Civil Engineering Infrastructures Journal, 51(2): 405 – 424, December 2018
Print ISSN: 2322-2093; Online ISSN: 2423-6691
DOI: 10.7508/ceij.2018.02.010

Numerical Evaluation of Progressive Collapse Potential in Reinforced Concrete Buildings with Various Floor Plans Due to Single Column Removal

Rezaie, F.[1*] , Fakhradini, S.M.[2] and Ghahremannejad, M.[3]

[1] Associate Professor, Department of Civil Engineering, Bu-Ali Sina University, Hamedan, Iran.
[2] M.Sc. in Structural Engineering, Department of Civil Engineering, Bu-Ali Sina University, Hamedan, Iran.
[3] Ph.D. Candidate, Center of Structural Engineering Research/Simulation, Department of Civil Engineering, University of Texas at Arlington, United States.

Received: 17 Feb. 2018; Revised: 02 May 2018; Accepted: 06 May 2018

ABSTRACT: 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. In this research, the potential of progressive collapse due to a sudden removal of vertical load-bearing elements in reinforced concrete buildings structures with different floor plans such as geometrical regular and irregular floor plans as well as floor plans with and without torsional irregularity were assessed. The buildings were designed according to ACI 318-14 provisions and Iranian seismic code. The progressive collapse potential of the structures was assessed following of a sudden column or shear wall removal in different locations in their first floor using nonlinear dynamic analysis (NDA). Displacement sensitivity and column sensitivity indexes were utilized to compare different cases of load-bearing element removal in each model. Results indicated that in all geometrical regular floor plan, floor plan with reentrant corner and floor plan with torsional irregularity, the most critical case of column removal was removing columns located in outer corners of the plan. In addition, removing external columns was more critical than internal columns. In buildings with shear walls, removing shear walls led to much more critical scenarios than removing columns. Furthermore, results revealed that buildings with torsional irregularity floor plan, designed according to Iranian seismic code, had a lower potential of progressive collapse rather than those buildings with no irregularity.

Keywords: Nonlinear Dynamic Analysis, Progressive Collapse, R.C. Buildings, Regular and Irregular Plans, Sudden Element Removal.

INTRODUCTION

Progressive collapse is the spread of an initial local damage to the entire structure or a large portion of it so that the final damage is disproportionate to the local damage that initiated the collapse. Progressive collapse can occurs because of various reasons including design and construction errors, foundation subsidence, fire, gas explosions,

* Corresponding author E-mail: freydoon.rezaie@gmail.com

bomb explosions and vehicular collisions. Progressive collapse as a structural engineering topic first came into notice when Ronan Point apartment tower collapsed in London in 1968 (Ellingwood, 2006). Ronan Point was a 22-story precast concrete apartment in which a gas explosion on the 18th floor caused a progressive collapse in southeast corner of the building (Pearson, 2005). There are two general approaches for reducing the possibility of progressive collapse: Indirect Design and Direct Design. With indirect design approaches, resistance to progressive collapse is considered implicitly through the provision of minimum levels of strength, continuity and ductility. Whereas direct design approaches include explicit consideration of resistance to progressive collapse during the design process (DoD, 2016). One of the methods for direct designing of structures to resist progressive collapse is alternate path (AP) method that recommended in UFC 4-023-03 (2016) and GSA (2013) guidelines. In this method, if a vertical load-bearing element was destroyed, the structure shall be able to bridge over the damaged element and alternative load paths must be considered for preventing progressive collapse.

So far, various studies have addressed progressive collapse and its potentials in structures. Some of these studies are pointed out as follows:

Marjanishvili (2004) discussed the advantages and disadvantages of four analytical procedures for assessment the progressive collapse including linear static, nonlinear static, linear dynamic and nonlinear dynamic analysis. Sasani and Sagiroglu (2008) studied progressive collapse in two building with ordinary reinforced concrete frames. One building was designed for a moderate level of lateral loads while the other was designed for a minimum level of lateral loads. The results showed that the maximum vertical displacement of the joint above the removed column in the weaker structure (designed for a minimum level of lateral loads) was approximately 3.5 times that of the stronger structure (designed for a moderate level of lateral loads). Yi et al. (2008) investigated progressive collapse of a reinforced concrete frame due to removal of the middle column on the first story of the frame by a static experimental study. They found that progressive collapse in reinforced concrete frames include three distinct phases: elastic, plastic, and catenary phases. According to the results the catenary phase in beams can be considered as an alternative load path to resist extra load. Kim and Kim (2009) studied progressive collapse resistance of steel moment frames using alternate path methods. In this study progressive collapse resistance of 3, 6, and 15-story steel frames was investigated using linear static, linear dynamic, and non-linear dynamic analysis. Results showed that with increasing number of floors progressive collapse potential decreased. Also, linear static analysis provided more conservative results for progressive collapse potential.

Helmy et al. (2012) evaluated progressive collapse resistance of a reinforced concrete structure due to columns and shear walls removal. Pachenari et al. (2013) investigated progressive collapse potential of a five-story regular building with intermediate RC moment frame by alternate path method. They used nonlinear dynamic and nonlinear static analyses in their investigation. Results from nonlinear dynamic analysis indicated that the structure is resistant to progressive collapse while nonlinear static analysis revealed that the structure needs some modifications in design sections. Hence they concluded that nonlinear static analysis led to more conservative results than nonlinear dynamic analysis. Rahai et al. (2013) studied progressive collapse in a regular RC structure due to instantaneous and gradual removal of columns. Results showed that in the instantaneous scenario both the maximum vertical displacement of the upper node of the removed column and the maximum axial forces at neighboring columns of the removed column are greater than the gradual scenario.

Ren et al. (2014) studied progressive collapse resistance of two typical 15-storey buildings. The first building had a weak wall-strong frame structure while the other had a strong wall-weak frame system. Results indicated different performance in progressive collapse prevention. The building with strong wall-weak frame system had insufficient resistance to progressive collapse and thus a special collapse prevention design is required.

Zahrai and Ezoddin (2014) compared advantages and disadvantages of different methods of progressive collapse analysis. They evaluated progressive collapse potential of two 5 and 10-story regular reinforced concrete buildings with intermediate moment-resistant frame with four analysis procedures including linear static, nonlinear static, linear dynamic and nonlinear dynamic analysis. Findings showed that dynamic analysis procedures gave more accurate results. Rezvani et al. (2015) investigated the effect of span length on progressive collapse potential of steel moment frames. Result showed that by decreasing the span length the strength of frames against progressive collapse increased.

Li and Sasani (2015) assessed the effects of seismic design and structural integrity requirements on progressive collapse resistance of reinforced concrete frame structures. In this study the relative importance of ductility capacity and strength were discussed for response of structures subjected to severe seismic ground motions and to loss of a column. They also examined the effects of span length on response of the structure after column removal. Results showed that for buildings with shorter spans at sites with low to medium seismic severity, designing for higher seismicity does not necessarily lead to a better performance.

Tavakoli and Kiakojouri (2015) studied fire-induced and threat-independent progressive collapse potential in 2D steel moment resisting frames. Results indicated that in fire-induced progressive collapse the most important parameter was the weight of the structure above the failure zone, whereas in threat-independent column removal alternative load paths had main role.

Abdollahzadeh et al. (2016) evaluated the probability of progressive collapse and the reliability of an important steel building with a special moment frame system under probable blast scenarios inside and outside the building. Results indicated that progressive collapse probability and reliability of the building are 57% and 43% respectively.

Arshian et al. (2016) investigated the effect of different nonlinear modeling approaches in progressive collapse response of reinforced concrete framed structures subjected to sudden column removal scenarios through alternate path analysis. For this purpose, the finite element model of a progressive collapse experimental test was developed in three approaches. The first finite element model was developed in Sap2000 using concentrated plastic hinges at beam–column end sections. Two other finite element models were developed in OpenSess framework using nonlinear force-based element and displacement-based element respectively, with distributed plasticity and fiber sections. Results showed that using concentrated plastic hinges approach led to larger vertical displacement at the top of the removed column compared to the fiber-based modelling approaches. Fiber-based modeling approaches can estimate maximum vertical displacement with inconsiderable error compared with the experimental measurement.

