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Effect of Structural Height on the Location of Key Element in Progressive Collapse of RC Structures

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ABSTRACT: 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. As the beginning of progressive collapse in a structure is more sensitive to the missing of an element, the location of that element is more important to be found. The most sensitive element is called the key element. In this paper, sensitivity analysis is modified following GSA and DoD guidelines and used for finding the key element of symmetric structures with different heights. Four structures with different heights have been analyzed for every column missing event and the load carrying conditions of the structures have been monitored. The results showed that the location of the key element in the plan and height of the structure is different in structures with different heights.

Keywords: Key Element, Modified Sensitivity Analysis, Progressive Collapse, Reinforced Concrete Structures, Tall Buildings.

INTRODUCTION

"It is estimated that at least 15 to 20% of the total number of building failures are due to progressive collapse" (Leyendecker and Burnett, 1976). Progressive collapse is a failure sequence that relates local damage to large scale collapse in a structure. As ASCE 7-10 defines "the spread of an initial local failure from element to element, resulting eventually in the collapse of an entire structure or a disproportionately large part of it". A notable example of such a failure is the Ronan Point building collapse (Griffiths et al., 1968). Local failure in a structure is defined as missing of the load carrying capacity of one or more structural components that are a part of a whole structural system, e.g. failure of one column in a structure (Hadianfard et al., 2012). After some structural components fail the structure should prepare an alternative load-carrying path. After the load is redistributed, each structural component will support different loads. In the new load-carrying path, if redistributed load exceeds the load carrying capacity of any member, it will cause another local failure. Such sequential failures can propagate through a structure. If a structure loses too many members, it may suffer partial or total collapse.

Over recent years, many researches have

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been done in the field of progressive collapse. Subjects of these researches can be categorized as follows: the phenomenon of progressive collapse (Tavakoli and Kiakojouri, 2015; Al-Salloum et al., 2016), the way of modeling the progressive collapse (Kheyroddin and Mehrabi, 2012; Izzuddin et al., 2008; Vlassis et al., 2008; Krauthammer and Cipolla, 2007), loading (Leyendecker et al., 1975; Ettouney et al., 2006), behavior of structures subjected to progressive collapse (Fu, 2009, 2010; Li et al., 2017; Tavakoli and Rashidi Alashti, 2013; Mashhadiali et al., 2016; Zahrai and Ezoddin, 2014), design of structure to control this phenomenon (Kheyroddin et al., 2012; Mashhadiali and Kheyroddin, 2013; Tavakoli and Kiakojouri, 2014; McConnella and Brown, 2011), and finding the most important structural element in progressive collapse (Khandelwal et al., 2009; Kim et al., 2011; Choi and Chang, 2009; Frangopol and Curley, 1987; Wada et al., 1989; Takumi and Toshinobu, 2014).

Al-Salloum et al. (2017) studied the progressive collapse of a high rise RC structure subjected to blast loading. Izzuddin et al. (2008) and Vlassis et al. (2008) proposed a simplified framework for progressive collapse assessment of multi-story buildings, considering sudden column loss as a design scenario. It offers a method for assessing the structural robustness at various levels of structural idealization, and it quantifies the factors influencing robustness away from the generalities.

Using 3-D and 2-D push-over analysis, Tavakoli and Rashidi Alashti (2013) studied the progressive collapse resistance of MRF steel structures that have been designed based on seismic codes. Kheyroddin et al. (2012) proposed a new and simple 5-step method to calculate the dynamic load amplification factor due to sudden column loss within a progressive collapse event in a structure. Using 3D analyses, Amiri et al. (2017) proposed a new formula to determine dynamic increase factor for progressive collapse analysis of RC structure, so the stress and deformation in the RC structures' members after column removal may be predicted.

Khandelwal et al. (2009) studied the progressive collapse resistance of seismically designed steel braced frames with 2-D models. He considered two types of bracing systems; special concentric and eccentric bracings. The simulation results showed that the eccentrically braced frame is less vulnerable to progressive collapse than the special concentrically braced frame.

Kim et al. (2011) studied the sensitivity of design parameters of steel buildings subjected to progressive collapse. Their results showed that yield strength is the most important design parameter in the moment resisting frame buildings while the column yield strength is the most important design parameter in the dual system building.

To design or rehabilitate a structure against progressive collapse, the most important part is to find the key element. Based on column loss scenario, some codes and guidelines have suggested that columns are the most important element of a structure in collapse progression (e.g. GSA, 2003; DoD, 2005). The main question is that omitting of which column can make the collapse begin and lead to progressive collapse. To find the answer to this question a sensitivity analysis must be done.

Modeling collapse progression, and design structures against it, are the main part of some building codes like GSA (2003), UFC 4-023-03 (DoD, 2005) and (DoD, 2009). UFC 4-023-03 (DoD, 2009) has been included changes for two times, in 2010 and 2013 (DoD, 2010, 2013).

UFC 4-023-03 (DoD, 2009), uses the Alternate Path Method (APM) for analysis and design of a structure subjected to progressive collapse. Three analysis procedures are employed in this method:

1. Linear Static procedure (LSP)
2. Nonlinear Static procedure (NSP)
3. Nonlinear Dynamic procedure (NDP)

It is suggested to perform APM analyses for column removal at:

1. First story above grade
2. Story directly below the roof
3. Story at mid-height
4. The story above the location of a column splice or change in column size

In addition to the elevation of the removed column, its location in the plan of the structure is another problem. DoD advises removing external columns in the middle of the short side, in the middle of the long side, and in the corner of the building. Engineering judgment should be used to recognize these critical column locations (DoD, 2009). For a complete risk analysis in a structure against any danger, three analysis are needed: threat analysis, impact analysis and vulnerability analysis (Krauthammer, 2008). Threat analysis shows the kind, magnitude, and place of the danger threatening the structure. Impact analysis shows effect, cost, and importance of the collapse. Vulnerability analysis declares the magnitude of the collapse caused by the failure in any kind (e.g. failure of any column in progressive collapse). As it is clear, the location of the removal column (suggested in codes) is based on the threat analysis and not based on the behavior of the structure itself (vulnerability analysis).

Some researches have been done on finding the most important element in the structure (Khandelwal et al., 2009; Kim et al., 2011; Choi and Chang, 2009; Frangopol and Curley, 1987; Wada et al., 1989; Takumi and Toshinobu, 2014). In each one, a procedure has been used, but no procedure in which all steps is clear, reliable and based on an accepted code has been presented. Also in most researches only missing of some of the columns have been checked. In this paper, at first, a procedure has been modified according to the reliable codes and has been used for all columns of a structure.

INTRODUCTION TO MODIFIED SENSITIVITY ANALYSIS (MSA)

Sensitivity analysis is the study of how the uncertainty in the output of a model or system can be apportioned to different sources of uncertainty in its inputs (Saltelli et al., 2008). Frangopol et al. (1987) and Takumi and Toshinobu (2014) studied how much the resistance of a structure would remain after structural components were destroyed by accidental action, and compared it with the resistance at the original state of the structure. It is regarded as the sensitivity index to the member's disappearance, denoted as S.I.

$$\text{Sensitivity Index: } S.I. = (\lambda_0 - \lambda_{\text{damage}}) / \lambda_0 \qquad (1)$$

where λ_0: represents the load carrying capacity of the structure in its original state and λ_{damage}: represents the load carrying capacity of the structure in its damaged state (one column omitted) (Frangopol et al., 1987).

If one or a set of structural members disappears suddenly, the building should remain standing against vertical gravity loads, dead and live, and should not completely collapse. Therefore, it must be examined how to attain such performance. For this purpose sensitivity analysis was modified in such way to be able to find the key element of the structure; in the normal push-down analysis, one point of a structure is pushed down step by step, until the element collapses (i.e. some plastic hinges are made in the element). In every step, the force is recorded and at last, a curve is presented.

