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Progressive Collapse Assessment of Multistory Reinforced Concrete Structures Subjected To Seismic Actions

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Abstract

Progressive collapse is a catastrophic partial or total failure of a structure that mostly occurs when a structure loses a primary component like a column. Some international standards have started to consider progressive collapse resistance in various approaches. In this study, the 'Unified Facilities Criterion' guidelines were used in assessing the structure; these guidelines represent one of the codes that discuss progressive collapse using sophisticated approaches. Three-dimensional nonlinear dynamic analyses using the 'Applied Element Method' were performed for a structure that lost a column during a seismic action. A parametric study was made to investigate the effect of different parameters on progressive collapse. In this study, a primary structural component was assumed lost during an earthquake. The studied parameters were the location of the removed column in plan, the level of the removed column, the case of loading, and the consideration of the slabs. For the study cases, it was concluded that the buildings designed according to the Egyptian code satisfies the progressive collapse requirements stated by 'Unified Facilities Criteria' (UFC) guidelines requirements with a safety factor of 1.97. Also, it was found that losing a column during a seismic action is more critical for progressive collapse than under gravity load. Finally, this study elaborated the importance of considering the slab in progressive collapse analysis of multistory buildings in order to include the significant catenary action developed by the slabs.

Keywords: *progressive collapse, seismic loads, applied elements, catenary action, UFC*

1. Introduction

Progressive collapse has become a hot research point as a result of several recent collapses. The first recorded collapse in the last century was the failure of the masonry 'St. Mark's Campanile' in July 1902. The failure was due to the deterioration of the structure after experiencing some cracks that resulted from a preceding fire. The first well-known collapse was on the morning of 16 May 1968 in the 'Ronan Point apartment', which was a 22-story building that failed due to a gas explosion on the 18th floor. This explosion caused a failure in the load-bearing precast concrete panels near the corner of the building. The structure experienced a chain of collapses until the ground floor. Finally, the most famous progressive collapse event ever was the failure of the twin towers of the 'World Trade Center' building on September 11, 2001. The reason for the failure was that when the 'Boeing 767 jetliner' crashed into the tower at high speed, the crash caused structural damage at and near the point of impact and set off an intense fire within the building. The fire weakened the steel structure until the trusses started to sag. This sagging converted the downwards pull of the trusses into an inwards pull and the weight and impact of the collapsing upper part of the tower caused a progression of failures extending downward all the way to the ground. In addition, many structures have experienced progressive collapse due to seismic actions in our modern history for instance, the dramatic collapse of 'The Kaiser Permanente Building' after the 'Northridge earthquake'. Moreover, in 2008, the Beichuan branch of the 'Agriculture Development Bank in China' (ADBC) suffered a progressive collapse after the Beichuan earthquake.

Progressive collapse is defined by the (ASCE/SEI 7-05, 2005) as "the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it". This loss could be because of a car accident, explosion of a service system, aircraft crash, bomb, missile in a military action, hurricane, tornado, or earthquake, the latter will be the scope of this study. Structures are exposed to interior loading, such as self-weight and occupancy weight, and to exterior loading, such as wind or seismic loads. Normal loads are usually considered directly or indirectly in the design process through existing codes and standards, while abnormal loads are not considered in many general design codes, and they are rarely considered in design practices, in spite of the probability for such loading to lead to catastrophic progressive collapses. Progressive collapse is a dynamic process wherein a collapsing system continually seeks alternative load paths in order to survive.

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Mechanisms that could contribute to the capacity of a system to resist collapse include: 1) Catenary action of slab and beams allowing gravity load to span adjacent elements, 2) Vierendeel action from the moment frame above a damaged column, 3) support provided by non-structural elements, such as partitioning walls. In this study, the contribution of non-structural elements will be neglected due to the difficulty in monitoring their quality and uniformity within different buildings.

2. Objective

Progressive collapse due to seismic actions has not received much attention in spite of its importance and repeated occurrence. The objective of the current research is to study and assess the progressive collapse of reinforced concrete structures designed according to the 'Egyptian code for Design and Construction of Reinforced Concrete Structures' (EC, 2007). The limits for deformations were defined by the 'Unified Facilities Criteria' (UFC, 2009) guidelines for buildings designed to resist progressive collapse. The study also investigates the effect of different parameters on the deformations resulting from the column loss.

3. UFC Guidelines

(UFC 2009) guidelines introduced two design approaches. The first one is the direct design approach, which includes the 'Alternative Path Method' (APM) and the 'Specific Local Resistance' (SLR), while the other one is the indirect design approach, which is called the 'Tie Force method' (TF). In APM, the structure should be capable of bridging over missing structural elements, while the Tie Force approach enhances continuity, ductility, and structural redundancy by requiring ties to keep the structure together in the event of an abnormal loading. Concerning the SLR, it was found from the literature that it is one of the most computationally expensive approaches. It was stated by Magallanes *et al.* (2015): "Specific Local Resistance (SLR) Approach – is a direct design approach that is

Table 1. Materials Properties

Material	Young's modulus (MPa)	Compressive strength (MPa)	Tensile strength (MPa)	Yield stress (MPa)	Ultimate strength (MPa)
Concrete	22,135	25	2	---	---
Steel reinforcement	200,000	---	---	360	520

Table 2. Mesh sensitivity discretization for each mesh group

Mesh group	Total Number of elements	Beams discretization	Columns discretization	Slabs discretization
Group 1	6925	5 × 1 × 1	5 × 1 × 1	4 × 4 × 1
Group 2	15558	10 × 2 × 2	8 × 2 × 2	8 × 8 × 1
Group 3	28382	16 × 3 × 3	12 × 3 × 3	14 × 14 × 2
Group 4	44036	20 × 4 × 4	16 × 4 × 4	18 × 18 × 2