Ghahremannejad and Park (2016) studied

the effect of the number of floors in progressive collapse of reinforced concrete structures due to sudden column removal. They analyzed the results in terms of two indexes: column sensitivity index and displacement sensitivity index. Results showed that when the number of floors increased potential failure of the neighboring columns increased. However, the vertical displacement of the top node of the removed column decreased. Shan et al. (2016) examined experimentally the interaction between the infill walls and the reinforced concrete (RC) frame members in the progressive collapse process.

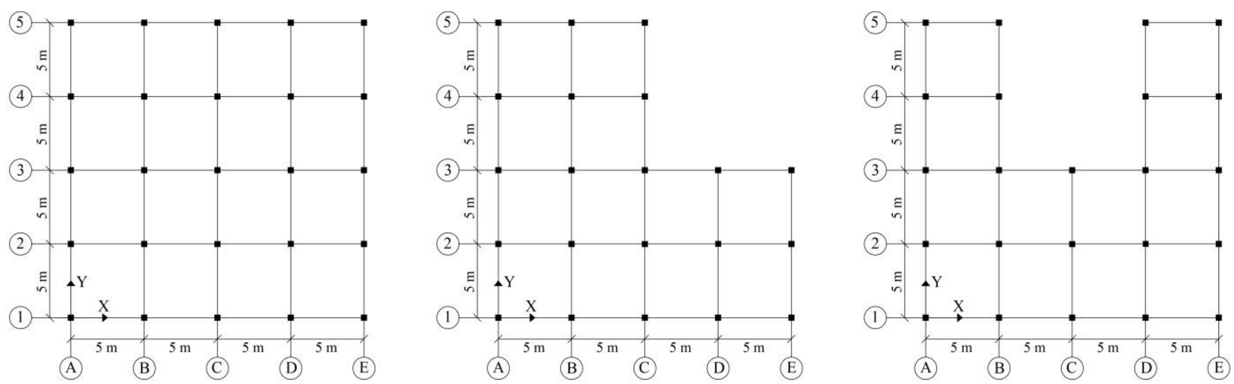
Arshian and Morgenthal (2017) studied the 3D nonlinear dynamic response of reinforced concrete structures subjected to sequential column removal scenarios. Results indicated that the time-lag between the column removals had significant effect on the 3D redistribution of gravity loads. Kordbagh and Mohammadi (2017) studied the effect of building height and designing base shear on progressive collapse resistance of steel moment frames. Results indicated that as the building height increased, potential of progressive collapse decreased. Also, the potential for progressive collapse is decreased by increasing the designing base shear.

The literature review appears the demand of studying the progressive collapse potential due to sudden removal of vertical load-bearing elements in irregular building structures especially those with reentrant corner and torsional irregular floor plans. Architectural concerns lead to having many buildings with reentrant corner and torsional irregularities floor plans. According to Iranian seismic code (2014), a reentrant corner irregularity is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 20% of the plan dimension of the structure in the given direction. Also, torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with torsional amplification factor equal to 1.0, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. As such, the present study aims to investigate progressive collapse potential of reinforced concrete buildings with reentrant corner, torsional and no irregularities that designed based on ACI 318-14 (2014) provisions and Iranian seismic code (2014), due to a sudden removal of the vertical load-bearing elements. Progressive collapse potential of structures is evaluated using nonlinear dynamic alternate path method based on GSA (2013) guideline.

BUILDINGS CHARACTERISTICS

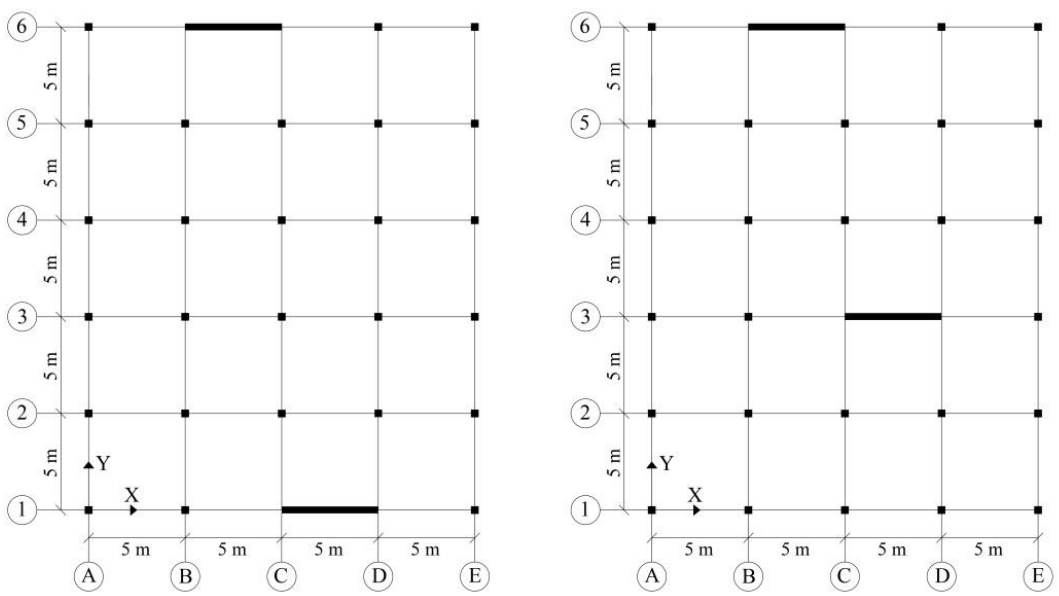
In order to evaluate the progressive collapse potential in reinforced concrete (R.C.) building structures with geometrical regularity as well as irregularity, three floor plans consist of a square, L and U shaped were investigated, shown in Figure 1. The lateral force-resisting systems for these buildings were intermediate bending-moment frames. Figure 2 illustrates two floor plans with and without torsional irregularity. A dual system comprised of intermediate bending-moment frames associated with special structural walls in one direction and intermediate bending-moment frames in the perpendicular direction acted as the lateral force-resisting systems for the designated buildings. All structures were designed for residential occupancy in a high seismic zone of Iran with a design acceleration of 0.3g and presumed that the structures were on soil type 2 (the average shear wave velocity to a depth of 30 m is 375-750 m/s). Response modification coefficient is taken to be 5 for an intermediate bending-moment frame system and 6.5 for a dual system comprised of intermediate bending-moment frames associated with special structural walls, based on Iranian seismic code (2014). In all models the spans length was 5 m and the stories height was 3.2 m. The compressive strength of concrete was f'_c= 25 MPa, and the yield strength of the reinforcement was f_y= 400 MPa. A dead load of 6 kN/m^2 was applied to the roof, and 5 kN/m^2 to other floors. A live load 1.5 kN/m^2 was applied to the roof and 2 kN/m^2 to other floors. In addition, a partition load of 1 kN/m^2 was applied to all floors except in roof level. Also, dead loads of 5 kN/m and 2 kN/m were applied to the perimeter beams of floors and roof; respectively, as the weight of perimeter walls.

In plans a, b and c the dimensions of beams were 400×500 mm in the first two stories and 400×400 mm in the other stories. In plans c and d the dimensions of beams along X axis of the plans were 400×400 mm in all stories. Also, the dimensions of beams along Y axis of the plans were 400×500 mm in the first two stories and 400×400 mm in the others. In all plans the width of beams was 400 mm. All beams contain two #6 bars as continuous reinforcement on the top and bottom of their sections. Also, extra #6 was used in the section of beams wherever needed. Column and shear wall types and sections details in all plans are given in Tables 1 to 3 and Figure 3.



