



## Research Article

# Numerical investigation of reinforced concrete frame behavior subjected to progressive collapse

Mohammad Bagher Paripour\*, Ahmet Budak, Oğuz Akın Düzgün

Department of Civil Engineering, Atatürk University, 25240 Erzurum, Turkey

## ABSTRACT

Progressive collapse is defined as the spread of an initial local failure of a structure. This phenomenon, caused by the removal of one or more load-bearing element, is followed by a chain of failures through the structure and ultimately leads to partial or even full collapse of an entire structure. As a result, an accurate understanding of structural behavior subjected to large displacements, caused by progressive collapse, is essential to ensure a safe structural design. A progressive collapse in buildings often starts with the removal of one or more columns and continues with the collapse of adjoining structural elements. Experimental studies on progressive collapse are generally not recommended because of its cost and safety reasons. Today, as a result of progress in computer technology, more complicated problems can be investigated numerically. In this study, a numerical model is used for nonlinear analysis of a reinforced concrete (RC) frame behavior subjected to progressive collapse. It is obtained that there is a good agreement between the results with those of the experimental study given in the literature. According to the results, it can be predicted numerically the response of an RC frame to progressive collapse at a highly accurate level.

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## 1. Introduction

In structural engineering, it is always aimed to predict some factors that significantly affect the durability and strength of a structure during its lifespan. These factors should be taken into account by the design engineer during the design process to ensure the structural stability under progressive collapse. The progressive collapse is defined as the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or disproportionately large part of it. The key characteristic here is that the total damage at the end is not proportional to the original cause (ASCE, 2016; Bažant and Verdure, 2007; Starossek, 2009). The first progressive collapse event which attracted engineers' and researchers' attention was the partial collapse of Ronan Point (a 22-story tower block) in 1968, initiated by a gas explosion in the 18th story, causing local progressive collapse of all stories (Pearson and Delatte, 2003). The final state of the collapse of the Ronan

Point apartment building is shown in Fig. 1. The progressive collapse of Alfred P. Murrah Building in 1995 was another big event that attracted the second wave of attention (Corley et al., 1998). Due to the high mortality and economic losses from attacks on the twin towers of the World Trade Center (WTC) in 2001, progressive collapse once again attracted different entities to address this phenomenon more carefully and seriously (Reeve, 1999).

The National Building Code of Canada (NBCC) (2005) has set out the requirements for the design of principal elements, the connection of elements, and alternative load path methods. The ACI 318 (2014) imposes the structural integrity requirements to prevent entire structural collapse subjected to a local failure. The ASCE 7 (2016) provides a design method and a tool for determining the load combinations. It also addresses structural integrity such as ACI 318. Recently, the General Services Administration (GSA) (2003) and the Department of Defense (DoD) (2009) of the United States developed functional

\* Corresponding author. Tel.: +90-539-2018550 ; E-mail address: mohammadbagher.paripour12@ogr.atauni.edu.tr (M. B. Paripour)

design guidelines to reduce the risk of progressive collapse. These guidelines present the alternative load path (ALP) method as the best technique to assess building vulnerability to progressive collapse. The dominant view in ALP is the use of alternative load path for handling gravity load in case of failure in the initial load path.



**Fig. 1.** Ronan Point apartment building after collapse (Pearson and Delatte, 2003).

Many studies have conducted investigations on the progressive collapse in recent years. Kaewkulchai and Williamson (2003) investigated a 2D model to compare the static and dynamic analyses of progressive collapse. According to their findings, the static analysis alone cannot represent the actual behavior of a structure subjected to progressive collapse. As a result, the dynamic behavior following the removal of the column has a significant effect in this regard. Iwankiew and Griffis (2004) investigated the progressive collapse mechanism in different structures and concluded that the architectural plan of a structure has a significant effect on its progressive collapse resistance performance. To obtain real results using static analysis, Ruth et al. (2006) proposed the use of increase factor of 1.5 for imposed loads. In addition, the relevant recommendations of these codes were compared. Bao (2008) developed 20 macro models to simulate the nonlinear behavior of beam-column connections in an RC frame and showed that the use of macro models is a suitable method for progressive collapse analysis by comparing the obtained results with experimental ones. Yi et al. (2008) investigated a 2D RC frame with four spans and three stories. During the experiment, the middle column of the lower story was removed. This experiment showed that the frame failure occurs due to the tensile rebar failure. Yu and Tan (2013)

