AUT Journal of Civil Engineering

AUT J. Civil Eng., 6(2) (2022) 263-282
DOI: 10.22060/ajce.2023.21169.5795

A Finite Element Analysis of Slab Opening Effects on the Column Removal Scenarios in Large Buildings

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ABSTRACT: Rigid diaphragms play an important role in structural integrity against progressive collapse scenarios. Several studies have been conducted to evaluate the effect of floor slab details on progressive collapse resistance. The main intention of this paper is to investigate the behavior of medium-raised steel buildings with various slab opening conditions in progressive collapse. Numerical studies were carried out by considering different locations and dimensions of the slab openings using the ABAQUS FE software. The highlight of this work is to utilize a nonlinear dynamic analysis to survey the diaphragm condition in load redistribution for column removal scenarios. The axial force variations of columns and diaphragm displacements are compared for different slab opening sizes and locations. The validity of the finite element method is examined using the moment rotation results of an experimental program conducted on a 3D frame covered by a reinforced concrete slab. Generally, the openings placed in the middle and the corner of the structure govern a different after-shock behavior on the structure. In contrast to the middle openings, the corner openings lead to negative effects on the load-resisting capacity of the structure. Moreover, the vulnerability of the columns placed in the vicinity of the corner openings is more than the column around the middle openings. The effect of the opening dimension depends on the opening location.

Review History:

Received: Feb, 28, 2022
Revised: Feb, 22, 2023
Accepted: Mar, 04, 2023
Available Online: Mar, 14, 2023

Keywords:

Progressive collapse
Finite element method
Slab opening
Opening location
Large building

1- Introduction

The increasing number of progressive collapse cases due to terrorist attacks and blast sources led researchers to study this issue in recent years. The overall procedure of progressive collapse can be stated as sudden damage or failure of a member leading to the failure of other members and eventually the collapse of the entire or large part of the structure[1].

For this reason, when one of the principal members of the structure is failed to complete its action, the associated members are affected. According to the literature review, the destruction rate of the progressive collapse mechanism is so faster than the failure of the first elements contributing to the progressive collapse procedure [2]. Nowadays, the modern methods of architectural design and implementation of high-performance material results in achieving an advanced construction system. The modifications in the structural design codes lead to more ductile and resistant structures [3].

One of the well-known cases of the progressive collapse is the PLASCO building in Tehran in 2016. As depicted in Fig. 1, the progressive collapse in this steel building occurred due to an initial blast and subsequent combustion. The progressive collapse is initiated by combustion-evolved local damage in the PLASCO building. Subsequently, local damage develops throughout the structural system and led to the entire collapse of the building. The fire flames in the middle floors caused to produce dust and high temperature, and then the thermal effects of skin fire led to large deformations in the main structural members [4]. The columns lost their resistance and the adjacent slabs sagged toward the lower floors, the procedure continued with consecutive devastation of the columns and slabs, which ultimately led to the complete collapse of the building.

The design of a high-rise building to resist fire using a performance-based approach requires a comprehensive study of the thermal behavior of the structure. The main purpose of the tests is to provide suitable data to validate and develop numerical models and enable the researchers to study different structural and combustion scenarios [5]. Astaneh Asl et al. studied the progressive collapse due to column removal caused by an explosion in a steel building with a composite slab. The study results indicate that after removing the middle columns, the progressive collapse does not take place in the floor slab against live and dead loads because of the chaining response of joists and girders [6].

Kaewkulcha et al. suggested a beam element formula and a method for dynamic analysis of the progressive collapse. Some recommendations were provided based on

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Fig. 1. The progressive collapse of the PLASCO building in Tehran in 2016 [4].

their research results for further studies [7]. Powell et al. investigated the general principles of the progressive collapse analysis using the Alternate path method [8]. Khandewal et al. investigated the progressive collapse of a steel structure with the special moment frame system. The results demonstrate that the structures with seismically resisted peripheral frames might not be robust enough to resist unexpected loads caused by removing a gravitationally loaded column. The analysis results revealed that the reduction of the beam flange width for RBS connections increases the vulnerability of the system to progressive collapse [9].

Antonius et al. carried out an innovative study on the assessment of seismic-resistant steel buildings against progressive collapse. Some experiments were conducted on the special moment frame and the prestressed frame in the steel structures against seismic excitation. Both cases withstood a load level 3 times more than the most critical live and dead loads without any severe damage, But the special moment frame was weaker and less flexible than the prestressed one [10].

Fu et al. conducted a study on the progressive collapse of high-rise buildings. They implemented the ABAQUS software to simulate the behavior of 20 story building. According to the analysis results, to resist the sudden added load due to the removal of the adjacent column, the column should be designed for an axial load of about twice of column capacity. Also, more displacement is obtained for the case of the removal of the columns in the upper floors because of the elimination of the lower stories in the energy dissipation process [11]. Tavakoli and Afrapoli's study discussed the energy method advantages against stiffness and base shear methods for the case studies under seismic loads. Their study results prove the excellence of the CBF system in progressive collapse scenarios [12]. Amiri et al. designed several 3D models of reinforced concrete buildings to achieve a new dynamic increase factor (DIF) for rising the accuracy of the static progressive collapse analysis. Their novel DIF formula has the capability of predicting the stress and deformation in the members adjacent to the column removal [13]. Li et al. presented a novel technique to compare the behavior of the different steel moment frames in the progressive collapse procedure [14]. Weng et al. proposed a series of damage indexes that consider the interaction of the shear, axial and flexural damages in the RC buildings. They also discussed the role of the damage constitutive models in the better prediction of progressive collapse development [15]. Jiang et al. conducted an experimental program to determine the progressive collapse resistance under localized fire. They also presented a numerical study to investigate the effect of the damping and strain rate parameters on the column buckling due to localize fire [16]. Yang et al. surveyed the behavior of composite floor systems under progressive collapse. The simulations were enhanced by using the experimental results of the composite floor components. The numerical method had been then validated against the results of a column removal test on the composite slab. Their numerical study was focused on the variations of geometric properties of the composite slab. Their studies demonstrated that the aspect ratio of slabs play an important role in improving the structural behavior in the column removal scenarios [17]. Lin et al. presented a new method for evaluation of the steel frames exposed to blast loads. A comprehensive procedure for modeling and loading of the structure under catastrophic explosions were provided. The proposed method was compared to other conventional methods through a typical building example. They proved that the damage initiation in the first floors is significantly more influential than the sudden event that occurred on the upper floors. They confirmed that the new method gives trustworthy predictions of the progressive collapse behavior of the buildings under blast loads [18]. Fu et al. studied the failure mode and load-transfer mechanisms of structures under progressive collapse using the FE method. The location of the removed column was investigated using different quasi-static and dynamic analyses. Finally, they proposed new dynamic increase factors (DIFs) for the quasi-static results

[19]. Bredean et al. made an effort to identify the effects of geometric properties of beams and slabs on the progressive collapse behavior of the structures under sudden accidents. A series of nonlinear analyses were performed to indicate the role of slab stiffness in load redistribution mechanisms [20]. Fu et al. compared two-dimensional and three-dimensional models to study effective parameters in progressive collapse. Their numerical study results indicated that the (DIFs) obtained for the 3-D models are smaller than that for the 2-D models [21]. Lu et al. conducted an experimental program on five scaled reinforced concrete (RC) frames. The effect of geometric and reinforcing properties of slab and beams on structural behavior was studied under sudden removal of columns using plastic strain and deformation variables. As theirs study results show, the RC slabs improve the structural capacity up to 1.5 times of the test models without slab. The increase of seismic reinforcement in beams is considerably more influential than augmenting the slab edge rebar's [22]. Peng et al. concentrated on dynamic collapse in single-story flat-plate substructures through an experimental program. The punching failure of internal columns due to peripheral columns removal was reported as the main failure mode of the test models in this program. The test results proved the key role of the continuous bottom reinforcement of the slabs around the connection zone for prevention of the chain reaction of the failures [23]. Pham et al. established a series of tests to investigate the beam-slab systems under two different load distribution patterns. The conventional method of the point load distribution was compared with realistic uniform load distribution. Their research results demonstrated that the beam-slab system is considered vulnerable against side column removal when accurate uniform loading is applied instead of the traditional point load method [24]. Sideri et al. completed a comprehensive numerical study on steel frame buildings with different seismic resistant systems of the internal rigid core and the peripheral moment resisting frames (MRFs). The conducted study shows a considerable robustness increase for buildings strengthened by the peripheral MRFs against the progressive collapse scenarios [25].