(a) Regular floor plan (Plan a) **(b)** L-shaped floor plan (Plan b) **(c)** U-shaped floor plan (Plan c)

Fig. 1. Geometrical regular and irregular floor plans

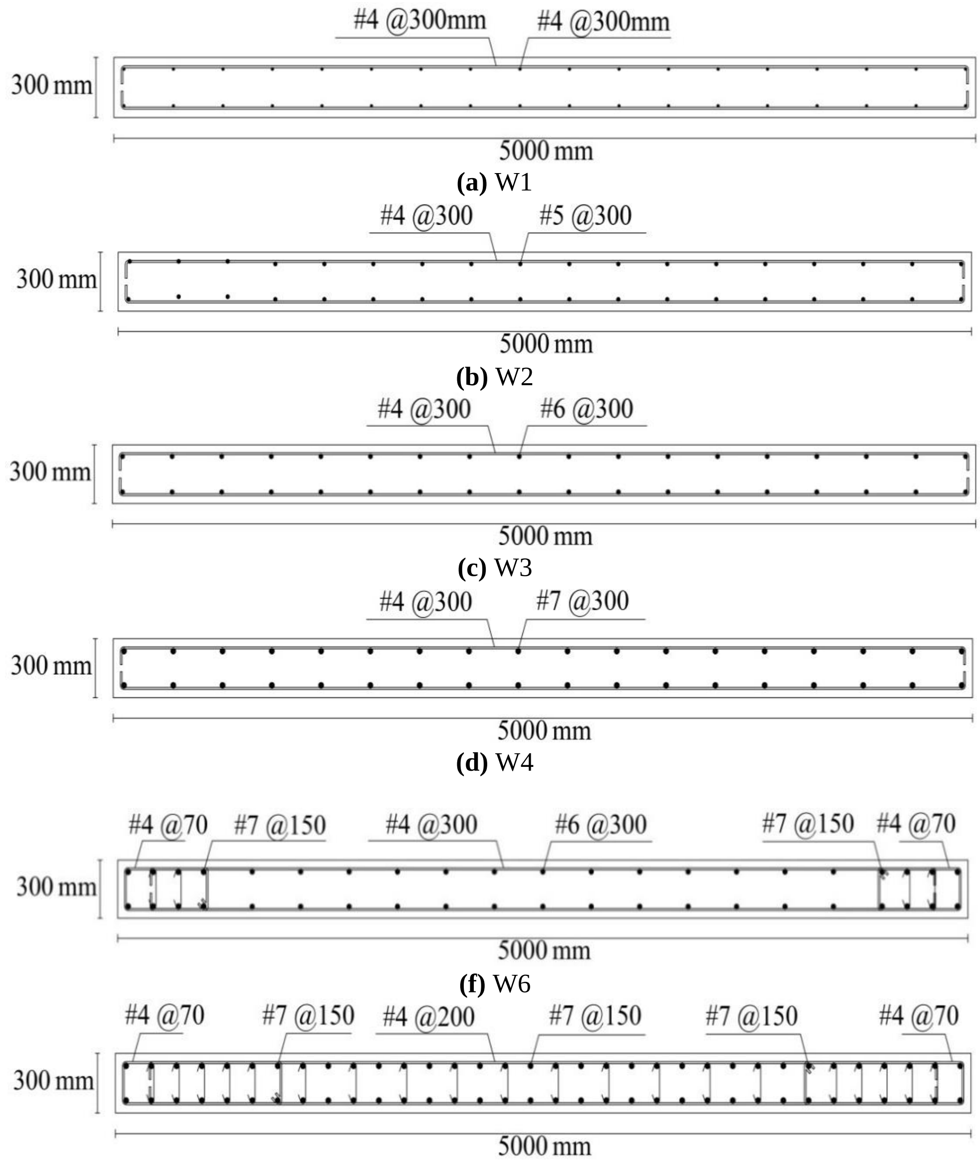


(a) Regular floor plan (Plan d) **(b)** Torsional irregular floor plan (Plan e)

Fig. 2. Floor plans with and without torsional irregularity

Table 1. Dimensions and reinforcement of column sections

Section	Dimensions (mm×mm)	Reinforcement	Section	Dimensions (mm×mm)	Reinforcement
C1	350×350	8 #6	C5	500×500	12 #6
C2	400×400	8 #6	C6	500×500	16 #6
C3	400×400	12 #6	C7	550×550	16 #7
C4	450×450	12 #6			



(a) W1

(b) W2

(c) W3

(d) W4

(f) W6

(g) W7

Fig. 3. Sections of shear walls

Table 2. Column types for geometrical regular and irregular floor plans

Story	Plan (a)		Plan (b)		Plan (c)	
	External Columns	Internal Columns	External Columns	Internal Columns	External Columns	Internal Columns
1	C4	C6	C4	C6	C4	C6
2	C4	C3	C4	C3	C4	C4
3	C4	C3	C4	C3	C3	C3
4	C2	C1	C2	C1	C2	C1
5	C1	C1	C1	C1	C1	C1

Table 3. Column and shear wall types for floor plans with and without torsional irregularity

Story	Plan (d)			Plan (e)		
	External Columns	Internal Columns	Shear Walls	External Columns	Internal Columns	Shear Walls
1	C4	C4	W4	C7	C6	W7
2	C3	C4	W3	C5	C4	W6
3	C3	C2	W1	C4	C4	W5
4	C1	C1	W1	C3	C2	W2
5	C1	C1	W1	C3	C1	W1

Analytical Modeling

For evaluating of progressive collapse potential of structures, 3D numerical models were developed in OpenSess (Mazzoni et al., 2016) framework and were analyzed using nonlinear dynamic AP method. The nonlinear dynamic analysis is one of the most accurate method for progressive collapse analysis since it takes into account material and geometric nonlinearities as well as dynamic effects (Ren et al., 2014). The beams, columns and shear walls of the structures were modeled by force-based nonlinear beam-column elements with distributed plasticity. Nonlinear analysis requires the consideration of material and geometric nonlinearities. In order to take into account geometric nonlinearity, P-Delta transformation was used. For material, Concrete01 and Steel02 from OpenSees (Mazzoni et al., 2016) materials library were used to define concrete and steel behavior, respectively. The stress–strain relationship of concrete and steel are shown in Figures 4 and 5, respectively. According to GSA (2013) guideline, in alternate path analysis, appropriate over-strength factors must be applied to materials strength to translate lower-bound material properties to expected strength material properties. Therefore concrete compressive strength was multiplied by 1.5 and yield strength of reinforcement steel was multiplied by 1.25 according to suggestion of ASCE-41 (2006). As such, strength of reinforcement steel was considered 500 MPa in numerical modeling. Also, the strain-hardening ratio of steel was 1% and modulus of elasticity was 200 GPa. Table 4 presents the expected strength material properties of confined and unconfined concrete calculated according to Mandar et al. (1988). Cross section of the elements were modeled by fiber section. Each fiber section of beams and columns were included two portions: core and cover. The confined concrete was assumed for the core of the sections to consideration of the effects of confinement; while, the cover of the sections were supposed unconfined concrete.

To model the cross section of the shear walls without boundary elements, all the concrete material of the sections were modeled using unconfined concrete. Whereas the walls with boundary elements, confined concrete was considered to model confinement at boundary elements; while, in other concrete parts of the walls unconfined concrete was used (Martinelli, 2009).

The gravity loads were applied to the entire structures according to the following load combination based on GSA guideline:

$$G = 1.2D + 0.5L \tag{1}$$

where D: is dead load, and L: is live load.

In alternate path method, the vertical load-bearing element is removed and the capability of the structure to bridging over the removed element is evaluated. The technique for modeling a load-bearing element removal used in this study included several steps: first, the structure was analyzed under the applied gravity loads and the internal forces of the selected load-bearing element were obtained. Then, the selected load-bearing element was removed and its internal forces along with the gravity loads were applied to the top node of the removed element. Finally, sudden removal of the load-bearing element was modeled by performing nonlinear dynamic analysis and applying forces with the same magnitude and opposite to the forces that were applied in previous step to the top node of the removed element in a very short duration. The nonlinear dynamic analysis continued until the structure reaches a stability state or collapses (Sasani and Sagiroglu, 2008; Pachenari et al., 2013; Ren et al., 2014). The time steps of the nonlinear dynamic analysis were 0.01 sec. The Newmark method with parameters of $\gamma = 0.5$ and $\beta = 0.25$ was used as the numerical integrator. The damping ratio of 5% was used in dynamic analysis.

Evaluation Indexes

In all models progressive collapse potential was investigated in different scenarios including sudden removal of corner columns, internal columns, external columns and also sudden removal of shear walls in the first floor. The results of the analysis including the vertical displacement of the top node of the removed element and variations of axial force in the adjacent columns of the removed element were examined.