investigated the progressive collapse in an RC frame by testing two samples (one vibrational and one non-vibrational) based on the ALP method. In this way, they studied the effect of vibrational design on the frame behavior under progressive collapse. Sasani et al. (2007) investigated the behavior of a 9-story RC frame subjected to progressive collapse and concluded that the Vierendeel frame action is the dominant mechanism in load redistribution and that the greater flexural rebar strength results in greater progressive collapse resistance.

The present study investigated progressive collapse in an RC frame to examine the critical effects of removing a vertical supporting element on the response of an RC structure. As a result, a numerical model was developed in ABAQUS (2012) to perform nonlinear analysis of the RC frame behavior under a progressive collapse. Results of an experimental study given in the literature were compared to those of the present study to ensure the accuracy of numerical results.

## 2. Finite Element Modeling

In this study, experimental results given by Sagioglu (2012) were used to model progressive collapse mechanism and evaluate the accuracy of the finite element model. The specifications of the RC frame and mechanical properties of the concrete and steel materials were considered to be the same as those of the experimental study. The structure was designed in compliance with ACI. The concrete damaged plasticity model was used to define the concrete. In this model, cracking occurs when tensile stresses exceed the tensile strength of concrete. The stress-strain curve of the concrete in the compressive area was developed by the experimental study (Sagioglu, 2012). The compressive strength of the concrete was equal to 25 MPa. The tensile strength of the concrete was considered to be  $0.62\sqrt{f'_c}$ , where  $f'_c$  is the compressive strength of the concrete. The tensile stress-strain curve of the concrete was drawn based on the model proposed by Hillerborg (Kwak and Filippou, 1990). The stress-strain equation of the concrete was considered according to the curve in Fig. 2(a).

The following properties for the steel material were used in the modeling: density of  $7860 \text{ kg/m}^3$  modulus of elasticity of  $2.1 \times 10^5 \text{ N/mm}^2$ , Poisson's ratio of 0.3, yield stress of 380 MPa, and ultimate stress of 530 MPa. The stress-strain curve of the steel materials was considered bilinear with strain hardening in the plastic area. Figure 2B represents the stress-strain curve of steel material. In the modeling, a 3D 8-node element type that uses the integration by reduction formula to solve the integrations was selected for modeling the concrete elements, while 3D 2-node element type was used for modeling longitudinal and transversal rebars, which were embedded in the concrete (ABAQUS, 2012).

According to GSA (2003), a dead load plus 25% live load were introduced to the beams. The dead load, live load, and load from surrounding walls were considered to be  $690 \text{ kN/m}^2$ ,  $482 \text{ kN/m}^2$  and  $690 \text{ kN/m}^2$ , respectively. Meshing was carried out through the regular meshing technique. The mesh size was selected in a way

that the analysis results became relatively independent of the grid. In addition, the analysis speed was considered in the determination of grid size. To remove the

column in the dynamic stage, the \*Remove Element was used and the middle column of the first story was removed in 0.002 seconds.