Panahi et al. investigated the effect of typical concrete building plans (rectangular and square) on progressive collapse. A 3D model of the building was analyzed by LS-DYNA software in which, the corner columns were removed considering the alternate path method [26]. Their study shows that rectangular buildings are more vulnerable to progressive collapse. Praxedes et al. proposed a novel robustness assessment methodology for better design of reinforced concrete frames against progressive collapse [27]. A new risk-based robustness index has been developed in this study. Moreover, they used a unique directional simulation technique. Finally, their assessment can help us make sure whether or not the enhancement design is warranted for such a low-probability-high-consequence event. Peng et al. used an alternate path method to evaluate the progressive collapse resistance of multi-story modular buildings [28]. These buildings are constructed using corner-supported composite modulus. In this study suitable dynamic increase factors are identified and recommended. Hu et al. investigated the reasons behind the collapse of the Florida international university pedestrian bridge by reproducing a virtual scenario of this collapse using the finite element numerical simulation and the alternate path method [29]. They showed that the collapse of the building was due to the destruction of local nodes. Mucedero et al. investigated the progressive collapse of framed buildings with partially encased (PE) composite beams [30]. They evaluated the vertical load-carrying capacity of framed buildings with PE beams after column loss. Thai et al. analyzed the robustness of modular high-rise buildings utilizing time-history analysis and alternate path method [31]. They developed a numerical model in which, a typical modular tall building is subjected to a corner module removal scenario.

The slabs play a key role in the structural resistance against progressive collapse. Resembling the girders, the force redistribution in slabs reduces the vertical displacement and prevents the development of the adverse effects of the column removal on other structural members. Since most of the researches are studied the behavior of the girders and columns in the progressive collapse procedure, the current paper is focused on the role of the diaphragm system. Also, the effect of the slab opening is investigated using the alternate path method. A three-dimensional finite element model of a 6-story steel structure is considered to evaluate the failure potential of the structure due to the different dimensions and locations of the slab openings. The analytical results demonstrate that the slab opening cause to prevent the transmission of the exerted loads due to the column removal to the adjacent columns and girders.

2- Methodology

The process in which, the collapsing system looks for alternative load paths for the purpose of dealing with the loss of a crucial structural member, is known as Progressive collapse which is a highly complex dynamic process. To evaluate the structure against progressive collapse, the alternate load path method is widely used among engineers and researchers. In this method, it has to be shown that the structural system has sufficient resistance to progressive collapse. Performing this method requires using one of the following three procedures: linear static, nonlinear static, and nonlinear dynamic [32].

2- 1- Linear static procedure

In this procedure, two load cases are considered for a linear static model. Deformation-controlled actions are calculated using the first load case and by using the second load case, force-controlled actions are computed. To evaluate deformation-controlled actions, the applied load is increased by a load increase factor (LIF) to be approximately eligible for both dynamic and nonlinear effects. After modifying the linear static model by removing the vertical load-bearing member (column, wall, etc.), the enhanced load is applied to it. Next, internal member forces which are attributable to the increased loads are calculated and compared with

Table 1. The Brittle cracking model for concrete slabs.

Direct Stress after Cracking	Direct cracking strain
3100000	0
0	0.001

Table 2. The shear retention factor for concrete slabs.

Shear retention Factor	Crack opening Strain
1	0
0	0.001

the expected member capacities. A capacity increase factor (CIF) which is like the m-factor in ASCE 41, is used to enhance the expected member capacity (This accounts for the expected ductility). The results are then compared with the deformation-controlled actions. For force-controlled action analysis, different LIF which only takes into account the inertial effect is used to reanalyze the model. The calculated demand is finally compared directly with the unmodified member capacity.

2- 2- Nonlinear static procedure

At first, a model which is both materially and geometrically nonlinear is built. The loads are then magnified by a dynamic increase factor (DIF). This factor only accounts for inertia effects. The model is modified by removing the vertical load-bearing member and then, these forces are applied to it. To analyze deformation-controlled actions, based on our desired performance level, the deformation limits are set and the resulting member deformations are compared with them. Analyzing force-controlled actions requires a comparison between the member strength which is not modified and the maximum actions.

2- 3- Nonlinear dynamic procedure

In this approach, it is required to apply the unmodified-load case directly to the model which is geometrically and materially nonlinear. Under the applied-load case, the structure is allowed to reach equilibrium (first phase). Then, in the second phase, a vertical load-bearing element which is a wall or column is removed instantaneously. The resulting motion of the structure is calculated in this phase. In a process similar to a nonlinear static case, calculated maximum deformations are compared with deformation limits. The member strength and maximum internal member force are compared with each other to analyze the force-controlled actions. According to the fact that inertial effects and nonlinearity are considered exclusively in this approach, it is not required to use correction factors. The Nonlinear dynamic procedure has been used in this research to survey the role of openings of slabs in a series of progressive collapse scenarios.

3- Progressive Collapse Analysis

3- 1- Finite element method

A 3D building model was provided using *Beam and *Shell elements for the frames and slabs in ABAQUS software [33]. The frames and slabs were fully constrained to each other in the joints and structural component interfaces. The slab was simulated using the four nodes *Shell element having bending and membrane stiffness terms available with the ABAQUS software [33]. The REBAR elements were used to model reinforcing in each shell element. The nonlinearity was considered in material behavior. The selected software can handle extreme deformations due to explosion. The material properties of all the structural steel components were defined as elastic-plastic material with isotropic hardening. The plastic zone of the steel material was defined as the true stress (σ) versus the plastic strain (ε). The ABAQUS approximates the smooth stress-strain behavior of the material with a series of straight lines. The concrete material was assigned to the model using the Brittle cracking constitutive law. This allows the analysis to follow the development of the full elastic-plastic behavior of the section at each integration point along the beam. All columns of the ground floor are assumed to be fixed to the support. The mesh size is justified based on the verification model to ensure the accuracy of the output results.

3- 1- 1- Material behavior

A Brittle cracking model [33] was used to define material constitutive law for slab elements. The concrete Poisson's ratio and modulus of elasticity are 0.2, and 24757 MPa, respectively, and the mass density of concrete is 2400 kg/m3. A Brittle cracking model of strain type was selected then the direct stress after cracking and Direct cracking strain was assigned based on Table 1. The compressive strength of concrete is assumed to be 30 N/mm2.

The amount of shear transfer across a crack is specified using the shear retention factor therefore the brittle shear state is defined using the shear retention factor per crack opening strain. The brittle shear was defined according to Table 2.

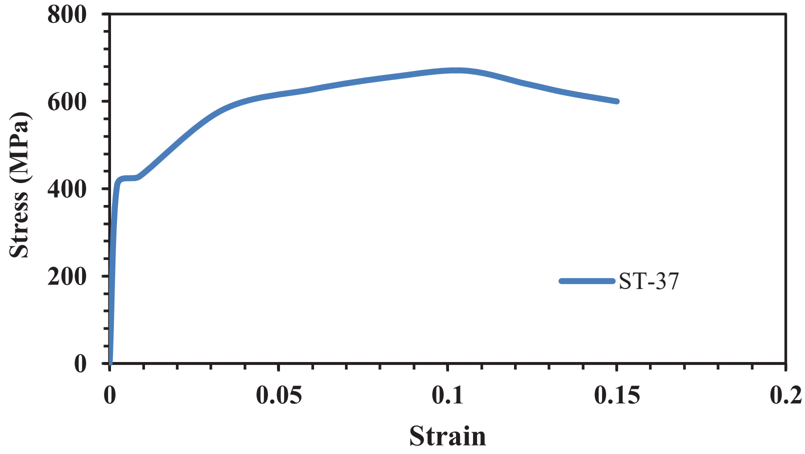


Fig. 2. Stress-strain relationship of steel components.

