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Progressive collapse assessment of precast reinforced concrete beams using applied element method

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ABSTRACT

Progressive collapse is defined as either partial or overall failure of the structure due to losing one of the main structural elements. In order to control this chain reaction, it is important to study the main structural elements behavior under column removal. Precast concrete structures become widely used recently due to the quality control assurance, economical aspects and time saving construction. Due to this many researchers studied the precast concrete structures behavior under earthquake loading, observing the failure patterns, weak points, and how to overcome all those parameters, however, regarding progressive collapse, Precast concrete structures need intensive researches to cover all the parameters that will affect the structure's behavior due to accidental loading. One of the main parameters that still ambiguous is the Precast beam span lengths and its behavior on the overall structure When subjected to progressive collapse. In this paper, the influence of different span length of precast beams is studied under different column removal scenarios. A precast concrete structure case study is adopted and designed according to Precast/ Prestressed Concrete Institute and ACI 318-14 and a multiple 3D models, for different span lengths, are modeled in Extreme Loading of Structures software based on the Applied Element Method. Non-linear dynamic time dependent analysis is conducted on two case studies ; bare frame structure without any slab contribution (Case1), and full structure with slab contribution (Case2). Column removal scenarios are applied according to the UFC regulations, partial collapse took place in case1 while case 2 showed high resistance to progressive collapse. Observations are reported in terms of failure cause for case 1 and the resisting mechanism that took place in case 2. Rotational ductility redistributed applied loads for beams and columns are obtained for case 2. A comparison took place between the rotations obtained in the case study and the rotation limits specified by the UFC and found that the system is satisfying the UFC limits, and no additional consideration need to be done in resisting progressive collapse.

1. Introduction

The term of the progressive collapse was not well defined until the late nineties due to the occurrence of many structures' failures caused by accidental events. Ronan point building failure considered as one of the famous incidents which took place in 1968 due to a natural gas explosion that leads to the collapse of one of the structure's floors as a result of a pancake

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progressive collapse of the southeast corner of the whole building [1]. Another incident took place in 1995 in Oklahoma City, a terrorist attack using a truck filled with explosives targeted Murrah Federal Office building, as a result of the progressive collapse of half of the structure [2].

In the past few years, many incidents took place such as; gas explosion, bombing, and airplane attacks that led to the local failure of one of the main structural elements as columns which leads to the total or partial structural collapse. Thus, many researches are held to study structural behavior under the removal of different vertical structural elements. The observations helped in finding the adequate approach in preventing the partial or total collapse of the structure and limit the structural failure by increasing the ductility and the redundancy of the structure. Also, many approaches are suggested for either enhancing or increasing the structure's efficiency in resisting the collapse by designing the most critical parts in the structure to overcome the accidental action.

Precast structures became widely used in the recent few years because of their high manufacturing quality in a well-controlled environment as well as its fast and high precision assembly. However recently, precast concrete structures are subjected to full and/or partial collapse as a result of various unintended events for some of the structural elements. The progressive collapse influence should be taken into consideration. Thus, many standards codes and regulations introduced an acceptance of certain criteria and limitations in analysis while adopting the load combination.

The overall behavior of the precast concrete structure under several column loss scenarios studied by few researchers. Shi [3] evaluated the behavior of the moment frame precast concrete structure against progressive collapse when subjected to a bombing accident. The influence of different beams' behavior has been investigated for different column removal scenarios and an enhancement technique has been developed and proposed for beam resistance against several column removal scenarios. Li et al. [4] conducted a study on two reinforced concrete frames using the tie method proposed by several codes. It is concluded that the tying method is inaccurate when ignoring the major factors such as; dynamic effect and the 3D load redistribution internal force correction. As a result of proposing an analytical approach for tie force calculation, as a result of reaching a reliable results and an effective technique in simulating the actual structural behavior.

A full-scale of 10-story prototype precast concrete structure is tested for a moment frame assembly and a detailed non-linear finite element model is investigated to record the overall behavior [5]. Nimse et al. [6] and Nimse et al. [7] investigated various precast concrete connections, dry and wet, for a reduced one-third scaled structure under column loss scenarios. The study focused on the behavior of different beam span lengths under different column loss scenarios. While the behavior of different scaled assemblies and forms of dry connections have been studied under static and dynamic loading by Qian et al. [8].

Qian et al. [9] tested experimentally the behavior of beam-column connection of post-tensioned precast reinforced concrete beams under the internal column removal scenario. The use of UPS showed that most of the damage occurred and concentrated near to the connection of the middle column while relieved at the connection of the side column. Also, Qian et al. [8] investigated numerically and experimentally the Progressive Collapse Resistance of Post-Tensioned Precast Concrete Beam-Column Subassemblages. Moreover, the dynamic behavior of precast concrete beam-columns sub-assemblages with high performance connections subjected to sudden column removal scenario has been studied by Qian et al. [10].

A risk assessment is applied and improved through studying the contribution and efficiency of supplementary progressive collapse resisting mechanisms [11,12]. The influence of the slab and beam design on the development of the resisting mechanisms of progressive collapse for RC framed structures investigated by Bredean & Botez [13]. Botez et al. [11] applied two different techniques and methods to assess the progressive collapse of a reinforced concrete frame structure using Finite and Applied Element Methods, and validated using experimental results from Gamble et al. [14].

Shi Yanchao et al. [15] propsed a simulate the scenario for losing columns and was applied on a three-story with a two-span reinforced concrete frame.

Attia et al. [16] investigated a numerical prototype 10-story reinforced concrete structure with a flat slab system against progressive collapse using the applied element method. As a result of the Vierendeel action and the Catenary Action (CA), the flat slab showed high resistance towards progressive collapse. the post-tensioned flat slab behavior are investigated by Keyvani & Sasani [17] against progressive collapse experimentally and numerically.

Rahai et al. [18] studied and investigated the gradual and instantaneous progressive collapse behavior under the removal of column scenarios for a five-story reinforced concrete structure. Helmy et al. [19] Helmy et al. [20] used the General Services Administration [21] guidelines and the Alternative Path Method (APM) and Unified Facilities Code (UFC) guidelines to assess a 10-story RC frame structure behavior to progressive collapse under the removal of primary vertical support using non-linear dynamic analysis. Extreme Loading for Structure software is validated by Ehab et al. [22] the experimental tested specimens made by Nimse et al. [6] are modeled and analyzed the numerical modelind showed good agreement with the experimental testing. A post-tensioned reinforced concrete flat slab structures are assessed by Mahrous et al. [23] using AEM. Limited experimental studies have been performed on the progressive collapse behavior of precast beams [24]. Biagi et al. [25] introduced A Simplified Method for Assessing the Response of RC Frame Structures to Sudden Column Removal.

2. Research significance

The main objective of this paper is to investigate and study the resistance of typical precast reinforced concrete framed structures, with respect to different span lengths, to progressive collapse initiated by loss of different column scenarios. The

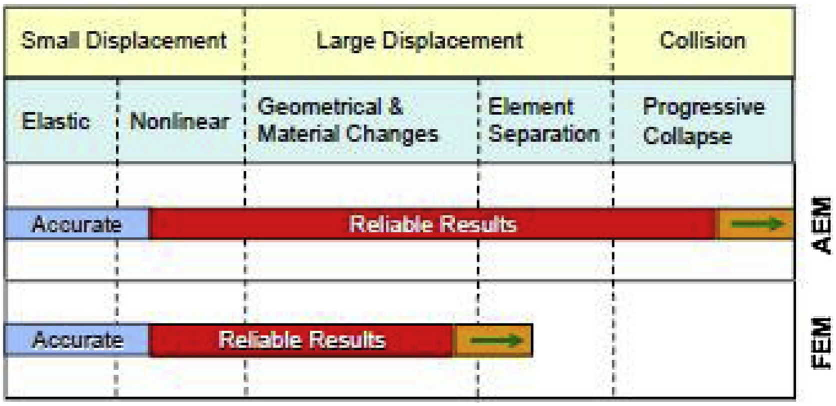


Fig. 1. The Analysis Domain of Applied Element Method (AEM) and the Finite Element Method (FEM) [27].

structures are designed according to ACI 318-14 requirements. According to recent research, the influence of different beam span lengths, in precast reinforced concrete industryis never carried out before. The importance of the outcome of this study will provide an overall idea of the behavior of the collapse of the different beam span lengths for either new or existing structure during the loss of one of its primary support. The case study is conducted using the Applied Element Method that adopts the discrete cracking concept [26], [27], and [28]. The material modeling is applied using complete non-linear constitute models for reinforced concrete. An elasto-plastic model and fracture model are adopted for compression concrete [29]. The AEM proved to model the structure against a progressive collapse in an accurate manner and shows, higher results, and accuracy when compared to the FEM as shown in Fig. 1. Complete non-linear constitutive models for reinforced concrete are adopted in the AEM as shown in Fig. 2.

Many analytical techniques already established by researchers focus on the aspect predicting critical loads resulting in initial damage after initiation of collapse [30] or structural collapse [31–34]. Various implementations of the FEM and modifications, e.g. by integration techniques [35] and/or specialized elements [36], are implemented in most cases. Such analyzes never try to model the final debris heap. Because of its strongly non-linear behavior and the high level of computational effort, successful modeling of the final and collapsed condition of large structures with FE is still an issue for current research and only a few examples have been found in literature, for instance Michaloudis et al. [37] Blankenhorn et al. [38], and Luccioni et al. [39]. Unlike the continuum-based methods, discrete approaches such as the Discrete Element Method (DEM) [40], the Rigid Bodies Spring Method [41], the Finite Particle Method [42], and further derivatives have some inherent advantages in considering the simulation of debris heaps. A comprehensive and detailed overview has been discussed and provided by Bakeer on the methods used in the context of masonry collapse [43]. But it is not easy to model the change from an initially intact structure to discreet debris pieces, especially where complex failure mechanisms are involved,

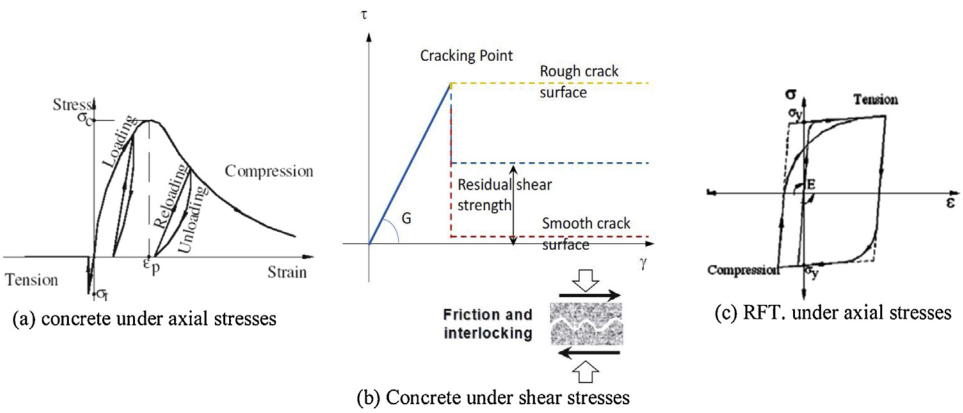


Fig. 2. Concrete and steel constitutive models adopted in AEM [27].