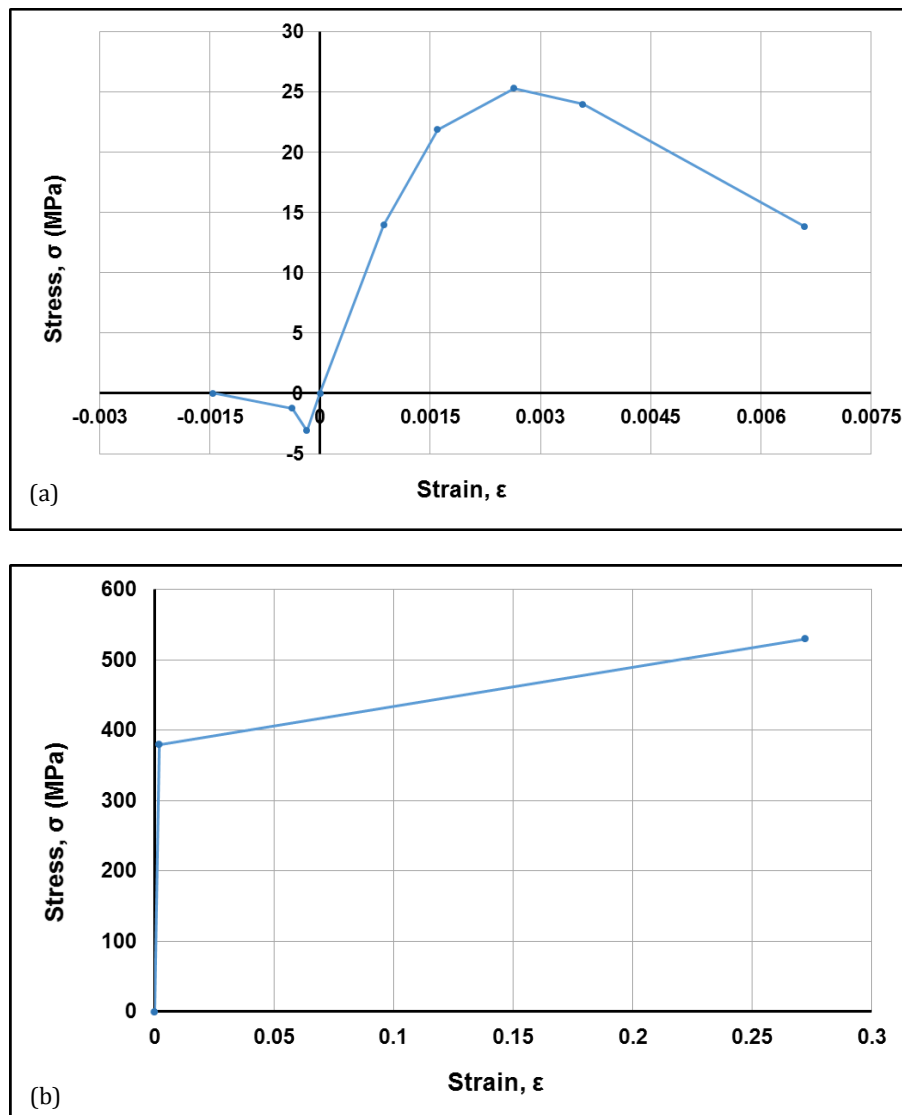


Fig. 2. (a) Concrete stress-strain curve for concrete damaged plasticity model; (b) Stress-strain curve of steel.

### 3. Details of the Investigated Model

In this study, an RC frame with three stories and four spans was considered as provided by Sagioglu (2012). To meet the instantaneous column removal conditions in Sagioglu's experiment (2012), the middle column of the first story was built with glass. In this way, the RC frame resistance to progressive collapse could be investigated with sudden smash of this glass column. Fig. 3 presents the investigated RC frame. This experimental test was carried out in two stages. In the first stage (dynamic stage), the static load was introduced to the beams and the dynamic behavior of the structure after sudden removal of the column was evaluated. Fig. 4 presents the mechanism of loading at this stage. The beam weight was considered as its dead load without applying other loads.

In the second stage (static stage), the load introduced to the beams in the first stage was eliminated, and a linear incremental displacement along the column was removed and introduced to the roof level. In this way, the performance of RC frame subjected to progressive collapse was evaluated. Fig. 5(a) shows dimensional properties and details of the beam and column reinforcement in the first and second stories. Fig. 5(b) depicts dimensional properties and details of the beam and column reinforcement in the third story. The beam has been subjected to uniformly distributed loads. To simulate the boundary conditions, all columns were fixed on the ground. To this end, all transitional and rotational degrees of freedom in finite element software was supposed to be zero. Fig. 6 presents the developed model with its loading and boundary conditions.



