Anbar Journal of Engineering Science (AJES)
P-ISSN: 1997-9428; E-ISSN: 2705-7440

Vol. 12 , No. 1 (2021) , pp. 10 ~ 19
DOI:10.37649/aengs.2021.171194

Unviersty of Anbar

Anbar Journal Of Engineering Science©

journal homepage: http:// www.uoanbar.edu.iq/Evaluate/

Progressive Collapse Analyses of Buildings Subjected to Earthquake Loads

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PAPER INFO

Paper history:
Received 13/12/2020
Revised 19/01/2021
Accepted 22/02/2021

Keywords:
Progressive collapse, Concrete buildings, Seismic load, Nonlinear dynamic analysis, Plastic hinge.

ABSTRACT

Progressive collapse is a partial or total failure of a building that mostly occurs when the building loses primary structural elements (typically columns) due to accidental or natural hazards. The failure of structures due to an earthquake is one of the most important and frequent types of progressive collapse. In this study, the finite element method is used to assess the response of multistory reinforced concrete buildings subjected to column loss during an earthquake. Three-dimensional nonlinear dynamic analyses are carried out using SAP2000 V.20 program. The effects of different parameters on the progressive collapse behavior are investigated, namely: the location of the removed column within the ground floor; the method of column removal (sudden, in two-steps, and in four-steps) and the removal timing during the earthquake. It is demonstrated that the collapse occurs when all or most of the hinges at the bases of the ground floor columns reach their collapse level. The chosen column removal timing and policy affect the structural behavior considerably. It is realized that, the risk of building collapse increases when the removal timing harmonizes with the peak ground acceleration timing. Based on the adopted earthquake characteristics and building configurations, it is found that, the two steps removal scenario is the most dangerous one.

1. Introduction

Progressive collapse happens when one or more primary structural component (typically vertical load-carrying members) is failed. This failure could be because of an explosion, earthquake, vehicle impact, fire, or other human errors or natural hazards. The failure of a member in the structure leads to redistribution and transfer the load to neighboring members. If these members are not capable to resist the additional load, that part of the structure will be failed. This action continues in the structure until whole structural elements collapse or a large part of it. As a result, the overall damage is disproportionate with the initial damage [1, and 2]. Since the partial collapse of the Ronan Point Building in England in 1968 due to a gas explosion near the corner of the building on the 18th floor and the collapse of Alfred Murrah Federal building in USA in Oklahoma City 1995 due to a terrorist attack, many codes and standards discussed progressive collapse under abnormal loads. Each of them presented different approaches and design guidelines to prevent or mitigate the potential for progressive collapse in buildings. British standard (BS 8110) was the first standard to address progressive collapse. It had presented several approaches to enhance ductility, continuity, and structural redundancy [3]. American

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code (ACI 318-08) included re`quirements in the details of reinforcement recommendations to improve structural integrity and enhance ductility in the structure [4]. (ASCE 7-05) introduced a section entitled (General structural integrity), the intent of it is to improve the general structural integrity in order to carry loads around the damaged members [5]. The main two guidelines that have presented an explicit approach for progressive collapse analysis and design are General Services Administration (GSA 2003) and Department of Defense (DOD 2005). Alternate load path method (ALP) was adopted in both guidelines for progressive collapse assessment in which the structure should be capable to transfer load over the removed vertical member. According to this method, progressive collapse analysis can be linear static, nonlinear static, linear dynamic and nonlinear dynamic analysis [1, and 2].

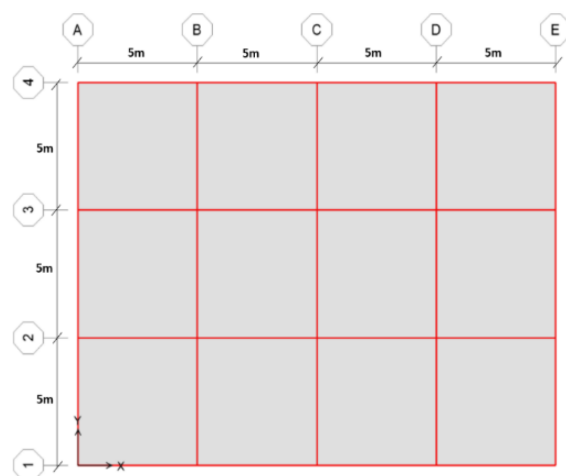
Marjanishvili, et al. [6] compared the four analysis methods for progressive collapse by analyzing 9-storey steel moment-resisting frame building using SAP2000. The results showed that the dynamic analysis method was more accurate compared to the static method. Tsai, et al. [7] evaluated the potential of an earthquake-resistant reinforced concrete building for progressive collapse under four threat independent column removal conditions by using the linear static analysis method as per GSA. The results concluded that the building had a low potential for progressive collapse. Said, et al. [8] conducted a comparison between GSA (2003) and DOD (2005). Seven-story reinforced concrete building was analyzed using a nonlinear static method. The results showed that the load percentages at which the collapse occurred adopting the DOD procedure were less than their counterparts from the GSA procedure. Patel [9] performed nonlinear static and nonlinear dynamic progressive collapse analyses for a 15-storey moment resistant reinforced concrete building designed according to Indian standards. The results showed that the nonlinear dynamic analysis procedure was the most efficient method for progressive collapse analysis. Elshaer et al. [11] studied the progressive collapse of a ten-story reinforced concrete building designed according to Egyptian code. Three-dimensional nonlinear dynamic analyses using (Applied Element Method) were conducted for a building that lost a column during an earthquake. It was proved that, the column loss under seismic load was more critical than the gravity load. The study also illustrated the important effect of including the catenary action of the slab in progressive collapse resistance.