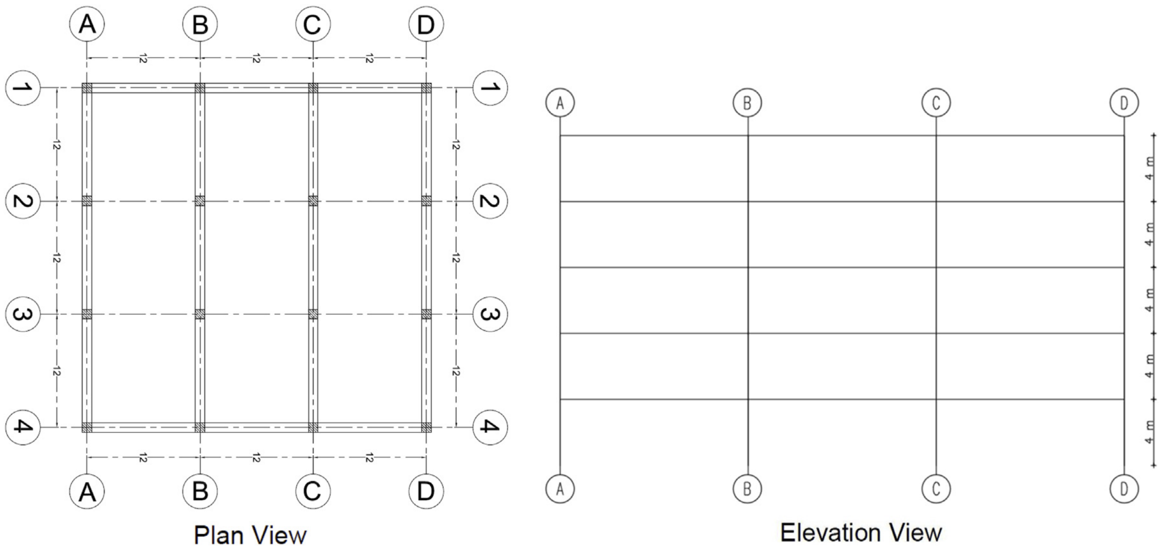


Fig. 3. Typical General Structure Dimensions.

such as in RC structures. The Applied Element Method (AEM) is a hybrid of the Discrete Element Method (DEM) and the Finite Element Method (FEM).

Many scientific research papers on the successful use of AEM over the FEM to model the collapse of full structural systems were published as discussed previously and in Dinu et al. [44] Zerin et al. [45]Cismasiu et al. [46]Elshaer et al. [47] Garofano & Lestuzzi [48]S alem et al. [49] Salem & Helmy [50] Park & Suk [51] Khalil [52], Sasani [53], and Salem et al. [54].

3. Numerical case studies

Two studies are adopted, multiple ordinary frame system with different beam spans are designed to resist gravity loads. The assumption of choosing the beam span lengths are dependent on the its applicability in precast RC systems. The minimum span lengths used are 6 m while the maximum is 9 m. This is due to the fact that a prestressing system need to be used for beams more than 9-meter span. The first case study adopted is a skeletal structures without slab contribution and the second study is considering the slab effect. The structure's applied loads are the self-weight, live load, flooring material, and the non-structural wall partitions distributed throughout the slab. It is assumed that the structure is a residential type. According to the ACI 318-14, for each partition, the applied dead load (floor cover = 2 kN/m2, precast floor slab own weight = 3.3 kN/m2 partitions and superimposed dead load = 2.5) taken as 7.83 kN/m2 and live load of 2 kN/m2. Lateral loading are excluded in this study for reducing the problem size, however, it will be take into consideration in future research work. The combination of load used in modeling is (1.2D.L. + 0.5 L.L.) according to UFC regulations and using nonlinear dynamic analysis for the building. Different main vertical support removal scenarios are implemented with time-dependent function to study the influence of different span lengths in precast reinforced concrete beams.

3.1. Prototype structure description and details

Two structures with 5 typical floors are adopted as shown in Fig. 3. Different bay lengths are applied. One with 6 m and the other is 9 m span length. The floor area of the 6 m and 9 m buildings are 324 m^2 and 729 m^2 respectively. The precast connection used is a wet connection which means that some gaps are left hollow in the pre-cast beams and columns to be filled after assemblage with cast-in-situ concrete to ensure connections integrity. A 100-mm gap is left at the beam topside to be used for extending additional top reinforcement between the beams through the columns. A gap of the same level as in beams are left in columns for the same purpose.

The beams are rested on the RC corbel using a bearing pad to resist beam bearing stresses, dowels are projected from the RC corbel through a hollow gap at the beam bottom side. All the gaps are filled with cast-in-situ concrete. The compressive strength of concrete is taken 40 MPa while the yield stress is taken 420 MPa for all reinforcing bars.

The design of different structural components in the prototype structure is illustrated in detail as shown in Table 1. The reinforcement detailing of the typical structural elements with a plan of the prototype structure is shown in Fig. 4 and 5.

Table 1
Beam Dimensions and Reinforcement.*.

Structure	Beam section	Flexure RFT	TOP RFT (Sec. A / Sec. B) Stirrups	Shear RFT	Torsion RFT Longitudinal RFT		Ledge RFT Closed Ties	Section Dimensions
S =6 m	L- Beam	4Φ16	5Φ16 / 4Φ12	Φ8@100		8Φ12	Φ18@100	400*550*150(upstand) *275(bw)
	IT Beam	5Φ16	5Φ22 / 2Φ12	Φ8@100	None		Φ18@100	500*550*150(upstand) *250(bw)
S =9 m	L- Beam	5Φ22	5Φ25 / 2Φ16	Φ8@100		No. Long. RFT. is Required	Φ20@100	500*800*200(upstand) *350(bw)
	IT Beam	5Φ28	5Φ32 / 3Φ18	Φ8@100	None		Φ20@100	600*800*200(upstand) *350(bw)

* All dimensions are in mm.

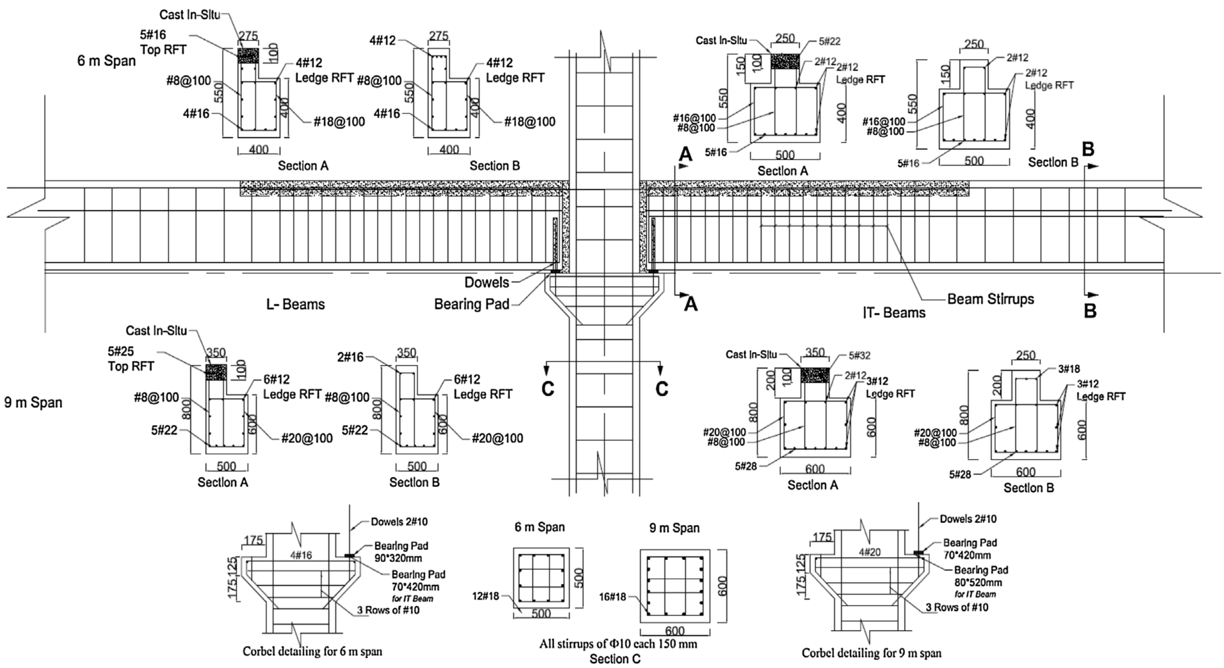


Fig. 4. Geometry and reinforcement details of the structure components.

For the second case study considering slab contribution, a precast flat slab of 150 and 200 mm is designed on concise beams with 6Φ12/m and 7Φ25/m rebars and six strands of diameter 15.2 mm for 6 m, and 9 m span structures as illustrated in Fig. 5. The precast flat slabs are rested on 20 mm neoprene pads on the precast beams' ledges. According to ACI section 4.10, to ensure Structural integrity for enabling tie force and diaphragm action, a longitudinal rebars of 1Φ12 per flat slab panel and transverse rebars of 5Φ12 and 8Φ12 per span for 6 m and 9 m structures are anchored between the slab and beam junctions respectively. The 50 mm and 100 mm gaps that are left between slabs and beams For peripheral reinforcement purpose, are filled with cast in situ concrete and forming a concrete topping of 50 mm that are reinforced with 5Φ10 /m to ensure structural integrity.