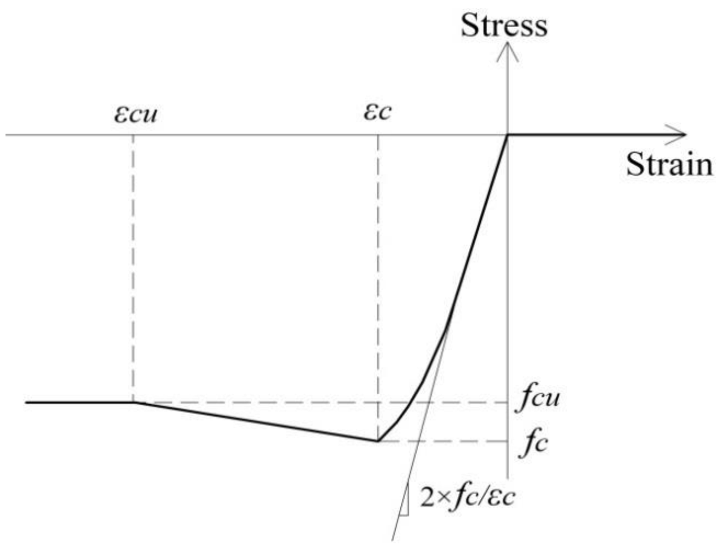


Fig. 4. Stress-strain relationship of concrete (Mazzoni et al., 2016)

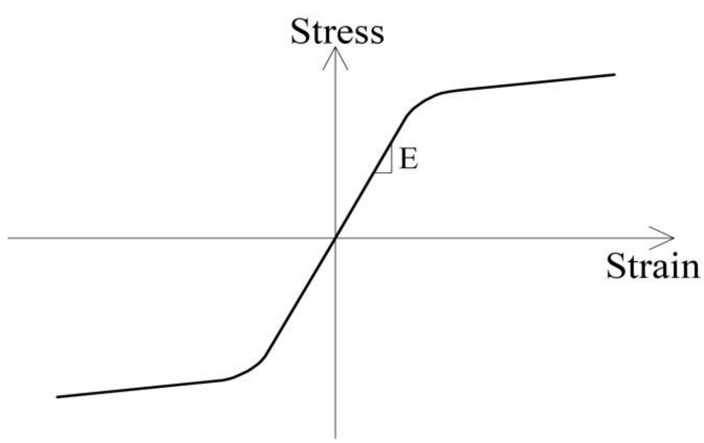


Fig. 5. Stress-strain relationship of steel (Mazzoni et al., 2016)

Table 4. Concrete properties

	Maximum Compressive Strength f_c (MPa)	Strain at Maximum Strength ε_c	Ultimate Strength f_{cu} (MPa)	Strain at Ultimate Strength ε_{cu}
Unconfined concrete	37.5	0.002	0	0.005
Confined concrete (beams and columns)	41.5	0.0035	8.3	0.018
Confined concrete (Shear walls)	47	0.0056	8.4	0.03

In order to evaluate and compare the various scenarios of load-bearing element removal in each model, two dimensionless indexes including column sensitivity index and displacement sensitivity index were also employed. The column sensitivity index or β index is related to columns and displacement sensitivity index or λ index is related to beams. The β index is defined as follow (Ghahremannejad and Park, 2016):

$$\beta = \frac{\textit{Maximum applied axial load to neighbor column after column removal}}{\textit{applied axial load to neighbor column before column removal}} \tag{2}$$

The greater value of β indicates that the column is in a more critical condition. The λ index is also defined as follow (Ghahremannejad and Park, 2016):

$$\lambda = \frac{\textit{ultimate vertical displacement of the top node of the column after column removal}}{\textit{vertical displacement of the top node of the column before column removal}} \tag{3}$$

The λ index compares the gravity stiffness of the structure at the node of the removed column. Gravity stiffness of a frame structure is defined as the summation of the bending stiffness of the elements and axial stiffness of the columns which effect the vertical displacement of a certain node. The greater value of λ denotes that the structure loses more gravity stiffness when a column removes, and condition of the structure is worse. β and λ are good indices for finding the key elements of a structure (Ghahremannejad and Park, 2016).

Analysis Results

In Figure 6, variations of the vertical displacement of the top node of the column C1 in Plan (a) as well as variations of the axial force of the external and internal adjacent columns before and after sudden removal are shown. It reports that after the sudden column removal, the vertical displacement of the top node of the removed column and the axial force of the adjacent columns were increased immediately and reached their maximum values. After that the structure vibrated until reached its ultimate stage.

The vertical displacement of the top node of the removed columns and axial force of adjacent columns for all column removal scenarios in all plans were presented in the following sections.

Geometrical Regular and Irregular Floor Plans

Table 5 expresses the location of the removed columns in the geometrical regular and irregular floor plan as well as the vertical displacement of the top node of the removed columns, and the values of λ index.

According to Table 5, in Plan (a), the largest value of the maximum vertical displacement and ultimate vertical displacement is associated with the corner column removal. In this scenario, the maximum vertical displacement is about 15%

larger than the other scenarios. Also, the maximum value of λ index is associated with the corner column removal scenario. Therefore, the most critical column removal scenario was associated with the removal of corner columns since the structure lost more gravity stiffness than the other scenarios. The vertical displacement due to removal of external column and internal column were almost similar. However, the value of λ index in the external column removal scenario was larger than the internal column removal scenario. Thus, removing external columns was more critical than removing internal columns.

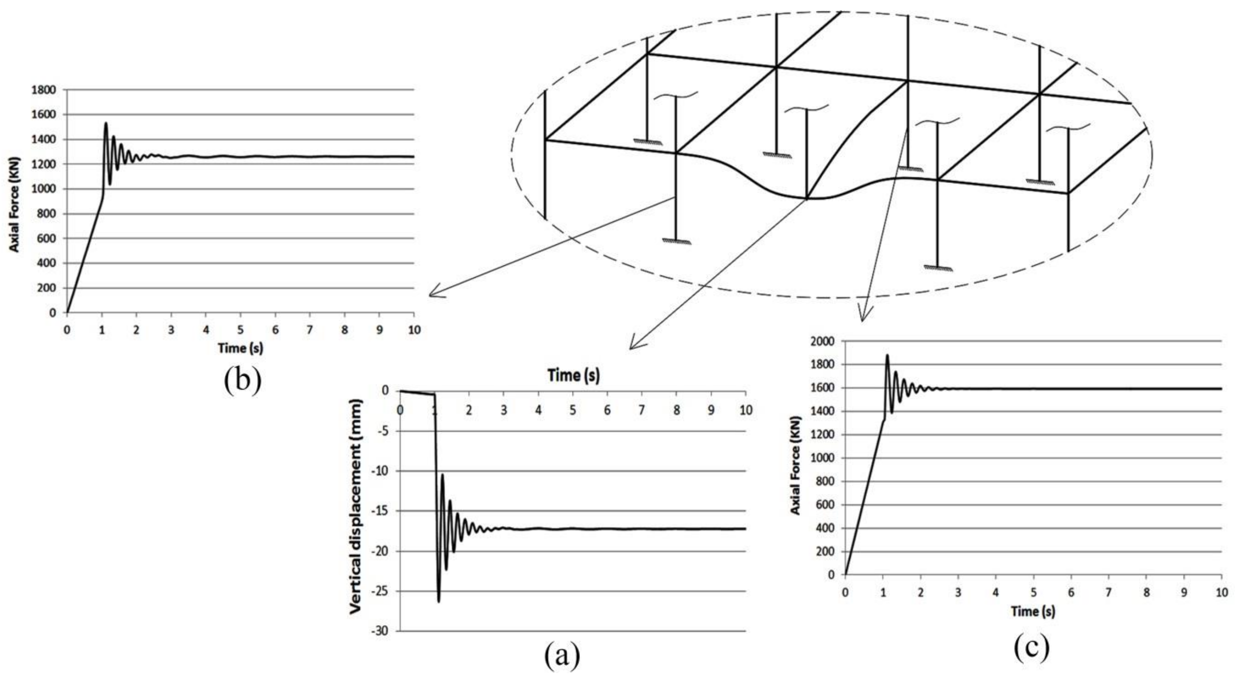


Fig. 6. a) Variations of the vertical displacement of the top node of the column C1, b) variations of the axial force of the column B1, c) C2 before and after sudden column removal in Plan (a)

Table 5. Vertical displacement of the top node of the removed columns and "λ" index for Plans (a, b and c)

Plan	Removal Location	Before Removal (mm)	After Removal Maximum (mm)	After Removal Ultimate (mm)	λ
(a)	A1	0.280	30.53	19.90	71.1
	C1	0.443	26.29	17.20	38.8
	C3	0.525	26.78	17.90	34.1
(b)	A1	0.279	31.01	20.29	72.7
	C5	0.281	31.25	20.97	74.6
	C1	0.443	26.64	17.42	39.3
	C4	0.435	25.49	16.94	38.9
	B2	0.538	29.01	19.12	35.5
	C3	0.593	24.06	15.97	26.9
(c)	A1	0.277	31.48	20.55	74.2
	B5	0.280	33.17	22.91	81.8
	C1	0.439	26.92	17.63	40.1
	C3	0.432	24.89	16.52	38.2
	C2	0.530	27.14	18.09	34.1
	B3	0.592	24.28	16.13	27.2