In this study, using modified sensitivity analysis (MSA), the ratio of vertical load carrying capacity of the structural system before and after the disappearance of a certain member has been evaluated. At first, the whole structure is loaded in the gravity direction. This loading is increased in steps until the structure collapses (i.e. plastic hinges are made in some elements and the collapse criteria of the whole structure appear). Ultimate load carried by the structure is called the ultimate load.

Then for intact structure, one specific column will be omitted and the structure will be loaded using DoD (2009) load pattern (i.e. twice the normal load, on the panels related to the removed column which are marked using grey color, and normal loading on the other panels which are marked without any color and load name G, Figure 1).

The whole structure and plastic hinges are monitored in all steps. Two kinds of collapse are considered: local collapse and global collapse. Local collapse is defined as the collapse of panels related to the removed column and global collapse is defined as the collapse of the structure according to GSA 2003 (Figure 2). In Figure 2, exterior and interior considerations refer to the consideration of failure caused by removal of an exterior and interior column, respectively. Load of the step in which local collapse occurs is called local damage load and a load of the step in which global collapse occurs is called global damage load. The local collapse is important when some sensitive instruments or facilities are used in these areas.

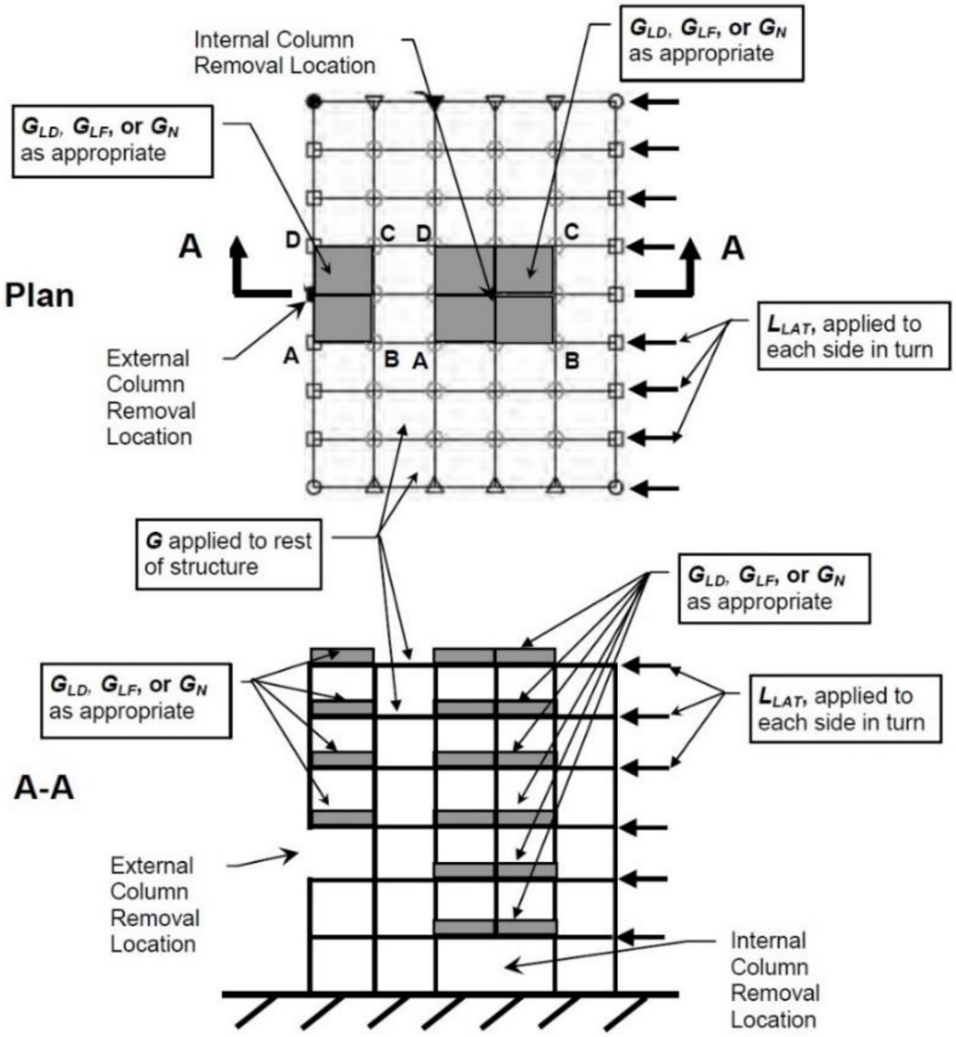
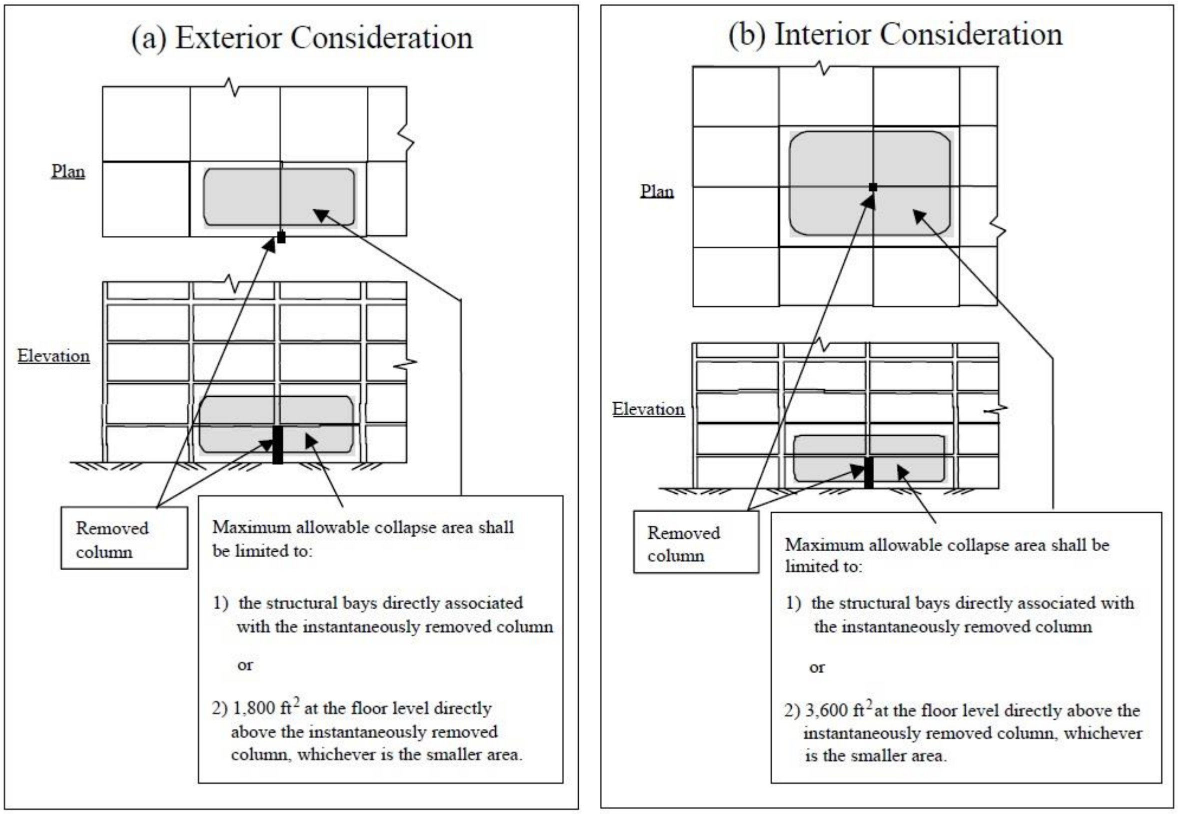


Fig. 1. Loads and load locations for column removal based on DoD (2009)



Maximum allowable collapse area shall be limited to:

1) the structural bays directly associated with the instantaneously removed column

or

2) 1,800 ft^2 at the floor level directly above the instantaneously removed column, whichever is the smaller area.