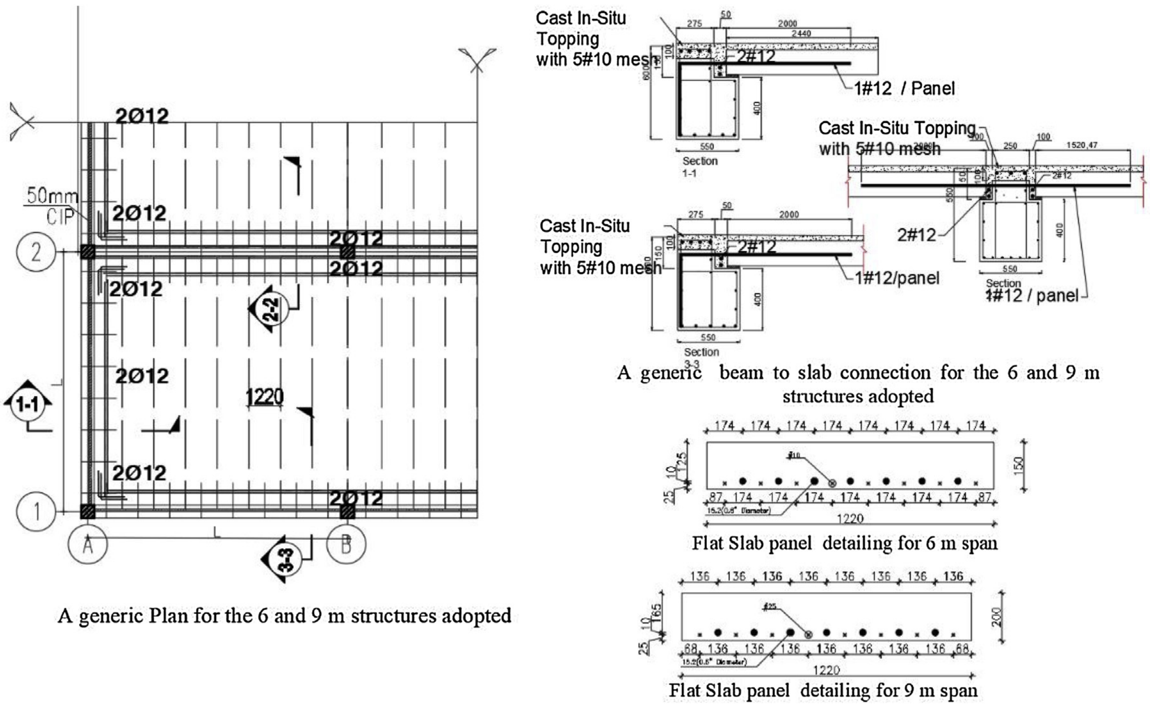


Fig. 5. Geometry and reinforcement details of the structure components.

3.2. Numerical ELS modelling and mesh sensitivity

The adopted designed structures are modeled in the ELS software environment by applying the assumed material properties as shown in Table 2. The precast wet connections are modeled in detail as shown in Fig. 6. To ensure a real representation of the cast in situ and the precast beam connectivity, a material interface between the two materials are take automatically the weakest of the two concretes modelled.

In compliance with the UFC guidelines, analysis cases will Corner, Edge, and Internal column removal for a typical multi-story structure system. For each case, the support removal was carried out three times; on the ground floor, on the third floor, and the floor just below the roof (5th floor). One support is removed at a time. The locations of removed supports are shown in Fig. 7. The typical floors are removed for model clarification.

A parametric study for the mesh sensitivity analysis is done for the determination of the most optimum mesh size that will be used in the analysis of the ELS Model, Different mesh discretization is used under one column removal scenario is shown in Table 3 to obtain the optimum mesh size corresponding to the Maximum deflection at the removed column position. as shown in Fig. 8, the difference between both meshing set # 3 and # 4 could be neglected; we chose mesh set # 3 to be used in all analyzed cases.

4. Numerical analysis results and discussion

The aim of this study is to take into account the structure's 3D impact during the column loss for different beam spans. The column loss scenarios are applied by time dependent stage analysis. The numerical analysis took place on two stages, the first stage is applying all gravity loading, and the second stage, it is time dependent non-linear stage analysis, is removing the specified column at zero time. The investigation is done on two levels. Case (1): is to model the entire structure in detail as a skeletal structure without slabs contribution. Case (2): focuses on the entire structural behavior with slabs contribution. This research is essential for observing the beam behavior with respect to its length and precast wet connectivity due to different column removal scenarios specified by the UFC regulations. A three-dimensional model is represented in Fig. 9 as a notation of the column removal scenarios (corner column, edge column and inner column removal scenarios) used throughout the analysis results representation.

Table 2
Material Properties.

Material	Young's modulus (MPa)	Compressive strength (MPa)	Yield stress (MPa)	Ultimate strength (MPa)
All Elements Concrete	29725.4	40	–	–
Slab Concrete	34856.13	55	–	–
Cast in Situ Concrete	27805.6	35	–	–
Reinforcement[a]	200,000	–	420	588
Strand 15.2 mm	195,000	–	1650	1860

[a] Main Rebars Reinforcement for beams and columns.

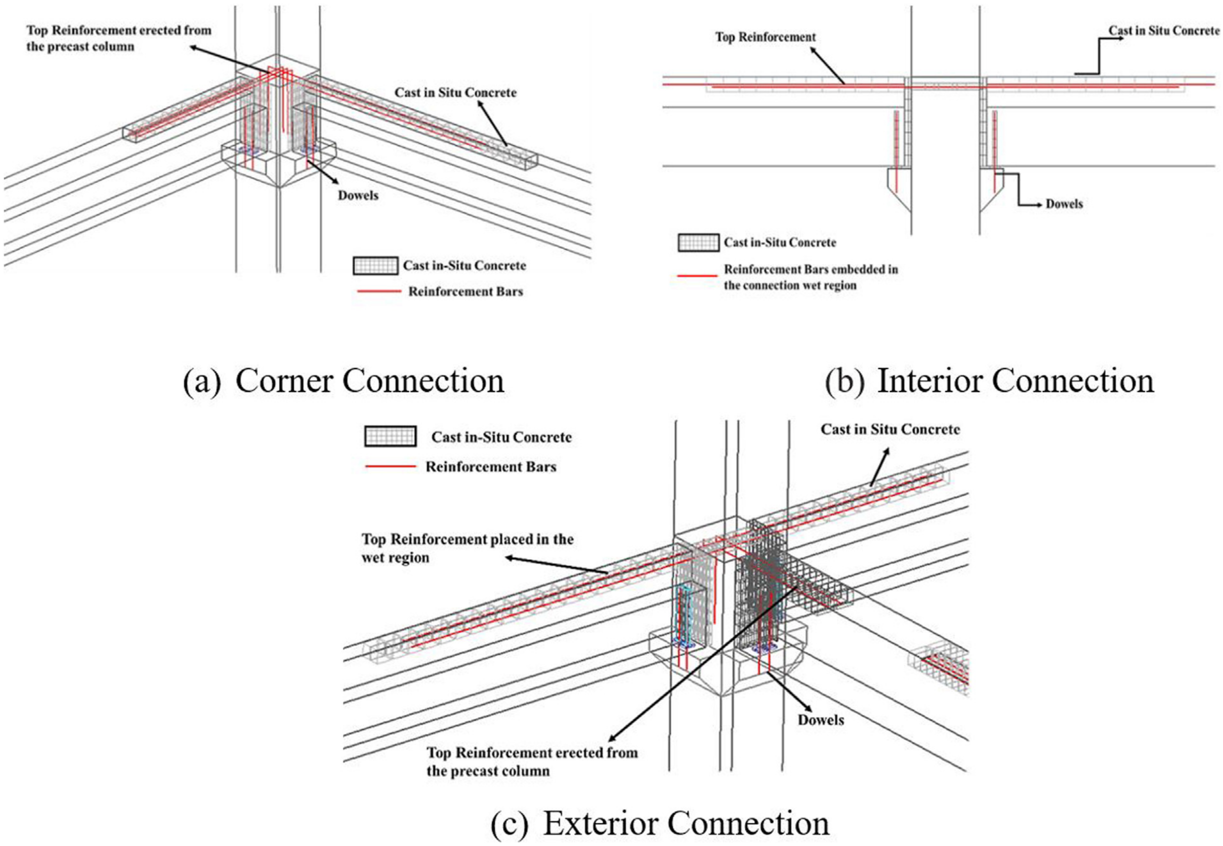


Fig. 6. ELS models for the studied connections (Corner, Interior, and Edge respectively).

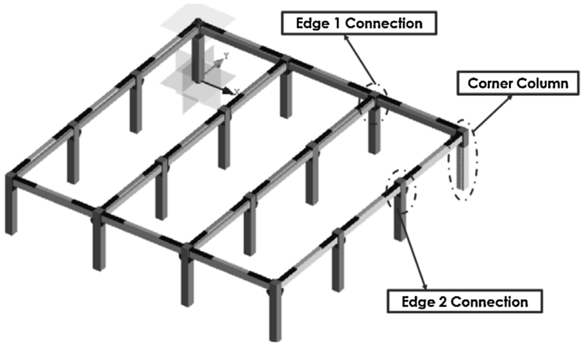


Fig. 7. 3D Model for the Removed Corner Column and the Adjacent Column.

Table 3
Mesh Sensitivity Study Details.

