-story** | |
| **Story** | Column | Beam |
| 1-2 | Box 50x2 | W 12x96 |
| 3 | Box 45x2 | W 12x96 |
| 4-7 | Box 40x1.5 | W 12x96 |
| 8-9 | Box 35x1.5 | W 12x65 |
| 10-12 | Box 35x1 | W 12x65 |
| 12-13 | Box 25x1 | W 12x50 |
| 14-15 | Box 20x1 | W 12x30 |

## FINITE ELEMENT MODEL

Steel structures with moment frames system are used in the present study. Structures have a square plan with, 4-meter bays and the height of stories are 3.2 meters. Structures are designed in three types of 4, 8 and 15-story, on the basis of the fourth edition of Standard No. 2800 (BHRC, 2005). The soil type is III and the seismic hazard is so high. The dead load of floors is 500 kgf/m$^2$ and the Live load is 200 kgf/m$^2$.

For the structural design, steel with the yield strength of Fy=240 Mpa is used. Cross sections of structural elements are shown in Table 1. At first, structures are designed, using a linear analysis, in SAP2000 and subsequently, the nonlinear analysis is applied to them in Perform 3D. For the nonlinear modeling, concentrated plastic hinges are used. The models are shown in Figure 1.

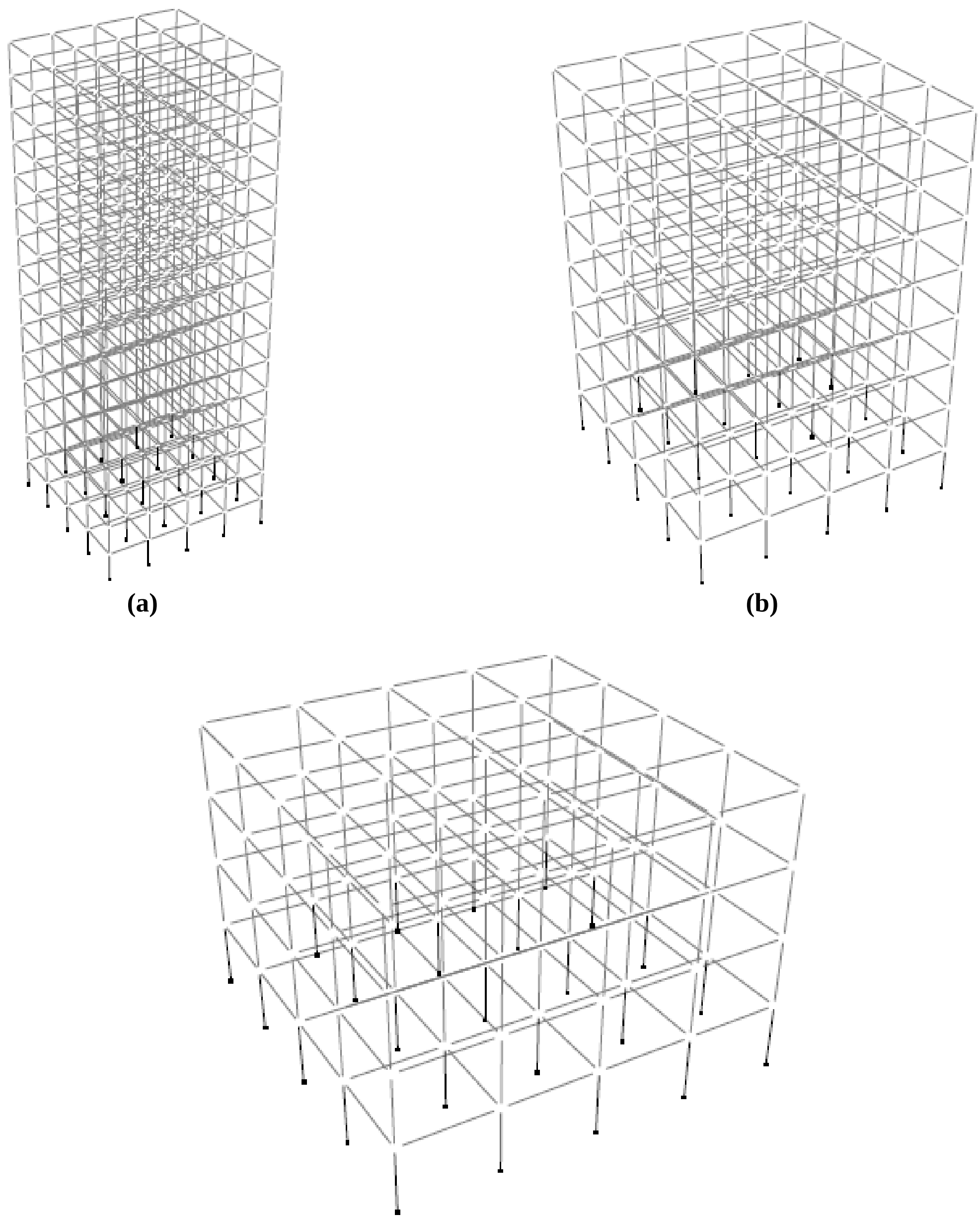
For analysis using the alternate path method, it is assumed that one of the corner columns of structure has been damaged at the beginning of lateral loading and it has been extremely weakened so that its presence or absence has no effect on lateral resistance of the structure. The mentioned column has been removed in accordance with the GSA regulations and the structure has been under lateral loading without this member. There are lots of choices in selecting the type and the number of elements in order to induce a local damage in structure. According to GSA, the corner columns are one of the critical progressive collapse scenarios in structures. In fact, the first story columns especially the corner column has the highest probability of omission considering external explosion or impulse of vehicles.