2. Objective

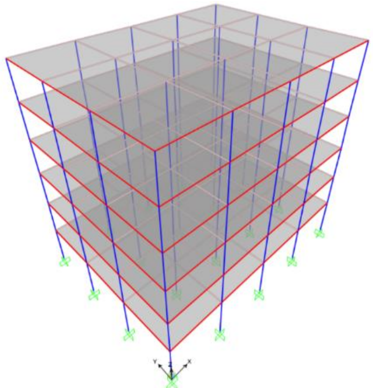
Although the progressive collapse due to seismic action is important and repeatedly occurred, still it has not received much attraction. The main objective of this study is to investigate the response of a multi-story reinforced concrete buildings subjected to sudden and gradual column loss during an earthquake. Nonlinear dynamic analysis (time history analysis) is carried out using SAP2000 program to follow the step by step progressive collapse of the building during seismic loading, and to evaluate the deformations resulting from column removal. The effects of various parameters on building response are studied such as the location of the removed column, method of column removal, and time of column removal during the earthquake.

3. Building Description

The building used in this study is a six-story reinforced concrete frame building (G+5), with four bays of (5 m) in X-direction and three bays of (5 m) in Y-direction, as shown in Figure 1. The size of the building in plan is (20 m x 15 m). The height of the ground floor is (3.5 m) and the other stories height is (3 m), giving a total height of (18.5 m). The building supports at the base are assumed as fixed supports. Concrete compressive strength is (f′c =30 N/mm^2), the yield stress of steel reinforcement is (fy = 420 N/mm^2), and the modulus of elasticity for concrete and steel are (Ec = 25.74 GPa) and (Es =200 GPa), respectively.



a. Plan dimensions



b. Three-dimensional model

Figure 1. Model for the studied building.

In addition to the self-weight of the structural components, the gravity loads applied to the building are presented in Table 1.

Table 1. Gravity loads

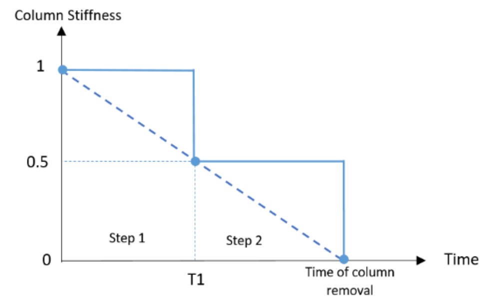
Load	Typical stories	Roof story
Dead load	2 kN/m^2	2 kN/m^2
Live load	4 kN/m^2	1.5 kN/m^2
Wall load	12 kN/m	5 kN/m

The structure is designed via SAP2000 according to ACI 318-11. The dimensions of beams and columns for all stories are taken as (300 mm x 500 mm) and (500 mm x 500 mm), respectively. Slab thickness is taken as (150 mm) with two layers of reinforcement, to study the effect of slab on the stiffness and response of the structure as well as to include catenary action provided by the slab in progressive collapse.