Maximum allowable collapse area shall be limited to:

1) the structural bays directly associated with the instantaneously removed column

or

2) 3,600 ft^2 at the floor level directly above the instantaneously removed column, whichever is the smaller area.

Fig. 2. GSA allowable extent of collapse from column removal

For both local and global damage conditions, the sensitivity index is calculated using Eq. (1). The results lead us to find the key element whose sensitivity index is greater than others.

For defining the capacity of elements, reinforced concrete beams and joint requirements of DoD (2009) should be used: "for new and existing construction, the design strength and rotational capacities of the beams and beam-to-column-to-beam joints shall be determined with the guidance found in ASCE 41 (2014), as modified with the acceptance criteria provided in DoD (2009)".

For modification of the analyses, both GSA and DoD codes have been used. For loading and acceptance criteria DoD is more updated due to satisfy modern structural findings and codes. But in some cases such as criteria of collapse for the whole building, DoD has deficiencies compared to GSA. Hence for a complete evaluation, the procedure must be defined using the advantages of both codes. The advantage of the MSA, presented here is that all steps and criteria of the MSA are clearly based on reliable codes and completely applicable using commercial programs like SAP2000. Because of the 3D nonlinear pushdown analysis, used in MSA, results are highly reliable.

Hence modified sensitivity analysis (MSA) contains some steps as follows:

1- A uniform load should be applied to all floors and increased step by step until the structure collapsed. This load is called the ultimate load (intact load).

2- One of the columns should be removed from the intact structure, and a uniform load to be applied to the whole area of the structure regarding an accepted load pattern such as the DoD.

3- Uniform load of the second step should be increased until disproportionate collapse occurs, around the removed column.

4- The sensitivity index for the removed column to be calculated. Perform steps 2, 3 and 4 for all other columns.

5- In the final step, a comparison between sensitivity indexes of all columns must be performed. The result clarifies the elements which the structure is more sensitive for their loss.

As DoD (2009) uses computer program SAP2000 for modeling progressive collapse in its appendixes, in this study SAP2000 was used to develop four 3D Finite Element models of the structures. Beam elements were modeled as L and T sections and attached to the shell element of the slabs, using 10 nodes in any side of the slabs for connection of beam and shell (slab) elements. Considering the effects of the slab, L and T sections have been used according to the seismic design codes ASCE 41-06 (Santafe et al., 2011). For this purpose, three times the slab thickness was taken as the effective flange width of the beam, on each side.

According to experimental studies (Choi and Kim, 2011; Sadek et al., 2011), generally the flexural failure mode of beams govern the collapse of RC framed structures. As shown in Figure 3, for application of the nonlinear analyses, the plastic hinge model was assigned to the both ends of beams.

These hinges were placed at locations of high stress as recommended by DoD (2009). Beam elements included plastic hinges at the midspan and ends of the members whereas hinges for the columns were added only at the ends.

The properties for the hinges were defined using the built-in hinge assignments for SAP2000. SAP2000 that uses the Federal Emergency Management Agency (FEMA) designations for these hinges, specifically Table 5-6 of FEMA 356 for structural steel hinge properties. ASCE 41-13 and the DoD (2009) recognize these as the standard properties for plastic hinges and reference this table for their own hinge definition procedures.

In Figure 3, the maximum allowable rotation in plastic hinges (point C on the *M-θp* curve), which corresponds to the “Collapse Prevention” performance level has been increased from 0.02 rad. to 0.035 rad., according to the GSA (2003) recommendations for RC frames. The slope from point B to C has been taken as 10% of the elastic slope for strain hardening, in accordance with the seismic code ASCE 41-013 (2014), which indicates that the slope should be taken as a small percentage between 0% and 10%. Point D represents the residual strength ratio of 0.2. A value of 0.07 rad. Has been taken for point E as the failure limit, which has been considered as an average value (0.04 rad., ..., 0.10 rad.) given by the DoD (2009).

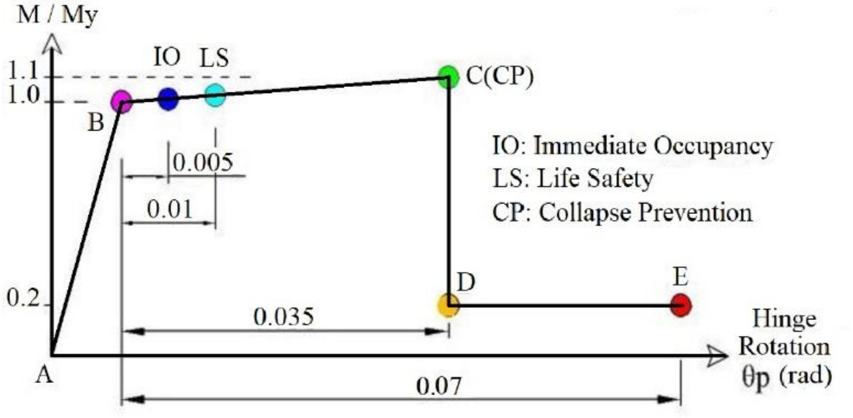


Fig. 3. Plastic hinge model assigned to beams ends

MODELING AND ANALYSIS

In this paper, in order to find the location of the key elements in the structure, four reinforced concrete structures with different heights were designed and analyzed using the program SAP2000. All the structures have a square plan of 5×5 bays and have been named as S10, S20, S30, and S40 which have 10, 20, 30 and 40 stories respectively (Figure 4).

The structures have been designed according to Iranian national building codes (BHRC, 2013a, b, c). The structures are supposed to be as commercial buildings, with RC special moment resisting frames, the system of floors is 15 cm thick RC slab and located in Tehran, on a soil of type II (with shear wave velocity $350 < v_s < 750$ m/s). Material properties in the structures are for concrete, f_c = 250 kg/cm^2 and for reinforcement f_y = 4000 kg/cm^2.

The aspect ratio in the plan (ARp) is the ratio of the length of the structure to its width, and aspect ratio in height (ARh) is the ratio of the height of the structure to one of its plan dimensions in height. In all structures, each bay is 5 meters and each floor has 3.5 meters in height. The aspect ratio in the plan (ARp) and the aspect ratio in height (ARh) for all the structures are as follows:

The intact structural models are shown in Figure 4. In this figure, the prototype models (the model with no column omitted) are shown.

Table 1. Structural aspect ratios

Name of the structure	Length of the structure (m)	The width of the structure (m)	The height of the structure (m)	ARp	ARh
S10	25	25	35	1.0	1.4
S20	25	25	70	1.0	2.8
S30	25	25	105	1.0	4.2
S40	25	25	140	1.0	5.6