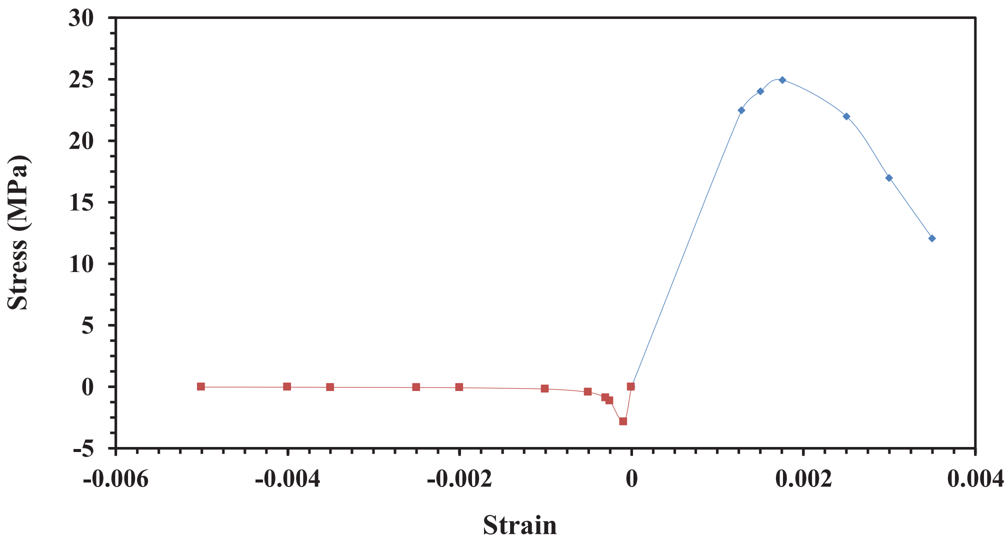


Fig. 3. Stress-strain relationship of concrete components.

The direct cracking failure strain was also assigned as 0.001. The shear retention model is assumed to be in a linear form in this research.

A strain rate-independent plasticity model with isotropic hardening has been considered as material behavior for steel components of the models. It's the most practical model for metal plasticity calculations based on the Mises yield surface [34] When the material is rate independent, its behavior is expressed as: (23. ABAQUS Inc., ABAQUS 6.5 Analysis User's Manual, 2004, SIMULIA.)

$$q = \sigma^0 \tag{1}$$

where is the yield stress and depends on the equivalent plastic strain. The nominal uniaxial stress-strain data should be converted to true stress and the logarithmic plastic strain in the isotropic material:

$$\sigma_{true} = \sigma_{nom}(1+\varepsilon_{nom}) \tag{2}$$

$$\varepsilon_{\ln}^{pl} = \ln\left(1+\varepsilon_{nom}\right) - \frac{\sigma_{true}}{E} \tag{3}$$

The plastic straining leads to changes in the yield stress uniformly in all directions which are discussed as isotropic hardening in the literature [34]. The stress-strain relationship is displayed in Fig. 2 for the steel material of the models. Fig. 3 displays the stress-strain relationship for the concrete slabs.

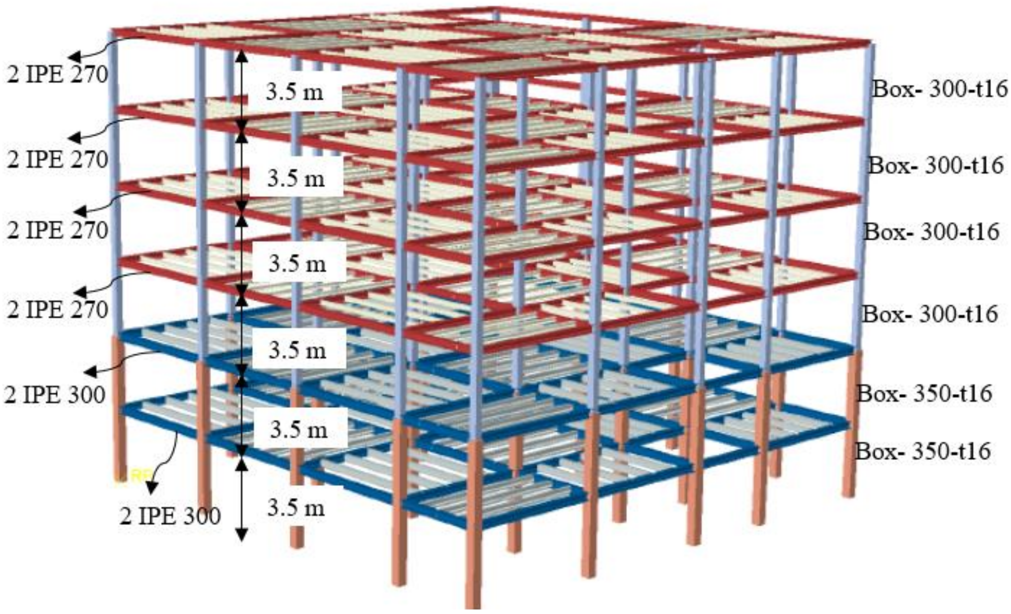


Fig. 4. 3D finite element modeling of a 6-story building.

3- 1- 2- Dimensioning of the models

Some efforts were made to study the effect of the slab openings on progressive collapse using the ABAQUS software. The numerical model includes a 6-story steel building with the same plan in all stories and a story height of 3.5 meters. The lateral load resisting system in both directions X and Y was assumed to be an intermediate moment Frame and the steel is of type ST37. The steel structure was loaded and designed conforming to ASCE 7-10 and AISC 360-16 using ETABS software [35, 36]. The panel zone effects were not considered in the prepared models.

At first, dimensioning of the structural members was carried out for stories of 3.5 meters in height. The main purpose of the structural design based on the described procedure is to ensure that the considered building performance can be completely representative of a typical structure in townships. The lower stories of the building include the first and second floors assigned to Type 1 of members profile sections and the higher floors are grouped into Type 2. Box sections of B350x16 and B300x16 were utilized for Type 1 and Type 2, respectively. The main girders of 2 IPE300 and 2 IPE270 were determined for Type 1 and Type 2. A steel deck including joists of IPE270 and IPE180 was considered for Type 1 and Type 2. The structural members were merged in the joints to act as a moment frame during the analyses. The "Tie" constraint technique was assigned to common joints of structural members and slabs to form integrated rigid diaphragms in the floors. In this paper, no special design has been considered for an alternative load transfer path to assess the capability of the conventional design procedure for maintaining structural integrity against terroristic threats.

Eight different cases were considered to analyze the steel structure of the model (Fig. 4), the parametric studies consist of the slab opening's location (corner or middle) and the slab opening's dimension (12x6 and 4x10). The progressive collapse potential was evaluated by the alternate path method. This method estimates the capability of the structure to substitute another alternate path for the load transfer to the ground when the normal load paths are degraded due to removal or damage in the main structural system. Two states of the progressive collapse potential were investigated in the current study. In the first one, the finite element model was analyzed without column removal, and in the latter one, two corner columns were removed on the ground floor (Fig. 5).

The load combinations of the UFC provisions were implemented for all cases [30]. The dynamic strain rate effect on the strength of materials was considered by multiplying static values by increasing coefficients.

3- 1- 3- Mesh size and boundary condition

Linear quadrilateral elements of type S4R have been selected for slabs and linear line elements of type B31 have been selected for beams and columns of the models. The mesh size has been determined based on the verification trial-error process and is considered to be 1 meter for both shell and beam elements. All rotational and translational degrees of freedom has been considered to be restrained for the end joints of the columns at the base level.

Table 3. Geometric properties of case studies.

Case	Number of stories	Column Remove Location	Slab opening location	Slab opening dimension
1	6	Intact structure	Corner	12X6
2	6	Intact structure	Middle	12X6
3	6	Two corner col.(E1,D1)	Corner	12X6
4	6	Two corner col.(E1,D1)	Middle	12X6
5	6	Intact structure	Corner	10X4
6	6	Intact structure	Middle	10X4
7	6	Two corner col.(E1,D1)	Corner	10X4
8	6	Two corner col.(E1,D1)	Middle	10X4

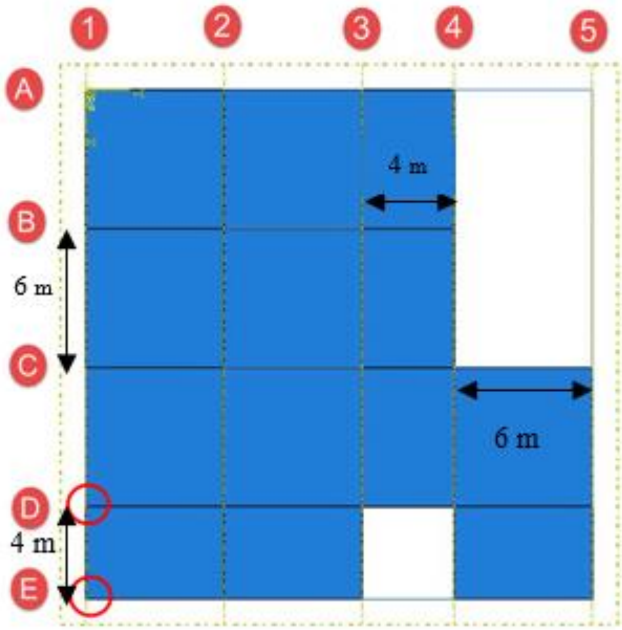


Fig. 5. The location of the removed columns in the numerical study.