In Figure 2, the 4-story structure with localized failure has been shown. To apply lateral loading and calculation of robustness index, increasingly static and dynamic nonlinear analysis in structural models has been used. Nonlinear static analysis has been done under triangular, modal and uniform load patterns. In the increasing dynamic analysis, the structure has been subjected to two earthquakes; Elcentro and Northridge, with specifications of Table 2.

In this study it is assumed that the considered column is already omitted from the structure due to a happening and its effect on the earthquake has been considered. In fact, in this investigation the vibrations caused by omission of a column (which can be assessed independently in dynamic pushdown analysis) is ignored.

**Table 2.** Specifications of applied earthquakes

| Record/Component | Station | Magnitude | Distance (km) | PGV (cm/s) |
|---|---|---|---|---|
| El Centro | 5052 Plaster City | 6.5 | 31.70 | 5.4 |
| Northridge | 24576 Anaverde Valley-City R | 6.7 | 38.40 | 5.5 |



(a) (b)

(c)

**Fig. 1.** Structural modeling; a) 15-story; b) 8-story; c) 4-story

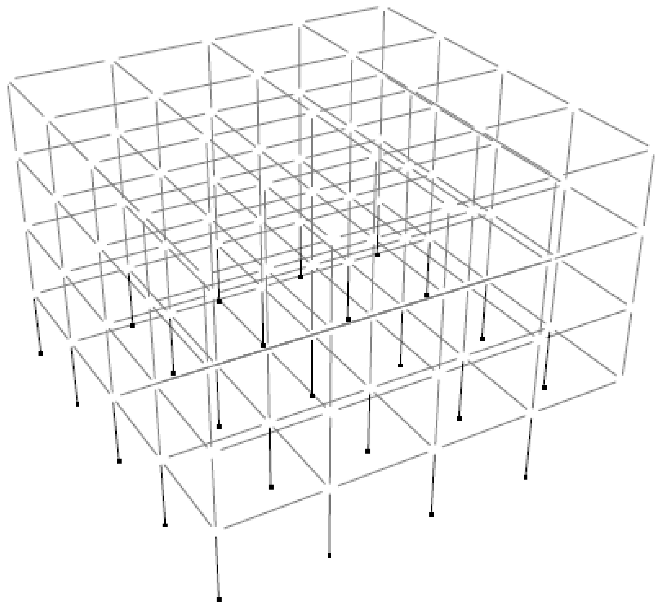


**Fig. 2.** The 4-story structure with localized damage

In selecting type and number of elements for specifying the direction of inducing progressive collapse in structures, there are many ways. According to GSA, the corner column is one of the critical scenarios for progressive collapse in structures. In fact, the columns of the first story have the possibility of omission regarding external explosion or vehicle strike, especially the corner columns.

## RESULTS

### Assessment of Stiffness-Based Robustness Measuring

In this study, the robustness of structures under static lateral load is investigated firstly. The structure has been pushed under different triangular, modal and uniform lateral load patterns. Then, the robustness of them has been calculated based on equation 1. Results are shown in Figure 3.

As it is clear in Figure 3, by adding the number of floors the robustness index in moment frame system increases. In fact, by increasing the number of floors, elements and also degrees of indeterminacy, redistribution of forces is done better and the structure can exhibit more resistance compared with shorter structures. Robustness index obtained based on the stiffness method only represents the ratio of stiffness in an intact structure to a damaged one.

Actually this index shows the reduction in structural stiffness, while robustness expresses the resistance of structure against localized failure. Robustness index achieved from this method has no considerable dependence upon the initial cause of the localized failure. Also, it cannot express the probability of failure in structures and its location or type.

### Assessment of Robustness Based on the Base Shear- Measure

After assessment of robustness index based on stiffness method and expressing its weaknesses, robustness index of structures is investigated based on the base shear method. After applying the localized failure, structures have been under Elcentro and Northridge earthquakes and the robustness index has been calculated in different PGAs based on Eq. (4).

The drift of 0.1 has been considered as the range of general structural failure in IDA analysis. That means when the maximum drift of floors reaches this value, the analysis will stop and total failure will happen in the structure. In perform-3d software it is needed to specify a limit state for stopping analysis. Because the objective of this research was not investigating the performance of a structure according to standards such as FEMA, the limit state for the maximum drift considered the maximum value of the software itself, 0.1, with which the analysis continues without stopping.

Robustness values calculated based on base shear method is shown in Figure 4. The results indicate that the robustness index using this method depends on earthquake applied load, so that in different earthquakes and PGAs various values have been achieved for the robustness index. According to Figure 2, the four-story damaged structures had drift ranges less than 0.1. This drift happened before PGA exceeds 0.9g in Elcentro earthquake or 0.8g in Northridge earthquake.