Table 3. List for the Progressive Collapse Safety Factor for All Analyzed Cases

Column Type	Loading Case	Elevation of removed column	Beam rotation degree	Column rotation degree	Joint rotation degree	S.F. for Beam Rot.	S.F. for Column Rot.	S.F. for Joint Rot.
Interior	Gravity	Ground	0.4720	0.0586	0.0317	7.63	14.16	36.28
		Fifth	0.4700	0.0517	0.0722	7.66	16.05	15.93
		Eighth	0.5020	0.0410	0.1035	7.17	20.24	11.11
		Tenth	0.5550	0.0187	0.1721	6.49	44.39	6.68
	Seismic	Ground	0.8820	0.2712	0.3134	4.08	3.06	3.67
		Fifth	0.6950	0.3808	0.2788	5.18	2.18	4.12
		Eighth	0.6350	0.3813	0.3115	5.67	2.18	3.69
		Tenth	0.5650	0.2491	0.5036	6.37	3.33	2.28
Edge	Gravity	Ground	0.8290	0.1136	0.0816	4.34	7.31	14.09
		Fifth	0.6900	0.0958	0.0992	5.22	8.66	11.59
		Eighth	0.8030	0.0621	0.1219	4.48	13.37	9.43
		Tenth	0.8120	0.0340	0.1930	4.43	24.41	5.96
	Seismic	Ground	1.1750	0.3085	0.3684	**3.06**	2.69	3.12
		Fifth	1.0530	0.4129	0.3785	3.42	2.01	3.04
		Eighth	0.9820	0.4214	0.3440	3.67	**1.97**	3.34
		Tenth	0.8630	0.2788	0.5293	4.17	2.98	**2.17**
Corner	Gravity	Ground	0.4990	0.0111	0.0275	7.21	74.77	41.82
		Fifth	0.4730	0.0106	0.0525	7.61	78.30	21.90
		Eighth	0.4440	0.0080	0.0720	8.11	103.75	15.97
		Tenth	0.4900	0.0030	0.1341	7.35	276.67	8.58
	Seismic	Ground	0.7030	0.2175	0.2504	5.12	3.82	4.59
		Fifth	0.6550	0.2992	0.2771	5.50	2.77	4.15
		Eighth	0.5810	0.3052	0.2617	6.20	2.72	4.39
		Tenth	0.5540	0.2034	0.2260	6.50	4.08	5.09

often the least employed in engineering practice ... The reason of the difficulty in running these computations lie in (1) understanding the input, assumptions, and capabilities of the model, (2) handling and transferring the GB's of data generated in the simulation, and (3) interpreting and understanding the output. While for the TF method, it is usually assumed that the surrounding beams connected to the remaining part of the column are capable of carrying a tension force resulted from the axial load that was carried by the column. The main drawbacks of this approach: (1) it is not easily implemented in dynamic analyses, which is the case for earthquake and (2) it does not consider the slab contribution (i.e. neglecting the catenary action). Thus, the authors believe that the APM should be the only rational method to investigate the progressive collapse of the reinforced concrete structures subjected to seismic actions. Based on the UFC guidelines, the load combination depends on the analysis type, static or dynamic. In the current study, the analysis type was dynamic. Therefore, for the dynamic analysis, the gravity load combination for the entire structure will be [(0.9 or 1.2DL) + (0.5LL or 0.2S)] in addition to a lateral load with a value of [0.002 × (sum of the gravity loads (DL + LL))] (UFC, 2009), and (ASCE, 2005), where DL is the dead load, LL is the live load, and S is the seismic load.

For every load case, four analyses should be performed. In each analysis, the lateral load will be applied in one of the main directions, i.e. east to west, west to east, north to south and south to north. The analysis stated by UFC assumes a sudden removal of a primary supporting element like a column. The removed column has different locations depending on the structural system. In the structural system used in this study, three types of columns should be checked: interior, edge, and corner columns. The removal should not affect the beam-column connections to maintain the continuity of the horizontal elements attached to the column at the floor level. APM analyses should be carried out for the parking story, story with a public area, first story, story directly below the roof, story at mid height, and story above the location of a column splice or change in column size. For each analysis, the rotations of each of the beam, column and joint must be checked. The beam rotation is checked using Table (4-1) in the UFC guidelines, while the column and joint rotations are checked using Tables (6-8) and (6-9) in the (ASCE-41, 2006) guidelines for the seismic rehabilitation of existing buildings.

4. Applied Element Method (AEM)

AEM is a modeling method adopting the concept of discrete cracking. As shown in Fig. 1(a), the structural elements in the AEM are modeled as an assembly of elements connected together along their surfaces through a set of normal and shear springs. The two elements shown in Fig. 1(b) are assumed to be

Table 4. Effect of Load Case on the Deformations of the Structure

Column Type	Elevation of removed column	Loading Case	Deflection	Beam rotation	Column rotation	Joint rotation	Seismic / Gravity			
			mm	degree	degree	degree	Deflection	Beam Rotation	Column Rotation.	Joint Rotation.
Interior	Ground	Seismic	44.596	0.8820	0.2712	0.3134	1.5	**1.87**	4.6	**9.9**
		Gravity	29.492	0.4720	0.0586	0.0317				
	Fifth	Seismic	42.350	0.6950	0.3808	0.2788	1.3	1.48	7.4	3.9
		Gravity	33.217	0.4700	0.0517	0.0722				
	Eighth	Seismic	38.110	0.6350	0.3813	0.3115	1.1	1.26	9.3	3.0
		Gravity	35.119	0.5020	0.0410	0.1035				
	Tenth	Seismic	37.300	0.5650	0.2491	0.5036	1.0	1.02	13.3	2.9
		Gravity	37.028	0.5550	0.0187	0.1721				
Edge	Ground	Seismic	53.221	1.1750	0.3085	0.3684	1.5	1.42	2.7	4.5
		Gravity	36.645	0.8290	0.1136	0.0816				
	Fifth	Seismic	44.648	1.0530	0.4129	0.3785	1.1	1.53	4.3	3.8
		Gravity	39.902	0.6900	0.0958	0.0992				
	Eighth	Seismic	42.405	0.9820	0.4214	0.3440	1.0	1.22	6.8	2.8
		Gravity	41.734	0.8030	0.0621	0.1219				
	Tenth	Seismic	43.129	0.8630	0.2788	0.5293	1.0	1.06	8.2	2.7
		Gravity	42.359	0.8120	0.0340	0.1930				
Corner	Ground	Seismic	65.895	0.7030	0.2175	0.2504	**2.0**	1.41	19.6	9.1
		Gravity	32.900	0.4990	0.0111	0.0275				
	Fifth	Seismic	48.367	0.6550	0.2992	0.2771	1.3	1.38	28.2	5.3
		Gravity	36.323	0.4730	0.0106	0.0525				
	Eighth	Seismic	48.360	0.5810	0.3052	0.2617	1.3	1.31	38.2	3.6
		Gravity	36.795	0.4440	0.0080	0.0720				
	Tenth	Seismic	48.195	0.5540	0.2034	0.2260	1.1	1.13	**67.8**	1.7
		Gravity	43.209	0.4900	0.0030	0.1341				