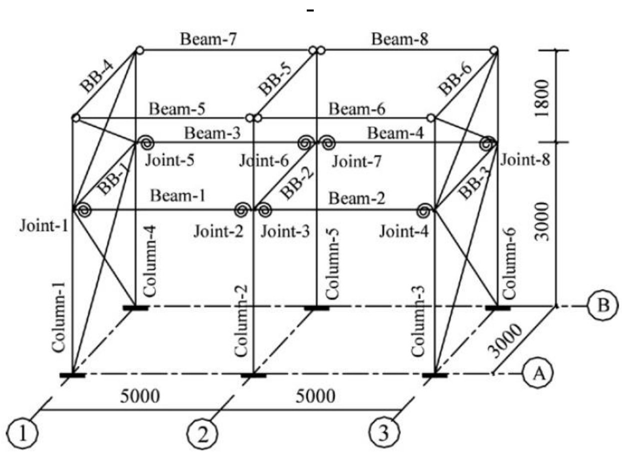
4- Verification of the FE Method

4- 1- Verification of numerical methods according to Fu et al. experimental results [37]

As illustrated in Fig. 6, the experimental program conducted by Fu et al. on a 2-story building with a steel-concrete composite frame and composite slab system was selected to validate the numerical method. Fig. 7 shows the composite frame in the Fu et al. test setup [37]. As the numerical study of progressive collapse is conducted through a dynamic explicit analysis using ABAQUS software therefore the test setup introduced by Fu et al. has been selected because of its similarity to our main models. Although the main purpose of the Fu et al study is to study the flexural behavior of composite building frames, all essential parts of their experimental model are so similar to our selected models.

The FE modeling technique described in the Feng Fu research report was implemented to simulate the experimental frame in the ABAQUS software [33]. Fig. 7 illustrates the dimensions and the section properties and the location of the applied loads in the frame model. The moment resisting connection was considered for the beam-to-column joints and the columns connection to the foundation was assigned to be fixed. The Fu et al. study results on the (frame “A”) were used to validate the FE method of the current study. Frame “A” was modeled and loaded to reach the ultimate capacity of the frame [37]. The material properties of the test model are the same as those explained in section 3.1.1 of this paper. As the main purpose of this experiment was to determine the capacity of steel frames under vertical loading, therefore the test model was loaded at the points illustrated in Fig. 7 till the

Fig. 6. Fu et al.'s test setup [37].



(a) General layout, Units in mm.

Steel column:

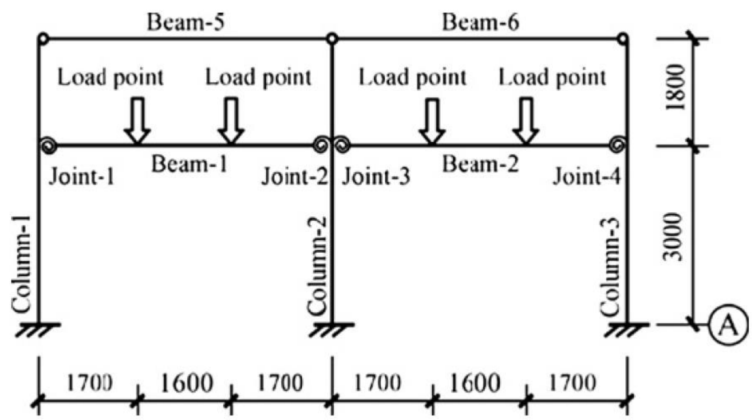
Column-1~6:H-250X250X9X14

Steel beam:

Beam-1~4:H-300X150X6.5X9

Beam-5~8: H-200X100X5.5X8

Beam-1~4: H-200X100X5.5X8



(b) Loading point places at both main frames of the 3D test model, Units in mm.

Fig. 7. Frame size and the dimension of the full-scale frame tested by Fu et al. [37].

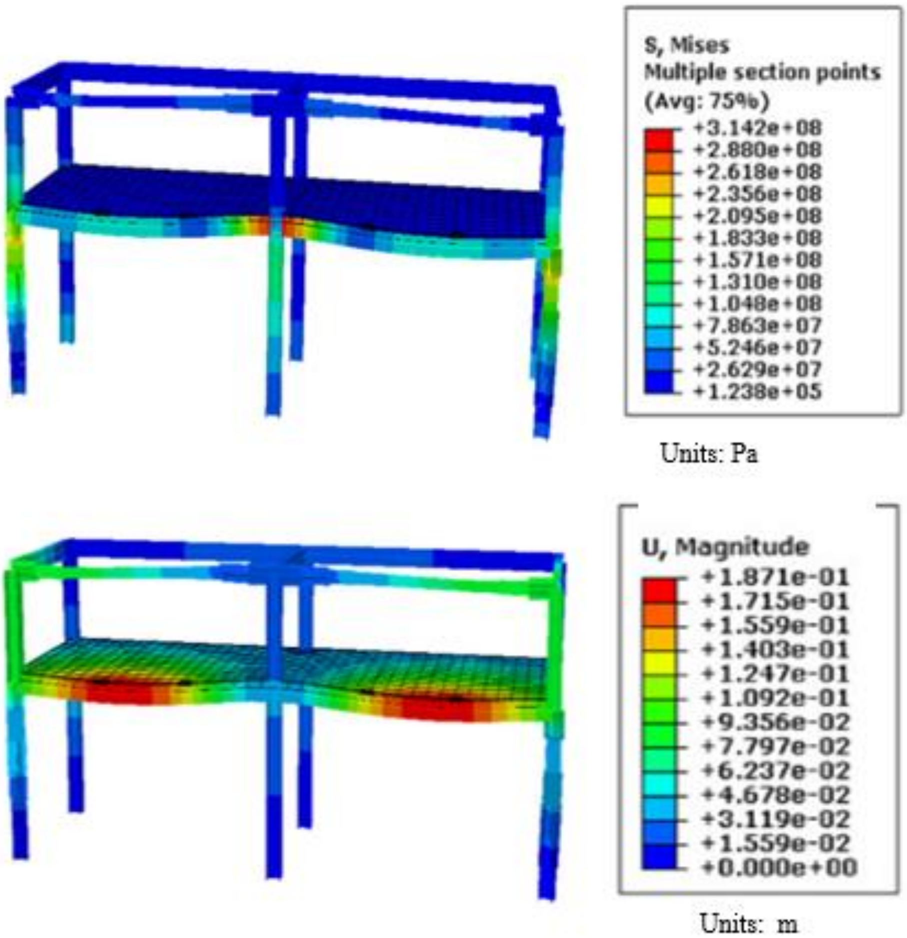


Fig. 8. Stress and displacement results of the verification model in the ABAQUS software.

failure of the structural steel connections.

Fig. 8 displays the general numerical results of Fu et al. experimental experience which has been used to determine the mesh size of the beam and shell elements and also to ensure the material model definition for concrete slab and steel components. Fig. 9 compares the FE model and experimental results for the moment rotation diagram of joint 1. The main purpose of this comparison is to determine the accuracy of the selected FE method to analyze the 3D composite steel-concrete frames. The total error of the numerical results concerning test results has been calculated based on the under-curve parameter and is 7.2 percent. As shown in Fig. 9, there is a fair agreement between the experimental and numerical results for the maximum moment capacity and ultimate rotation. And also, the variation of the finite element model results is similar to the experimental ones.

4- 2- Verification of numerical method through Jiang et al. test results [16]

A summary of geometric features of the considered steel frame and vertical loading properties are presented in Fig. 10. The steel frame section and loading weights are given in Table 4.

The experimental program was conducted based on a column removal scenario using thermal loading on the selected column. Fig. 11 compares the experimental set-up of the Jiang et al. study and the nonlinear analysis of the test model using ABAQUS software. The displacement versus temperature is reported for the point above the removed column based on the test results and the verification numerical model in Fig. 12. The comparison of the curves of Fig. 12 shows fairly good agreement with the baseline of the test results.