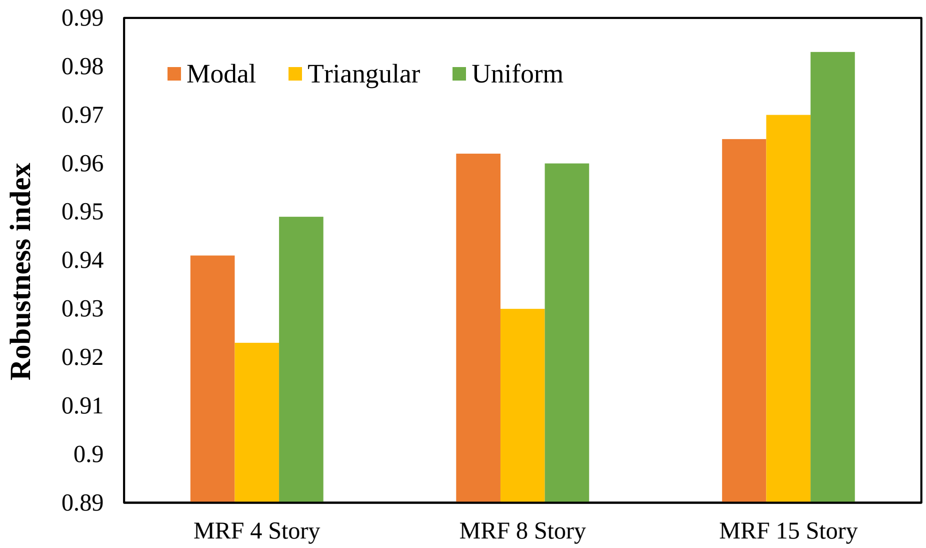
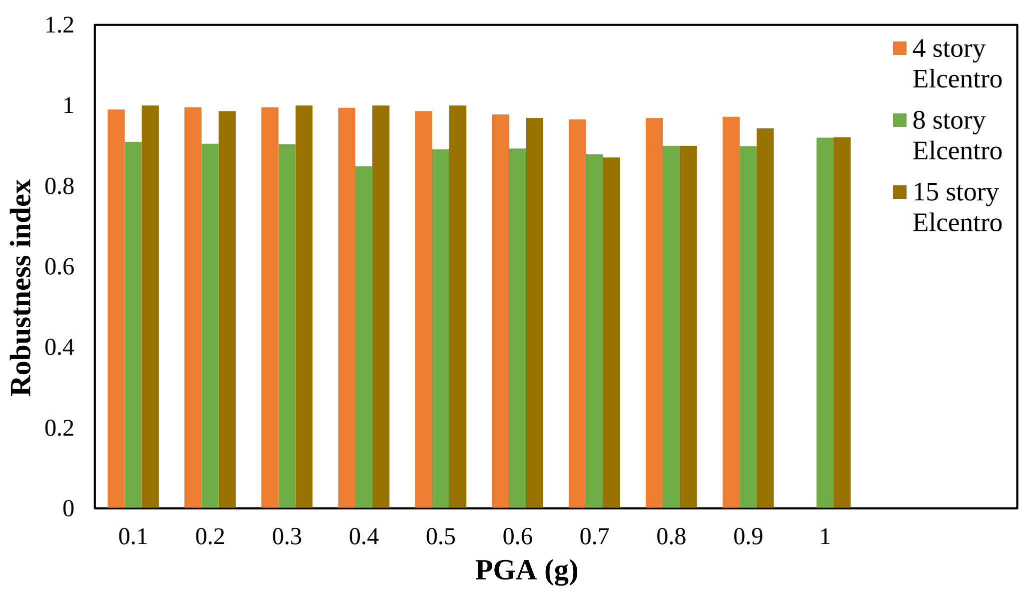
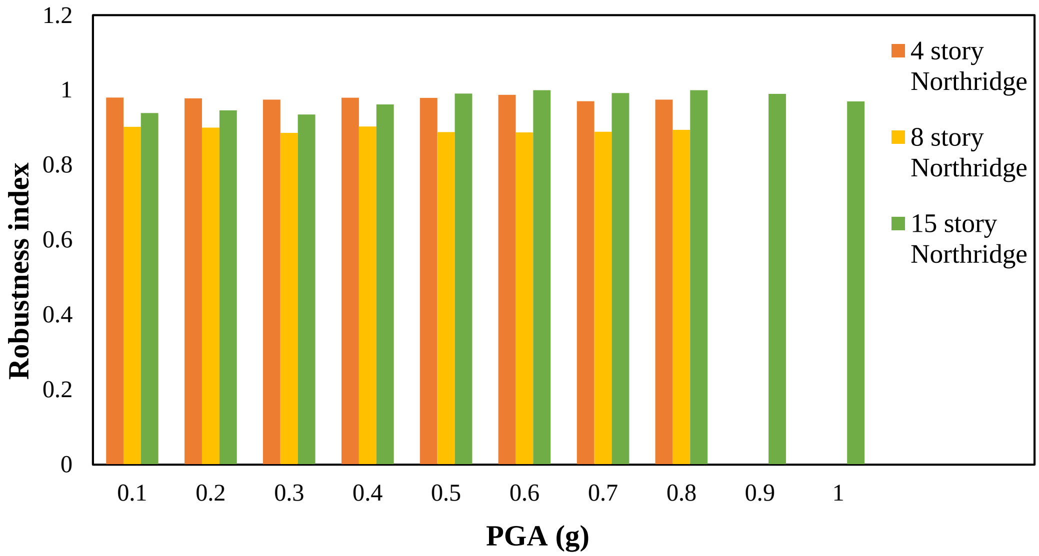



**Fig. 3.** Robustness index values based on the stiffness method for 4, 8 and 15-story buildings




**(a)**




**(b)**

**Fig. 4.** Robustness index values calculated for 4, 8 and 15-story structures, based on the base shear method: a) under Elcentro earthquake; b) under Northridge earthquake

The important point in evaluating robustness index based on base shear method is that by increasing the PGAs, robustness index values have not decreased considerably, although it is expected that the structural sensitivity to localized failure grows and structural strength reduces after seismic load increasing. Actually, this method only represents the ratio of the maximum base shear during seismic load applying. Base shear method is not that much reliable in Robustness calculation. The issue of question is one of its weaknesses which are mentioned in the text. It seems by increasing PGA, the base shear will increase in damaged and undamaged structures but these increasing are different and they do not obey specific rules. So, the values have significant differences.

## Introduction of Energy-Based Robustness Measure under Seismic Loading

As mentioned before, calculation of robustness index based on stiffness and base shear methods have disadvantages and ambiguities. If robustness index is defined as the insensitivity of structures to localized failure, a few questions will come up: On what basis the failure sensitivity should be considered? Is it reasonable that robustness index in different collapses (e.g. Pancake and Zipper collapses) are considered equally? Do robustness indexes based on base shear and stiffness show distribution of damage in structure? An index that does not show the distribution of possible damage in structures, how can be a measure of sensitivity to the localized failure in them?