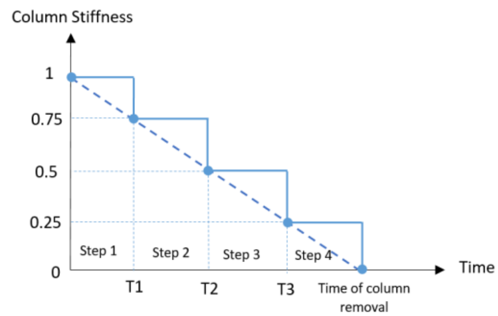
4. Progressive Collapse Analyses

4.1. Column Removal Modeling

Two column removal scenarios are used to simulate the losing of column during an earthquake (sudden and in steps). The nonlinear staged construction feature in SAP2000 is adopted to model both scenarios. This feature allows modifying the structure during nonlinear analysis. The gradual removal of the column during an earthquake is approximated by removing the column in several steps. This is done by replacing the column to be removed with many secondary columns with equivalent flexural and shear stiffness. Removing the secondary column at a specific time represents the reduction in the stiffness of the column due to the earthquake. Therefore, the removal of the last secondary column means the total failure of the column is caused by the earthquake. In this work, the column is removed in two and four steps, with a linear reduction in flexural and shear stiffnesses of the column to be removed in each step, as shown in Figure 2.



a. remove column in 2-steps



b. Remove column in 4-steps

Figure 2. Remove column in steps.

5. Analysis Procedure

Nonlinear dynamic analysis (time history analysis) is used to investigate the response of the structure for losing a column during the earthquake. In addition to seismic load, the load combination for the gravity loads is used as per (GSA 2013) and (DOD 2005) guidelines which is (1.2 DL + 0.5 LL) where (DL) is the dead load and (LL) is the live load. The earthquake used is an actual ground motion record of El Centro, California in 1940, North-South component. The building is subjected to the acceleration values in the X-direction during (10 sec) at a time interval of (0.02 sec). The time of column removal during the earthquake is assumed based on the significant change in acceleration direction. Two column removal time is used to study the effect of removing time on the response of the structure. The first time of a complete column removal is assumed at (1.2 sec) and the second time at (1.6 sec) as shown in Figure 3.

The location of the removed column is specified according to the General Services Administration (GSA) guidelines. The columns are removed from the ground floor one at a time (external column, corner column, and internal column) as shown in Figure 4. Therefore, the studied cases will be as shown in Table 2.

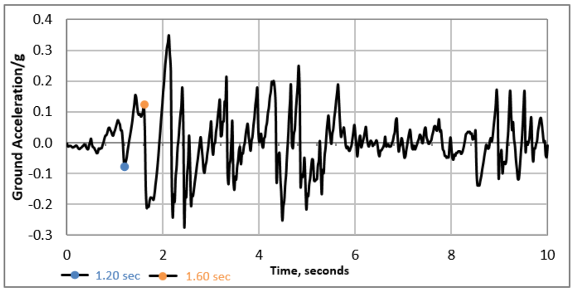


Figure 3. El Centro earthquake, California in 1940.

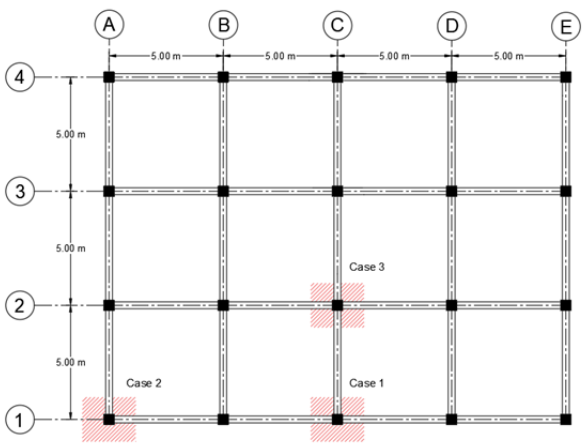


Figure 4. Location of the removed columns.

Table 2. Studied cases.

Case No.	Location of the removed column	Time of column removal	Column removal scenario
Case-1	External column (C-1)	1.2 sec	Sudden
			In 2-steps
			In 4-steps
		1.6 sec	Sudden
			In 2-steps
			In 4-steps
Case-2	Corner column (A-1)	1.2 sec	Sudden
			In 2-steps
			In 4-steps
		1.6 sec	Sudden
			In 2-steps
			In 4-steps
Case-3	Internal column (C-2)	1.2 sec	Sudden
			In 2-steps
			In 4-steps
		1.6 sec	Sudden
			In 2-steps
			In 4-steps

6. Acceptance criteria

The plastic hinge rotation capacity of beams and columns is used as an acceptance criterion to evaluate the nonlinear dynamic analysis results. The rotation capcity depends on cross-section properties and reinforcement details at hinge location. The plastic hinges are assigned at both ends of beams and columns. The plastic hinges stages can be defined by a force-displacement (moment-rotation) curve introduced by FEMA 356 [12] and ASCE 41 [14], as shown in Figure 5. SAP2000 uses (ASCE 41-13) [14] to calculate the plastic hinge rotation at each performance level. Where (IO) is immediate occupancy level, (LS) is life safety level and (CP) is collapse prevention level, and (C) is a collapse point. Points D and E represent the residual capacity and the total failure of the hinge respectively.