Table 5. Reduction in Progressive Collapse Safety Factor Due to Seismic Action

Column Type	Elevation of removed column	Loading Case	Beam rotation	Column rotation	Joint rotation	S.F. on Beam Rot.	S.F. on Column Rot.	S.F. on Joint Rot.	Overall Safety Factor	% of Reduction in S.F.
			degree	degree	degree					
Interior	Ground	Seismic	0.8820	0.2712	0.3134	4.08	3.06	3.67	3.06	-59.9
		Gravity	0.4720	0.0586	0.0317	7.63	14.16	36.28	7.63	
	Fifth	Seismic	0.6950	0.3808	0.2788	5.18	2.18	4.12	2.18	**-71.5**
		Gravity	0.4700	0.0517	0.0722	7.66	16.05	15.93	7.66	
	Eighth	Seismic	0.6350	0.3813	0.3115	5.67	2.18	3.69	2.18	-69.6
		Gravity	0.5020	0.0410	0.1035	7.17	20.24	11.11	7.17	
	Tenth	Seismic	0.5650	0.2491	0.5036	6.37	3.33	2.28	2.28	-64.8
		Gravity	0.5550	0.0187	0.1721	6.49	44.39	6.68	6.49	
Edge	Ground	Seismic	1.1750	0.3085	0.3684	3.06	2.69	3.12	2.69	-38.0
		Gravity	0.8290	0.1136	0.0816	4.34	7.31	14.09	4.34	
	Fifth	Seismic	1.0530	0.4129	0.3785	3.42	2.01	3.04	2.01	-61.5
		Gravity	0.6900	0.0958	0.0992	5.22	8.66	11.59	5.22	
	Eighth	Seismic	0.9820	0.4214	0.3440	3.67	1.97	3.34	1.97	-56.1
		Gravity	0.8030	0.0621	0.1219	4.48	13.37	9.43	4.48	
	Tenth	Seismic	0.8630	0.2788	0.5293	4.17	2.98	2.17	2.17	-51.0
		Gravity	0.8120	0.0340	0.1930	4.43	24.41	5.96	4.43	
Corner	Ground	Seismic	0.7030	0.2175	0.2504	5.12	3.82	4.59	3.82	-47.1
		Gravity	0.4990	0.0111	0.0275	7.21	74.77	41.82	7.21	
	Fifth	Seismic	0.6550	0.2992	0.2771	5.50	2.77	4.15	2.77	-63.6
		Gravity	0.4730	0.0106	0.0525	7.61	78.30	21.90	7.61	
	Eighth	Seismic	0.5810	0.3052	0.2617	6.20	2.72	4.39	2.72	-66.5
		Gravity	0.4440	0.0080	0.0720	8.11	103.75	15.97	8.11	
	Tenth	Seismic	0.5540	0.2034	0.2260	6.50	4.08	5.09	4.08	-44.5
		Gravity	0.4900	0.0030	0.1341	7.35	276.67	8.58	7.35	

Table 6. Effect of the Removed Column Level on Progressive Collapse Safety Factor

Loading Case	Column Type	Elevation of removed column	Max. Beam rotation	Max. Column rotation	Max. Joint rotation	Min. S.F. for Beam Rotation	Min. S.F. for Column Rotation	Min. S.F. for Joint Rotation	Overall Safety Factor	% of variation in SF
			degree	degree	degree					
Seismic	Interior	Ground	0.8820	0.2712	0.3134	4.1	3.1	3.7	3.1	41
		Fifth	0.6950	0.3808	0.2788	5.2	2.2	4.1	2.2	
		Eighth	0.6350	0.3813	0.3115	5.7	2.2	3.7	2.2	
		Tenth	0.5650	0.2491	0.5036	6.4	3.3	2.3	2.3	
	Edge	Ground	1.1750	0.3085	0.3684	3.1	2.7	3.1	2.7	37
		Fifth	1.0530	0.4129	0.3785	3.4	2.0	3.0	2.0	
		Eighth	0.9820	0.4214	0.3440	3.7	2.0	3.3	2.0	
		Tenth	0.8630	0.2788	0.5293	4.2	3.0	2.2	2.2	
	Corner	Ground	0.7030	0.2175	0.2504	5.1	3.8	4.6	3.8	**50**
		Fifth	0.6550	0.2992	0.2771	5.5	2.8	4.2	2.8	
		Eighth	0.5810	0.3052	0.2617	6.2	2.7	4.4	2.7	
		Tenth	0.5540	0.2034	0.2260	6.5	4.1	5.1	4.1	
Gravity	Interior	Ground	0.4720	0.0586	0.0317	7.6	14.2	36.3	7.6	18
		Fifth	0.4700	0.0517	0.0722	7.7	16.1	15.9	7.7	
		Eighth	0.5020	0.0410	0.1035	7.2	20.2	11.1	7.2	
		Tenth	0.5550	0.0187	0.1721	6.5	44.4	6.7	6.5	
	Edge	Ground	0.8290	0.1136	0.0816	4.3	7.3	14.1	4.3	**20**
		Fifth	0.6900	0.0958	0.0992	5.2	8.7	11.6	5.2	
		Eighth	0.8030	0.0621	0.1219	4.5	13.4	9.4	4.5	
		Tenth	0.8120	0.0340	0.1930	4.4	24.4	6.0	4.4	
	Corner	Ground	0.4990	0.0111	0.0275	7.2	74.8	41.8	7.2	12
		Fifth	0.4730	0.0106	0.0525	7.6	78.3	21.9	7.6	
		Eighth	0.4440	0.0080	0.0720	8.1	103.8	16.0	8.1	
		Tenth	0.4900	0.0030	0.1341	7.3	276.7	8.6	7.3	