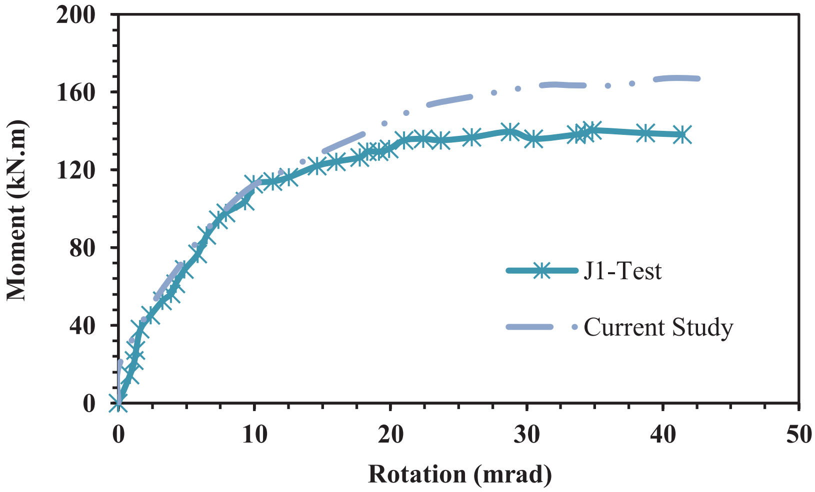


Fig. 9. Moment rotation of finite element and laboratory model [37].

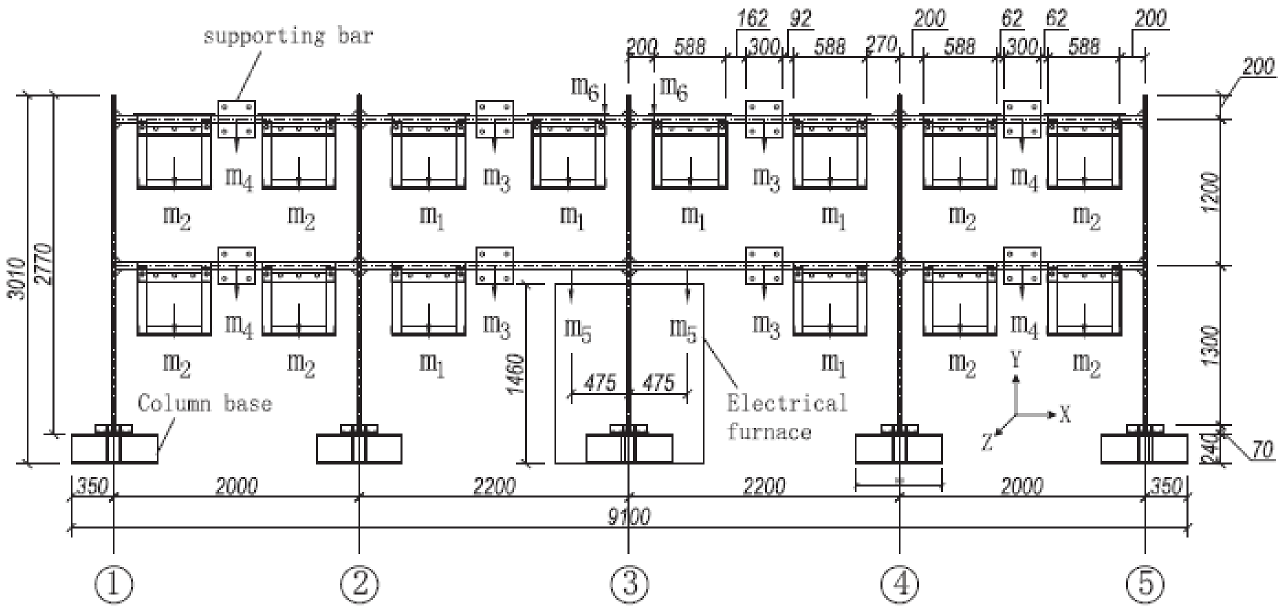


Fig. 10. Jiang et al.'s test setup [16].

Table 4. Cross-section details of members and loading weights [16].

Column	Middle bay beam	Side bay beam
Box-50x30x3	Box-150x50x5	Box-60x40x3.5

m_1(N)	m_2(N)	m_3(N)	m_4(N)	m_5(N)
2910	1760.7	701.3	73.9	74

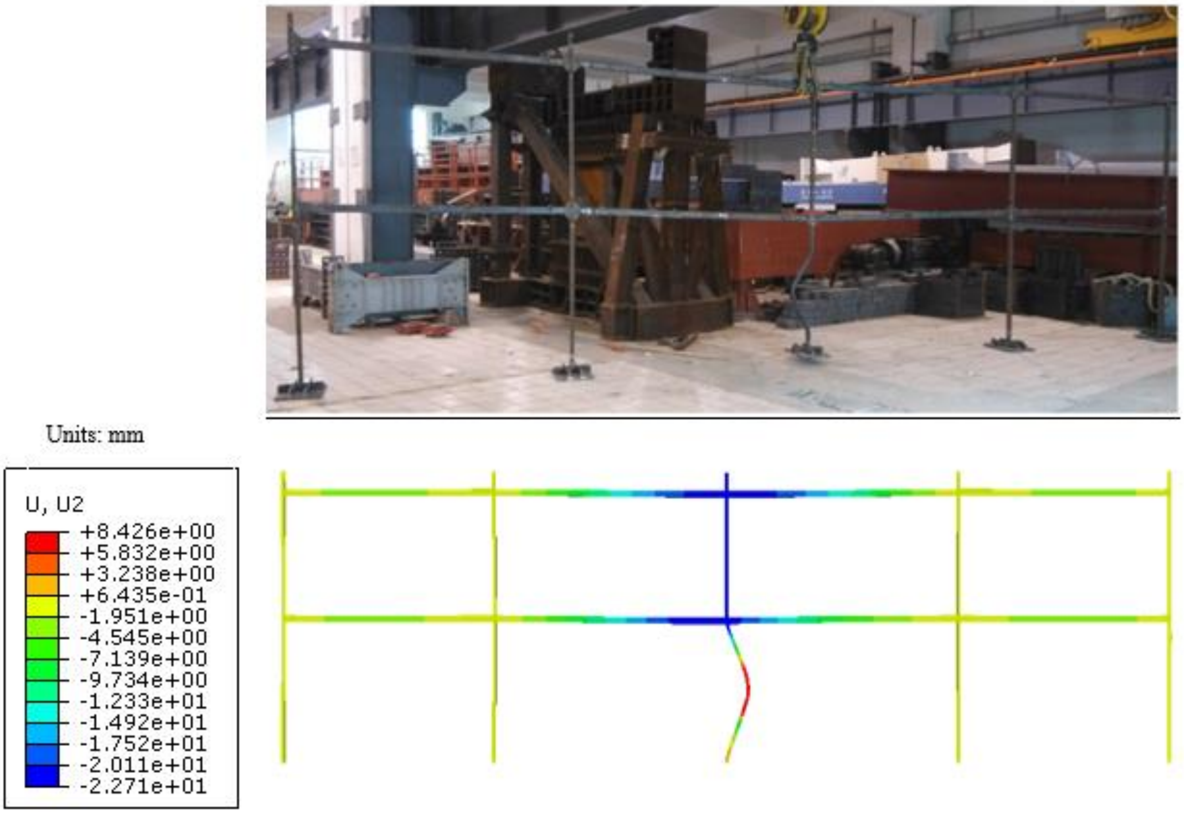


Fig. 11. Final status of the test sample and analysis displacement contour.

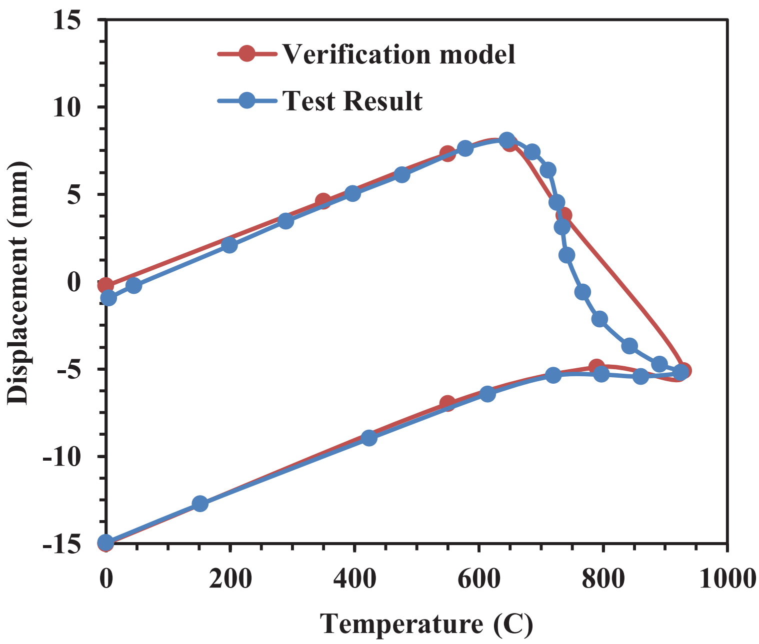


Fig. 12. Displacement result for the point located at the top of the removed column.