Due to the weaknesses of the above methods, robustness index calculation has been investigated based on energy. According to the mentioned notes, it is essential that robustness index which is a function of input seismic loads, be a comparison between the intact and the damaged structure a criterion of the intended failure and its type. So the robustness index for seismic load based on the type of failure has been suggested as Eqs. (5) and (6). The basis of these equations is redistribution of plastic strain energy and the flow of it in intact and damaged structures. If the intended failure created in a structure due to removal of a damaged column occurs as a soft story, the robustness index will be defined based on the Eq. (5). In this type of failure, because seismic loading type and also direction of failure are lateral, this type of failure are called Pancake collapse.

$$R_p = 1 - \frac{E_i - E_d}{E_T} \quad (5)$$

where $E_i$: is plastic strain energy in the intended area of the intact structure, $E_d$: is plastic strain energy in the damaged structure and $E_T$: is plastic strain energy for causing the intended failure in the area of study.

Based on plastic strain energy, Eq. (5) can be used for an element or a particular group of elements under seismic load. The main challenge in this equation is calculation of $E_T$. While an acceptable criterion to determine the amount of dissipated energy during failure of elements still does not exist in this equation it is supposed that the mentioned area is pushed to reach the failure criterion of this study which is the drift of 0.1. The dissipated energy in this area, is considered as the plastic strain energy causing the failure. Since the amount of plastic strain energy causing a failure depends on the number of reciprocating cycles, this estimation will certainly be approximate. In fact, Eq. (5) represents the structural sensitivity to localized failure towards structural failure under a going round and dissipated energy under a cyclic loading for failure is ignored. To use this equation, it is enough that whole the energy is taken as the energy needed for demolishing two considered elements.

If the investigated failure of structure is as the collapse of upper region of the removed column, the robustness index will be considered based on the Eq. (6). In this type of failure, because the seismic load has been applied laterally and the failure occurs vertically, it is called Zipper collapse.

$$R_z = 1 - \frac{E_d}{E_T} \quad (6)$$

where $R_z$: is the robustness index for Zipper collapse. $E_d$: is dissipated plastic strain energy in intended area for the damaged structure under seismic loads and $E_T$: is dissipated plastic strain energy for causing the considered failure. In this equation, the damaged area for calculation of $E_T$ has been pushed down in order to have all beams of this area in the performance level of CP (Collapse Prevention). Calculated plastic strain energy is considered as $E_T$ (Tavakoli and Moradi, 2018). The energy needed for an element to be destroyed depends on various parameters. There is not a unique relationship yet, so, in this research for specifying the energy need for destruction of elements the number of cycles is neglected. It is natural that by increasing the number of elements this value will increase but it is not clear in what number of cycles it reaches the considered performance level for energy calculation.

As a result of removing columns, their upper beams have downward rotation and during seismic loading, the damaged area has no remarkable upward movement and often moves downward. So, the dissipated energy for the failure of the damaged area, with a proper approximation, can be considered equal to the dissipated energy for failure under pushdown. In this research column bulking is ignored and the lost energy in lateral performance of them is considered. Dissipated plastic strain energy is considered only for one cycle and it is assumed that the structure experiences local damage under one cycle. If the energy loss of cyclic behavior is considered this value increases.

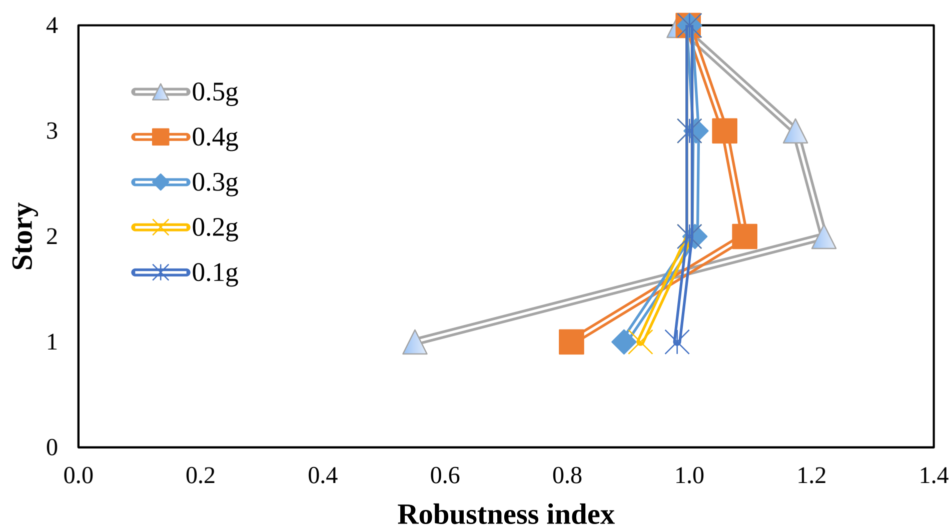
According to Szyniszewski investigation, when the plastic strain energy in structures increases suddenly, the progressive collapse happens in the structure. In this research with having the assumption of plastic rotations criteria for elements, it is assumed that when elements rotations reach CP performance level, elements are destroyed and their energy is considered as $E_T$.