Mesh Sensitivity Study Details [a]								
	Mesh discretization							
Analysis	Girder	Column	Hz. Cast in Situ	Vl. Cast in Situ	Dowels	Bearing Pad	Corbel	Total Number of Elements
1	L 10 × 1 × 2 T 10 × 2 × 1	10 × 2 × 2	1 × 3x1	1 × 3 × 3	1 × 2 × 2	L 1 × 3 × 2 T 1 × 2x4	3 × 2 × 2	15080
2	L 15 × 2 × 3 T 15 × 3x1	20 × 2 × 2	2 × 5 × 2	1 × 4x4	1 × 3 × 3	L 1 × 4 × 3 T 1 × 2 × 5	4 × 2 × 3	20805
3	L 25 × 3 × 5 T 25 × 4 × 2	25 × 3 × 3	2 × 10 × 3	2 × 5 × 4	2 × 5 × 3	L 2 × 5 × 3 T 2 × 2 × 16	5 × 3 × 3	41355
4	L 35 × 3 × 6 T 35 × 5 × 2	30 × 4x4	2 × 10 × 3	2 × 6x4	2 × 6 × 3	L 2 × 6 × 3 T 2 × 3x7	6 × 3x4	57780

[a] The number of elements used in the models in both vertical directions for beams and columns and cross-section and the cast in situ elements numbers in both the depth of it and plan.

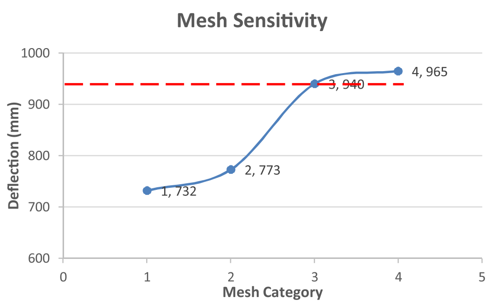


Fig. 8. Relation the Maximum Deflection Above the Removed Column and the Mesh Category.

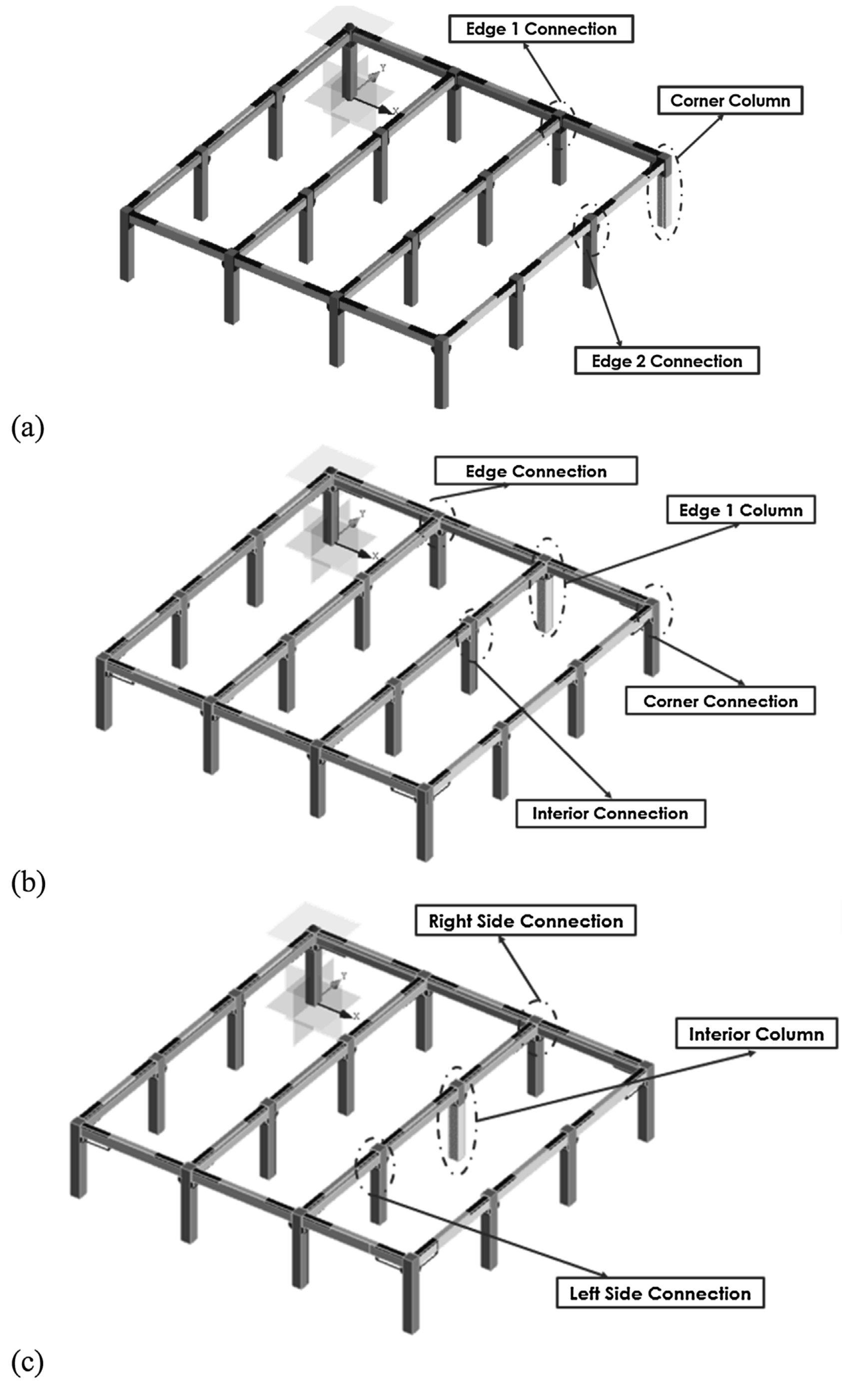


Fig. 9. Groun floor 3D model ELS model showing different column removal scenarios at : (a) corner, (b) edge and (c) interior.

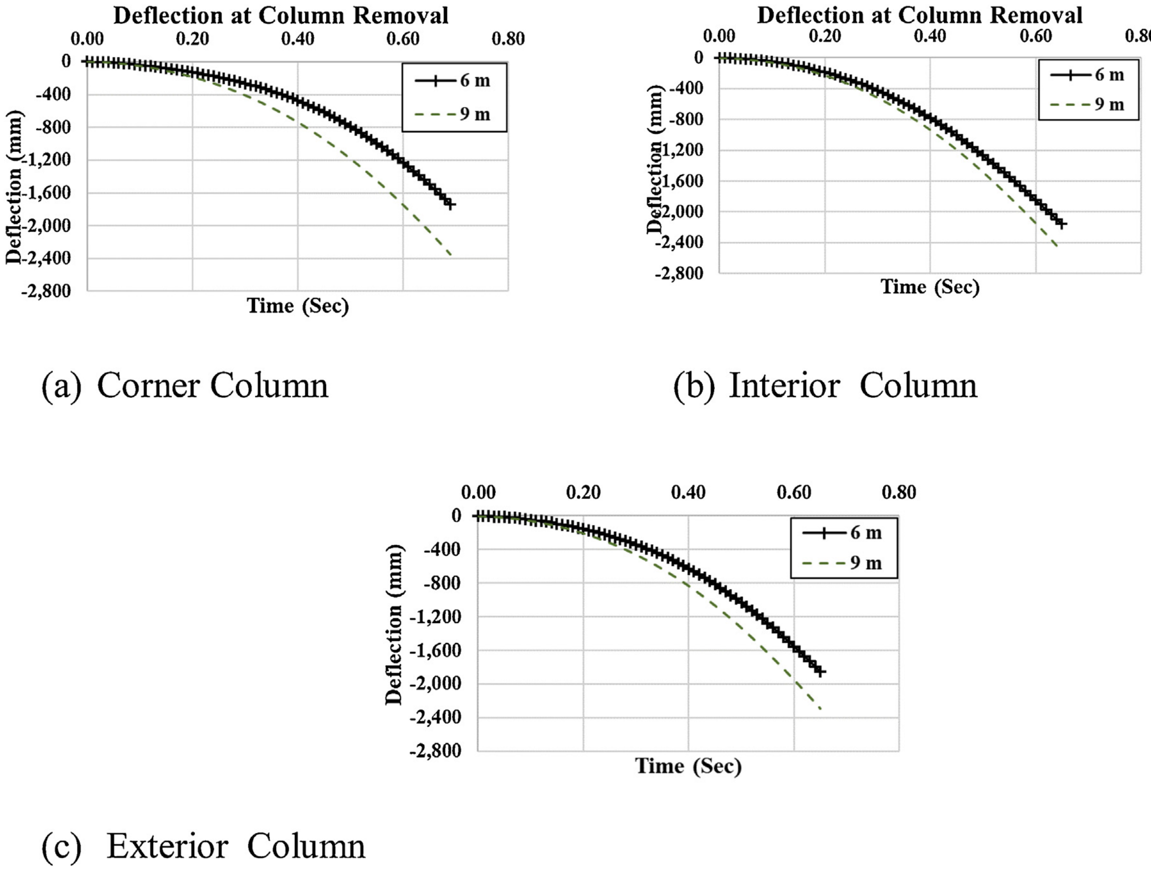


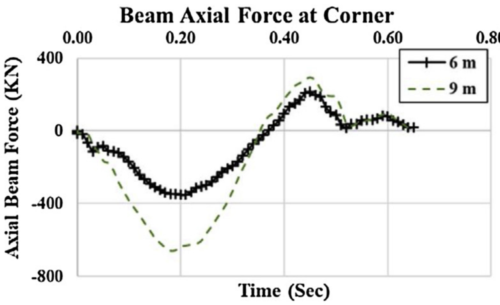
Fig. 10. Deflection at column removal for diferent scenarios at : (a) corner, (b) edge and (c) interior.

4.1. Case 1: 3-D bare frames behavior (Without Slabs)

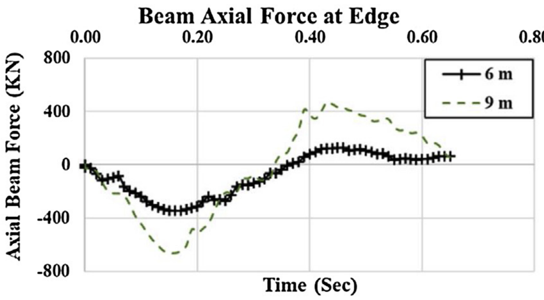
All analysis scenarios for the 3-D bare frame showed a structural partial collapse. The beam deflection is obtained; the failure of the 9 m beam span is more rapid compared to the 6 m span as shown in Fig. 10. The internal axial force in beams are obtained as shown in Fig. 11. It is observed that the beam resistance for the interior column removal, for both span lengths, encountered a compression arching and catenary action of maximum values of 150 K N and 200 K N consequently. However, for the edge and the corner column scenarios, the maximum compression arching and catenary action forces are 700 K N and 400 K N consequently.