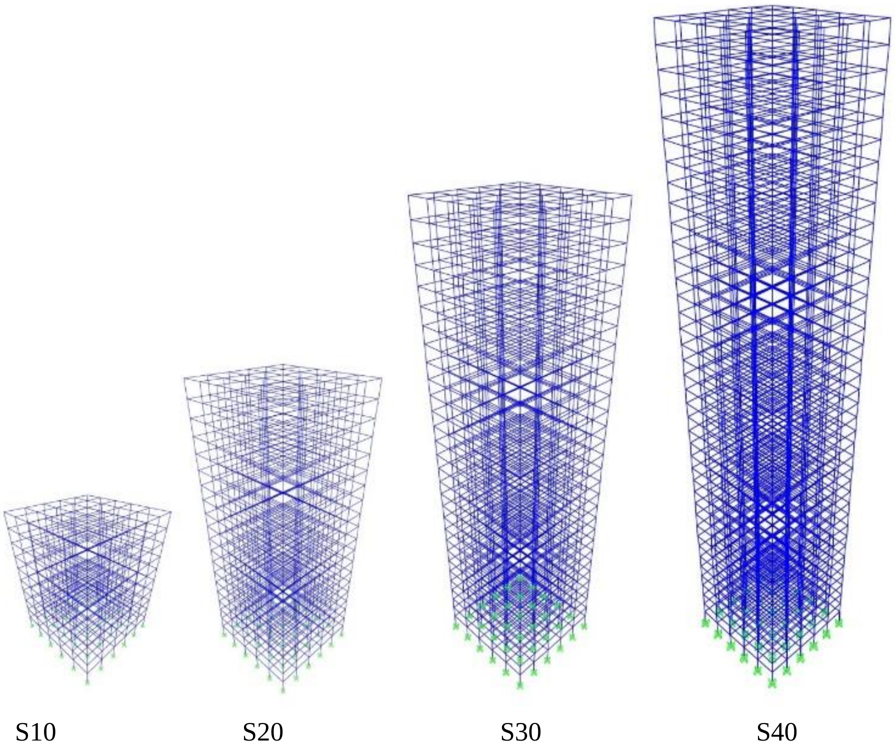


Fig. 4. Four different structures modeled and analyzed

Naming the axis of columns in structures is shown in Figure 5. For example column B2 refers to the column which is common in the axis B and axis 2 in X and Y directions, respectively. When a column is omitted, the structure is named as for example S30-A2@28 that refers to the 30-story structure in which the column, which is common in axis A and 2 is omitted at the story 28. For more information, see Figure 5.

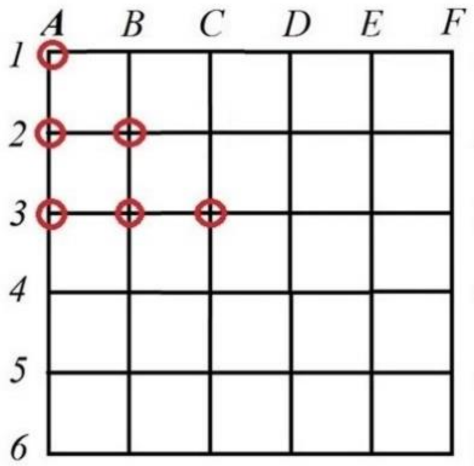


Fig. 5. Plan of the structures and naming of the axis in X and Y directions

In this paper for studying effects of all columns, 3600 3D nonlinear pushdown analysis were needed, but because of the eccentricity of structures 904 3D nonlinear pushdown analysis were done. Number of analysis needed for each structure is presented in Table 2.

Table 2. Number of analysis needed for each structure

	S10	S20	S30	S40
Number of needed analysis	360	720	1080	1440
Number of done analysis	90	180	270	360

Figure 6 shows a schematic view of the local collapse of the structure; red signed beams show the beams that if they collapse, cause local collapse. The yellow circle shows the panels that if collapse, cause local collapse.

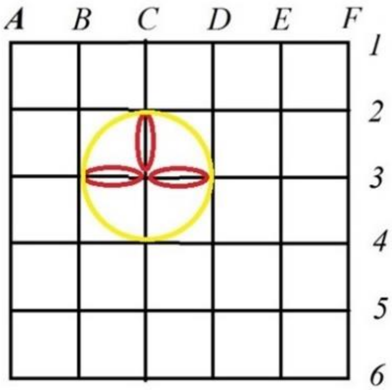


Fig. 6. The schematic view of the local collapse

In the first step, a uniform load is applied on all floors and increased step by step until the structure collapsed. The criteria of collapse were the same as the criteria of GSA 2003. The ultimate load for all the intact structures are presented in Table 3.

Table 3. The ultimate load of the structures (uniform load)

	S10	S20	S30	S40
The ultimate uniform load (kg/m^2)	1960	1990	2105	2165

All five steps of modified sensitivity analysis (MSA) were done step by step for all structures and all their columns.

RESULT DISCUSSION

Doing 904 3D nonlinear pushdown analyses to find the sensitivity index of the elements, the pushdown curve can be drawn. Figure 7 shows the pushdown curve for omitting columns in 1st floor of the structure S30, where it can be observed that for the corner column and the adjacent-to-corner column, the area under the curve is more than the others; the area under the curve for column A1 is 20% more than the area under the curve for column C3 in S30.

It should be noted that in the pushdown curves, displacement has been monitored is the place of the missing column.

The sudden drops of the load in the pushdown curves, as it is observed in Figure 7, occur simultaneously with the plastic hinges occurring in the first beam that transfers loads of the floor. Based on the results of the analysis, outer beams of the structure (peripheral beams in the structure plan) are more ductile than inner beams. Hence collapse of the structure subjected to the loss of an inner column is more sudden than the collapse of the structure subjected to the loss of a peripheral column. Because of the place of the omitted column, one, two or four slabs are connected to the beams that are connected to the column, which shows the effect of the slabs in the integrity of the whole panel. Also as mentioned before the effective flange width on each side of the beam is taken as three times the slab thickness. Hence it seems to be necessary to pay more attention to the role of the slabs in preventing progressive collapse.

In Figure 8 the pushdown curves for S10-A1@01, S20-A1@01, S30-A1@01, and S40-A1@01 is shown. These curves show the pushdown displacement of the structure when the column A1 (corner column) from the first story is missing.

As it is seen in Figure 8, for the structure S40, area under the curve in more than the others and it relates to the height of the structure. When the height of the structure increases the area under the pushdown curve increases too. It is because of the effects of structural height on the size of the structural elements. As for controlling loads and the lateral displacement of the structure, size of structural elements especially beams must be increased. In tall structures such as S30 and S40, for controlling torsion of the structure (caused by accidental eccentricity), structural mode shapes and shear lag, the beams on the corner of the structure should be bigger in size and have some differences in the designation. These issues lead to totally different beams in the corner of the structure whose pushdown curve is significantly different. As the whole structure must be designed as a special moment resisting frames, the connections of these beams are applying more ductility.

In Figure 9 the pushdown curves for S10-C3@01, S20-C3@01, S30-C3@01, and S40-C3@01 is shown. These curves show the pushdown displacement of the structure when the column C3 (center column) from the first story is missing.