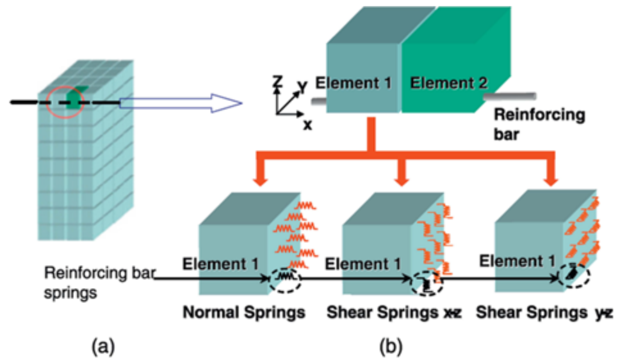

Fig. 1. Modeling of a Structure with AEM

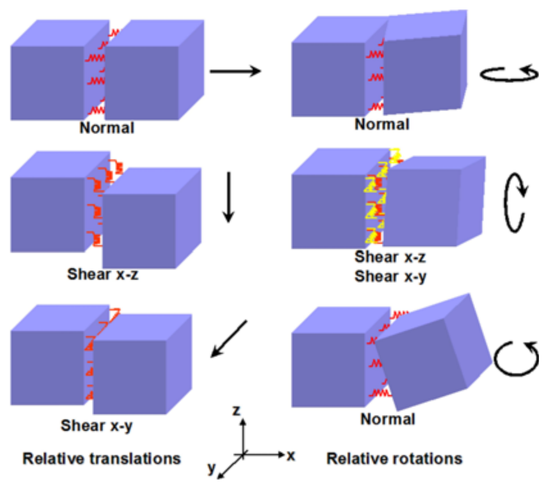

Fig. 2. Stresses in Springs Analytical Model

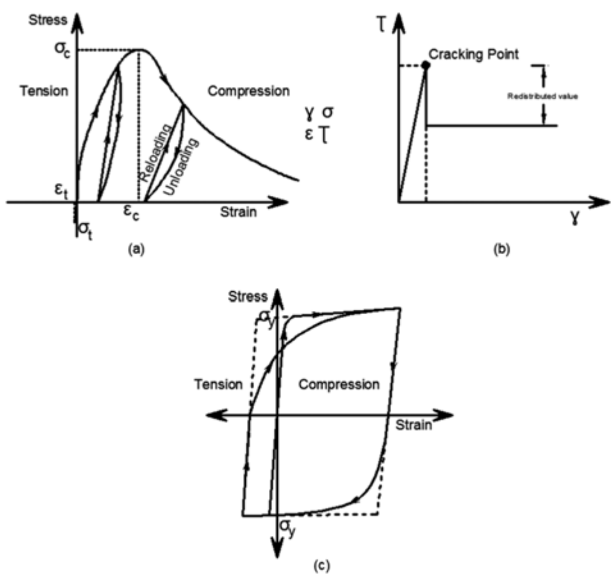

Fig. 3. Constitutive Models for Concrete and Reinforcing Bars: (a) Concrete Under Axial Stress, (b) Concrete Under Shear Stress, (c) Reinforcement Under Axial Stress

connected by normal and shear springs located at the contact points, which are distributed on the element faces. These connecting springs represent the state of stresses, strains and connectivity between elements. They can represent both concrete and steel reinforcing bars. For large displacements and deformations of structures up to collapse, AEM theory provides reliable estimations for the structures' behavior compared to the 'Finite Element' Method. It has been extensively validated and showed good agreement with real cases. For instant, Tagel-Din *et al.,* 2000 utilized AEM for studying dynamic loading. He found that AEM provides high accuracy throughout small deformations as well as large deformations up to failure. Sasani (2008) evaluated the building behaviour of the simultaneous removal of two adjacent exterior columns using AEM. He found good agreement with the experimental results of a Bi-directional Vierendeel system with walls Also, Helmy *et al*. (2009) studied the progressive collapse of a typical 10-story building according to (ACI 318-08). The study adopted AEM for the nonlinear dynamic analyses and proposed some modifications to the ACI code in order to meet the UFC limits. Other numerical work could be found in literature that studies progressive collapse using AEM, such as (Tagel-Din *et al.*, 2004), (Sasani, 2008), (Wibowo *et al.*, 2009), (Park *et al.*, 2009), (Galal *et al.,* 2010), (Salem *et al.*, 2011), and (Salem, 2011).

Each single element has six degrees of freedom: three for translations and three for rotations. Relative translational or rotational motion between two neighboring elements causes stresses in the springs located at their shared face as shown in Fig. 2. Two adjacent elements can be totally separated once the springs connecting them fail. Fully nonlinear path-dependent constitutive models are used in the AEM as shown in Fig. 3. For concrete in compression, an elasto-plastic and fracture model of (Maekawa *et al*., 1983) is adopted as shown in Fig. 3(a). When concrete is in tension, a linear stress-strain relationship is adopted until cracking of the concrete springs, where the stresses then drop to zero. Then, the residual stresses are redistributed in the next loading step by applying the redistributed force values in the reverse direction. For concrete springs, the relationship between shear stress and shear strain is assumed to remain linear until the cracking of the concrete. Then, the shear stresses drop down as shown in Fig. 3(b). The level of drop of shear stresses depends on the aggregate interlock and friction at the crack surface.

For reinforcement springs, the model presented by (Ristic *et al.*, 1986) is used as shown in Fig. 3(c). The tangent stiffness of reinforcement is calculated based on the strain in the reinforcement spring, loading status (either loading or unloading) and the previous history of the steel spring, which controls the Bauschinger's effect. The solution for the dynamic problem adopts an implicit step-by-step integration (Newmark-beta) method as in (Bathe, 1982), and (Chopra, 1995). The equilibrium equations represent a linear system of equations for each step. The equilibrium equations are commonly solved using Cholesky upper-lower decomposition. Once elements are separated, the stiffness matrix becomes singular. However, the stability of the overall system of equilibrium equations is kept because of the existence of the mass matrix. Separated elements may collide with other elements. In that case, new springs are generated at the contact points of the collided elements. In this study, 'Extreme Loading for Structures software' (ELS) is used for analysis of the case study, which uses the AEM.

5. Analytical Approach

The current study targets an assessment of progressive collapse of multistory reinforced concrete buildings due to seismic action. A to the UFC guidelines for progressive collapse, a column is assumed to be lost under gravity loads. The cause of potential loss of a column in such a case could be column overstressing, column deficiency or potential gas line explosion during the seismic action. The column would be assumed to be lost at the time of the peak ground acceleration. For comparison purposes, the progressive collapse assessment is carried out twice; once for column loss due to gravity loads, while the other for column loss due to seismic action.