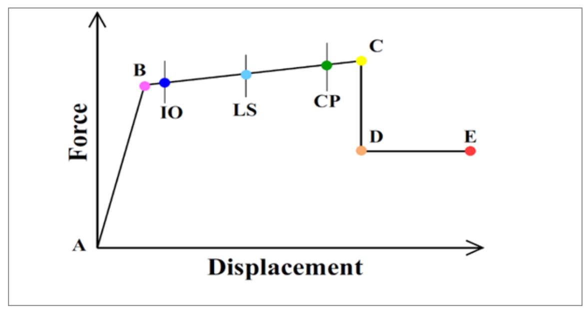


Figure 5. Plastic hinge performance levels.

7. Results and discussion

The progressive collapse analysis results of the studied cases are given as follows:

1. The distribution of plastic hinges and their performance levels at different times until the end of the earthquake or building collapse for the case of external column removal with different scenarios, and based on column removal times of (1.2 sec) and (1.6 sec) are shown in Figures 6 and 7, respectively. The failure mechanism of the other two cases, corner column and internal column removal, are similar to case-1 but the differences are in the collapse time so it is not shown.
2. The maximum base shear in X-direction, for the three locations of the removed column and removing scenarios and based on column removal times of (1.2 sec) and (1.6 sec), are shown in Figure 8. For the collapsed buildings, the values represent the maximum base shear before the collapse.
3. The maximum story displacements in X-direction before the building collapse, for the three locations of the removed column and removing scenarios and based on column removal times of (1.2 sec) and (1.6 sec), are given in Figure 9.

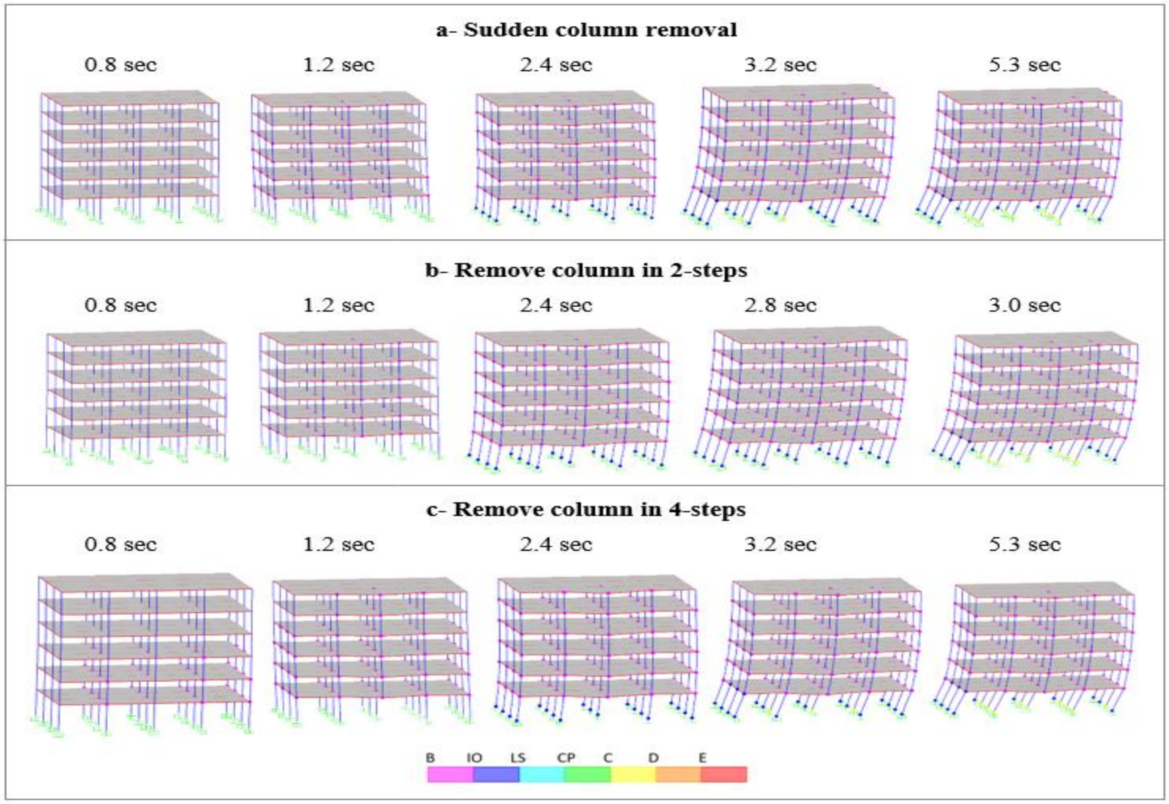


Figure 6. The distribution of plastic hinges and their performance for the column removal time of 1.2 sec (Case-1).