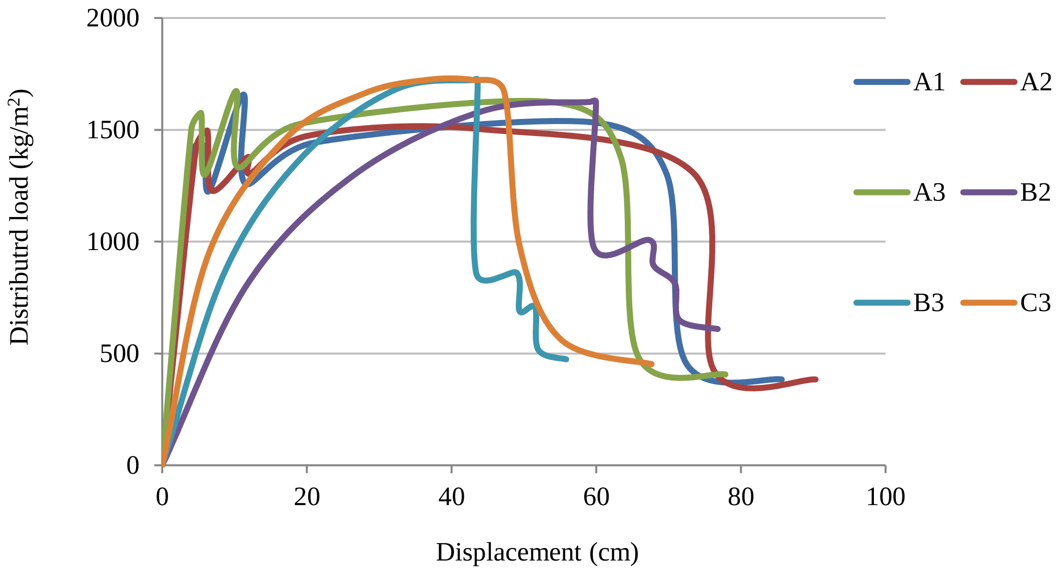


Fig. 7. The pushdown curve for different column loss of S30 at first floor

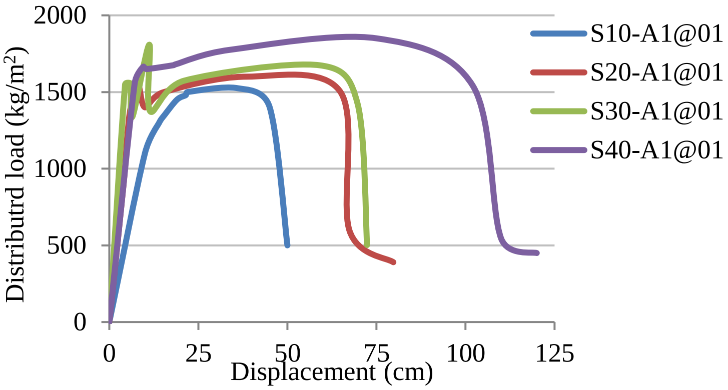


Fig. 8. The pushdown curve for S10-A1@01, S20-A1@01, S30-A1@01, and S40-A1@01

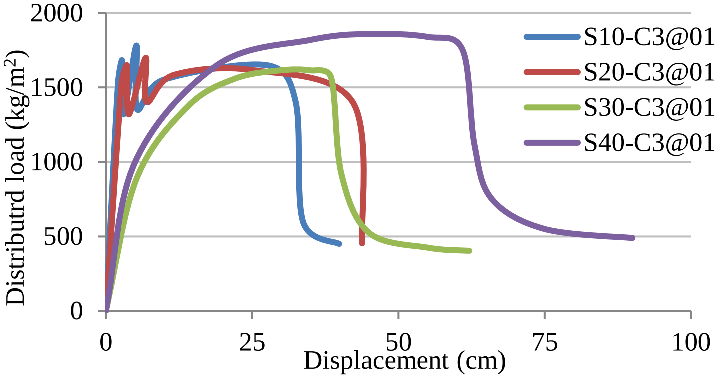


Fig. 9. The pushdown curve for S10-C3@01, S20-C3@01, S30-C3@01, and S40-C3@01

As it is seen in Figure 9, for the structure S40, the area under the curve in more than the others. But for the structure S30 and S20 the area under the curve is likely, however, their shapes are different. The area under S10 is the least. It is because of the effects of structural height on the structural elements.

Sensitivity index of all columns loss for all structure is shown in Figures 10 to 17. Figures 10 to 13 show the sensitivity index for local collapse (SIL) and Figures 14 to 17 show the sensitivity index for global collapse (SIG).

As observing the sensitivity index for local collapse (SIL), it differs in the plan of the structure as the height of structure differs. In the structure S10, the column at the corner of the structure (A1) is the most sensitive element, whereas in the structure S20 SIL of the adjacent-to-corner column (A2) behaves like the corner column. As the height of the structure increases the sensitivity of the structure to the adjacent-to-corner column (A2) increases and in the structure S40 it becomes the key element of the structure. Also as the height of the structure increases the sensitivity of the structure to the column A3 increases and in the structures, S30 and S40 its SIL behaves like the corner column. This can be because of the effects of shear lag in the design of tall structures that leads to a different situation in the design of the corner columns.

In structure SIL increases when the height increases and the most sensitive elements (key elements) are placed in the upper stories. The key element is located at the story under roof. All structures are less sensitive to the loss of inner columns.

Hence, for ordinary structures, the key elements in local collapse are the corner column, but for tall structure, the key elements in local collapse are the adjacent-to-corner column in lower stories and the corner column in upper stories.

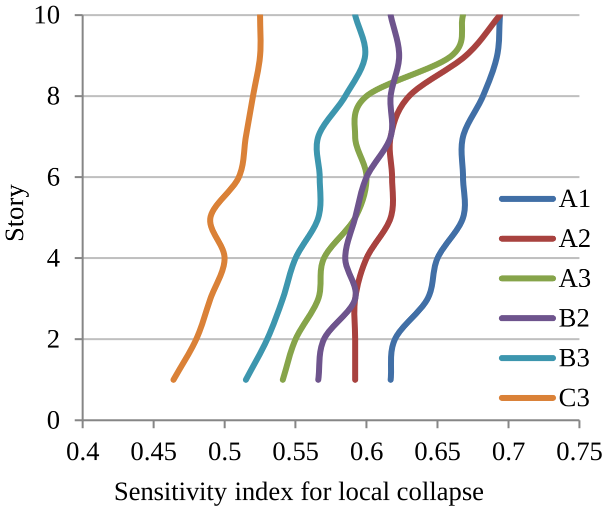


Fig. 10. Sensitivity index for local collapse (SIL) of all columns of the structure S10

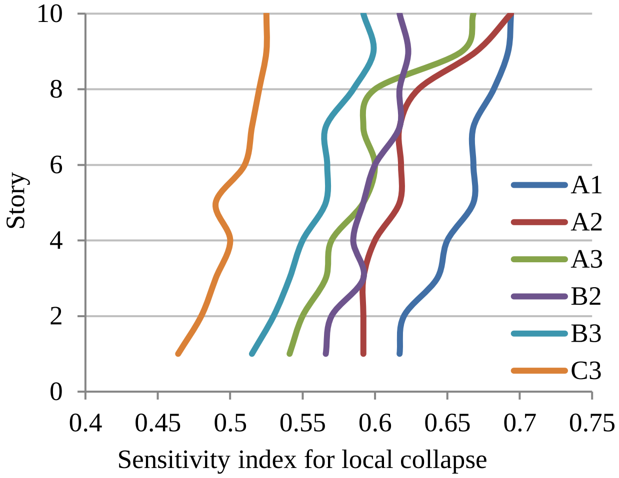


Fig. 11. Sensitivity index for local collapse (SIL) of all columns of the structure S20

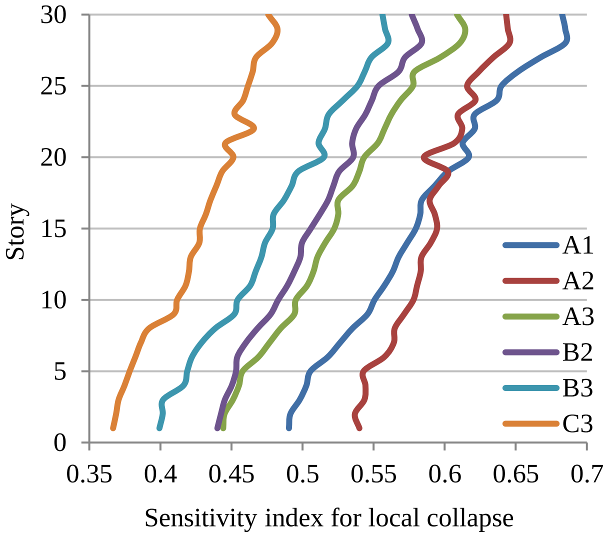


Fig. 12. Sensitivity index for local collapse (SIL) of all columns of the structure S30