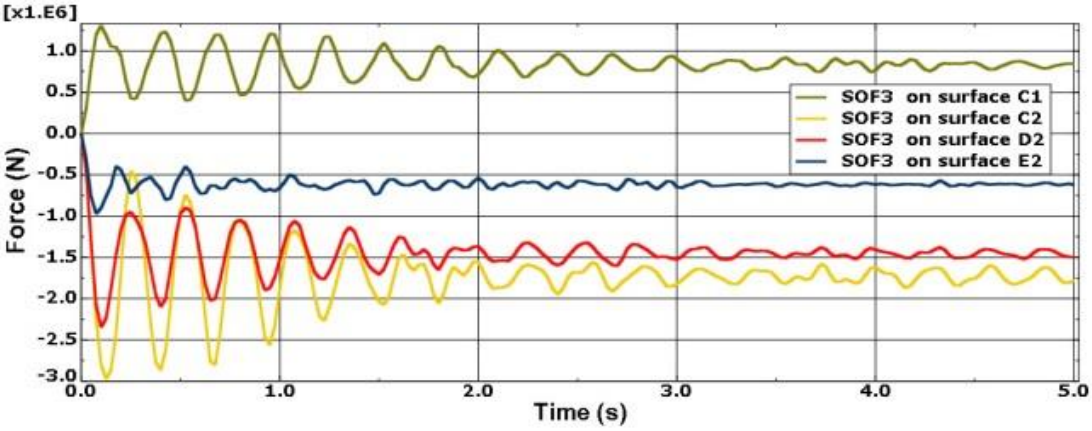


Fig. 13. The exerted load in the members located in the vicinity of the removed column (case 1).

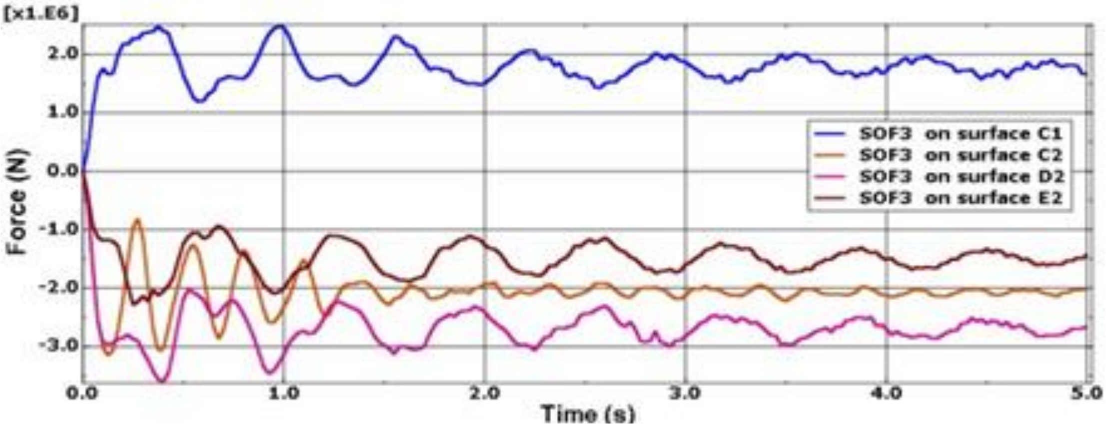


Fig. 14. The exerted load in the members located in the vicinity of the removed column (case 3).

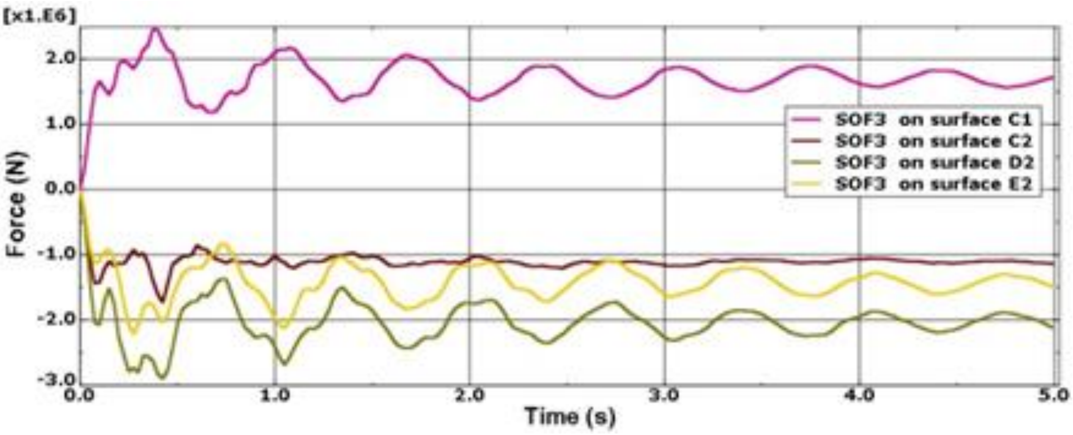
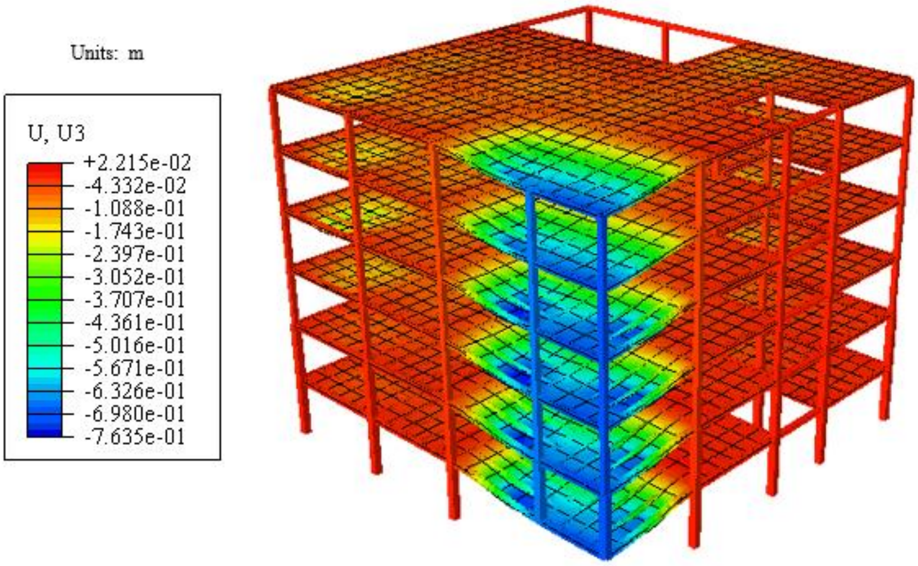


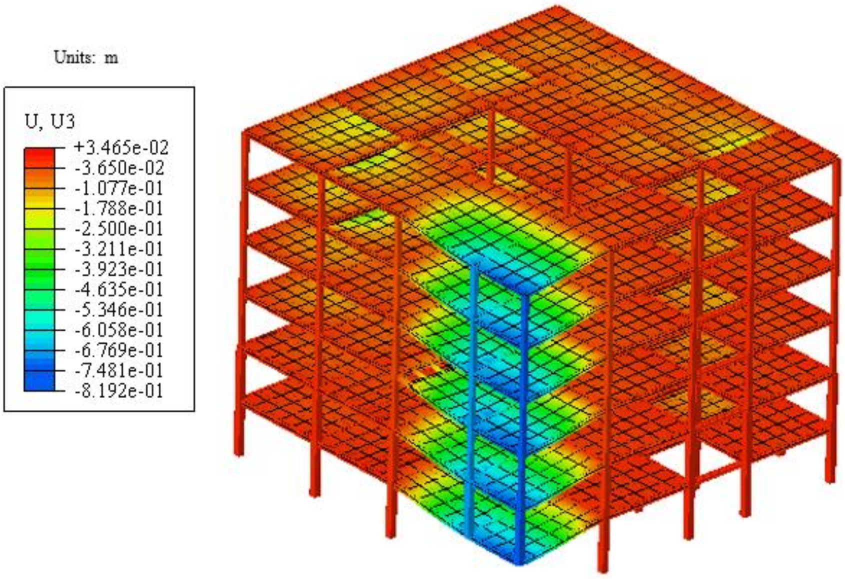
Fig. 15. The exerted load in the members located in the vicinity of the removed column (case 7).