**Fig. 3.** RC frame tested by Sagioglu (2012).



**Fig. 4.** Mechanism of static loading in Sagioglu's experiment (2012).

#### 4. Comparison of Results from Numerical Modeling and Experimental Study

The history of vertical displacement at the roof level was obtained along the removed column and presented in Fig. 7. According to experimental results, there is a peak vertical displacement of 10.6 mm during the vibration, and a permanent displacement of 9.9 mm after approximately 1.25 seconds. According to the numerical results, there is a peak vertical displacement of 11.5 mm during the vibration, and a permanent displacement of 10.4 mm at the end of vibration. The difference between experimental and numerical results based on

the permanent displacement after vibration was approximately 5%. Therefore, it can be said that the finite element modeling results are well consistent with experimental data.

In the second stage, with the introduction of linear incremental loading, the middle column at the roof level was displaced downward by 412 mm. Fig. 8 presents the frame deformation in the ultimate displacement during the experiment. Fig. 9 presents the frame deformation in the finite element model during the experiment. A good agreement was observed between the experimental and numerical results in terms of the mechanism of RC frame failure and the location of plastic hinge formation.

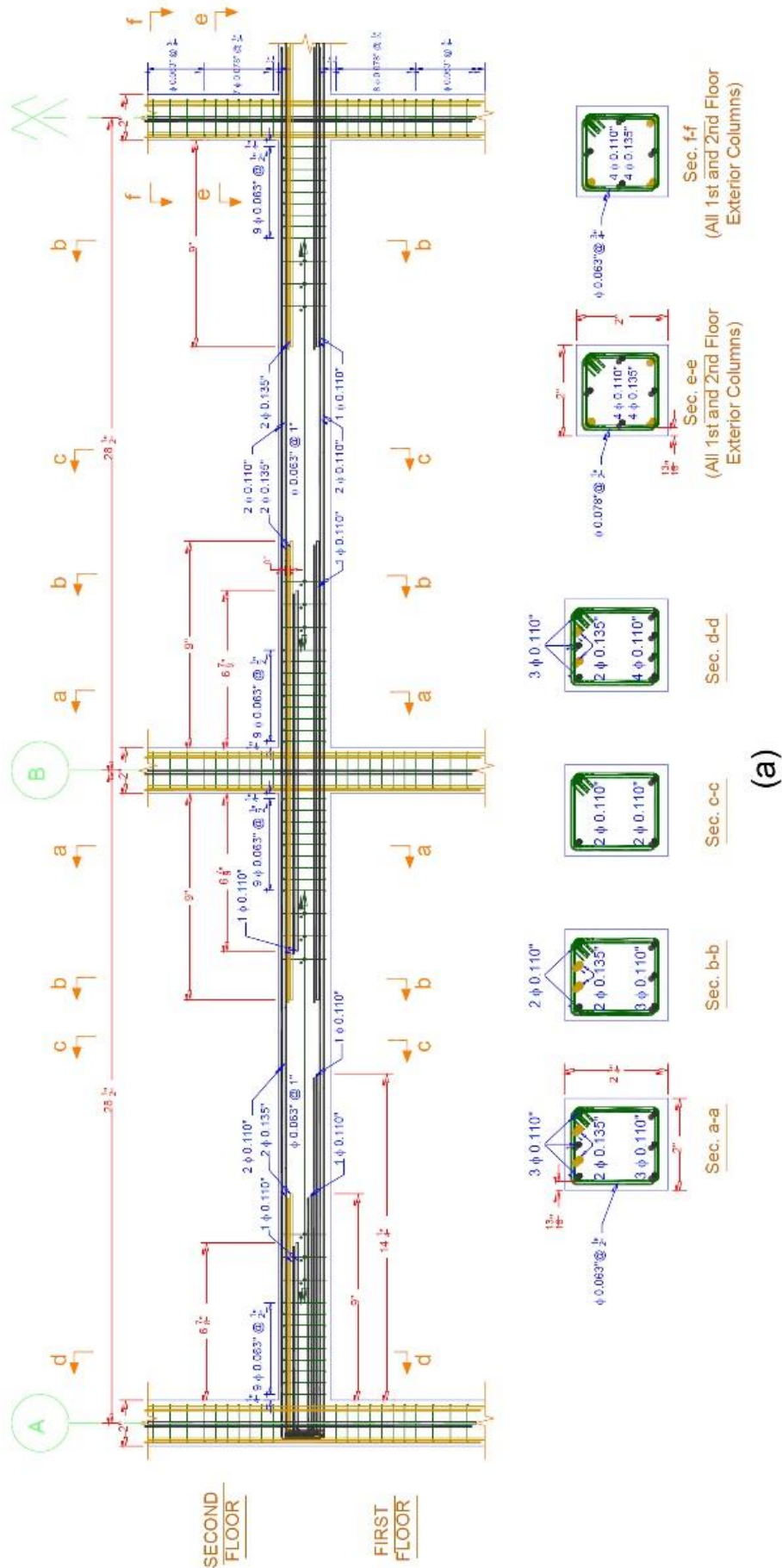


Fig. 5. (continued)



Fig. 10 presents the experimental and numerical results pertaining to vertical load changes based on vertical displacement of the middle column in the second stage of the experiment. At this stage, the resisting force increased to 8.1 kN and the beam bending failure took place due to the flexural strength of beams at both sides of the middle span. After the bending failure of the beams, the resisting force increased gradually and slightly and reached its peak level (8.81 kN) at vertical displacement of 119mm.

After this stage, a sudden reduction was observed in the resisting force with formation of the plastic hinge in the beam. This reduction was up to 5.89 kN for vertical displacement of 228mm. After the formation of plastic hinge in the beams, the resisting force increased again, because of the catenary action, and finally increased to 9.42 kN for the peak vertical displacement of 412 mm. Accordingly, there is a good consistency between the results from finite element modeling and experimental data.

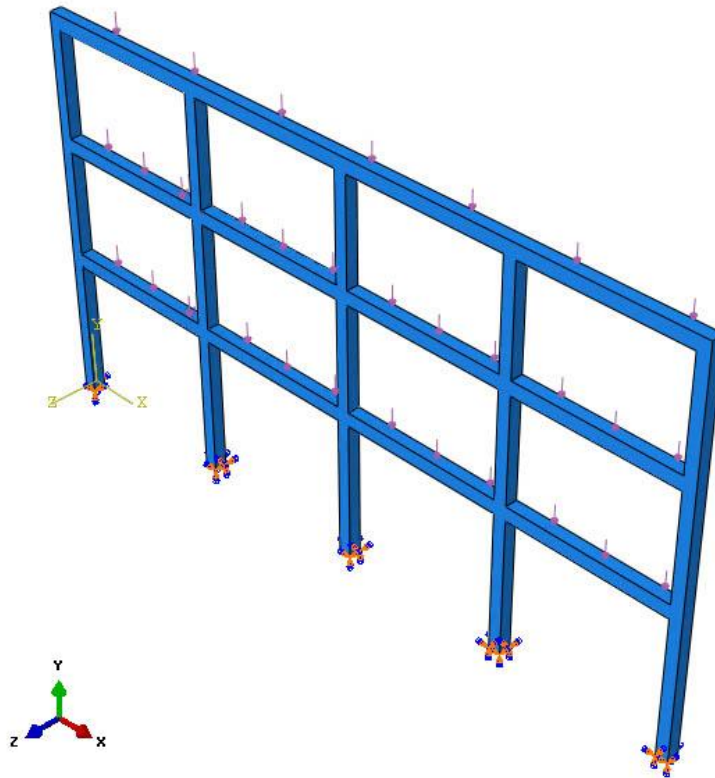


Fig. 6. Developed model with loading and boundary conditions.