In Plan (b), removing columns at the outer corners of the plan (A1 and C5) resulted in the largest vertical displacements. The maximum vertical displacement in the case of removing columns at the outer corners (A1 and C5) was about 16% larger compared to the one in removing the external column C1, 22% larger compared to the one in removing the external column C4, 7% larger compared to the one in removing the internal column B2, and about 30% larger compared to the one in removing column C3 at the inner corner of the plan. In addition, the maximum value of λ index was associated with the case of removing the outer corner columns. Therefore, the most critical column removal scenario in Plan (b) was associated with the removal of outer corner columns. The vertical displacement of the top node of the removed column in the case of internal column removal was larger compared to the removal of external columns. But the value of λ index for external columns was larger than the one for the internal column, meaning that the structure lost more gravity stiffness in case of the removal of external columns compared to the removal internal column. Therefore, in Plan (b), removing external columns was more critical than removing internal columns. The minimum vertical displacement and the minimum value of λ index were associated with the removal of column C3 at the inner corner of the plan.

In Plan (c), the maximum vertical displacement and the maximum value of λ index was associated with the removal of columns at the outer corners of the plan (B5 and A1). Thus, the most critical removal scenario belonged to removal of columns at the outer corners of the plan. The maximum vertical displacement in the case of corner columns removal was associated with removing column B5. The maximum vertical displacement in this removal case was 23% larger than the one in the case of removing external column C1, 33% larger than the one in the case of removing external column C3, 22% larger than the one in the case of removing internal C2 and 36% larger than the one in the case of removing column B3 at the inner corner of the plan. Also, λ index associated with the removal of external columns was larger compared to the λ index associated with the removal of internal columns. Thus, removing external columns was more critical than removing internal columns. The minimum displacement and the minimum value of λ index were associated with the removal of column B3 at the inner corner of the plan.

By comparing the maximum and ultimate vertical displacement of the top node of the removed columns with identical conditions and location in all three Plan (a), Plan (b) and Plan (c) (such as columns A1 and C1), it was observed that the maximum and ultimate vertical displacement of the top node of the removed columns were similar in all plans and did not significantly change. For instance, the maximum vertical displacement due to removing column A1 in Plan (a), Plan (b) and Plan (c) were only about 2% and 3% different, respectively. Thus, changing the geometric shape of plans and inserting reentrant corner irregularities in plans, did not significant effect on the progressive collapse potential of building structures.

After a sudden load-bearing element removal, the loads were redistributed to the adjacent members. In this section, variation of axial force in the adjacent columns of the removed element was investigated. Table 6 shows the axial force in the adjacent columns before and after columns removal. In addition, the β index was calculated for adjacent columns in all scenarios that can be used to identify the most critical adjacent columns. According to Table 6, in Plan (a) and Plan (b), the maximum value of the β index is associated with the adjacent external column in the case of removing external column. In other words, the most critical load redistribution occurs when an external column was removed. In the Plan (c), the maximum value of β index belonged to adjacent column in case of removing corner column. It was found that when a removed column had both internal and external adjacent columns, the β index for external adjacent columns was larger than the one in internal adjacent columns. In other words, the load redistribution in external adjacent columns was more critical than in internal adjacent columns.

After a sudden column removal, performance of its adjacent columns was assessed by axial-moment interaction curves shown in Figure 7. Any point inside these curves represents the combination of moment and axial force that does not result in column failure. According to the ultimate values of axial forces and moments of adjacent columns of the removed columns, the corresponding points of ultimate axial force and moment are inside the axial-moment interaction curves. Hence after sudden columns removal and load redistribution, the adjacent columns were not failed.

Floor Plans with and without Torsional Irregularity

Table 7 presents the locations of the removed columns, the vertical displacement of the top node of the removed columns, and values of the λ index for plans (d) and (e). In addition, Figures 8 and 9 display the vertical displacement of the top node of the removed shear walls after sudden removal in regular and torsional irregular models.

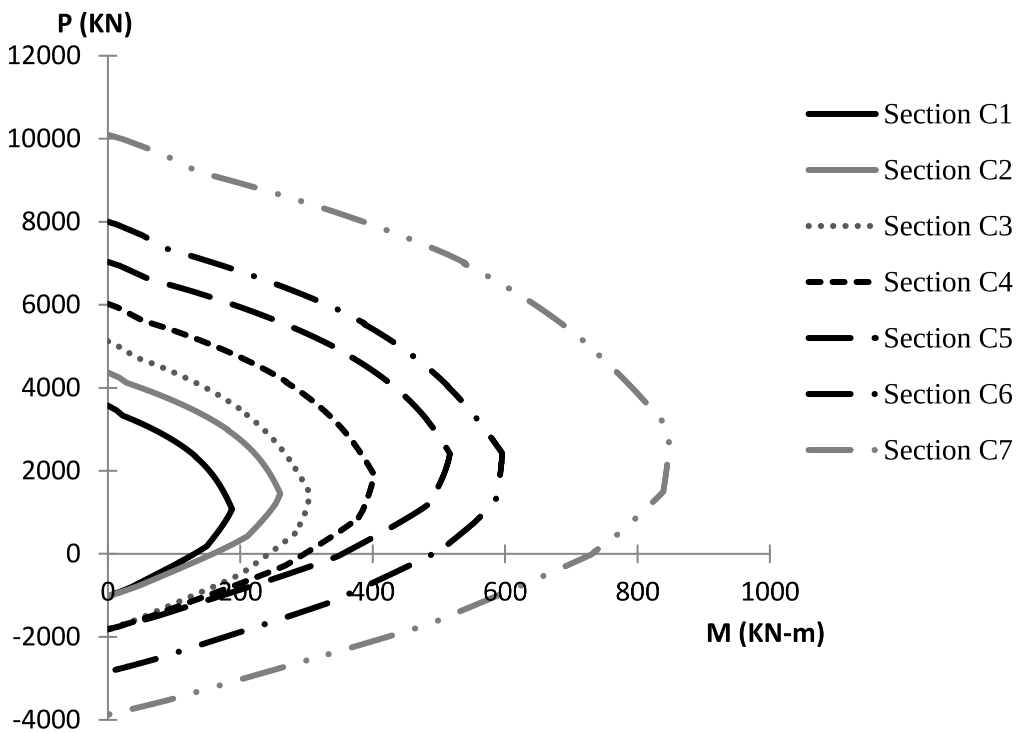


Fig. 7. Axial-moment interaction curves of the columns cross-sections

Table 6. Axial force of adjacent columns and β index for Plans (a, b and c)

Plan	Removal Location	Adjacent Column	Before Removal (KN)	After Removal Maximum (KN)	After Removal Ultimate (KN)	Ultimate Moment (KN-m)	β
(a)	A1	B1	902.6	1489.0	1242.0	31.2	1.65
	C1	B1	902.6	1532.5	1260.3	38.7	1.70
		C2	1308.1	1882.1	1539.2	40.8	1.44
	C3	B3	1338.2	1986.4	1680.9	50.5	1.48
		C4	1335.1	1972.9	1673.8	57.4	1.47
(b)	A1	B1	902.4	1490.5	1246.9	30.0	1.65
	C5	B5	907.5	1505.6	1297.1	18.8	1.66
		C4	906.4	1478.9	1245.8	34.4	1.63
	C1	B1	902.4	1534.1	1262.8	40.9	1.70
		C2	1307.9	1886.6	1594.4	36.8	1.44
	C4	C5	575.3	1108.0	893.0	52.1	1.93
		B4	1338.8	1908.6	1632.4	25.9	1.43
	B2	B1	902.4	1507.2	1229.4	59.5	1.67
		C2	1307.9	1985.4	1667.0	62.0	1.52
	C3	B3	1337.1	1953.0	1650.0	43.4	1.46
		C4	906.4	1474.6	1227.6	38.5	1.63
(c)	A1	B1	895.3	1428.1	1238.7	31.1	1.65
	B5	B4	906.4	1518.9	1305.9	24.6	1.67
		A5	574.1	1099.5	890.2	37.5	1.91
	C1	B1	895.3	1522.3	1253.0	39.5	1.7
		C2	1316.2	1883.8	1600.8	33.1	1.43
	C3	C2	1316.2	1882.3	1618.1	29.1	1.43
		B3	1212.8	1805.0	1534.9	29.3	1.49
	C2	C1	892.0	1461.1	1193.7	51.3	1.64
		B2	1347.7	2001.3	1695.6	49.6	1.48
	B3	B2	1347.7	1956.9	1659.8	41.8	1.45
		B4	906.4	1477.3	1229.3	38.8	1.63