According to UFC specifications for the typical structure of this study, the analyzed cases will be as follows:

1. Removal of a corner column.
2. Removal of an edge column.
3. Removal of an internal column.

For each case, the column removal was carried out four times, on the ground floor, on the fifth floor (middle height), on the eighth floor (change in column size) and on the floor just below the roof (10th floor). One column was removed in each analysis. Two load cases were used in the case study, namely, the gravity load case and seismic load case.

5.1 Gravity Load Case

In the current study, a nonlinear dynamic analysis was carried out. The relevant UFC load combination for the nonlinear dynamic analysis was used. This load combination is (1.2 D.L. + ½ L.L.), where D.L. is the dead load including loads that are relatively constant over time including the weight of the structure itself, and L.L. is the live loads including loads that are temporary, short duration, or moving loads. This load was applied to the whole building. The column was removed instantaneously at (t = 0.0 seconds) without affecting the beam-column connection in order not to affect the continuity of the horizontal members.

5.2 Seismic Load Case

The ultimate load combination of (EC, 2007) was used, which is (1.12 D.L. + ½ L.L. + S), where S is the seismic load. According to the (EC, 2007), the applied lateral acceleration due to seismic loads was applied in two perpendicular directions. The main earthquake was in the X-axis direction, while the other was in the Y-axis direction with a peak value of 0.3 of the main earthquake. This load combination was applied to the building for 20 seconds, which is a reasonable earthquake duration. The column removal was assumed to be carried out when reaching the peak ground acceleration at (t = 2.0 seconds) without affecting the beam-column connection in order not to affect the continuity of the horizontal members. Based on the location and the structural properties, the response spectrum of the studied structure had been generated according to the (EC, 2007) criterion as shown in Fig. 4(a). In order to perform a time-domain dynamic analysis, artificial earthquakes were generated based on the response spectrum using (Simqke_GR v.2.7) software as shown in Fig. 4(b). The (EC, 2007) requires generating three artificial earthquakes cases. According to the preliminary analysis results, no significant difference was found for the structural deformations due to the three artificial earthquakes, and hence, for simplicity, it was decided to use only one record for the rest of the analyzed cases.

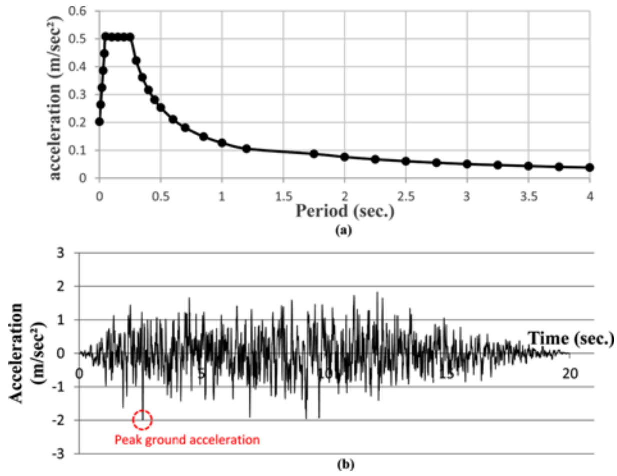

Fig. 4. Generating of an Artificial Time History Data for an Earthquake from the Response Spectrum: (a) The Response Spectrum Resulted from the (EC 2007), (b) Generated Earthquake from the Response Spectrum of the Structure

6. Study Case

6.1 Structure Details

The studied structure was assumed an administrative ten-story reinforced concrete structure with a 400-m^2 plan area. The structure consists of four equal bays in each direction; each bay spams five meters, which is a commonly used span in most residential and administrative buildings in Cairo, Egypt. All floors were three meters high. The structure was assumed located in Cairo, Egypt. Therefore, the building lies in earthquake zone number (3) according to the (EC 2007). Therefore, design peak ground acceleration for this structure is 0.15 g and the building will follow the response spectrum Type-1. It was assumed that the building has an importance factor (γ_I) of one, a damping coefficient (η) of 1, and sub-soil group type (B).

The structure was designed according to the Egyptian code for design and construction of reinforced concrete structures. Beams and columns were designed to resist gravity and lateral loading coming from seismic loads according to the (EC, 2007), while slabs were designed for gravity load only. Columns were

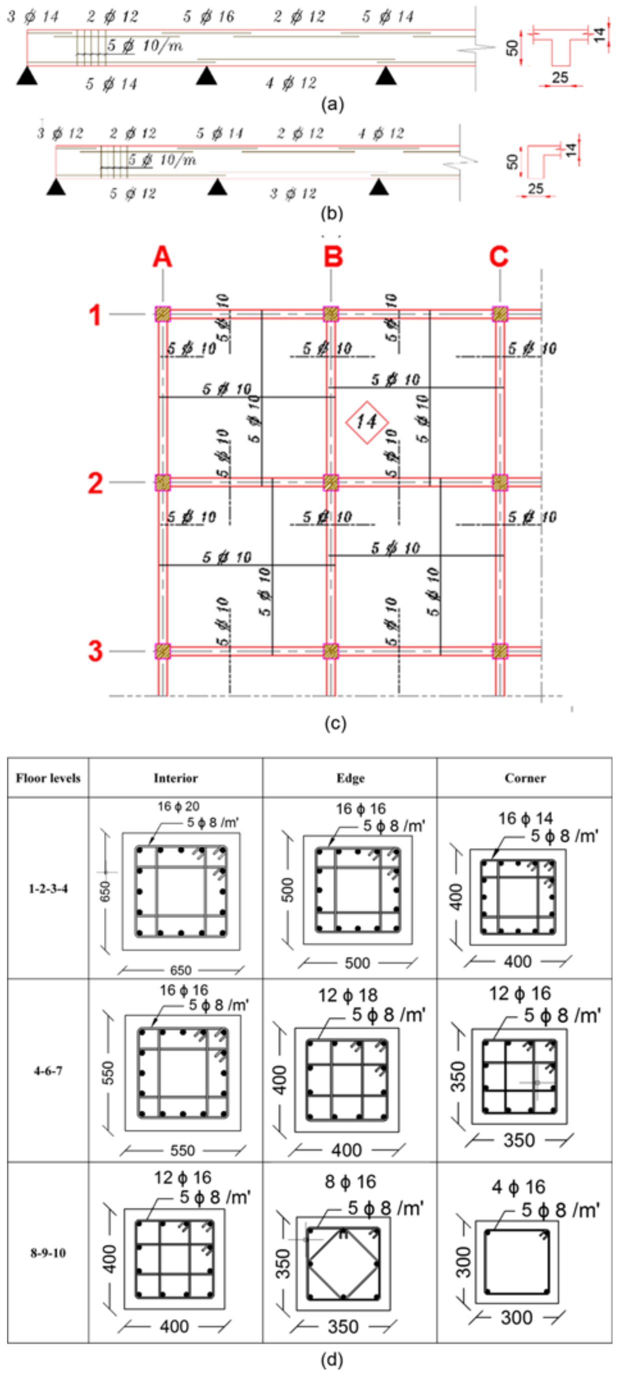

Fig. 5. Geometry and Reinforcement Details of the Structural Components for Interior and Exterior Frames and Slabs: (a) Interior Beams, (b) Edge Beams, (c) Slabs, (d) Columns