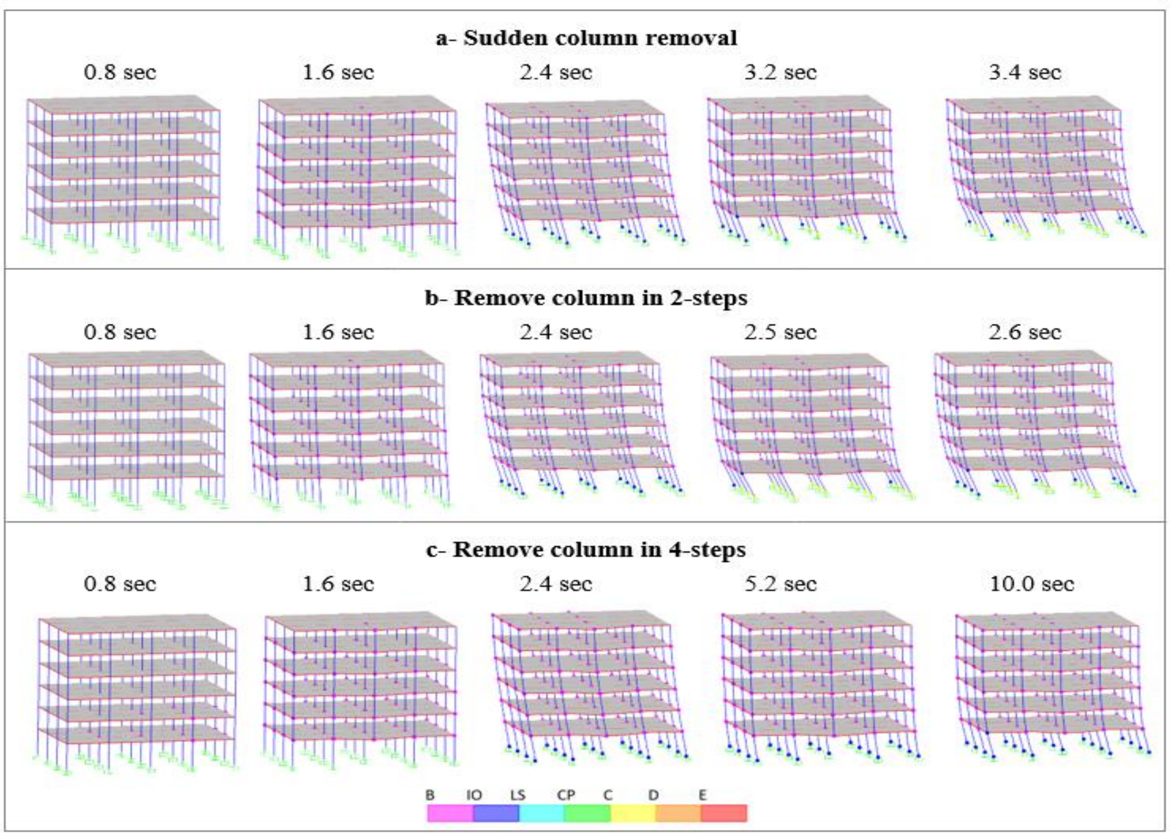


Figure 7. The distribution of plastic hinges and their performance for the column removal time of 1.6 sec (Case-1).

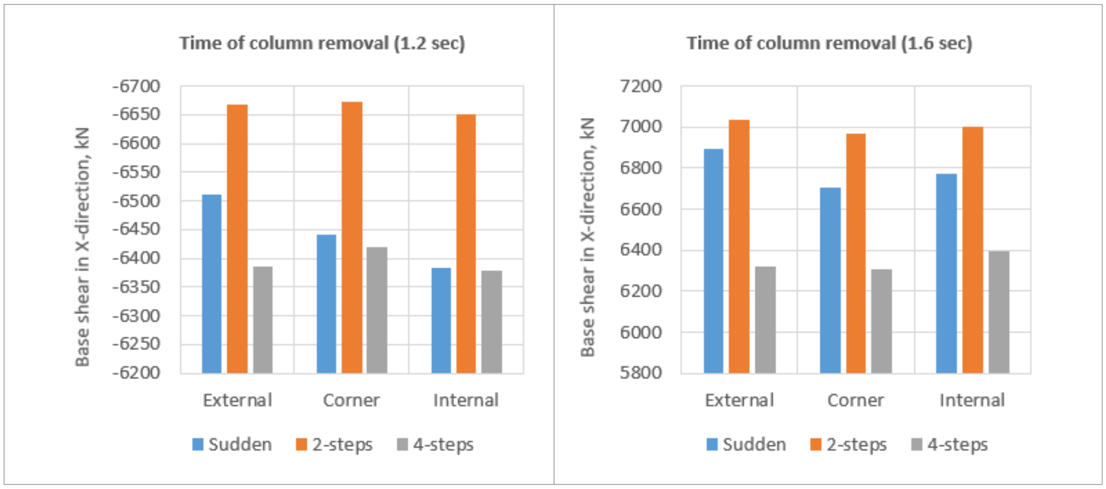


Figure 8. Maximum base shear in X-direction.