Table 7. Vertical displacement of the top node of the removed columns and "λ" index for Plans (d and e)

Plan	Removal Location	Before Removal (mm)	After Removal Maximum (mm)	After Removal Ultimate (mm)	λ
(d)	A1	0.271	39.15	26.47	97.7
	B1	0.425	35.64	25.17	59.2
	A3	0.433	29.36	19.18	44.3
	C4	0.645	31.64	21.39	33.1
(e)	A1	0.193	31.42	19.92	103.2
	B1	0.301	32.86	21.14	70.2
	A3	0.303	24.53	15.76	52.0
	C4	0.527	29.69	19.73	37.4

In Plan (d), the maximum and ultimate vertical displacement was associated with the removal of corner column. The maximum vertical displacement after removing the corner column was about 10% larger compared to the one in removing the external column B1, 33% larger compared to the one in removing the external column A3 and 24% larger compared to the one in removing the internal column C4. The maximum value of λ index was associated with the removal of corner column. Hence, removing the corner column was the most critical column removal scenario, since in this scenario the structure lost more gravity stiffness compared to the other scenarios. The maximum vertical displacement due to the removal of external column in the small dimension of the plan was 12% larger compared to the removal of internal column. Additionally, the maximum vertical displacement due to the removal of external column in the long dimension of the plan was 7% lower compared to the removal of internal column. However, λ index for external columns was larger than the one for internal columns. Therefore, removing external columns was more critical than removing internal columns.

In Plan (e), the maximum vertical displacement was associated with the removal of external column in the small dimension of the plan. The maximum vertical displacement after removing this column was almost 5% larger than the one in the case of removing corner column A1, 34% larger than the one in the case of removing external column A3, and 10% larger than the one in the case of removing internal column C4. Although, the maximum value of λ index was associated with the removal of corner column. Also λ index for external columns was larger than the one for internal columns. Hence, in Plan (e), the most critical column removal scenario belonged to removal of corner column. As such, removing external columns was more critical than removing internal columns.

The Iranian seismic code applies more rigorous provisions for designing torsional irregular structures than regular structures. For designing buildings with a torsional irregular floor plan, the value of redundancy factor ρ is considered 1.2 in accordance with provisions of this code. Thus, the designing seismic coefficient of a torsional floor plan is 20% larger than the one in the regular floor plan. Accordingly, cross-sections of columns and shear walls have undergone some modifications in a building with a torsional irregular floor plan in comparison to the regular floor plan. In addition, cross-sections of internal and external beams have undergone major modification and reinforcement bars of beams have increased. By modifying the cross section of the members, especially the cross-sections of the beams, the vertical displacement of the top node of the removed element decreased. In Plan (e), the maximum vertical displacement after removing the corner column A1 was about 20% lower than the one in Plan (d). Furthermore, the maximum vertical displacement after removing the external column A3, external column B1 and internal column C4 was about 16%, 8% and 6% lower than the one in Plan (d), respectively. In fact, progressive collapse potential of Plan (e) has decreased compared to Plan (d).

As shown in Figures 8 and 9 with sudden removal of shear wall in s (d) and (e), the vertical displacement of the top node of the removed shear wall increased continuously without converging and the structure could not reach a stable stage.

Comparison of the maximal vertical displacement of the top node of the removed columns with removed shear walls indicated that sudden removal of shear walls created more critical situations for progressive collapse compared to sudden columns removal. According to Tables 5 and 7 after sudden column removal in all plans the structures could converge to a ultimate stable stage whereas after sudden shear wall removal in Plans (d) and (e) the structures could not converge to a stable stage. Thus, there should be a special attention in designing reinforced concrete structures with shear walls to properly resist against progressive collapse.

The variation of axial forces in adjacent columns of the removed columns in Plan (d) and Plan (e) were investigated. Table 8 shows the axial force in the adjacent columns before and after columns removal. Also, the β index was calculated for adjacent columns in all scenarios.

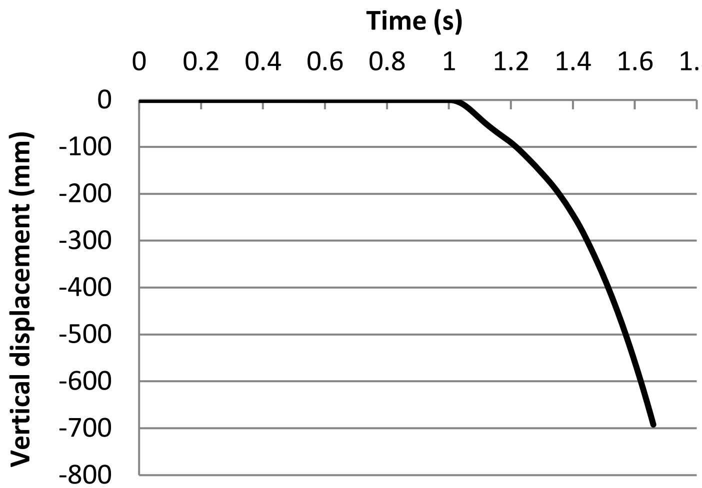


Fig. 8. Vertical displacement at the top node of the removed shear wall in axis 6 of Plan (d)

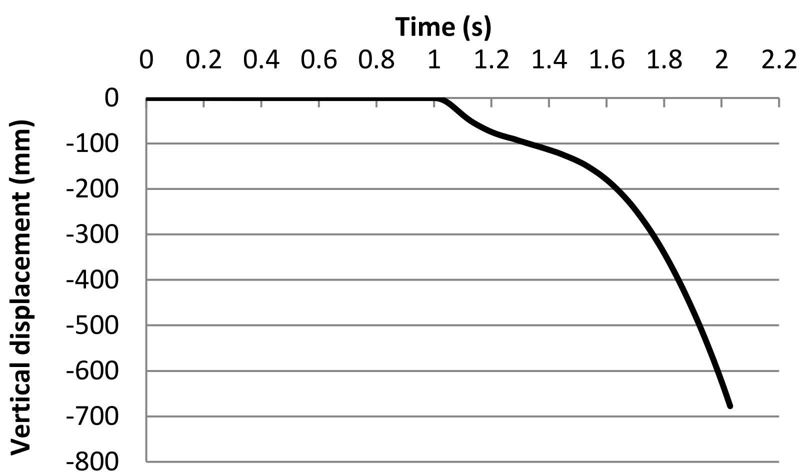


Fig. 9. Vertical displacement at the top node of the removed shear wall in axis 6 of Plan (e)

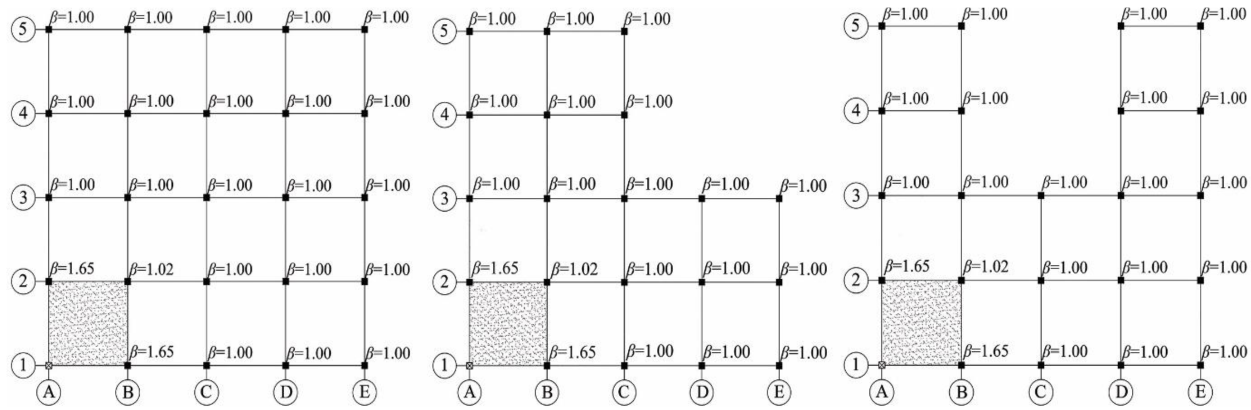
According to Table 8, the maximum value of β index in both Plans (d) and (e) was associated with the case of removing external columns. In other words, the most critical load redistribution occurred in the removal of external columns. Further, β index revealed that, in general the columns that had both external and internal adjacent columns, load redistribution in external adjacent columns was more critical than the one in internal adjacent columns.