**ASSESSMENT OF ENERGY-BASED ROBUSTNESS MEASUREING**

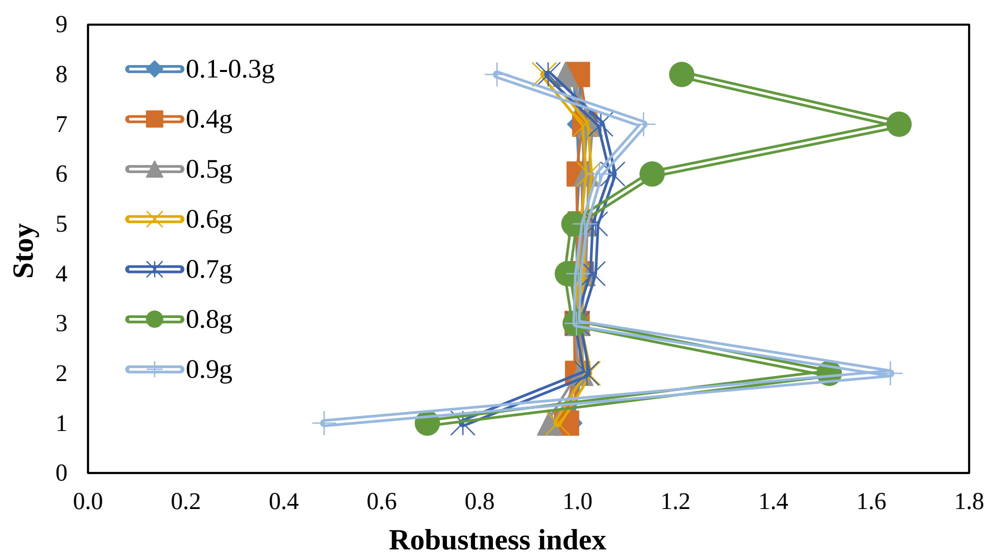
In order to assess robustness index by energy method, intended frames have been analyzed using mentioned assumptions. Robustness index is studied for two failure modes. In the first case, it is assumed that the total failure of the structure is in the form of soft story and robustness is calculated based on Eq. (5). In the second case, it is assumed that the failure has happened because of the collapse in upper part of the damaged area which has occurred due to the failure of upper floors beams (Zipper collapse). In this case, the robustness index is calculated by Eq. (6). In all models it is assumed that analysis continues when the drift is less than 0.1.

**Robustness Index Assessment in Pancake Collapse**

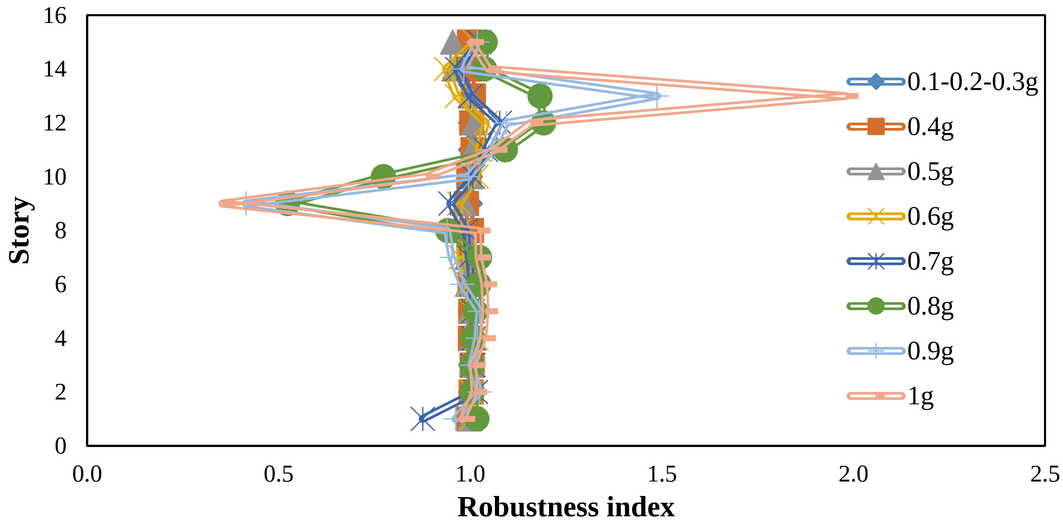
In Figure 5, robustness index values are shown under the total failure which has happened because of soft story in Northridge earthquake.




**(a)** 4-story building




**(b)** 8-story building




**(c)** 15- story building

**Fig. 5.** Robustness index under the failure because of soft story in Northridge earthquake

In Figure 5, robustness index for each floor have been showed in different maximum accelerations. Based on Figure 5a, increasing the maximum accelerations reduces robustness of the first floor. In other words, when the localized failure happens, the potential of soft story type failure at the first story is more than other stories. Robustness reduction in the first floor indicates that removing the column increases plastic strain energy in this floor and so, this floor is more sensitive to localized damage.

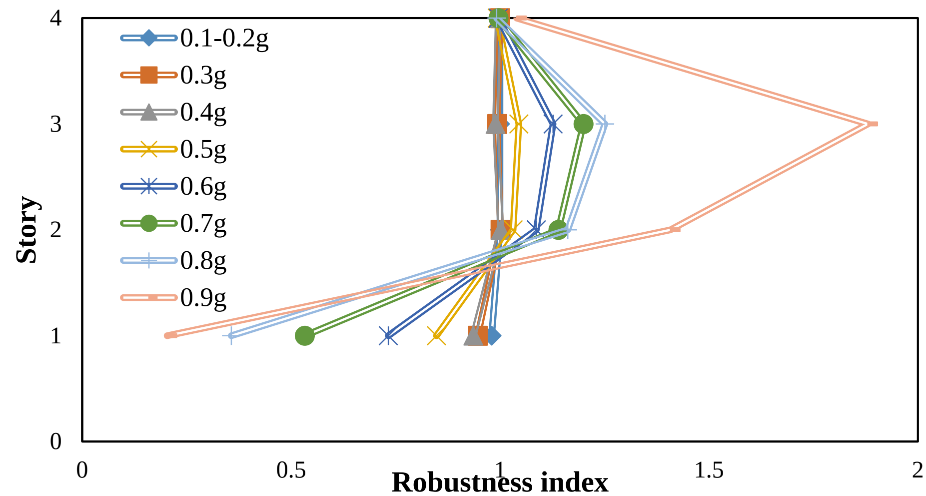
In maximum accelerations of 0.4g and 0.5g, robustness increases in the second and the third floors. This increasing shows that the localized failure created on the first floor causes the failure in second and third floors of the damaged structure and as a result, these floors dissipate less energy than an intact structure and will be more robust. Robustness index in the fourth floor in almost all cases is approximately equal to 1 that shows localized failure on the first floor has no effect on the robustness of the fourth floor.