assumed fixed to the foundation. Fig. 5 shows the geometry and reinforcement details of beams, columns and slabs. A three dimensional detailed model was built using ELS software considering all the structural elements.

6.2 Materials Properties

Table 1 shows the concrete and the reinforcement properties adopted in the analysis.

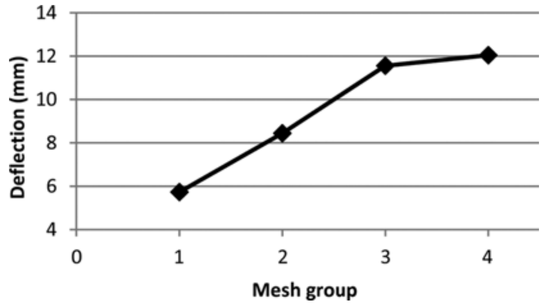

Fig. 6. Downward Deflection for an Element Just Above the Removed Column Corresponding Each Mesh Group

7. Results and Discussion

7.1 Mesh Sensitivity

A Mesh sensitivity study was carried out to find the suitable mesh size that will be used in all the analyzed cases. The study was carried out for the case of edge column removal. Four different mesh sizes were tested and compared as listed in Table 2. Fig. 6 shows the relation between the mesh groups and the downward deflection for an element just above the removed column. The change in the deflection from Mesh Group 3 to Mesh Group 4 is very small; therefore, Mesh Group 3 was used in the analysis.

7.2 Rotation of Structural Components

According to the UFC guidelines, the rotations of all beams, columns and joints due to column loss must be checked. For each analysis case, the rotation history of beams, columns, and joints have been checked that they do not exceed the allowable limit stated in the UFC guidelines.

7.3 Behavior of Frame-only System and Frame-slab System

The analyses were carried out for both gravity and seismic load cases without considering the slabs' contribution in resisting progressive collapse. All these cases collapsed after the column removal. Fig. 7 shows the structural collapse due to column loss on the ground floor for gravity and seismic loads. The cause of failure is that, after column removal, the beams acted in a way different from that for which they were designed. Some beams acted as cantilevers and therefore failed due to insufficient top reinforcement, like in the case of corner column loss. Others spanned two bays and therefore failed due to insufficient bottom reinforcement, like in the case of interior column loss. The modes of failure were obviously flexural ones where flexural cracks initiated at the most stressed sections followed by yielding and rupture of longitudinal reinforcement. Shear was not the predominant factor in the structural behavior, where low values for shear deformations were generally noticed with low level of stresses in stirrups. The collapsed areas were the bays directly connected to the removed column. The structure showed a high potential for progressive collapse and therefore it must be

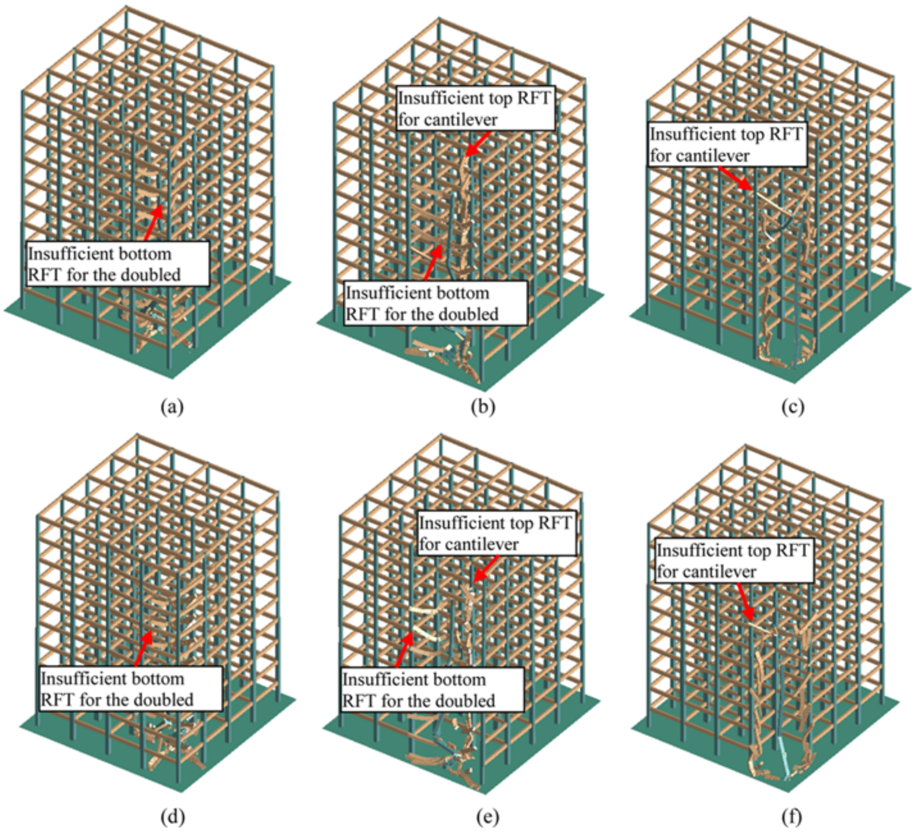

Fig. 7. Deformed Shape after Column Removal Under Gravity Load for: (a) Interior, (b) Edge, (c) Corner, and Under Seismic Load for, (d) Interior, (e) Edge, (f) Corner (frame system)