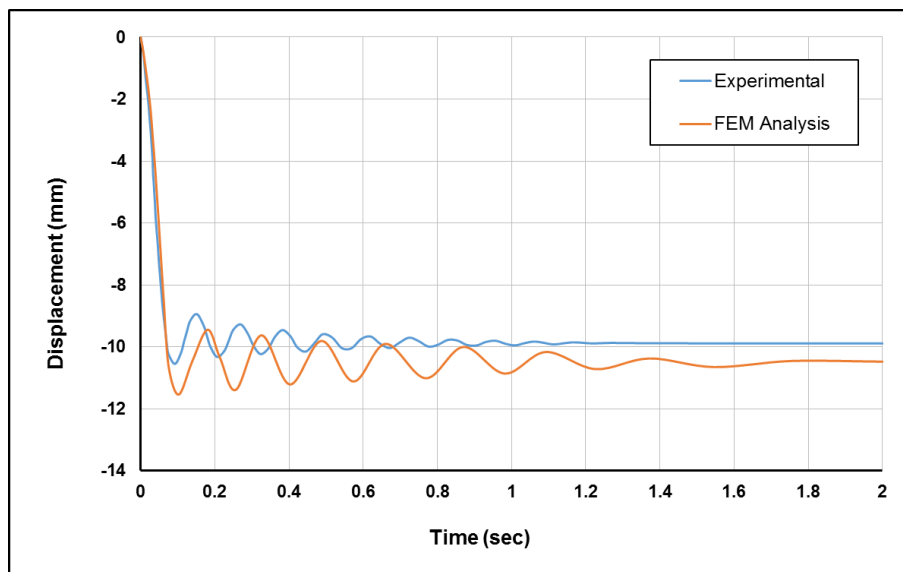


Fig. 7. Vertical displacement at the floor level of the middle column in the dynamic stage.

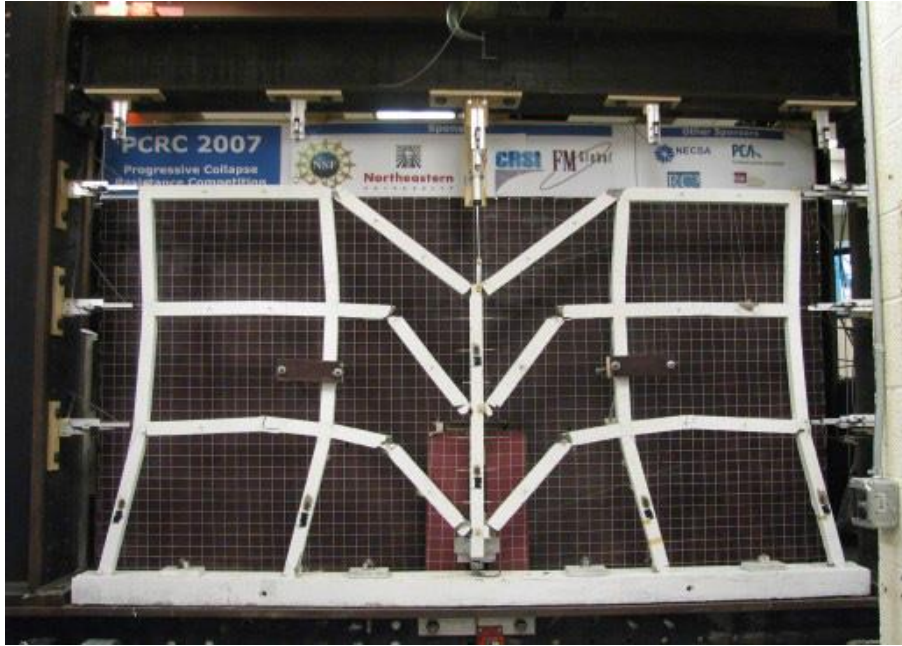


Fig. 8. Deformed frame in the experimental study (Sagioglu, 2012).