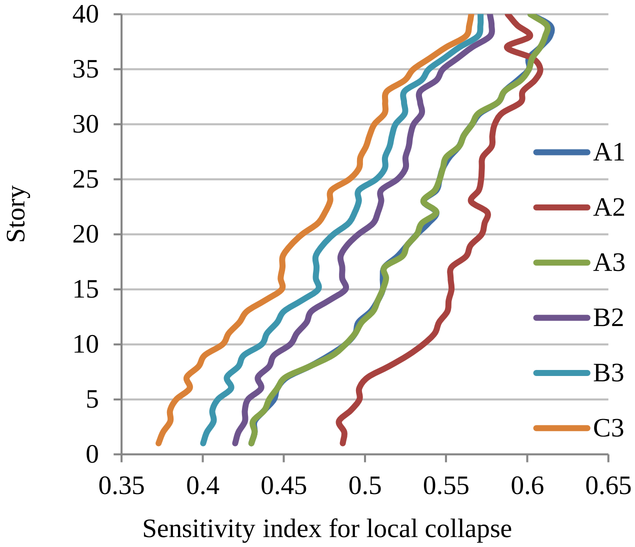


Fig. 13. Sensitivity index for local collapse (SIL) of all columns of the structure S40

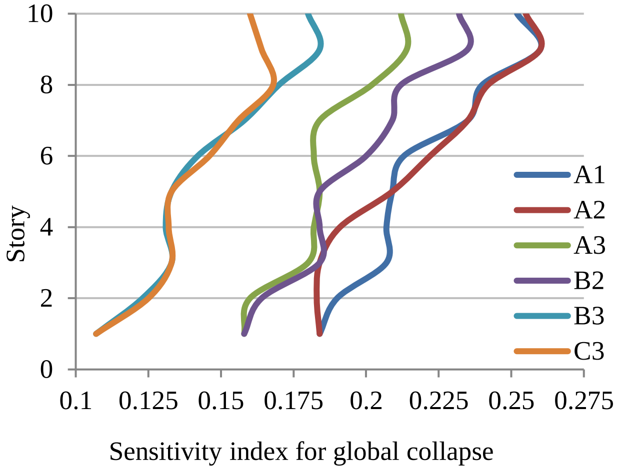


Fig. 14. Sensitivity index for global collapse (SIG) of all columns of the structure S10

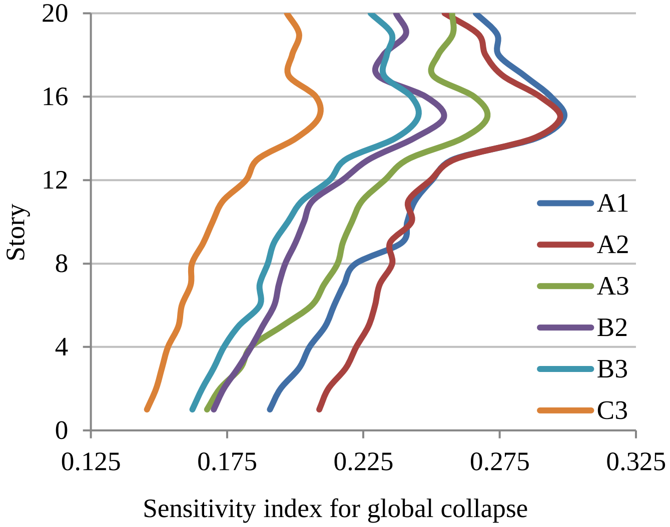


Fig. 15. Sensitivity index for global collapse (SIG) of all columns of the structure S20

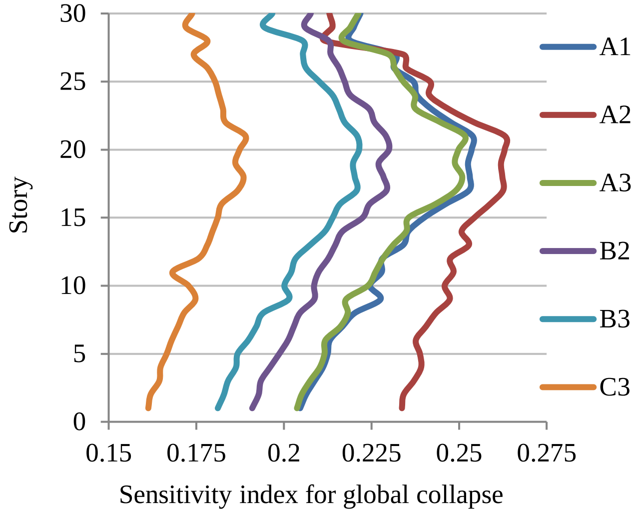


Fig. 16. Sensitivity index for global collapse (SIG) of all columns of the structure S30

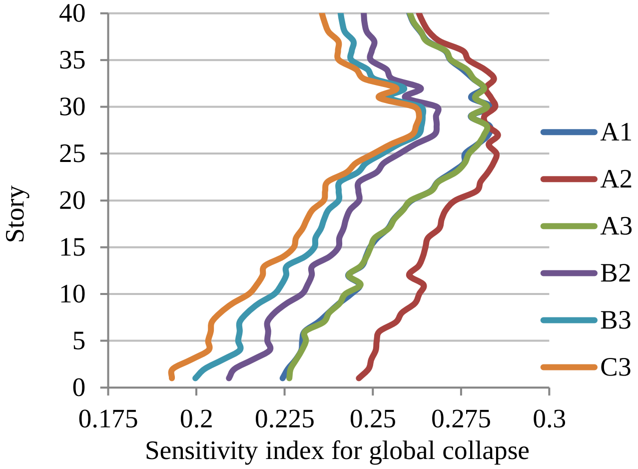


Fig. 17. Sensitivity index for global collapse (SIG) of all columns of the structure S40

As observing the sensitivity index for global collapse (SIG), it differs in the plan of the structure as the height of structure differs. In the structure S10, the column at the corner of the structure (A1) is the most sensitive element, whereas in the structure S20 SIG of the adjacent-to-corner column (A2) behaves like the corner column. As the height of the structure increases the sensitivity of the structure to the adjacent-to-corner column (A2) increases and in the structure S40 it becomes the key element of the structure. Also as the height of the structure increases the sensitivity of the structure to the column A3 increases and in the structures S30 and S40 its SIG behaves like the corner column.

In the structure, S10 SIG increases as the height increases, but in the structure S20 the biggest SIG places in the 15th story. For the structures S30 and S40 the biggest SIG places in the 20th and 30th story. It seems the by increasing the height of a structure, the story of the key element changes and from the top story encloses to the 2/3 of the height of the structure.

All structures are less sensitive to the loss of inner columns. Hence, for ordinary structures, the key elements for global collapse are the corner column, but for tall structure, the key element in local collapse is the adjacent-to-corner column in lower stories and the corner column in upper stories.

In Figure 18 sensitivity index for local collapse along the height of the structures are presented. Figure 19 shows the sensitivity index for global collapse along the height of the structures. In both Figures 18 and 19, the vertical coordinate shows the ratio of the height of the missing column (z) to the total structural height (H). In both Figures 18 and 19, the curve is only for the corner column of the structure (A1). It is notable that all Figures 18 to 21 are dimensionless.

From Figures 18 and 19, it is seen that for local collapse, as the ARh of the structure increases, the story of the key element is changing from the top of the structure to nine-tenths (9/10) of the height of the structure. But for global collapse, the story of the key element is changing from nearly the top of the structure to two-thirds (2/3) of the height of the structure, as the ARh of the structure increases.

In Figure 20 sensitivity index for local collapse along the plan of the structures at the first story, for the axis A are presented. Figure 21 shows sensitivity index for global collapse along the plan of the structures at the first story, for the axis, A. Figures 20 and 21 can be presented for all stories of the structure, but here they are presented only for the first story. As the columns in the first story are more threatened for being removed (either by terrorist attacks or car accident). In both Figures 20 and 21, horizontal coordinates show the ratio of missing column location (*l*) to the total structural width (L).