Load Propagation Pattern after Sudden Column Removal

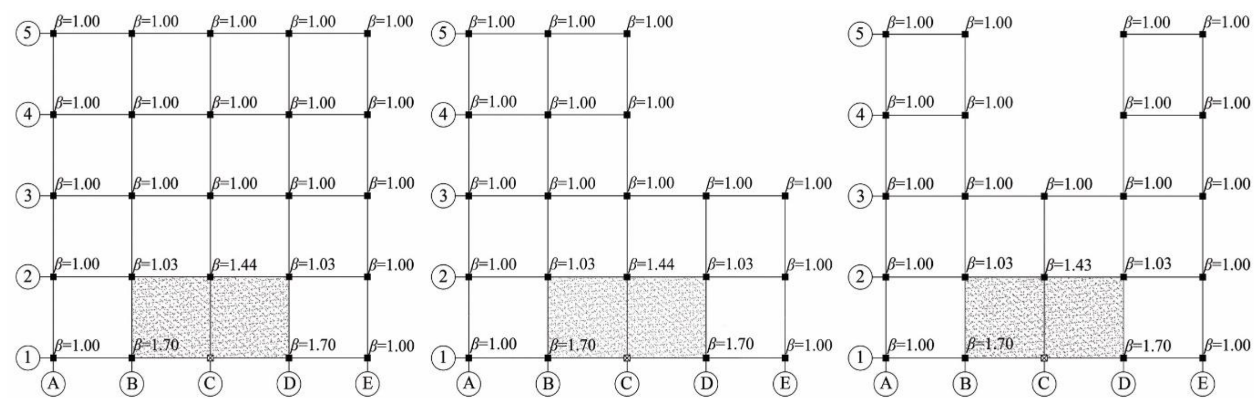
For investigating the load propagation pattern after sudden column removal in different plans, the β index was calculated for all columns of the plans after column removal. If the value of β index of every column in the plans was greater than 1, it indicated that it was affected by the load redistribution due to sudden column removal. For each plan, the load propagation pattern in different cases of column removal including sudden removal of corner columns, internal columns and external columns were examined.

In Figure 10 the value of β index for all columns of the plans after a sudden column removal and the area affected by the load redistribution are shown. According to Figure 10 the load propagation patterns in Plans (a),

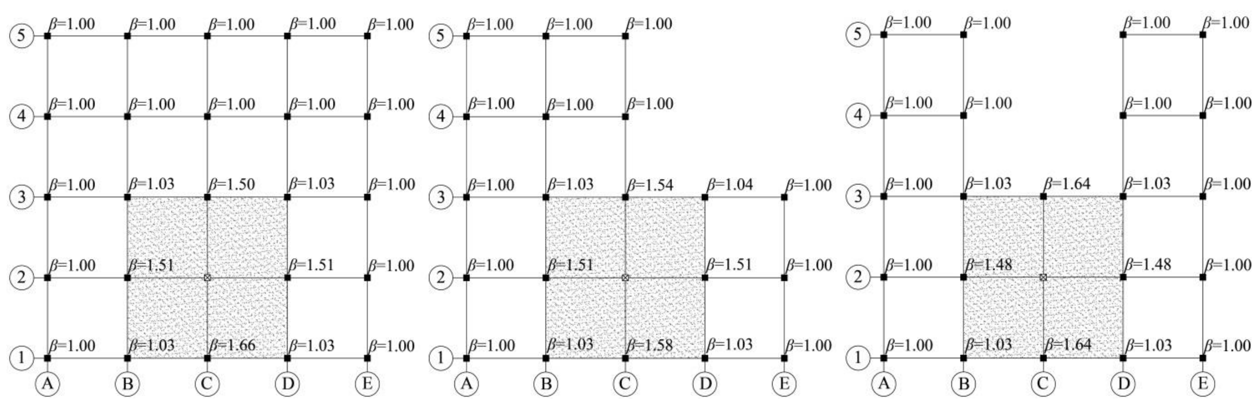
(b) and (c) as well as in Plans (d) and (e) in all column removal scenarios were similar. Hence, it can be concluded that regardless of the shape of floor plan, a single column removal only affected the connected panel(s) to removed column.



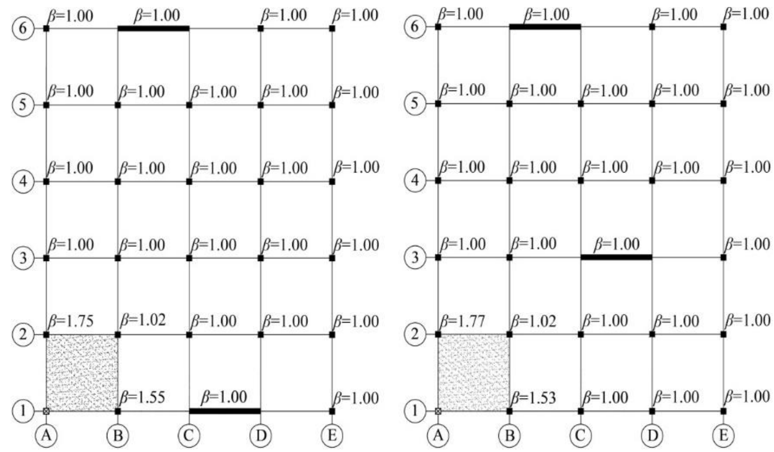
(a) The β index in geometrical regular and irregular floor plans when A1 column removed



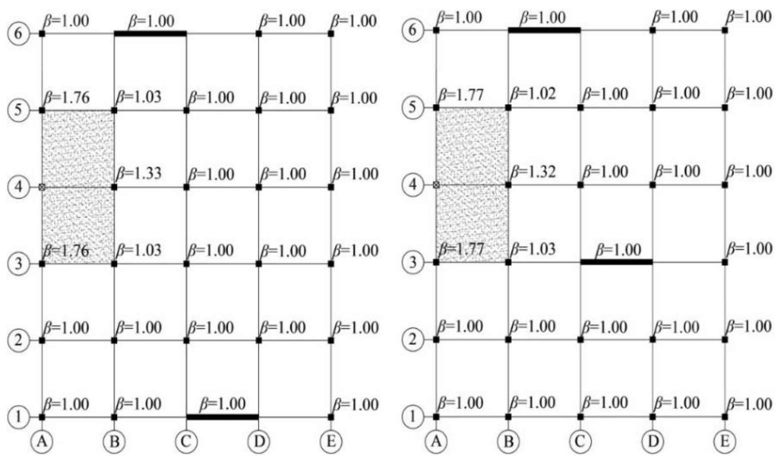
(b) The β index in geometrical regular and irregular floor plans when C1 column removed



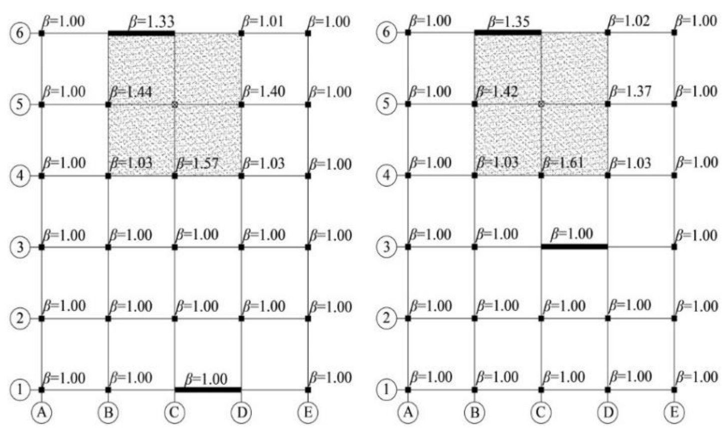
(c) The β index in geometrical regular and irregular floor plans when C2 column removed