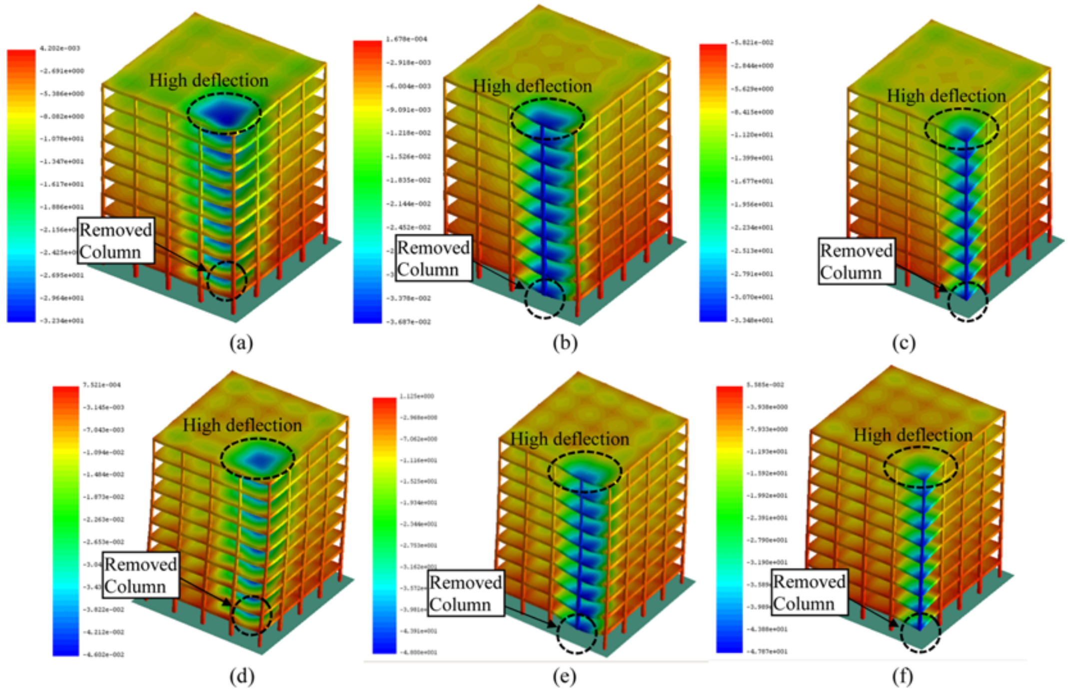

Fig. 8. Deformation Contours after Column Removal Under Gravity Load for: (a) Interior, (b) Edge, (c) Corner, and Seismic Load for, (d) Interior, (e) Edge, (f) Corner (frame-slab system)

redesigned according to UFC guidelines.

On the other hand, the cases that considered the slabs' contribution in collapse resisting did not collapse as shown in Fig. 8, and that agrees with the findings of (Helmy *et al.*, 2012)

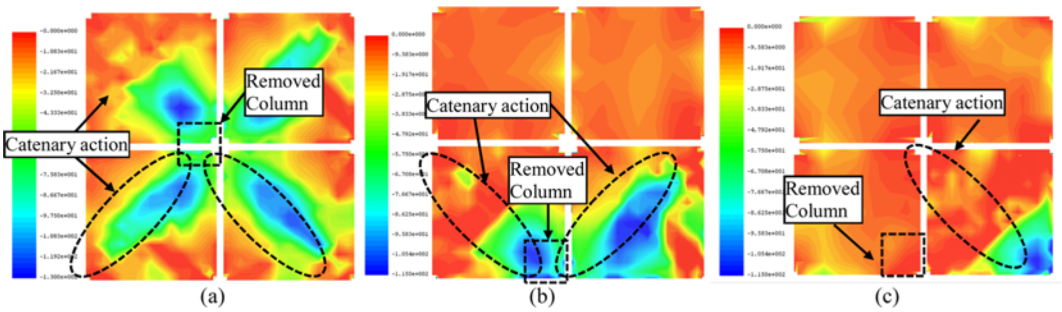

Fig. 9. Major Principal Stress Contours in the Slabs After the Column Removal for; (a) Interior, (b) Edge, (c) Corner (frame-slab system)

for progressive collapse under gravity loads. The explanation of the increased collapse-resistance is that the slabs' catenary action was able to reduce the deflection above the removed column and enable the structural components, slabs, beams and columns above, to act together transferring the gravity load to the surrounding columns. Fig. 9 shows the major principal stress contours that show the maximum tensile stresses developed in the slabs after the column removal. As can be seen, the resultant catenary forces developed in slab reinforcement appeared as diagonal tension tendons, and thus, constituted an alternative load-carrying path and prevented the structure from collapse.

These results proved that the catenary action developed only by the beams is insufficient to prevent the collapse due to the column removal under gravity or seismic loads, while the catenary action formed by the slabs has the ability to do it. Therefore, the analysis of the progressive collapse without considering the slabs and simulating the structure as a 3D frame, gives misleading results, and leads to uneconomical design for progressive collapse resistance.

7.4 Check of Satisfaction of the UFC Guidelines Requirements

In all the analyzed cases that consider the slabs' contribution, the structure did not collapse, and the rotations in beams, columns and joints did not exceed the limits stated by the UFC guidelines. The safety factor due to rotation can be calculated by dividing the UFC guidelines' limit by the maximum rotation measured in the model. The beam rotation did not exceed 1.175 degrees, which is less than the UFC limit (3.6 degrees) with a safety factor of 3.06. The column rotation did not exceed 0.4214 degrees, which is less than the UFC limit (0.83 degrees) with a safety factor of 1.97, while the joint rotation did not exceed 0.529 degrees, which is less than the UFC limit (1.15 degree) with a safety factor of 2.17. Therefore, it can be concluded that, for the studied cases, the structures designed according to the (EC 2007) meet the requirements of the UFC guidelines for progressive collapse resistance with a safety factor for the whole structure not less than 1.97 for all the analyzed cases.

7.5 Effect of Seismic Action on Progressive Collapse Safety Factor

As shown in Table 3, it could be noticed that in all the analyzed cases the column loss under seismic load gives higher values for deformations than the same analysis case under gravity load. For the downward deflection of the element just above the removed column, the seismic load cases cause maximum deflection up to double the value of maximum deflection for the same case under gravity load. Consequently, the elements' rotations caused due to seismic load cases also give higher values than that of gravity load cases. It was found that the seismic loads cause maximum rotation for beams, columns and joints of 1.87, 67.8 and 9.9 times that of those caused by only gravity loads, respectively.