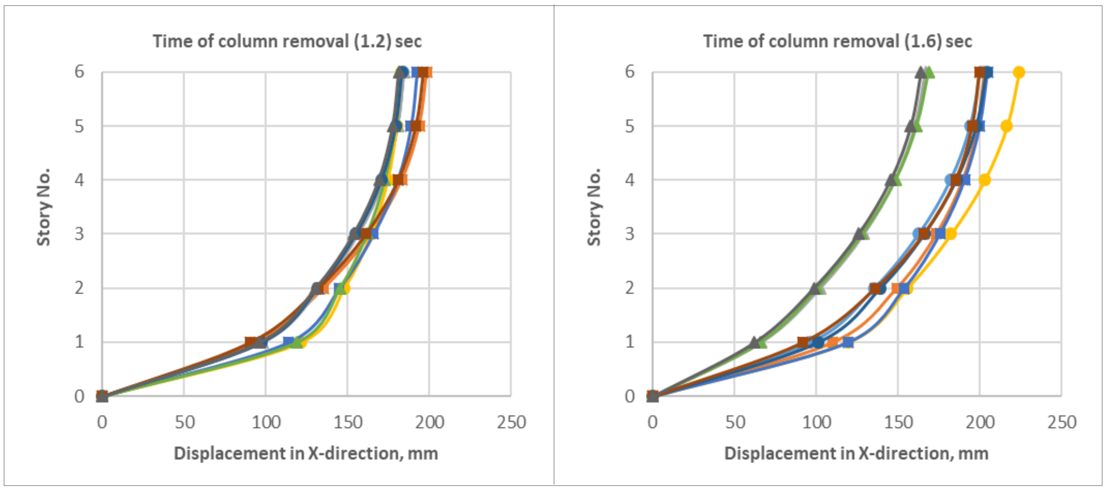


Figure 9. Maximum story displacements in X-direction.

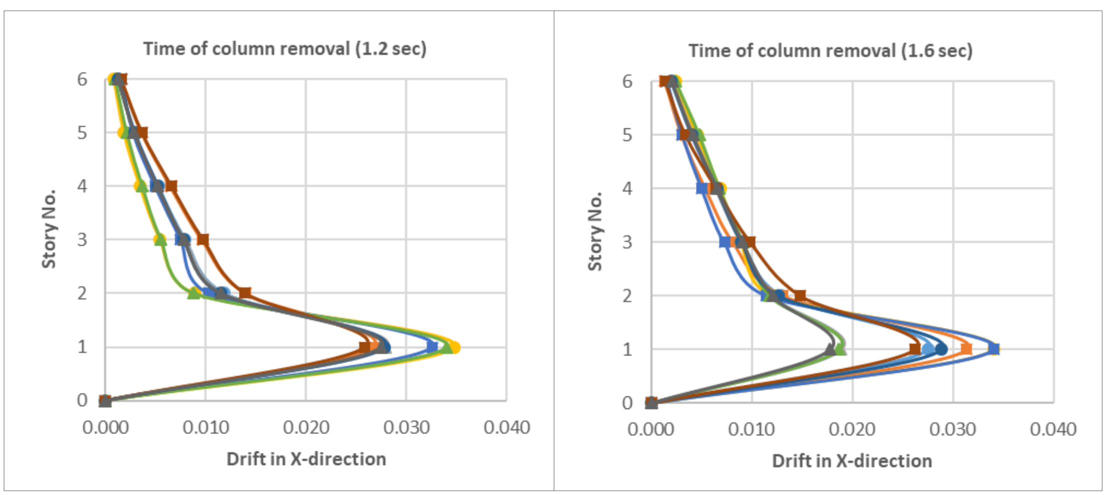


Figure 10. Maximum story drift ratio in X-direction.

Where:

4. The maximum story drift ratio in X-direction before the building collapse, for the three locations of the removed column and removing scenarios and based on column removal times of (1.2 sec) and (1.6 sec), are shown in Figure 10.