5- Results and Discussion

A parametric study has been performed on the slab opening size and location effects on the typical buildings located in the small metropolitan area. The main variable is the member's capacity, so, a reduction in demand-to-capacity ratios proves the efficient structural behavior against progressive collapse. The analytical results are presented for selected cases based on the adjacent column axial force variation and the vertical displacement contour. The results for case studies 1, 3, and 7 are provided in Figs. 13 to 15, respectively.



(a) Case 3



(b) Case 4

Fig. 16. Displacement contour for cases 3 & 4.

The behavior of the simulated structure was studied under sudden column removal using axial load redistribution. Also, the axial load variations of the columns adjacent to the column removal were calculated in the intact and damaged states for different opening locations and dimensions. As illustrated in Fig. 16, vertical displacement in case 3 is slightly less than in case 4. Since the opening size is equal; therefore, the difference is related to the location of the opening. Figs. 17 to 20 illustrate the maximum values of the column force variation for each case study described in Table 3. The comparison between the desired states and the parameters is provided in separate bar charts. Fig. 21 shows the plastic strain formation at the ends of the beams. The plastic region appears next to the columns in the vicinity of the removed columns. The results show that the maximum plastic strain has a more critical value for the case with middle openings.

The results comparisons are provided for different slab openings locations in Figs. 17 and 18. The corner opening leads to achieving the maximum axial force variation for the columns (E2 and C1) in the outer frame. Also, the middle openings of size (6X12 and 4X10) meters cause to make higher axial force variations in the interior frame columns (C2 and D2).

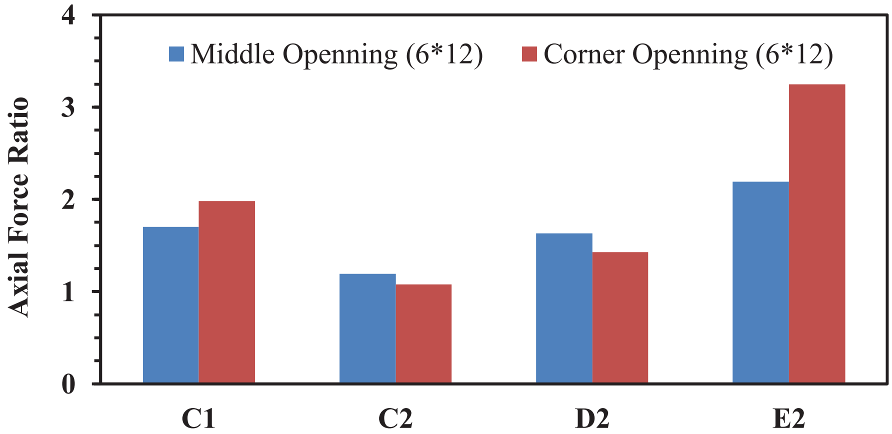


Fig. 17. Effect of the opening location on the openings of dimension (6*12 m).

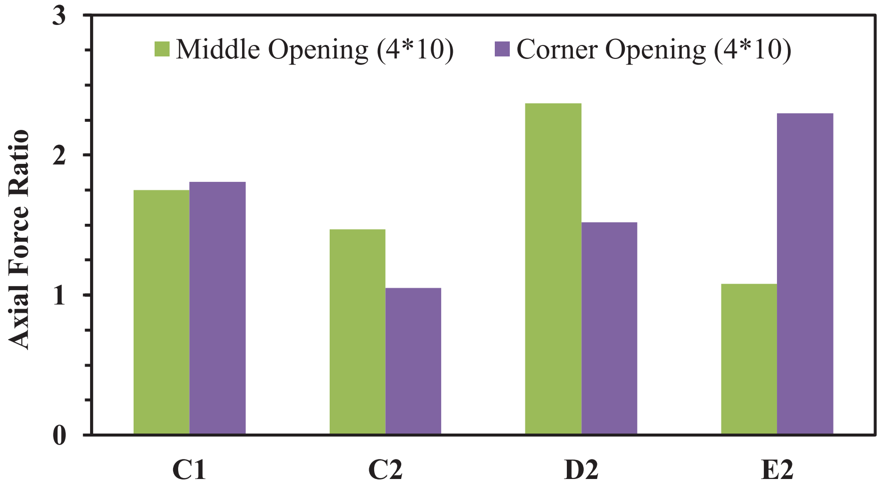


Fig. 18. Effect of the opening location on the openings of dimension (4*10 m).

Figs. 19 and 20 compare the results for different dimensions of the slab openings. As indicated in Fig. 19, a reduction in the size of the corner opening increases the bearing capacity of the structure and also reduces the axial force exerted on the columns adjacent to the removed column (E2). In contrast to the cases with the corner opening, if the opening is located in the middle of the structure plan, increasing the opening size will positively improve the structure's vertical bearing capacity. Fig. 20 describes the behavior of the structure in the cases with internal opening by the axial force variation of columns C2 and D2. For the cases discussed in Fig. 20, axial load variation values of columns C2 and D2 have been increased by widening the opening from 4X10 meters to 6X12 meters. However, for the cases with the corner openings, a reduction of the opening size leads to an increase in the load capacity of the columns placed in the

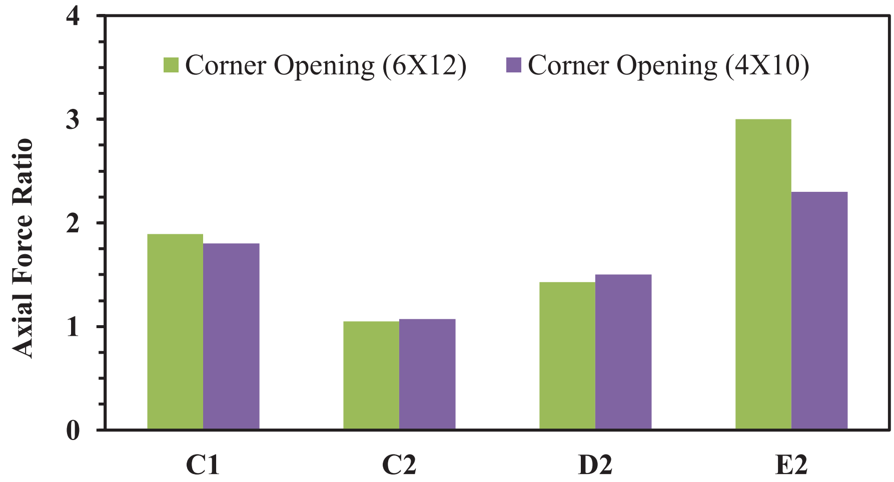


Fig. 19. Effect of the opening dimension on the corner openings.

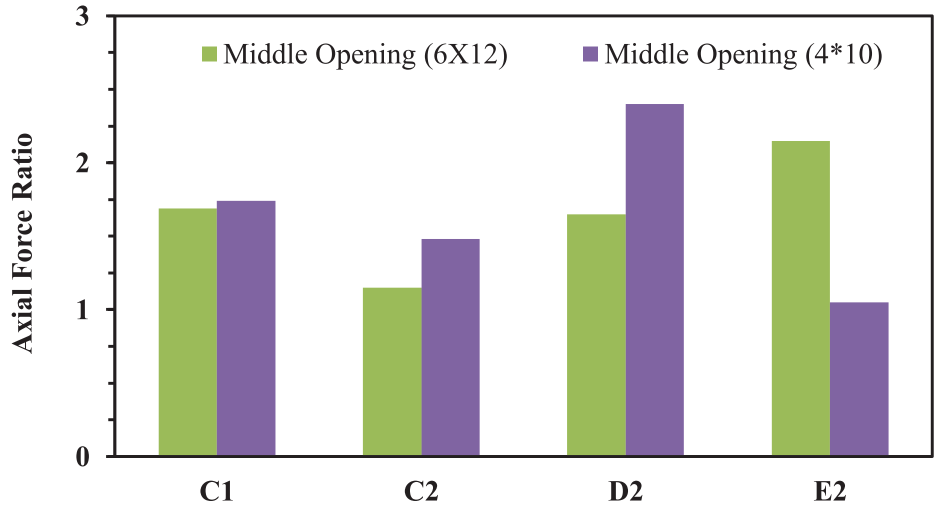
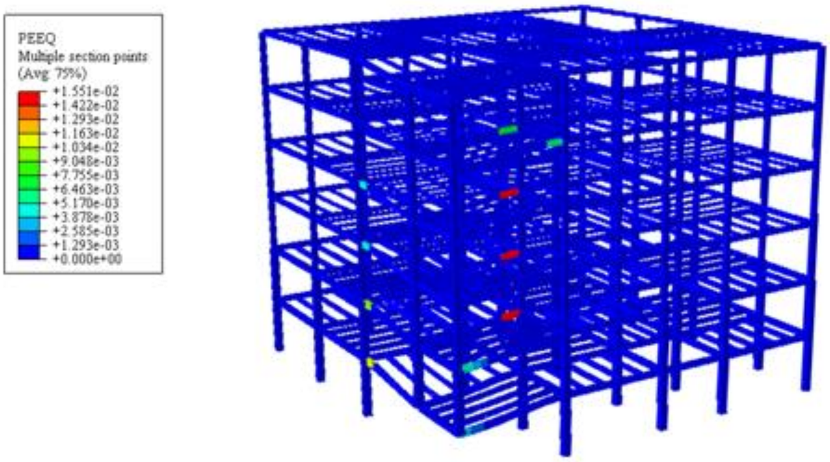