Robustness indexes for 8 and 15-story buildings are shown in Figures 5b and 5c, respectively. Figure 5c shows that the most sensitivity of the structure happens due to the localized failure on the first floor in maximum acceleration of 0.7g. In higher maximum accelerations, robustness increases on the first floor showing that the amount of dissipated strain energy in intact and damaged structures is that much high and close to each other that removing a column does not have considerable effect on the robustness index.

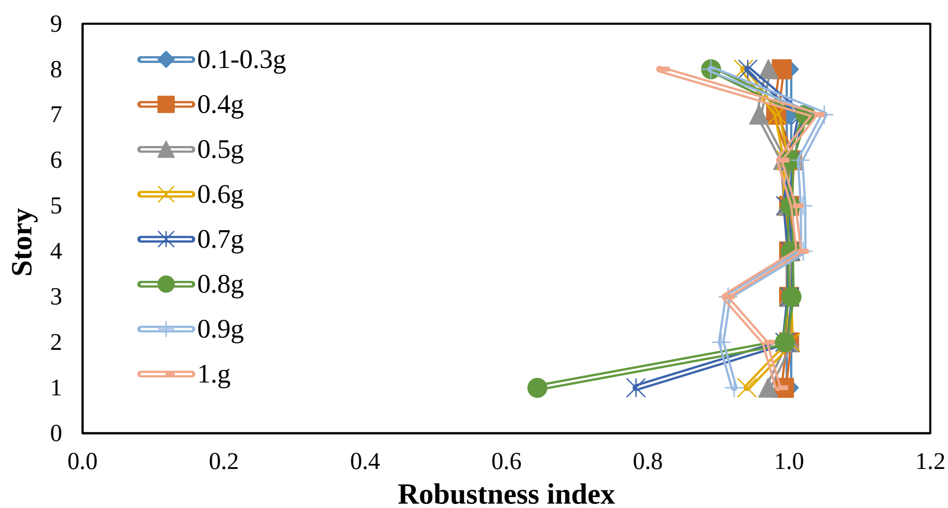
In the maximum accelerations of 0.1g, 0.2g and 0.3g, the localized failure does not have remarkable influence on the robustness of any floor. Also Figure 5c shows that the least robustness in the structure because of localized failure occurs in floor 9 in the maximum accelerations of 0.9g, 0.8g and 1g. In fact, removing a column from the first floor causes the potential of progressive collapse because of soft story to appear in the mentioned maximum accelerations in the ninth floor. Based on the Figure 5 it is easy to guess the plastic strain energy flow of an intact structure to a damaged one and also the type of redistribution of forces in the intended failure. In Figure 6, robustness index of structures under Elcentro earthquake are shown.

To present the total robustness of a structure under pancake collapse, the robustness index of floors can be used based on a statistical method. In this study, weighted average method is used for this purpose. In weighted average method, it is assumed that robustness indexes smaller than 1, weigh two, indexes equal to 1 weigh one and indexes greater than 1 weigh 0.25. It is also assumed that the importance of floor with the localized failure is twice as much as the other floors. Thus, the weight of robustness index at the first floor is considered twice as much as other floors. According to the expressed assumptions, the robustness index of the entire structure in pancake failure is considered as Eq. (7). In this equation, $R_{pi}$ is the robustness index for each floor, $\omega_i$ is the weight of each robustness index and $n$ is the number of all floors. Figure 5 shows the robustness index curve of the entire structure for pancake collapse. In this figure, a part of the curve ascends that represents two concepts; a) whether energy dissipation in an intact structure is so high that a column removal has no remarkable effect on increasing the strain energy in the damaged structure, b) or the pancake collapse has occurred in a floor except the first floor in which the localized failure has happened.

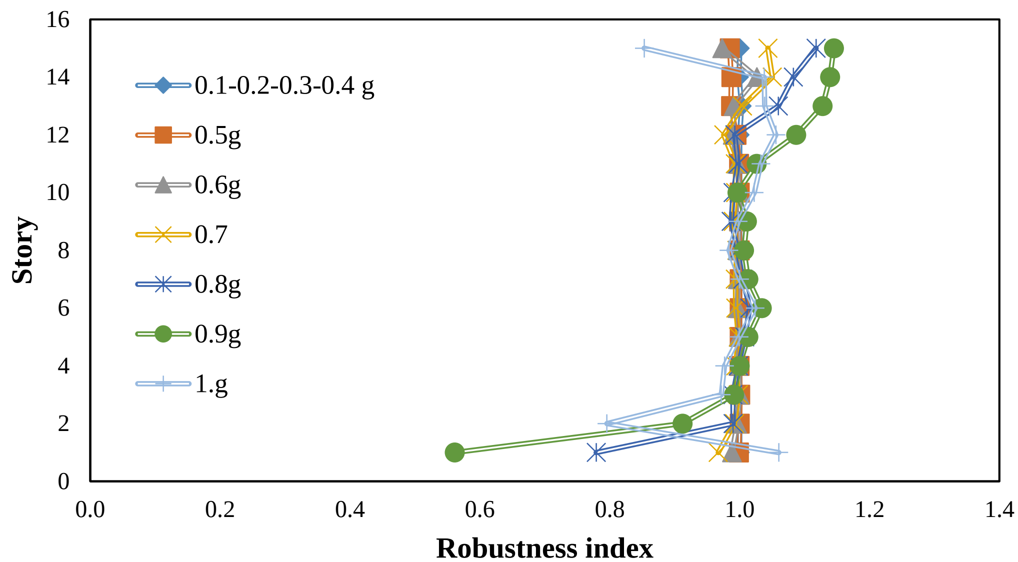
$$R_{pT} = \frac{\sum_{i=1}^{n} w_i R_{pi}}{\sum_{i=1}^{n} w_i} \quad (7)$$