5. The maximum vertical displacement at the top joint of the removed column, for the three locations of the removed column and removing scenarios and based on column removal times of (1.2 sec) and (1.6 sec), are given in Figure 11.

6. The remaining analysis results for the three studied cases base on column removal times of (1.2 sec) and (1.6 sec), which including (collapse time, roof horizontal displacement, maximum story drift ratio, number of plastic hinges, their performance levels before the building collapse, rotations of plastic hinges at the base of ground floor columns adjacent to the removed column that are presented in Table 3, and percentages of increase in axial load after column removal for the adjacent columns), are summarized in Table 4 and 5, respectively.

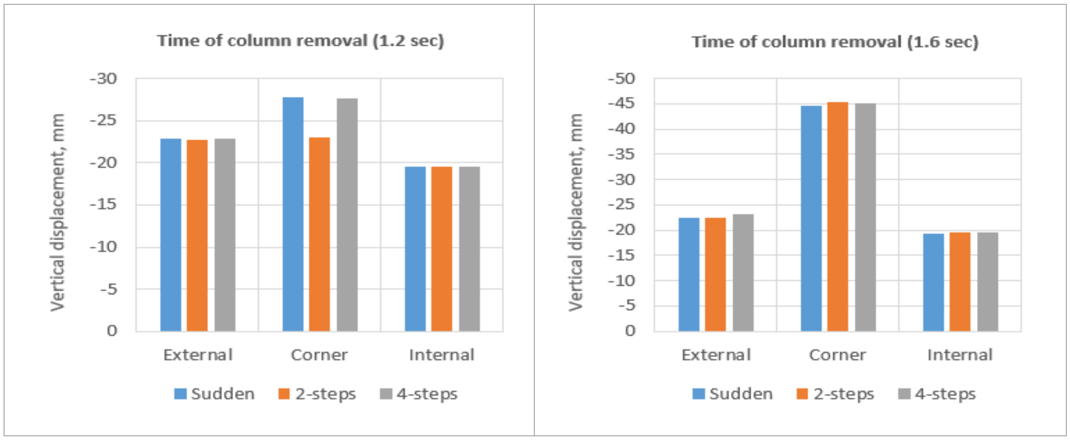


Figure 11. Maximum vertical displacement at the top joint of the removed column.

Table 3. Location of columns adjacent to the removed column.

Removed Column (x , y)	Neighboring columns. Left col. (x-5, y)	Right col. (x+5, y)	Above col. (x, y+5)	Below col. (x, y-5)
C-1	B-1	D-1	C-2	-
A-1	-	B-1	A-2	-
C-2	B-2	D-2	C-3	C-1

Table 4. Analysis results for the three studied cases base on column removal time of (1.2 sec).

Parameter		Column removal scenario								
		Sudden			2-Steps			4-Steps		
		1	2	3	1	2	3	1	2	3
Collapse time (sec)		5.3	8.3	5.3	3.0	3.6	3.0	5.3	8.4	5.3
Roof horizontal disp. (mm)		184	183	183	199	192	196	185	182	181
Max. story drift ratio		0.028	0.035	0.028	0.027	0.033	0.026	0.028	0.034	0.028
Number of plastic hinges	B-IO	281	266	286	269	257	281	273	262	281
	IO-CP	33	13	35	39	21	41	32	17	34
	> CP	9	26	10	8	22	6	9	22	10
Column at x-5 , y	Rotation (rad)	0.025	-	0.026	0.023	-	0.022	0.026	-	0.025
	Load increase (%)	28.06	-	21.05	28.29	-	19.76	22.52	-	15.52
Column at x+5 , y	Rotation (rad)	0.025	0.036	0.028	0.024	0.033	0.024	0.026	0.035	0.024
	Load increase (%)	41.22	48.33	26.88	35.55	40.00	22.75	33.05	42.72	20.02
Column at x , y+5	Rotation (rad)	0.027	0.033	0.028	0.025	0.031	0.024	0.027	0.032	0.027
	Load increase (%)	22.41	34.41	23.41	20.43	32.85	20.70	17.86	30.48	16.28
Column at	Rotation (rad)	-	-	0.028	-	-	0.024	-	-	0.028

x , y-5	Load increase (%)	-	-	31.16	-	-	29.39	-	-	22.41

Table 5. Analysis results for the three studied cases base on column removal time of (1.6 sec).