The removed column location affected the beams with respect to the applied loads and the structural system encountered after the column loss. As a result of changing the beams resistance behavior. In addition, the cross section design increase, the beam resistance capacity increases in terms of compressions arching and the catenary action effects. For the 9 m beam span, the compression arching is higher than the 6 m span by 75 % due the increase in concrete area that will contribute in the compression arching effect. For the edge and the interior beams, the catenary action effect is clearly indicated in the beam behavior due to the nature of the structural system change compared to the corner column loss.

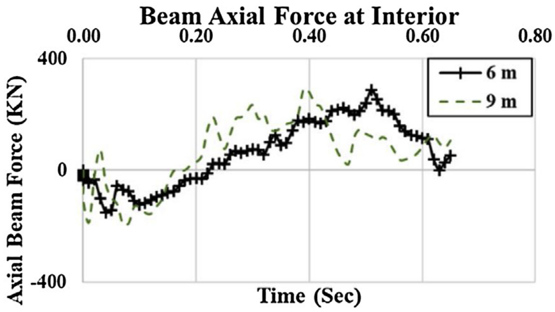
The collapse pattern for all collapsed cases are presented in Fig. 12, the failure took place locally in the spans surrounding the removed column location. This is initiated through dowels due to high tensile stresses that the dowels couldn't sustain. That resulted an excessive rotations for beam-column connection. The affected beams encountered a change in the



(a) Corner Column



(b) Edge Column



(c) Interior Column

Fig. 11. Internal axial force at column removal for diferent scenarios at : (a) corner, (b) edge and (c) interior.

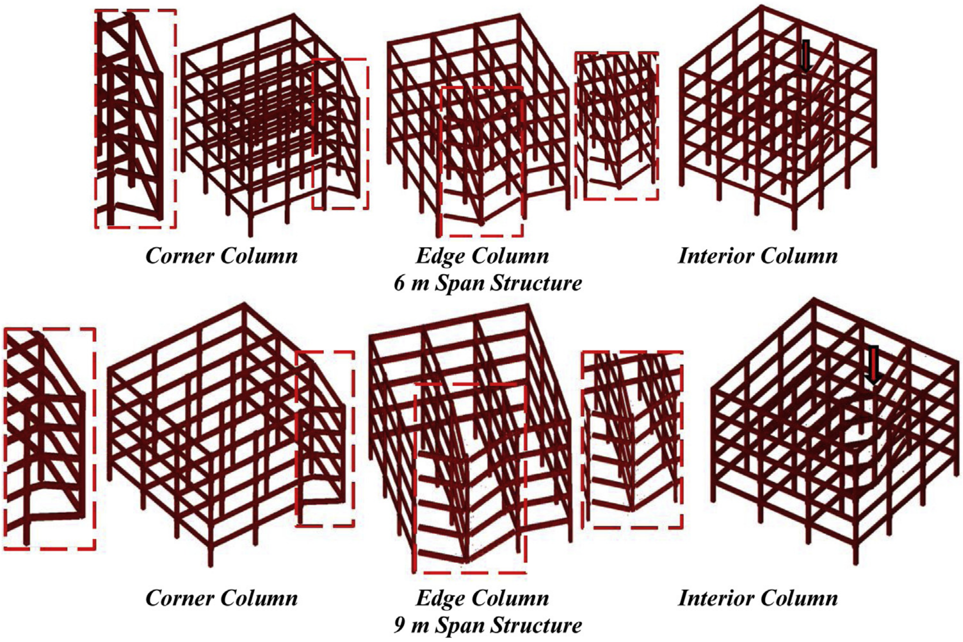


Fig. 12. Collapse after the support removal (3D bare frames).

Table 4
Axial loads in columns before and after ground column removal in KN.

Span	Location	Column Capacity	Axial Force before removal	Axial Force after removal	Increase in Axial Force (Absolute)
Corner Column					
6 m	**Edge 1**	5034	1970	2350	380
	Edge 2		1120	1570	450
9 m	**Edge 1**	7184	4980	5460	480
	Edge 2		2860	3450	590
Edge 1 Column					
6 m	**Edge**	5034	1980	2190	210
	Corner		1080	1280	200
	Interior		2050	2510	460
9 m	**Edge**	7184	4980	5570	590
	Corner		2620	2770	150
	Interior		5350	5120	NA
Interior Column					
6 m	**Edge 1**	5034	1970	2290	320
	Interior		2050	2520	470
	Edge 1*		1970	1850	NA
9 m	**Edge 1**	7184	4980	5320	340
	Interior		5390	5680	290
	Edge 1*		4950	5220	270

supporting systems as well as in the load distribution mechanism. Some beams converted to two bays span length and therefore failed due to insufficient bottom reinforcement. On the other hand, some behaved as a cantilever and therefore failed due to insufficient top reinforcement. The collapsed areas are located at the bays directly connected to the removed column. All cases have failed due to dowels' failure. The effect of the beam span length, with insufficient beam to column connectivity contributed in increasing the applied moment and deformations due to column loss.

However, the edge and corner scenarios have relatively small applied straining actions compared to the interior one. This difference delayed the failure of the dowels at column removal, but it was not enough to prevent collapse.

The failure of all cases can give us a wrong indication of beam behavior due to the absence of other parameters such as the beam to slab connectivity, as a result of expanding the research to consider the slab contributions and its effect in beam behavior towards progressive collapse.

A quick check is done on the column and the redistributed axial forces after column loss. Due to the small contribution of the Vierendeel in the absence of slabs, the redistributed axial forces are compared to the initial forces before column removal and to the ultimate column loads. No ultimate force exceedance, based on the ACI, occurred as shown in Table 4. The axial loads in columns did not exceed the ultimate design force limited by ACI.

*4.2. Case 2: coupled frame-slabs system behavior (*with slabs*)*

In all studied cases with column loss at the ground level, the structure showed a high potential to collapse resistance as shown in Fig. 13.

4.2.1. Precast beam behavior with respect to time

As shown in Fig. 14, the interior column removal encountered the highest deflection in both 6 m span and the 9 m span. The maximum deflections reached 8 and 10 cm respectively for the 6 and 9 m span lengths due to the high loading condition in the interior column. For the corner column removal, the maximum deflections reached average of 1.8 cm.

In all cases, the slab catenary action reduced the structure overall deflection at column loss location. The in-plane tensile stresses that occurred due to downward excessive deformations in the structural elements are shown in Fig. 15. It is noticed that the resultant of catenary forces developed in slab reinforcement appeared as diagonal tension struts, thus constituting an alternative load-carrying path and preventing the structure collapse

Table 5 summarizes the beams' internal axial forces which obtained for two different spans 6 m and 9 m for under different column removal scenarios. It is found that the axial forces are varied in these cases. This is due to many parameters

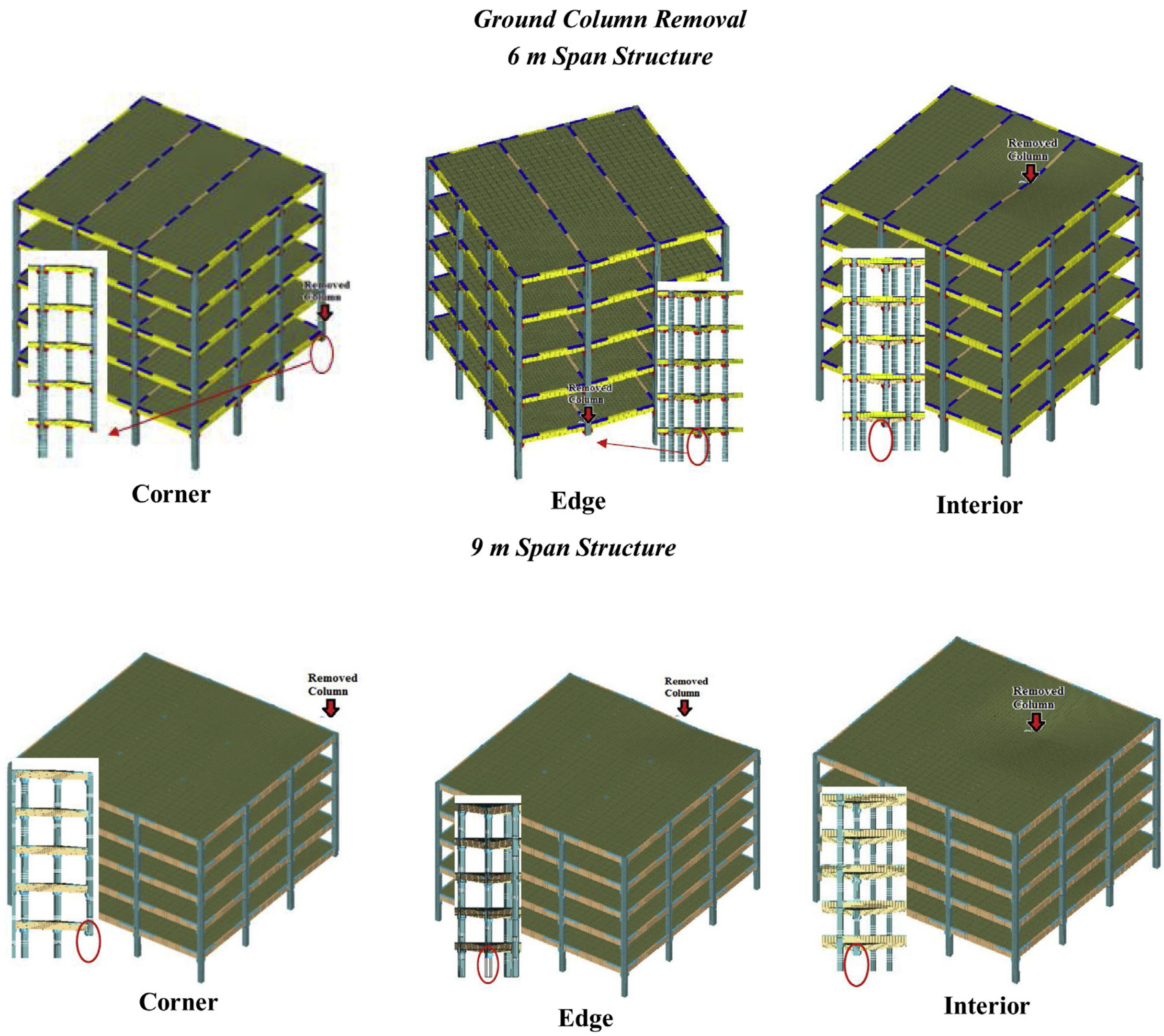


Fig. 13. 6 m and 9 m Spans Structures Overall Behavior After Removal of Corner Column in Ground Floor for all cases.