**(a)** 4-story building




**(b)** 8-story building




**(c)** 15-story building

**Fig. 6.** Robustness index under the failure with a soft story in Elcentro earthquake

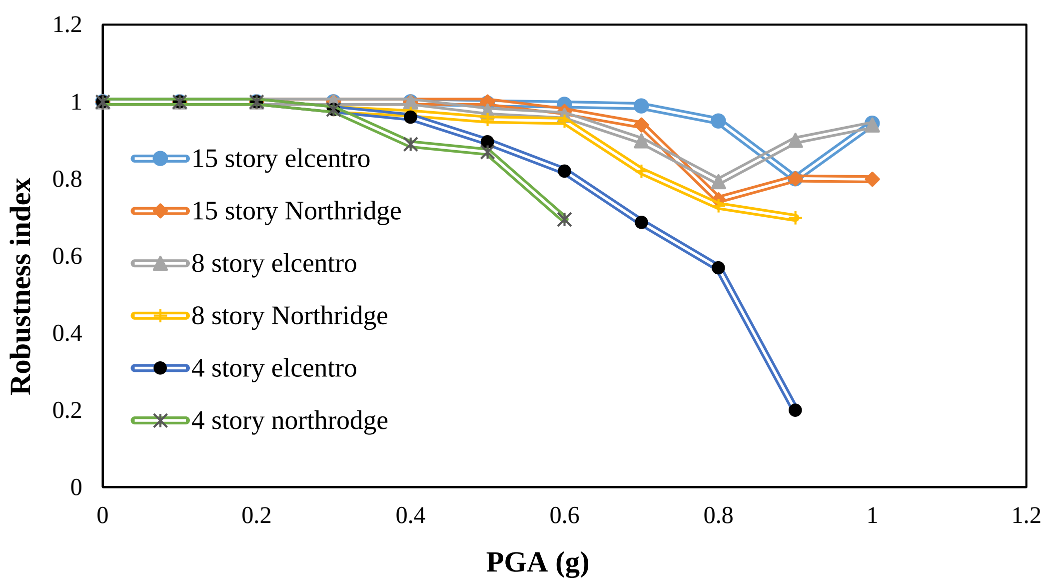



**Fig. 7.** Robustness index of the entire structure in pancake collapse

**Robustness Index Assessment in the Zipper Collapse**

In Figures 6 and 7, the robustness index are shown for zipper collapse under two Elcentro and Northridge earthquakes. The curves in Figures 6 and 7 show sensitivity of the structure to the type of zipper collapse. These curves indicate ratios of the amount of dissipated energy during earthquake to the required energy for total collapse of structure. According to Figures 6 and 7, the15-story structure has shown more robustness than other buildings under lateral failure. In fact, the potential of the zipper collapse in higher structures under seismic loading are more than shorter ones. It means that the sensitivity of a structure to zipper collapse increases by adding the floors; because the number of existing elements in the process of redistribution of forces increase.