Table 4 shows the percentage of reduction in safety factor for seismic loads compared to gravity loads. It was found that the seismic load case could decrease the safety factor of the structure against progressive collapse up to 71.5%, which means that the seismic load case is more critical in evaluating the structure against progressive collapse than gravity load cases.

7.6 Effect of Location of Removed Column on the Deformations of the Study Structure

As shown in Fig. 10, the maximum obtained beam, column,

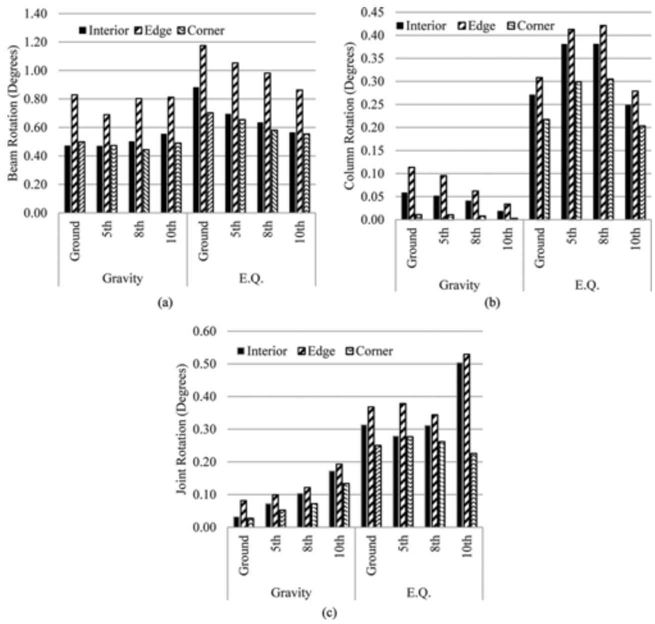

Fig. 10. Effect of Location of Removed Column (in plan) on: (a) Beam Rotation, (b) Column Rotation, (c) Joint Rotation

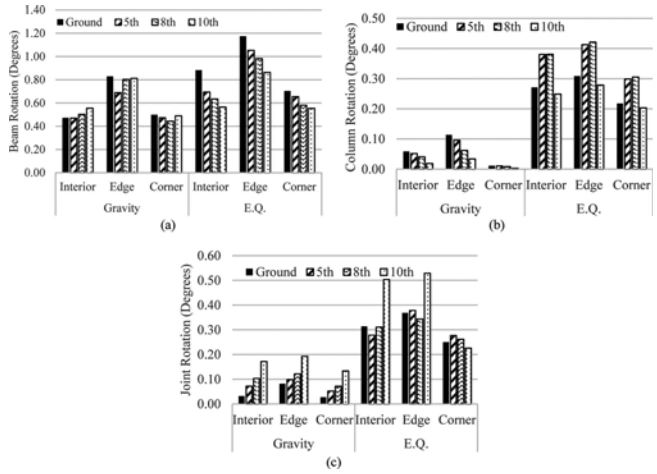

Fig. 11. Effect of Level of Removed Column on: (a) Beam Rotation, (b) Column Rotation, (c) Joint Rotation

and joint rotations reached were in the cases of the edge column loss. The edge column removal causes maximum rotation of beams, columns and joints of up to 66%, 94% and 157% more than rotations caused by the column removal from other locations for the same analyzed case. That can be explained by the fact that the edge column carries more axial loads, and moments in seismic load cases, than the corner one, while the interior column develops stronger catenary and Vierendeel actions than the edge column that will decrease the beam rotations due to the column removal.

Compared to the corner and interior columns, the edge column carries high axial loads and moments under seismic action, which causes higher values of rotation. On the other hand, due to the highly connected beam and slab framing of the interior bays after interior column loss, the rotation amounts are not as high as that of the cases of edge column removal.

7.7 Effect of the Level of Removed Column on Structural Deformations and Progressive Collapse Safety Factor

Figure 11 shows the effect of level of lost column on maximum rotation of beams, columns and joints. In general, the rotations of beams are three times higher than the rotation of columns and joints. There is no clear tendency for the effect of level of lost column except for beam rotation in the seismic loading case, where the maximum beam rotation decreases with the increase in the level of the removed column. This could be explained by the fact that, with the increase in the level of the removed column, the additional stresses exerted by the earthquake decrease, and hence the rotations of the beams connected to the removed column decrease.

Table 5 shows the effect of the level of removed column on the safety factor. It was found that the maximum variation in the safety factor is not significant under gravity loading, where the variation in the factor of safety within different levels for column removal does not exceed 20%. However, under seismic loading, that variation could reach as high as 50%.

8. Conclusions

Progressive collapse assessment for a multistory reinforced concrete building designed according to the (EC, 2007) was carried out. Based on the analysis of the results of the studied cases, the following conclusions can be obtained; however, for generic conclusions, more studies for different structural configurations and loadings should be carried out.

1. Considering reinforced concrete slabs, the structures designed according to the ‘Egyptian Code for Design and Construction of Reinforced Concrete Structures’ (EC, 2007) satisfy all the ‘Unified Facilities Criteria’ (UFC) guidelines requirements for progressive collapse resistance according to the APM with safety factor exceeding 3.06, 1.97, and 2.17 for beam, column, and joint rotations respectively.
2. The catenary action of the slab has a significant effect on progressive collapse resistance, and is capable of preventing the collapse of the structure when removing a column under both gravity and seismic loading cases. Neglecting the slabs’ contribution in progressive collapse analysis leads to incorrect simulation and uneconomic design.
3. The safety factor against progressive collapse for the structure subjected to seismic action could be up to 71.5% less than that when the structure is under the gravity load case.
4. Column removal under the seismic load case gives double the maximum deflection of that of the same case under gravity load. It causes maximum rotations of 1.87, 67.8 and 9.9 times that of the gravity load case for beams, columns and joints, respectively.
5. In both gravity and seismic load cases, the edge column removal is the critical case for progressive collapse analysis. It gives rotation values larger than other cases by the amount of 66%, 94% and 157% for beams, columns and joints, respectively.
6. The variation in the level of the removed column has a moderate effect on the factor of safety for resisting progressive collapse in seismic load cases. The variation in the factor of safety within different levels for column removal reaches as high as 50%. On the other hand, it has a slight effect for the case of gravity load, where the variation in the factor of safety within different levels for column removal does not exceed 20%.

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