that took place in the beam design. After the loss of vertical support, the change in axial loads does not exceed the ultimate capacity of the column and beams. This is because that the columns and beams are designed in accordance with the ACI 318-14) specifications, as the UFC guidelines implement lower factors, which used to improve the progress of structural collapse, than those used for the ACI *(1.2 D.L. + 1.6 L.L. instead of 1.2 D.L. + 0.5 L.L.)*. Additionally, the ultimate strain could define the failure point of reinforcement not the yield stress and there is not any strength reduction for the actual capacity for the sections. All of these reasons have assisted to increase both columns and beams' safety margin. Furthermore, the safety capacities of columns and beams are considered to be the strength reduction factors used in the ACI.

4.2.2. Distribution of axial load due to column removal

A redistribution of forces will take place after column removal. in corner column removal, axial loads is transferred to the nearby supports.; edge 1 and edge 2 columns increased by 33 % and 45 % respectively for 6 m span structure. While for 9 m span structure by 52 % and 59 % respectively.

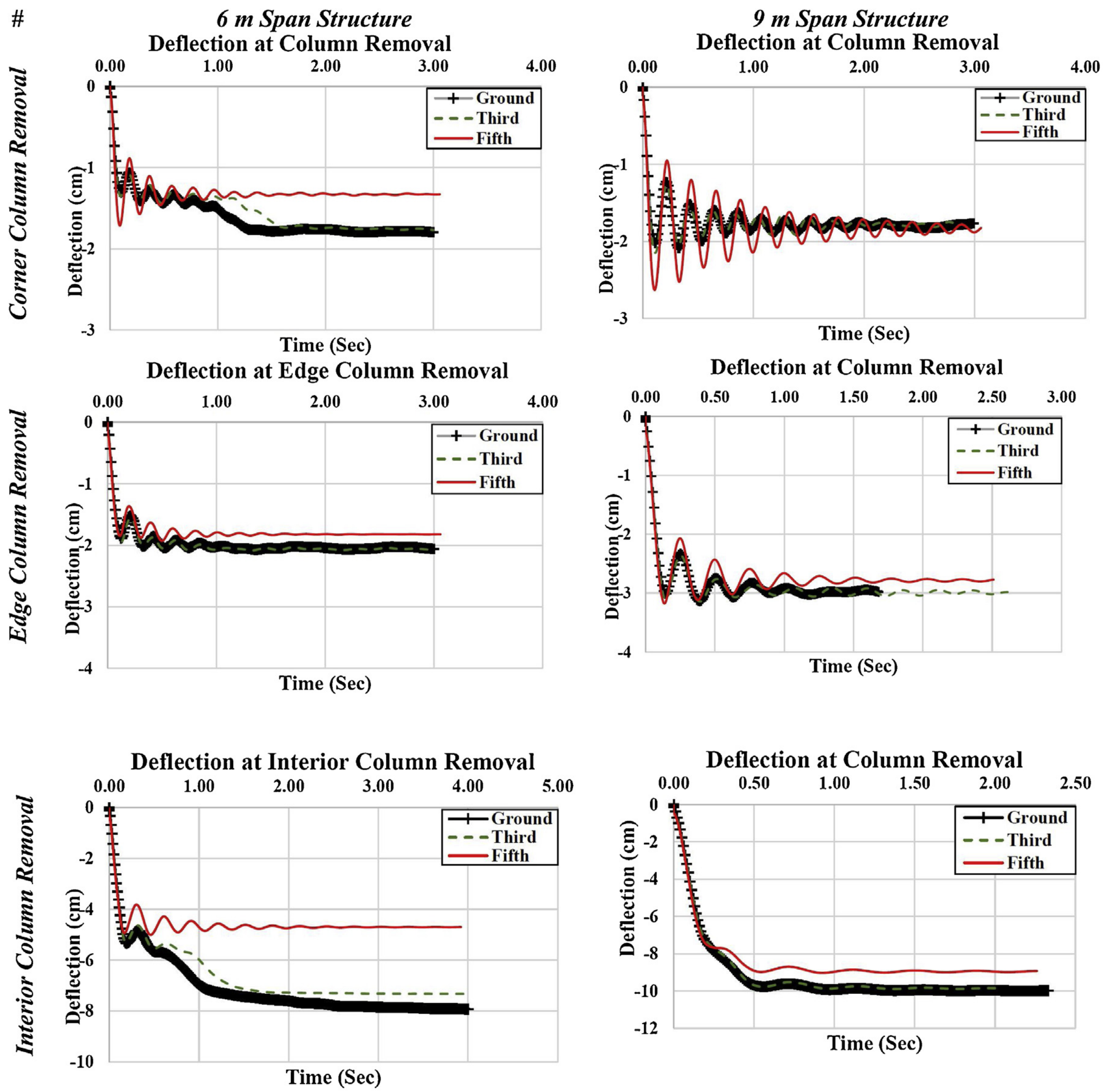


Fig. 14. Deflection at Column Removal for 6 m and 9 m Span Structures.

For the edge column in the first floor is removed, axial loads are transferred to the nearby supports; corner, interior, and edge columns increased by 66 %, 48 %, and 56 % respectively for 6 m span structure. While for 9 m span structure by 61 %, 64 %, and 62 % respectively.

While for the interior column in the first floor is removed, axial loads are transferred to the nearby supports; edge 1, interior, and edge1* columns increased by 65 %, 95 %, and 0 % respectively for 6 m span structure. While for 9 While for 9 m span structure by 73 %, 79 %, and 0 % respectively (Tables 6, 7, 8).

4.2.3. Satisfying the UFC limits

After checking the structure case studies adequacy under different column removal scenarios. An extensive check need to take place to ensure the structure safety according to the UFC regulations. Rotation value for beams and columns are

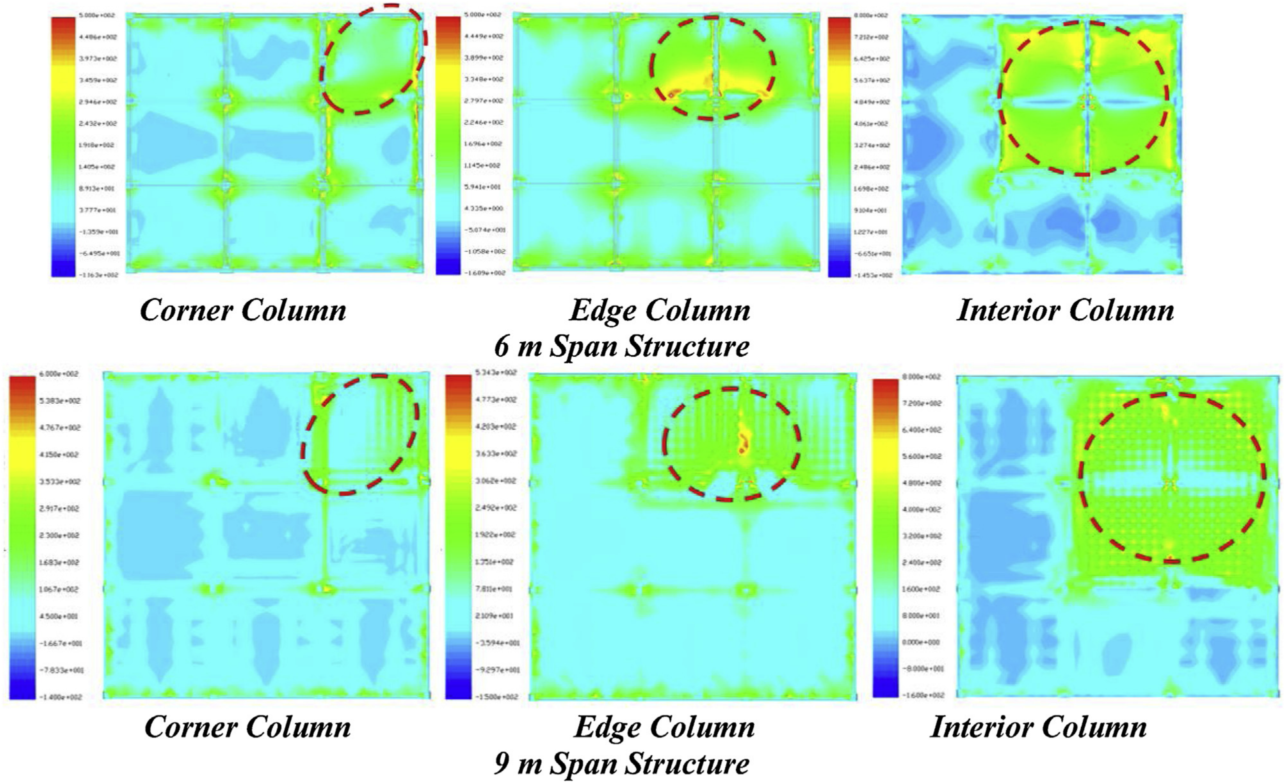


Fig. 15. Major principal stress contours in the slabs after column removal for ground floor for different column removal scnarios.

Table 5
Percentage in Increase for Beams Axial Force Due To Different Column Removal Scenarios.