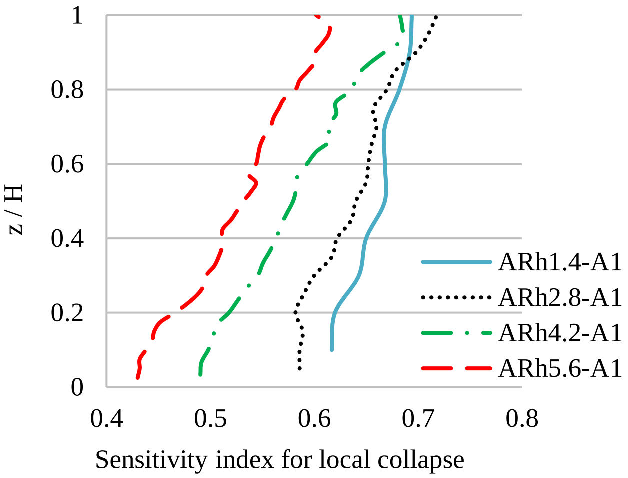


Fig. 18. Sensitivity index for local collapse (SIL) along the height of the structures

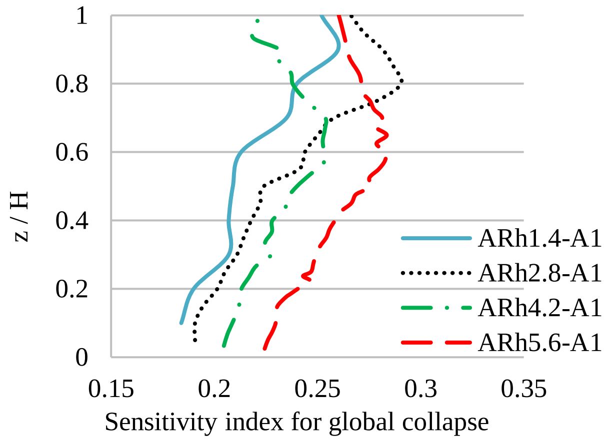


Fig. 19. Sensitivity index for global collapse (SIG) along the height of the structures

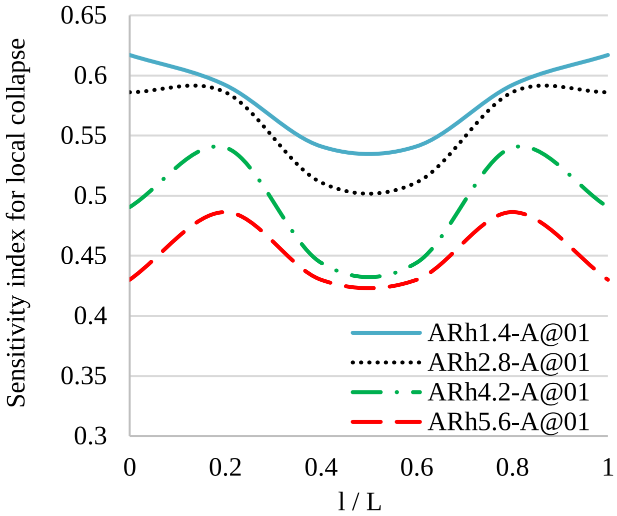


Fig. 20. Sensitivity index for local collapse (SIL) along the plan of the structures at the first story, for the axis A

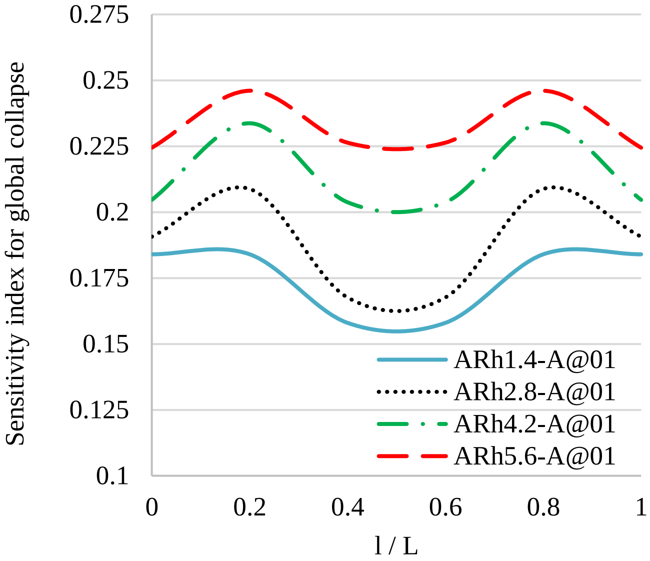


Fig. 21. Sensitivity index for global collapse (SIG) along the plan of the structures at the first story, for the axis A

From Figures 20 and 21, it is seen that for local and global collapse, as the ARh of the structure increases, the location of the key element changes from the edge of the plan of the structure to two tenths (2/10) of the width of the structure. Also, it is clear that when ARh increases, the structure becomes more sensitive to the loss of the columns in the middle of the plan.

CONCLUSIONS

In this paper, the modified sensitivity analysis (MSA) was applied to identify the location of the most critical column called the key element. For this purpose, four RC structures with different heights were studied. Doing 904 3D nonlinear pushdown analysis, the results are summarized as follows:

1- In the corner column and the adjacent-to-corner column, the area under the curve is more than the others i.e. collapse of the structure subjected to the loss of these columns are more ductile than the other columns.

2- For the tall structure S40, the area under the curve in more than for the other structures especially the corner column loss.

3- In ordinary structures (the structures with ordinary height), the key elements in local collapse are the corner columns, but as the height of the structure raises (ARh of the structure increases), the adjacent-to-corner column becomes more important. In tall structures, the key elements in local collapse are the adjacent-to-corner column. It means that vulnerability of the structure to loss of corner column in low rise structures and the adjacent-to-corner column in high rise structures are critical. This is different from the advice of the removal columns in DoD.

4- The key element in local collapse is located in the story under roof.

5- In normal structures, the key elements in global collapse are the corner columns, but as the height of the structure raises the adjacent-to-corner columns will become more important. In tall structures, the key elements in global collapse are the adjacent-to-corner column.

6- By increasing the ARh of structure, the story of the key element in global collapse changes and from the top story encloses to the 2/3 of the height of the structure.

7- All structures are less sensitive to the loss of inner columns.

REFERENCES

Abdollahzadeh, G., Nemati, M. and Avazeh, M. (2016). “Probability assessment and risk management of progressive collapse in strategic buildings facing blast loads”, *Civil Engineering Infrastructures Journal*, 49(2), 327-338.

Al-Salloum, Y.A., Abbas, H., Almusallam, T.H., Ngo, T. and Mendis, P. (2017). "Progressive collapse analysis of a typical RC high-rise tower", *Journal of King Saud University, Engineering Sciences*, 29(4), 313-320.

American Society of Civil Engineers (ASCE), (2010). “Minimum design loads for buildings and other structures”, SEI/ASCE 7-10, Reston, Va.

American Society of Civil Engineers (ASCE), (2014). “Seismic rehabilitation of existing buildings”, ASCE/SEI 41-13, Reston, Virginia, USA.

American Society of Civil Engineers (ASCE). (2006). *Seismic rehabilitation of existing buildings*, ISBN 970-0-7844-0884-1, Reston, Virginia, USA.

Amiri, S., Saffari, H. and Mashhadi, J. (2017). “Assessment of dynamic increase factor for progressive collapse analysis of RC structures”, *Engineering Failure Analysis*, 84, 300-310.

Choi, J.H. and Chang, D.K. (2009). “Prevention of progressive collapse for building structures to member disappearance by accidental actions”, *Journal of Loss Prevention in the Process Industries*, 22(6), 1016-1019.