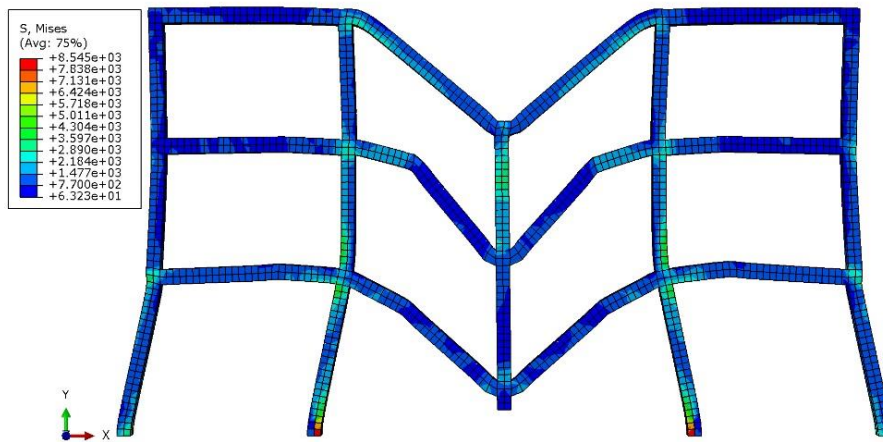


Fig. 9. Deformed frame in the finite element model.

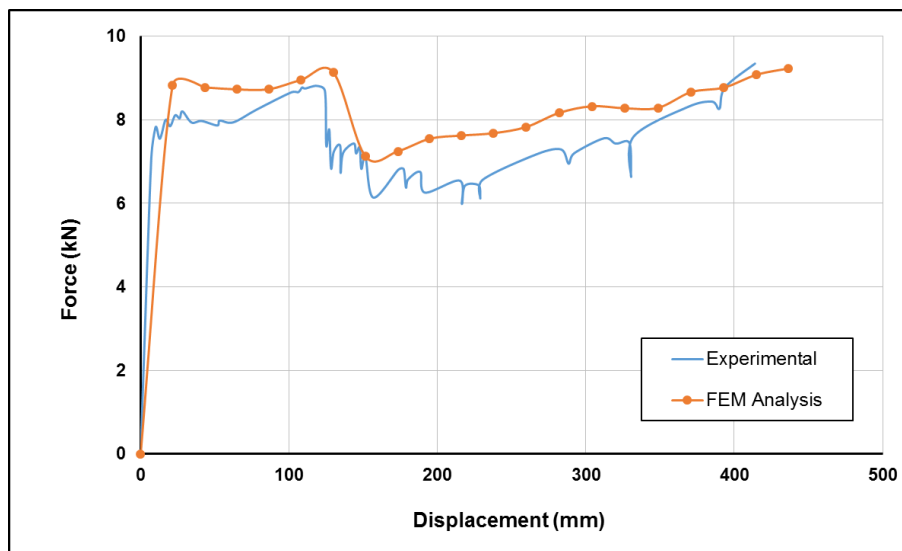


Fig. 10. Changes of resisting force-vertical displacement at middle column in static stage.

## 5. Conclusions

It is essential in numerical studies to ensure the accuracy of modeling results to turn the model into a reliable basis for future studies. Comparison of experimental and numerical results is a way to achieve this goal. As a result, similar experimental results were used to validate the numerical modeling solutions. Specifically, in the study of the behavior of concrete elements, the behavioral complexity of concrete materials, along with the combined function of concrete and steel materials, necessitates a method for validation of numerical analysis. With respect to progressive collapse of RC structures, the lack of adequate experimental results and high cost of experimental studies highlight the need for a reliable numerical model. In this study, a numerical model was developed for nonlinear analysis of the behavior of an RC frame subjected to progressive collapse. Comparison between the results of numerical analysis and those of the experiments on the investigated RC frame indicated a good agreement, suggesting an acceptable accuracy of the modeling. Investigating the effect of the third dimension elements, the effect of infill panel walls, the position of the removed column, etc. can be considered to be a basis for future studies on progressive collapse in RC structures. As a result, the proposed model can be used for conducting more relevant studies by keeping the modeling parameters constant.

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