Parameter		Column removal scenario								
		Sudden			2-Steps			4-Steps		
		1	2	3	1	2	3	1	2	3
Collapse time (sec)		3.4	3.5	3.4	2.6	3.2	2.5	-	-	-
Roof horizontal disp. (mm)		200	224	204	203	205	200	167	169	164
Max. story drift ratio		0.028	0.034	0.029	0.031	0.034	0.026	0.019	0.019	0.018
Number of plastic hinges	B-IO	286	290	291	282	285	290	298	303	304
	IO-CP	37	17	30	35	25	46	37	35	35
	> CP	10	25	16	20	26	6	0	0	0
Column at x-5 , y	Rotation (rad)	-0.025	-	-0.027	-0.031	-	-0.023	-0.016	-	-0.015
	Load increase (%)	29.69	-	21.97	26.34	-	19.23	33.68	-	20.57
Column at x+5 , y	Rotation (rad)	-0.025	-0.034	-0.029	-0.031	-0.035	-0.024	-0.016	-0.016	-0.014
	Load increase (%)	40.44	44.82	26.35	39.08	47.91	24.58	22.61	22.44	15.54
Column at x , y+5	Rotation (rad)	-0.027	-0.033	-0.029	-0.032	-0.034	-0.024	-0.014	-0.015	-0.014
	Load increase (%)	22.16	35.17	23.43	20.53	33.43	20.68	17.90	30.43	16.17
Column at x , y-5	Rotation (rad)	-	-	-0.029	-	-	-0.024	-	-	-0.014
	Load increase (%)	-	-	33.43	-	-	29.08	-	-	22.19

For all studied locations, times, and removal scenarios, it is observed that the mechanism of progressive collapse under the earthquake effect is started by the plastic hinges formation in the beams of the ground floor and their propagation towards the beams of the other stories, as the earthquake persist. Then, the plastic hinges are formed at the bases of ground floor columns. When these hinges are reached to immediate occupancy level (IO), the plastic hinges are started to form at the top joint of the ground floor columns and some columns of the other stories. The collapse plastic hinges are started to form at the bases of the ground floor columns first, when the top hinges reached to immediate occupancy level, where the building is collapsed when most hinges at the bases of ground floor columns are reached to collapse levels as shown in Figures 6 and 7. It is apparent from Figure 8 that, the maximum base shear for the (2-steps) removal scenario is greater than their counterparts for other scenarios for all locations of removed column and removal times. It is greater than sudden and (4-steps) removal scenarios with an average by (3.6%), for column removal time of (1.2 sec). It is greater than sudden and (4-steps) removal scenarios with an average by (3.1%) and (10.4%), respectively, for column removal time of (1.6 sec). It is evident from Figure 10 that, the maximum story drift before the building collapses is occurred at the ground floor in all times and scenarios of column removal, due to the formation of collapse plastic hinges at the column bases of this story. Corner column removal has given the maximum vertical displacement at the top joint of the removed column in all times and scenarios of column removal as shown in Figure 11. The vertical displacement at the top joint of the removed column has not affected much by the column removal scenario. It is observed form Tables 4 and 5 that, removing the column in two steps records the fastest collapse, for all studied locations of removed column and removal times. In case of removing the column in (4-steps) at (1.6 sec), no collapse hinges are detected and the building has not collapsed, regardless of the location of the removed column. The number and distribution of plastic hinges at different times of the earthquake, for the three-column removal locations and scenarios, are different. This is attributed to the intended stiffness reduction of the column to be removed and its synchronization with the applied acceleration. The rotation values of plastic hinges at the bases of columns adjacent to the removed column are approximately equal for each column removal scenario. The percentages of increase in axial load after column removal for the adjacent columns reflect the contribution of each column in the redistribution of the load carried by the removed column.

8. Conclusions

Based on the results that have been obtained from this study, the following conclusions can be drawn:

I- The selected column removal time and scenario greatly affect the response of building, because they are influenced by the variable applied excitation. It is not possible to isolate their effects, except for an idealized loading but not a random one like an earthquake.

II - Based on the used building configuration and the earthquake characteristics, the two steps column removal scenario is the most dangerous scenario for all locations of the removed column.

III- The building collapses during an earthquake when all or most of the hinges at the bases of the ground floor columns reached to collapse level.

IV- Irrespective of the removed column location, the formation of collapse plastic hinges is started from the bases of the internal columns of the ground floor. Those columns usually carry higher axial loads and have less rotation capacities.

V- Although, removing the column from long direction or the interior column, produces the most intensive effect, the slight differences in number of plastic hinges for the various locations of the removed columns, do not control collapse time.

VI- The roof horizontal displacements and the maximum drift ratios were almost similar. Removal of the corner column has the strongest effect on the vertical displacement of the top joint of the removed column.

VII - Column removal leads to slight and almost similar rotations and redistributed uneven load increases in the adjacent columns. The scenario of removal does not affect the rank of columns regarding load increase ratios.

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