Choi, H., and J. Kim. (2011). “Progressive collapse-resisting capacity of RC beam-column sub-assemblage”, *Magazine of Concrete Researches*, 63(4), 297-310.

Department of Defense (DoD). (2005). “Design of structures to resist progressive collapse”, UFC 4-023-03, Washington DC, United States.

Department of Defense (DoD). (2009). “Design of structures to resist progressive collapse”, UFC 4-023-03, Washington DC, United States.

Department of Defense (DoD). (2010). “Design of Structures to Resist Progressive Collapse”, UFC 4-023-03, 2009, Including Change 1, 27 January 2010, Washington DC, United States.

Department of Defense (DoD). (2013). “Design of structures to resist progressive collapse”, UFC 4-023-03, 2009, Including Change 2, 1 June 2013, Washington DC, United States.

Ettouney, M., Smilowitz, R., Tang, M. and Hapij, A. (2006). “Global system considerations for progressive collapse with extensions to other natural and man-made hazards”, *Journal of Performance of Constructed Facilities*, 20(4), 403-417.

Fu, F. (2009). “Progressive collapse analysis of high-rise building with 3-D Finite Element modeling method”, *Journal of Constructional Steel Research*, 65, 1269-1278.

Fu, F. (2010). “3-D nonlinear dynamic progressive collapse analysis of multi-storey steel composite frame buildings, Parametric study”, *Engineering Structures*, 32, 3974-3980.

Frangopol, D.M. and Curley, J.P. (1987). “Effects of damage and redundancy on structural reliability”, *Journal of Structural Engineering*, ASCE, 113(7), 1533-1549.

Griffiths, H., Pugsley, A. and Saunders, O. (1968). “Collapse of flats at ronan point, canning town”, Her Majesty’s Stationery Office, London.

Hadianfard, M.A., Farahani, A. and B-Jahromi, A. (2012). “On the effect of steel columns cross sectional properties on the behaviours when subjected to blast loading", *Structural Engineering and Mechanics*, 44(4), 449-463.

Iranian Road, Housing and Urban Development Research Center (BHRC). (2013a). *Building design codes for earthquakes*, Standard 2800, 4th Edition, Tehran, I.R. Iran.

Iranian Road, Housing and Urban Development Research Center (BHRC). (2013b). "Iranian national building code No. 6: Loading", INBC 6, Tehran, I.R. Iran.

Iranian Road, Housing and Urban Development Research Center (BHRC). (2013c). "Iranian national building code No. 9: Reinforced concrete structure design", INBC 9, Tehran, I.R. Iran.

Izzuddin, B.A., Vlassis, A.G., Elghazouli, A.Y. and Nethercot, D.A. (2008). "Progressive collapse of multi-storey buildings due to sudden column loss-part i: Simplified assessment framework", *Engineering Structures*, 30(5), 1308-1318.

Khandelwal, K., El-Tawil, Sh. and Sadek, F. (2009). "Progressive collapse analysis of seismically designed steel braced frames", *Journal of Constructional Steel Research*, 65(3), 699-708.

Kheyroddin, A. and Mehrabi, F. (2012). "Assessment of progressive collapse potential of steel frame due to sudden corner column loss", *Wulfenia Journal*, 19(10), 392-408.

Kheyroddin, A., Gerami, M. and Mehrabi, F. (2012). "Assessment of the dynamic effect of steel frame due to sudden middle column loss", *The Structural Design of Tall and Special Buildings*, 23(5), 390-402.

Kim, J., Park J.H. and Lee. T.H. (2011). "Sensitivity analysis of steel buildings subjected to column loss", *Journal of Engineering Structures*, 33, 421-432.

Krauthammer, T. and Cipolla, J. (2007), "Building blast simulation and progressive collapse analysis", NAFEMS World Congress, Quebec City, Canada.

Krauthammer, T. (2008). *Modern protective structures*, CRC Press, Taylor and Francis Group.

Leyendecker, E.V. and Burnett, E.F.P. (1976). "The incidence of abnormal loading in residential buildings", National Bureau of Standards, December.

Leyendecker, E.V., Breen, J.E., Somes, N.F. and Swatta, M. (1975). "Abnormal loading on buildings and progressive collapse: an annotated bibliography", National Bureau of Standards, May.

Li, L., Wang, W., Chen, Y. and Teh, L.H. (2017). "A basis for comparing progressive collapse resistance of moment frames and connections", *Journal of Constructional Steel Research*, 139, 1-5.

Mashhadiali N. and Kheyroddin, A. (2013). "Progressive collapse assessment of new hexagrid structural system for tall buildings", *The Structural Design of Tall and Special Buildings*, 23(12), 947-961.

Mashhadiali, N., Gholhaki, M., Kheyroddin, A., Zahiri-Hashemi, R., (2016), "Technical note: analytical evaluation of the vulnerability of framed tall buildings with steel plate shear wall to progressive collapse", *International Journal of Civil Engineering*, 14(8), 595-608.

Mashhadiali, N., Kheyroddin, A. and Zahiri-Hashemi, R. (2016). "Dynamic increase factor for investigation of progressive collapse potential in tall tube-type buildings", *Journal of Performance of Constructed Facilities*, 30(6), 04016050.

McConnella, J.R. and Brown, H. (2011). "Evaluation of progressive collapse alternate load path analyses in designing for blast resistance of steel columns", *Engineering Structures*, 33, 2899-2909.

Sadek, F., Main, J.A., Lew, H.S. and Bao, Y. (2011). "Testing and analysis of steel and concrete beam-column assemblies under a column removal scenario", *Journal of Structural Engineering*, 137(9), 881-892.

Saltelli, A., Ratto, M., Andres, T., Campolongo, F., Cariboni, J., Gatelli, D., Saisana, M. and Tarantola, S. (2008). *Global sensitivity analysis*, The Primer, John Wiley and Sons.

Santafe, I.B., Berke, P., Bouillard, P., Vantomme, J. and Massart, T.J. (2011). "Investigation of the influence of design and material parameters in the progressive collapse analysis of RC structures", *Engineering Structures*, 33, 2805-2820.

SAP2000. (2012). *Computer software*, Computers and Structures, Berkeley, CA.

Takumi, I. and Toshinobu, T. (2014). "Sensitivity analysis related to redundancy of regular and irregular framed structures after member disappearance", *International Journal of High-Rise Buildings*, 3(4), 297-304.

Tavakoli, H.R. and Kiakojouri, F. (2014). "Progressive collapse of framed structures: suggestions for robustness assessment", *Scientia Iranica, Transaction A, Civil Engineering*, 21(2), 329.

Tavakoli, H. and Kiakojouri, F. (2015). "Threat-independent column removal and fire-induced progressive collapse: Numerical study and comparison*", Civil Engineering Infrastructures Journal*, 48(1), 121-131

Tavakoli, H.R. and Rashidi Alashti, A. (2013). "Evaluation of progressive collapse potential of multi-story moment resisting steel frame buildings under lateral loading", *Scientia Iranica*, 20(1), 77-86.

The U.S. General Service Administrations (GSA). (2003). "Progressive collapse analysis and design guidelines for new federal office buildings and major modernization project", Washington DC.

Vlassis, A.G., Izzuddin, B.A., Elghazouli, A.Y. and Nethercot, D.A. (2008). "Progressive collapse of multi-storey buildings due to sudden column loss, Part II: Application", *Engineering Structures*, 30(5), 1424-1438.

Wada, A. and Wang, Zh. (1989). "A study on strength deterioration of indeterminate double-layer space truss due to accidental member failure", *Journal of Structural and Constructional Engineering, Transactions of AIJ*, 402, 89-99.

Zahrai, S.M. and Ezoddin, A. (2014). "Numerical study of progressive collapse in intermediate moment resisting reinforced concrete frame due to column removal", *Civil Engineering Infrastructures Journal,* 47(1), 71-88.