Floor	Span	Beam Location	Beam Tension Capacity	Max. Axial Force Just after removal	Avg. Axial Force after removal	Exceeding Capacity (%)
Beams Adjacent to Corner Column Removal						
Ground	**6 m**	**Edge 1**	661	287.5	210	NA
		Edge 2		274.3	197.5	NA
	9 m	**Edge 1**	1591	2460.5	1665.5	55%
		Edge 2		1681.4	1370	6%
Third	**6 m**	**Edge 1**	661	799.4	480.2	21%
		Edge 2		689.5	327	4%
	9 m	**Edge 1**	1591	2439	1680	53%
		Edge 2		1610	1325	1%
Fifth	**6 m**	**Edge 1**	661	861.3	587.7	30 %
		Edge 2		842.1	483	27%
	9 m	**Edge 1**	1591	2581.2	1760	62 %
		Edge 2		1812.6	1260	14%
Beams Adjacent to Edge 1 Column Removal						
Ground	**6 m**	**Edge**	661	1222.4	962.6	85%
		Corner		382.6	236	NA
		Interior	1061	929.2	446.3	NA
	9 m	**Edge**	1591	3088.2	2402	94%
		Corner		754.2	428.8	NA
		Interior	2593	2988	2268	15%
Third	**6 m**	**Edge**	661	1162	723.5	76%
		Corner		383.6	222.4	NA
		Interior	1061	918.9	443.8	13%
	9 m	**Edge**	1591	2973	2358	87%
		Corner		733.2	399.2	NA
		Interior	2593	2877	2157	11%
Fifth	**6 m**	**Edge**	661	1176.3	910.7	78%
		Corner		438.5	285.5	NA
		Interior	1061	880.9	448.6	NA
	9 m	**Edge**	1591	2958	2394	86%
		Corner		810.8	489.2	NA
		Interior	2593	2726	1861	5%
Beams Adjacent to Interior Column Removal						
Ground	**6 m**	**Edge 1**	1061	677.4	303	NA
		Interior		1587	205	50 %
	9 m	**Edge 1**	1591	1540	1100	3%
		Interior		4084	576	157%
Third	**6 m**	**Edge 1**	1061	710	232	NA
		Interior		1598	243	51%
	9 m	**Edge 1**	1591	1435	832	NA
		Interior		4210	120	165 %
Fifth	**6 m**	**Edge 1**	1061	533.8	155.4	NA
		Interior		1650	953	56 %
	9 m	**Edge 1**	1591	1420	1006	11%
		Interior		4141	556	160 %

All values in KN.

Table 6
Percentage in Increase for the Adjacent Column for the Removed Corner Column.

Location	Column Capacity (KN)	Axial Force B.R.* (KN)	Axial Force *A.R. (KN)	Increase in Axial Force (KN)	Increase in Axial Force (%)	Exceeding Capacity (%)
6 m Span Corner Column						
Edge 1	5034	1430	1900	470	33 %	NA
Edge 2		1310	1900	590	45 %	NA
9 m Span Corner Column						
Edge 1	7184	2940	4460	1520	52 %	NA
Edge 2		2730	4340	1610	59 %	NA

* B.R.: Before Removal, *A.R.: After Removal.

Table 7
Percentage in Increase for the Adjacent Column for the Removed Edge Column.

Location	Column Capacity (KN)	Axial Force B.R.* (KN)	Axial Force *A.R. (KN)	Increase in Axial Force (KN)	Increase in Axial Force (%)	Exceeding Capacity (%)
6 m Span Edge Column						
Corner	5034	680	1130	450	66 %	NA
Edge		1430	2120	690	48 %	NA
Interior		2320	3620	1300	56 %	NA
9 m Span Edge Column						
Corner	7184	1430	2300	870	61 %	NA
Edge		2920	4780	1860	64 %	NA
Interior		4720	7640	2920	62 %	7

*B.R.: Before Removal, *A.R.: After Removal.

Table 8
Percentage in Increase for the Adjacent Column for the Removed Interior Column.

Location	Column Capacity (KN)	Axial Force B.R.* (KN)	Axial Force *A.R. (KN)	Increase in Axial Force (KN)	Increase in Axial Force (%)	Exceeding Capacity (%)
6 m Span Interior Column						
Edge 1	5034	1440	2370	930	65 %	NA
Interior		2300	4490	2190	95 %	NA
Edge 1*		1440	1270	NA	NA	NA
9 m Span Interior Column						
Edge 1	7184	2940	5100	2160	73 %	NA
Interior		4710	8410	3700	79 %	17
Edge 1*		2990	2680	NA	NA	NA

*B.R.: Before Removal, *A.R.: After Removal.

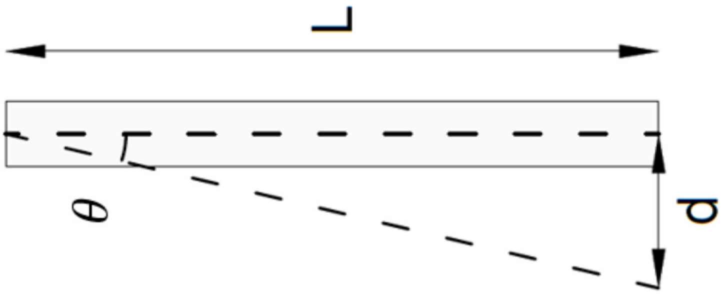
obtained for the different case studies. The rotation values are calculated by dividing the maximum structural element deflection by its length as shown in Fig. 16. Comparing the results obtained for the 6 m and 9 m structure, it's obvious that the values of beam rotations decrease when the span increases in all removal cases and this may be attributed to that the design of the 9 m structure will result in a more top reinforcement ratio embedded in the cast in situ part of the column beam junction compared to the 6 m structure. As a result of increasing the beam rigidity and decreasing the rotational values.

Histories of column rotation are presented in Fig. 17 for all analytical cases which showed no collapse. As per the UFC guidelines [55] and the *Seismic Evaluation and Retrofit of Existing Buildings* [56], acceptance criteria for the plastic rotations that for beams and columns for the mentioned structural system are introduced.

A comparison is held between the plastic rotations obtained from the analysis and the acceptance criteria according to the UFC code mentioned earlier. In all primary vertical support removal scenarios, the structure could resist structure progressive collapse. Any case which shows a partial failure of the structure does not meet the UFC requirements.

On the other hand, for the cases that resisted the support removal scenarios, the UFC specified limits for column and beam rotations. Column and beam rotation history is shown in Fig. 17 and 18 for various analytical cases. The acceptance criteria of the beam rotations by considering it as the primary element with the collapse prevention (CP) loading stage is 0.063 Radians (3.61 Degrees), while the columns will differ for each case based on the applied axial force, percentage of reinforcement, cross-section dimensions, and concrete compressive strength. All study cases according to the rotation limit as outlined by both Table 9 and Table 10 met the UFC criteria for both column rotation and beam rotation.

For all analyzed cases of interior primary support removal, column rotation histories showed a decrease in rotation values as the number of floors above it decreases. That is explained by the fact that the load carried by columns above the removed column decreases, hence leading to lower deflection in the upper floors and lower slab rotation values. Due to the reduced load subjected to the removed primary vertical support, the overall column rotation reduced as well, also the damping effect vanishes. As illustrated, all the structural elements rotations are found to be less than the UFC limits which means that no specific design is needed to overcome the progressive collapse due to the column loss.



$$\theta = \mathrm{Tan}^{-1}\left(\frac{d}{L}\right)$$

Fig. 16. Column Rotation Calculation Concept.

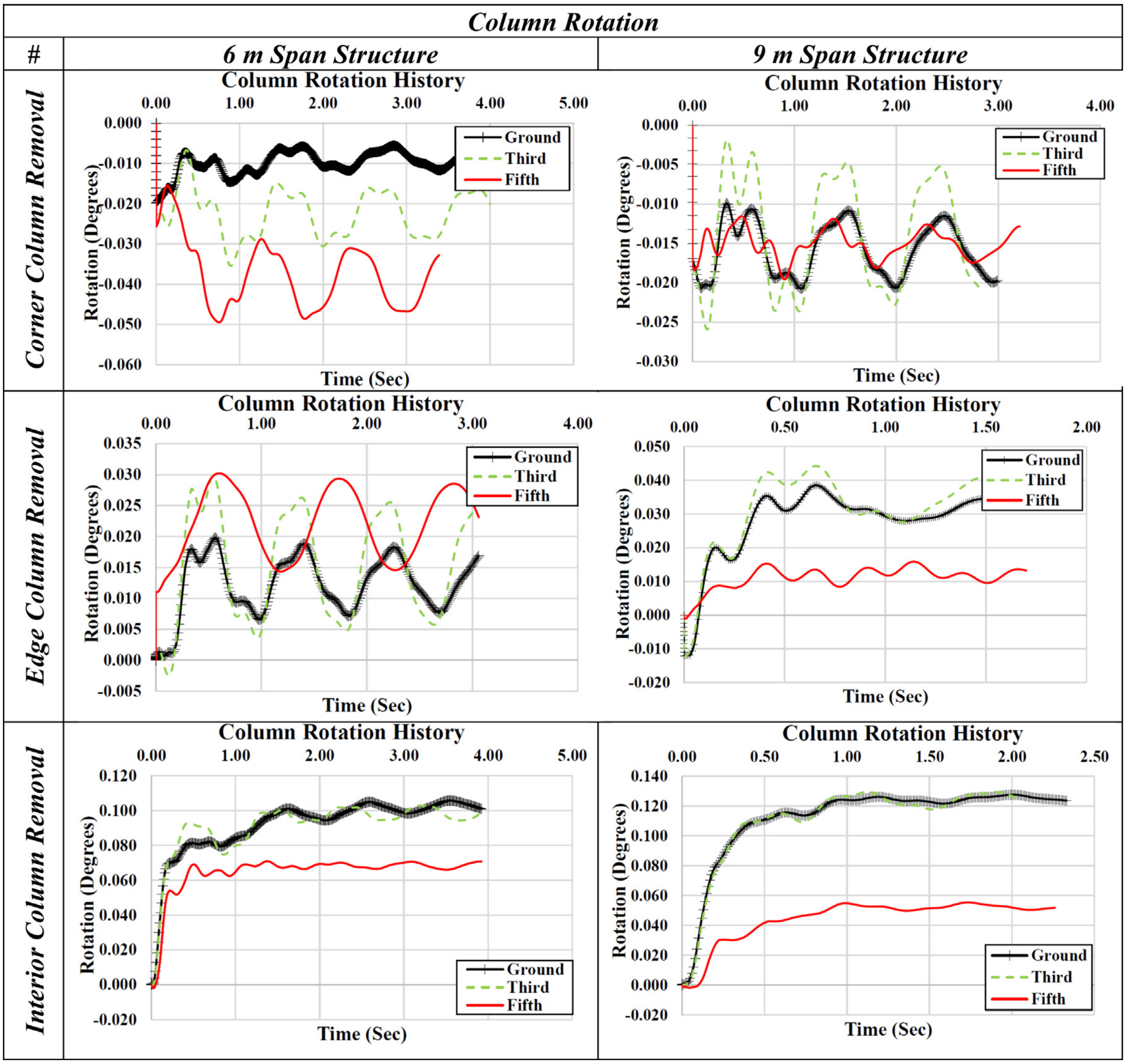


Fig. 17. Histories of Column Rotation in Case of Corner, Edge, and Interior Columns Removal Scenarios for 6 m and 9 m spans structures for Ground, 3rd, 5th floors.