(d) The β index in floor plans with and without torsional irregularity when A1 column removed



(e) The β index in floor plans with and without torsional irregularity when A4 column removed



(f) The β index in floor plans with and without torsional irregularity when C5 column removed

Fig. 10. The β index in plans after column removal

a) The β index in geometrical regular and irregular floor plans when A1 column removed
a) The β index in geometrical regular and irregular floor plans when C1 column removed
b) The β index in geometrical regular and irregular floor plans when C2 column removed
c) The β index in floor plans with and without torsional irregularity when A1 column removed
d) The β index in floor plans with and without torsional irregularity when A4 column removed
e) The β index in floor plans with and without torsional irregularity when C5 column removed

Table 8. Axial force of adjacent columns and β index for Plans (d and e)

Plan	Removal Location	Adjacent Column	Before Removal (KN)	After Removal Maximum (KN)	After Removal Ultimate (KN)	Ultimate Moment (KN-m)	*β*
(d)	A1	B1	864.2	1340.0	1124.9	11.6	1.55
		A2	885.1	1546.3	1278.1	45.7	1.75
	B1	A1	554.7	1003.4	808.6	14.0	1.81
		B2	1333.9	1986.6	1718.0	35.6	1.49
	A3	A2	885.1	1560.1	1261.3	49.9	1.76
		B3	1308.0	1756.0	1523.9	21.0	1.34
	C4	B4	1330.1	1830.8	1594.3	24.1	1.38
		C5	1316.3	2059.8	1728.8	40.2	1.56
(e)	A1	B1	905.1	1382.7	1166.2	17.7	1.53
		A2	912.2	1611.1	1300.8	42.0	1.77
	B1	A1	581.7	1083.0	858.7	17.7	1.86
		B2	1333.1	2084.1	1739.2	45.4	1.56
	A3	A2	912.2	1615.3	1296.9	52.5	1.77
		B3	1316.6	1751.1	1515.1	24.2	1.33
	C4	B4	1337.2	1815.9	1584.8	30.2	1.36
		C5	1329.9	2129.2	1766.9	51.3	1.60

CONCLUSIONS

In this research, progressive collapse potential due to a sudden removal of vertical load-bearing element was investigated in geometrical regular and irregular floor plans as well as floor plans with and without torsional irregularity in reinforced concrete buildings. Columns and shear walls were removed in the various locations of the studied buildings' first floor and analyzed using nonlinear dynamic analysis. Results of this study led to the following conclusions:

- In buildings with geometrical regular and irregular as well as with and without torsional irregularity floor plan, the most critical column removal scenario was associated with the removal of outer corner columns. Since, in this removal scenario the structure lost more gravity stiffness compared to the other removal scenarios. Further, removing external columns was more critical than removing internal columns.
- Sudden removal of shear wall in buildings with and without torsional irregularity indicated that the vertical displacement of the node of the removed shear wall increased continuously and irreversibility. A sudden shear walls removal created more critical situations for the building compared to sudden columns removal.
- By changing the geometric shape of the PlanS (a), (b) and (c) and inserting reentrant corner irregularities in plans, the progressive collapse potential of building structures did not vary significantly.
- Progressive collapse potential of a building with a torsional irregular floor plan which was designed according to Iranian seismic code, is lower than a building without torsional irregularity.
- When a removed column had both external and internal adjacent columns, the load redistribution in external adjacent columns was more critical than the one in internal adjacent columns.
- For buildings designed using ACI318-14 and Iranian seismic code, regardless of the shape of floor plan, a single column removal only affected the connected panel(s) to removed column.

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.

American Concrete Institute (ACI). (2014). *Building code requirements for structural concrete and commentary,* (ACI 318-14), Detroit, Michigan.

Arshian, A.H., Morgenthal, G. and Narayanan, S. (2016). "Influence of modelling strategies on uncertainty propagation in the alternate path mechanism of reinforced concrete framed structures", *Engineering Structures*, 110, 36-47.

Arshian, A.H. and Morgenthal, G. (2017). "Three-dimensional progressive collapse analysis of reinforced concrete frame structures subjected to sequential column removal", *Engineering Structures*, 132, 87-97.

ASCE/SEI41, (2006). *Seismic evaluation and retrofit of existing buildings (ASCE/SEI 41-06)*, American Society of Civil Engineers, New York.

Department of Defense (DoD). (2016). *Design of Building to Resist Progressive Collapse, Unified Facilities Criteria (UFC 4-023-03)*, Washington D.C.

Ellingwood, B.R. (2006). "Mitigating risk from abnormal loads and progressive collapse", *Journal of Performance of Constructed Facilities*, 20(4), 315-323.

General Service Administration (GSA). (2013). *Alternate Path Analysis and Design Guidelines for Progressive Collapse Resistance*, Washington D.C.

Ghahremannejad, M. and Park, Y. (2016). "Impact on the number of floors of a reinforced concrete building subjected to sudden column removal", *Engineering Structures*, 111, 11-23.

Helmy, H., Salem, H. and Mourad, S. (2012). "Progressive collapse assessment of framed reinforced concrete structures according to UFC guidelines for alternative path method", *Engineering Structures*, 42, 127-141.

Iranian Code of Practice for Seismic Resistant Design of Buildings. (2014). *Standard 2800*, 4th Edition. Building and Housing Research Center.

Kim, J. and Kim, T. (2009). "Assessment of progressive collapse-resisting capacity of steel moment frames", *Journal of Constructional Steel Research*, 65(1), 169-179.

Kordbagh, B. and Mohammadi, M. (2017). "Influence of seismicity level and height of the building on progressive collapse resistance of steel frames", *The Structural Design of Tall and Special Buildings*, 26(2), e1305.

Li, M. and Sasani, M. (2015). "Integrity and progressive collapse resistance of RC structures with ordinary and special moment frames", *Engineering Structures*, 95, 71-79.

Mander, J.B., Priestley, M.J. and Park, R. (1988). "Theoretical stress-strain model for confined concrete", *Journal of Structural Engineering*, 114(8), 1804-1826.

Marjanishvili, S.M. (2004). "Progressive analysis procedure for progressive collapse", *Journal of Performance of Constructed Facilities*, 18(2), 79-85.

Martinelli, P. and Filippou, F.C. (2009). "Simulation of the shaking table test of a seven-story shear wall building", *Earthquake Engineering and Structural Dynamics*, 38(5), 587-607.

Mazzoni, S., McKenna, F., Scott, M.H. and Fenves, G.L. (2016). *OpenSees command language manual*, Pacific Earthquake Engineering Research Center, Berkeley, University of California, http://opensees.berkeley.edu.

Pachenari, A., Keramati, A. and Pachenari, Z. (2013). "Investigation of progressive collapse in intermediate RC frame structures", *The Structural Design of Tall and Special Buildings*, 22(2), 116-125.

Pearson, C. and Delatte, N. (2005). "Ronan point apartment tower collapse and its effect on building codes", *Journal of Performance of Constructed Facilities*, 19(4), 172-177.

Rahai, A., Asghshahr, M.S., Banazedeh, M. and Kazem, H. (2013). "Progressive collapse assessment of RC structures under instantaneous and gradual removal of columns", *Advances in Structural Engineering*, 16(10), 1671-1682.

Ren, P., Li, Y., Guan, H. and Lu, X. (2014). "Progressive collapse resistance of two typical high-rise RC frame shear wall structures", *Journal of Performance of Constructed Facilities*, 29(3), 04014087.

Rezvani, F.H., Yousefi, A.M. and Ronagh, H.R. (2015). "Effect of span length on progressive collapse behaviour of steel moment resisting frames", *Structures*, 3, 81-89.

Sasani, M. and Sagiroglu, S. (2008). "Progressive collapse of reinforced concrete structures: a multihazard perspective", *ACI Structural Journal*, 105(1), 96.

Shan, S., Li, S., Xu, S. and Xie, L. (2016). "Experimental study on the progressive collapse performance of RC frames with infill walls", *Engineering Structures*, 111, 80-92.

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.

Yi, W.J., He, Q. F., Xiao, Y. and Kunnath, S.K. (2008) "Experimental study on progressive collapse-resistant behavior of reinforced concrete frame structures", *ACI Structural Journal*, 105(4), 433.

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.