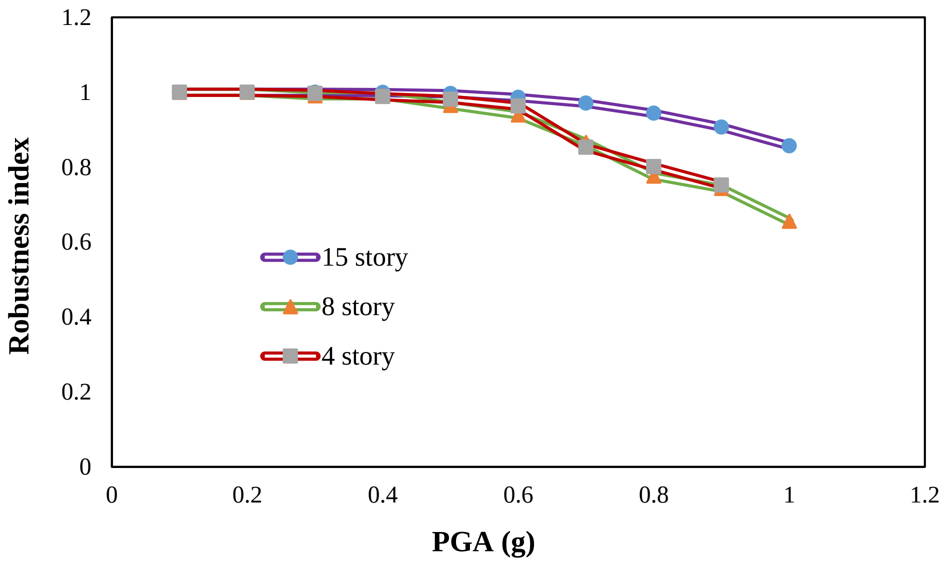



**Fig. 8.** Robustness index for the zipper collapse under Elcentro earthquake

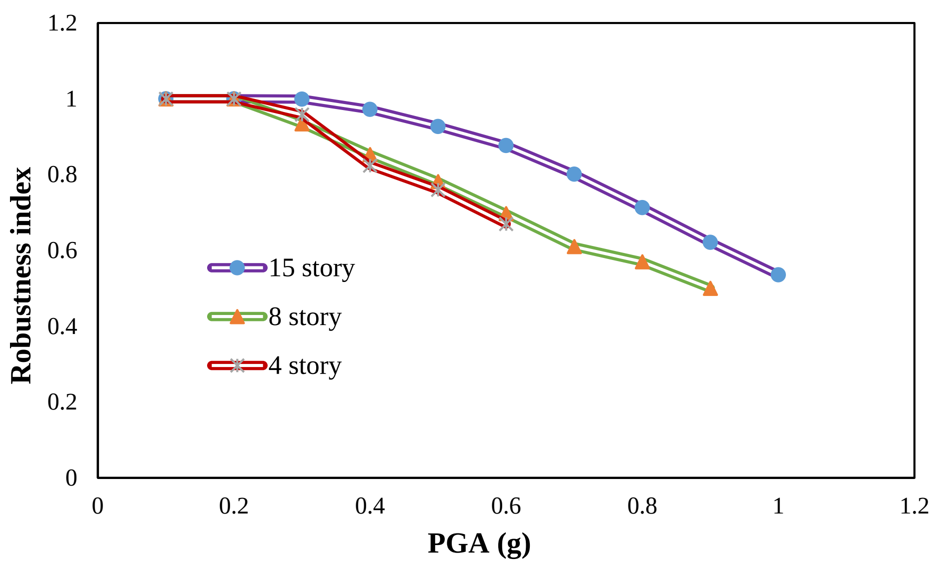



**Fig. 9.** Robustness index for the zipper collapse under Northridge earthquake

**Robustness Index Assessment of the Entire Structure for Zipper and Pancake Collapse**

In the previous sections, using the distribution of strain energy flow, some indexes have been presented for robustness according to the type of failure in the intended structures. In this section, the robustness index of the entire structure has presented by combining two indexes of $R_{pT}$ and $R_z$ for zipper and pancake collapses, respectively. The robustness index of the entire structure is presented by use of the weighted average method, assuming equivalent weights of robustness index for both pancake and zipper collapse based on Eq. (8). In this equation, $R_T$ : is the robustness index for the whole structure. Figure 10 shows the robustness index of the entire structure under pancake and zipper collapse.

$$R_T = \frac{R_{pT} + R_z}{2} \quad (8)$$

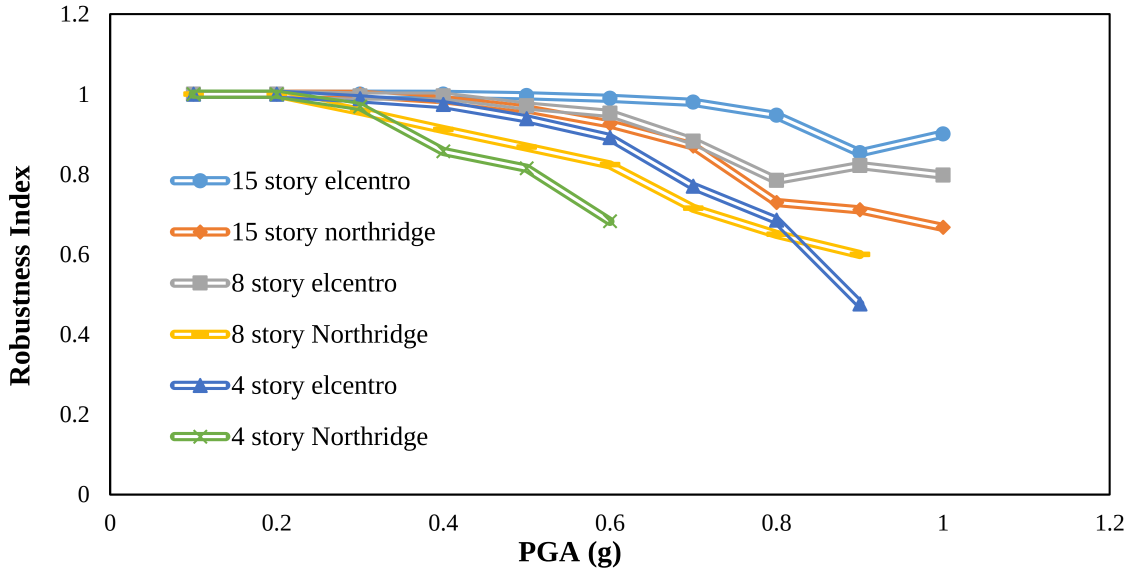



**Fig. 10.** The robustness index of the entire structure for Elcentro and Northridge earthquakes

## CONCLUSIONS

In the present study, the progressive collapse scenario has been investigated with attitude towards assessing the robustness index for steel structures with moment frame systems. For this purpose, the concepts of progressive collapse and evaluating methods for robustness index have been expressed initially. To investigate the robustness index, steel structures with moment frame systems have been used. Structures have been designed and modeled in three types of 4, 8 and 15-story buildings. Then, they have been analyzed under nonlinear static and increasing dynamic analysis, using Alternate Load Path method. The results have shown that the stiffness and base shear methods for assessing the robustness index have many weaknesses and basically, they represent just the amount of reduction in stiffness and base shear. Subsequently, the robustness index was investigated by energy method. To this end, equations presented for two types of pancake and zipper collapse. Finally, after combining those, the overall robustness index of structures in various earthquakes were examined. The results have shown that the energy method for evaluating the robustness index does not have the weaknesses of stiffness and base shear methods and it is also indicative of changes in strain energy flow, redistribution of forces and the place of vulnerability in structures.

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