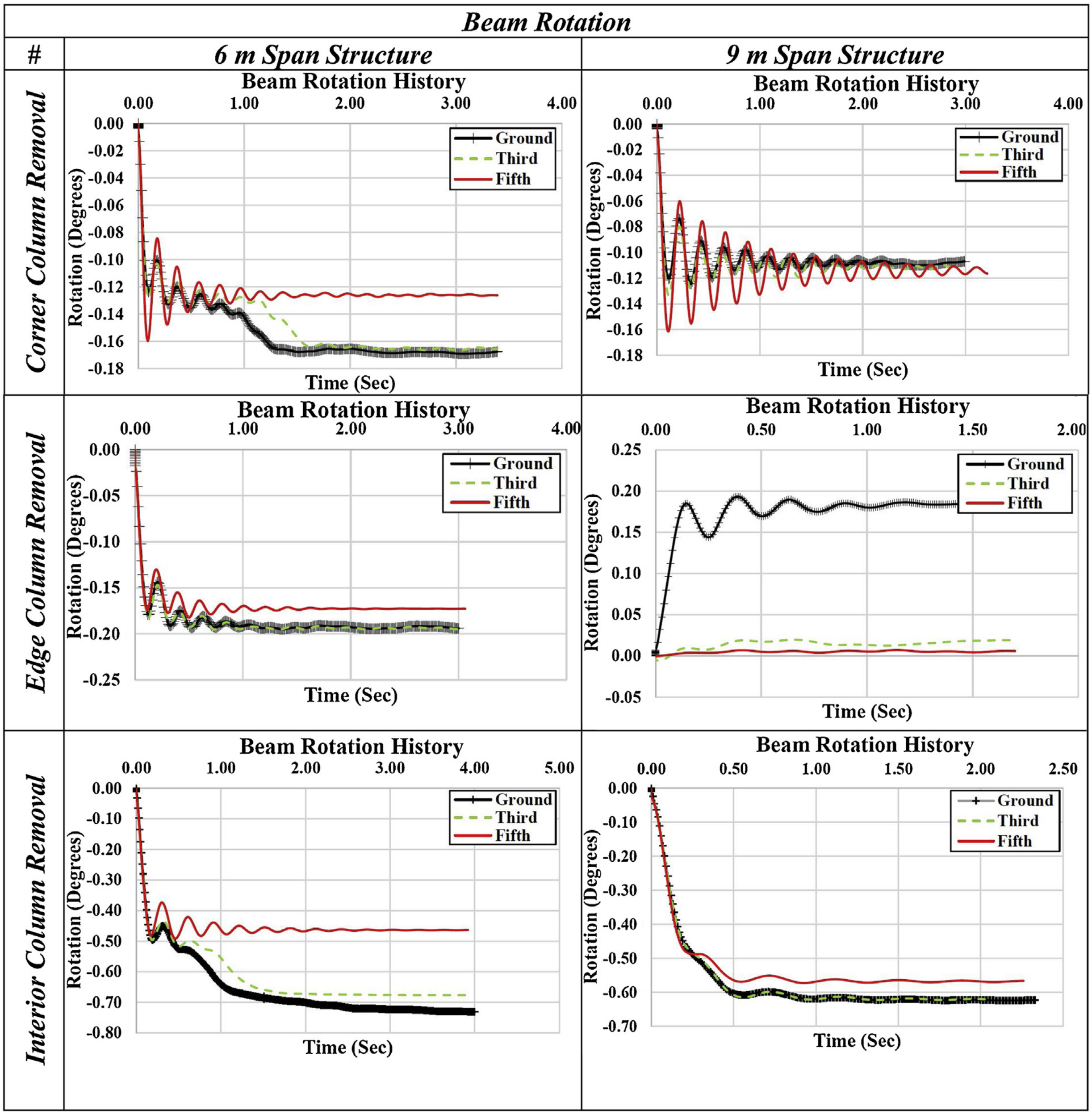


Fig. 18. Histories of Beam Rotation in Case of Corner, Edge, and Interior Columns Removal Scenarios for 6 m and 9 m spans structures for Ground, 3rd, 5th floors.

Table 9
Summary of Maximum Rotations in Different Column Removal Scenarios Checked Against ASCE/SEI 41-17.

Limits for Columns Rotations					
Removed Support Location	Structure	Level	UFC Acceptance Limits (°)	Actual Rotational Limits (°)	Acceptance Criterion
Corner Column	***6 m***	***Ground***	2.89	0.0199	SAFE
		Third	3.32	0.0358	SAFE
		Fifth	3.79	0.0494	SAFE
	9 m	***Ground***	2.20	0.0207	SAFE
		Third	2.91	0.0259	SAFE
		Fifth	3.65	0.0196	SAFE
Edge Column	***6 m***	***Ground***	2.75	0.0206	SAFE
		Third	3.26	0.0295	SAFE
		Fifth	3.59	0.0301	SAFE
	9 m	***Ground***	3.07	0.0392	SAFE
		Third	3.44	0.0444	SAFE
		Fifth	3.83	0.0165	SAFE
Interior Column	***6 m***	***Ground***	2.52	0.1017	SAFE
		Third	3.12	0.0974	SAFE
		Fifth	3.71	0.0716	SAFE
	9 m	***Ground***	1.86	0.1275	SAFE
		Third	1.96	0.0143	SAFE
		Fifth	3.59	0.0554	SAFE

Table 10
Summary of Maximum Rotations in Beams Checked Against UFC Limits.

Limits for Beams Rotations					
Removed Support Location	Structure	Level	Acceptance Limits (°)	Actual Limits (°)	Acceptance Criterion
Corner Column	***6 m***	***Ground***	3.61	0.1671	SAFE
		Third	3.61	0.1623	SAFE
		Fifth	3.61	0.1595	SAFE
	9 m	***Ground***	3.61	0.1158	SAFE
		Third	3.61	0.1114	SAFE
		Fifth	3.61	0.1095	SAFE
Edge Column	***6 m***	***Ground***	3.61	0.1958	SAFE
		Third	3.61	0.1910	SAFE
		Fifth	3.61	0.1719	SAFE
	9 m	***Ground***	3.61	0.1954	SAFE
		Third	3.61	0.1846	SAFE
		Fifth	3.61	0.1719	SAFE
Interior Column	***6 m***	***Ground***	3.61	0.7610	SAFE
		Third	3.61	0.6684	SAFE
		Fifth	3.61	0.4870	SAFE
	9 m	***Ground***	3.61	0.6238	SAFE
		Third	3.61	0.6200	SAFE
		Fifth	3.61	0.5666	SAFE

5. Conclusions

The AEM is used to evaluate the resistance of different beam spans for a precast reinforced concrete structure designed according to the ACI 318-14 and PCI codes against column removal scenarios specified by the UFC guidelines. In reference to the numerical results, two analysis cases were carried out for structural evaluation.

Firstly, a bare frame model will result in a vulnerable structure due to progressive collapse and extreme structure behavior due to column loss. For the analysis of the three-dimensional bare frame, the removal scenarios for corner, edge, and interior columns showed a partial collapse for the structure. The location of the column removal affected the beam behaviors in accordance to the compression arching and the catenary action effect. For the interior beam, the catenary action is the governing behavior compared to the other case studies. This is due to the high applied load nature and the precast connection nature compared to the monolithic structural element. The different span beam influence appeared in the resisting mechanism the beams showed for the different cases. As the cross section dimension increases in both edge and corner column removals, the compression arching resisting forces increases as well.

However, this mechanism and change diminished in the interior column removal scenarios. Another conclusion is that, experimental studies without considering the full structural effect and the column removal locations, could give misleading beam resistance results. As a result, extensive numerical research need to be relied on in the research for understanding the precast beam behaviors, and its different connectivity, with precast columns and compare it to the monolithic beams behavior and resistance.

Finally, Neglecting the slab contribution during the analysis phase will not represent the actual behavior and results in the structure progressive collapse

For the extensive study taking slab contribution into consideration, it is found that the structure succeeded in resisting different column removal scenarios due to hollow core slab contribution. Beams and column rotations are calculated and compared to the UFC rotation limits. All case studies found to be satisfying the UFC limitations, as a result of no need for further progressive collapse design. Different beam span influence only affected the rotational values of the beams. As the span increases the beam cross section and rigidity increases as a result of decreasing the rotational beam values. The nature of connectivity between the slabs and precast beams played a vital role in redistributing forces and resisting the failure.

Further studies need to be done to study the effect of the number of floors over the removed column, the seismic loading effect, pre-stressed beam systems, RC structures behavior with different height, irregular layouts, different precast connections and different structural systems such as hollow blocks, waffle slabs, and paneled beams.

Declaration of Competing Interest

The authors report no declarations of interest.

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