Fig. 20. Effect of the opening dimension on the middle openings.

vicinity of the column removal location. As the column web is assumed to be stiffened enough and there are no panel zone effects on the steel connections so as can be seen from Fig. 21, the plastic hinges are formed in the regions of the beams near the connections. Fig. 22 compares the effect of opening with different sizes and positions on the maximum sudden deflection due to column removals. The results show that middle openings can increase the sudden deflection by up to 11% concerning corner openings. As it is illustrated in Fig 23, the most critical plastic hinge is selected in the beam where located in the vicinity of the removed columns to obtain critical rotation of the beams due to the column removal. The maximum rotation of the beam reduces by changing the opening location from the middle part to the corner part of the plan. The higher location size also leads to higher rotations at the critical point on the beams. The rotation variation due to the above-mentioned parameter is so higher than the effect of these parameters on the displacements reported in Fig. 22. Changing the location of the openings increases the critical rotation up to 45 percent.



(a) Case 3

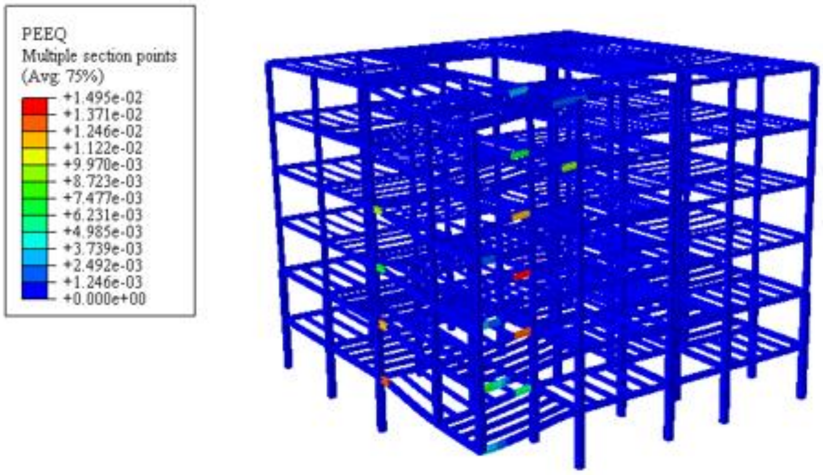


Fig. 21. Plastic Strain contour for cases 3 & 4.

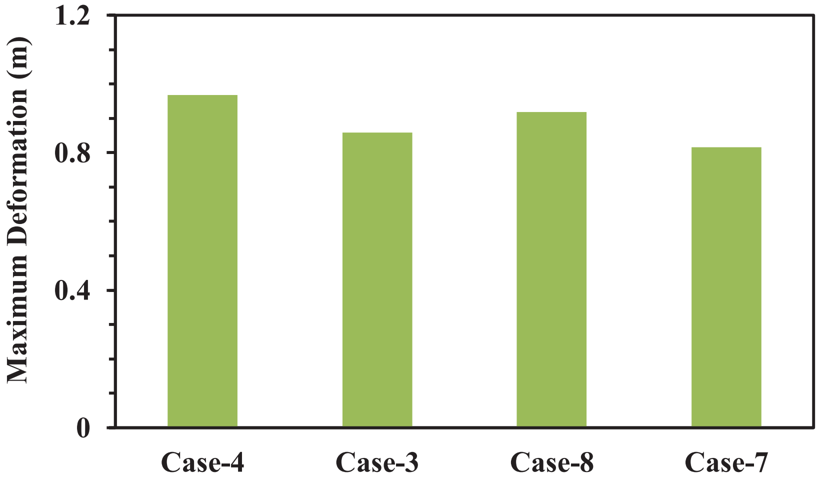
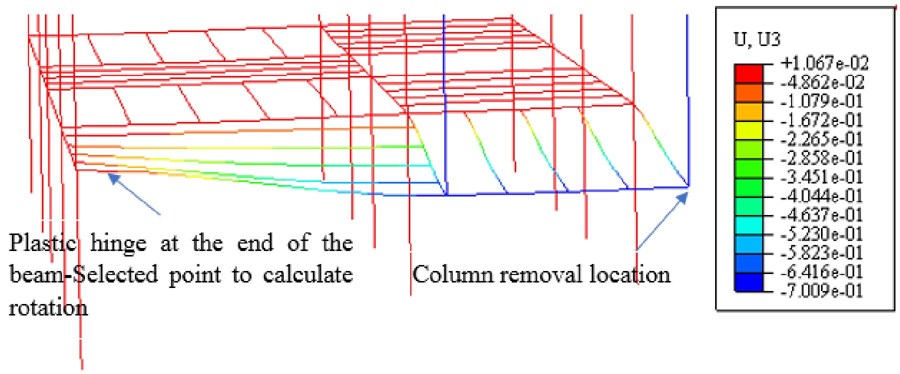
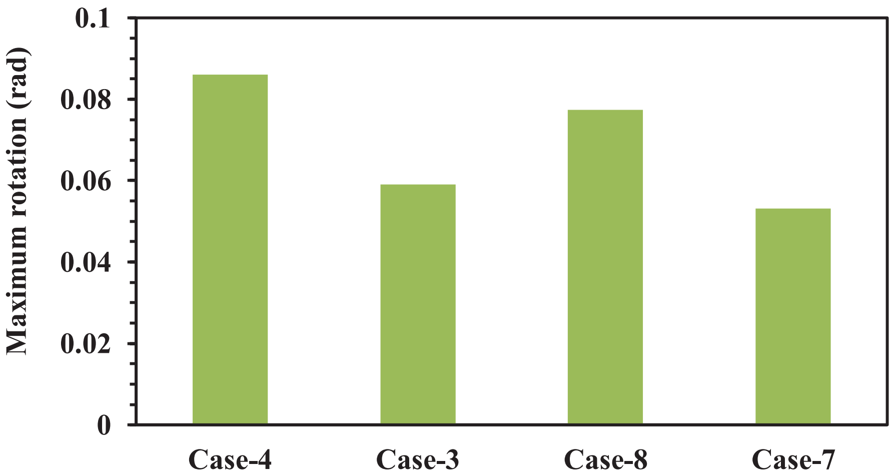


Fig. 22. Effect of the opening dimension and location on the maximum sudden deformation due to column removal.



(a) Selected point to obtain rotation value in case 3.



(b) Rotation values at the critical points in each case.

Fig. 23. Effect of the opening dimension and location on the maximum rotation of the critical beam due to the column removal.

6- Conclusion

In this paper, the progressive collapse development is investigated in the models with different slab opening sizes and locations using the Alternate path method. Several numerical analyses are carried out to evaluate the potential of the progressive collapse in the typical building under explosive loads. The analytical results are validated using reliable experimental results. No special consideration has been made for strengthening the structure against sudden events to measure the capacity of the typical structure for column removal scenarios. The ratio of the axial forces represents the improvement of the structural behavior in the progressive collapse (Capacity Control). The results of this paper are as follows:

1) Increasing the opening size in the middle of the structure plan leads to improving the structure load resisting capacity due to unloading the columns in the vicinity of the column removal.

2) Decreasing the corner opening size cause to produce more structural integrity between the outer frame columns and the slab-bearing system. Therefore, column removal is less destructive for the structural resisting system.

3) The dimension of the middle openings plays a more significant role in the progressive collapse development.

Finally, the numerical results demonstrate that the location and the dimensions of the openings can effectively change the structural behavior and bearing capacity. The special attention of the design provisions to the location and dimensions of the slab openings reduces the potential of progressive collapse development in the buildings.

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HOW TO CITE THIS ARTICLE

A. Shokoohfar, A. H. Gholizadeh, S. Jani, A Finite Element Analysis of Slab Opening Effects on the Column Removal Scenarios in Large Buildings , AUT J. Civil Eng., 6(2) (2022) 263-282.

DOI: 10.22060/ajce.2023.21169.5795