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TECHNICAL NOTE

Investigation of Progressive Collapse Resistance Mechanism in Reinforced Concrete Beam–Column Assembly

Mohammad Bagher Paripour[1] · Ahmet Budak[1] · Oğuz Akın Düzgün[1]

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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 elements, 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. In this study, the behavior of the beam–column assembly subjected to progressive collapse was investigated by using finite element software. The results were compared to those of the results published in the literature, and model validation was ensured. The progressive collapse resistance mechanisms in RC beam–column assembly as well as the effectiveness of each mechanism in progressive collapse prevention were investigated. According to the results, the four resistance mechanisms, i.e., flexural action, compressive arch action, plastic hinge formation, and catenary action, have a significant effect on structural behavior.

Keywords Progressive collapse · Resistance mechanisms · Reinforced concrete beam–column assembly · Finite element analysis

1 Introduction

Structural safety is among the most important factors in designing engineering projects. Recently, the phenomenon of progressive collapse, a structural failure mechanism, has attracted the attention of designers and engineers. The spread of an initial local failure from element to element, which ultimately leads to a major or even full collapse of an entire structure, is defined as progressive collapse (ASCE 2017). Human factors, like blasts, fire, and vehicle collisions, and/or natural disasters, such as an earthquake, represent the main causes that lead to progressive collapse of buildings. Progressive collapse was first presented after the collapse of Ronan Point (Pearson and Delatte 2005) in England, from which many relevant studies have been conducted. The assessment of progressive collapse potential in structures is essential as it can cause irreparable damages.

Different codes and guidelines have addressed progressive collapse, including ASCE7-16 (2017), GSA (2003), DoD (2009), NBCC (2005), Eurocode (2004, 2006) and BSI (1996, 1997, 2003, 2005). There are two essential prerequisites for progressive collapse to happen: first, an abnormal loading that may cause initial failure in structural elements and second structural inability to prevent the spread of progressive collapse. To control this phenomenon in structures, one of these factors should be controlled. In other words, the structural elements should be designed to resist either abnormal loading or the spread of failure following the failure of a load-bearing element. Therefore, continuity, ductility, and a sufficient degree of indeterminacy are required. The design methodology of building codes introduces indirect and direct methods against progressive collapse. There are two direct methods, namely the alternative load path (ALP) and specific load resistance. In the indirect design method, the reinforcement of the structure against progressive collapse is carried out in accordance with the guidelines provided by the building codes to increase the degree of indeterminacy of the structure (GSA 2003). Nevertheless, the majority of building codes use the alternative load path, a direct design method. In this method, the structure should be designed to ensure its stability by introducing ALPs to withstand the loads acted after the removal of gravity

✉ Mohammad Bagher Paripour
mbparipour@gmail.com

[1] Department of Civil Engineering, Engineering Faculty, Ataturk University, 25240 Erzurum, Turkey

load-bearing elements (Nair 2006; Brunesi and Nascimbene 2014; Brunesi et al. 2015).

In recent years, the development of laboratory and numerical simulation techniques has broadly directed the attention to progressive collapse in structural system, resulting in significant progressions. After the collapse of Ronan Point, Ferahian (1972) investigated the plausible changes in British and Canadian codes to prevent progressive collapse in buildings and believed that a seismic-resistant building can resist the spread of failure even after the removal of a load-bearing element. Laboratory studies by Yu and Tan (2010, 2013) investigated the resistance of two types of the RC beam–column assemblies with seismic and non-seismic details based on ALP. According to these studies, the seismic details have no significant effect on increased beam–column assembly resistance against progressive collapse. Many researchers have used finite element modeling process to investigate the progressive collapse phenomenon (Kripakovet al. 1995; Krauthammer 1999; Bao et al. 2008; Lee et al. 2009; Brunesi and Nascimbene 2014). Tsai (2012) investigated load-release and direct loading simulation approaches for progressive collapse analysis in structures subjected to sudden column failure. Bao and Li (2010) performed a numerical study to investigate the dynamic response and residual axial strength of RC columns. In this study, the effect of RC column parameters, such as rebar consumption, axial load, and column dimension, on the response of a structure subjected to blast was investigated. They proposed a formula to estimate the residual axial capacity ratio based on the mid-height displacement to height ratios.

Dusenberry and Juneja (2002) recommended further studies for better understanding of the progressive collapse resistance mechanisms. These studies may include investigations into the strength and flexibility of structural elements and systems in a finite state. Nair (2006) investigated three methods to increase structural resistance against progressive collapse. These methods include increased redundancy (or ALP), increased local resistance, and improved continuity. He also concluded that the current guidelines and codes are more concentrated on ALP, and the two other methods are rarely used. Starossek (2007) investigated the progressive collapse of structures and by classifying these failures, recommended different techniques for increasing structural stability such as by providing alternative load paths. Brunesi and Parisi (2017) proposed new progressive collapse fragility models based on pushdown analysis of low-rise, reinforced concrete-framed bare structures. Based on the results of this research, significant influence of both seismic design and secondary beams on the robustness of building classes has been observed. Parisi et al. (2018) presented a multilevel sensitivity analysis to characterize the progressive collapsed capacity of an RC-framed structure.

In a previous study (Paripour et al. 2018), a numerical model was developed in finite element software to perform nonlinear analysis of the RC frame behavior under a progressive collapse. In this study, progressive collapse mechanisms in an RC beam–column assembly are investigated by using ALP and nonlinear static analysis. So, a micro-model was developed, by using ABAQUS (2012), to analyze the behavior of RC beam–column assembly subjected to progressive collapse. To ensure the accuracy of numerical results, they were compared to the results of a laboratory specimen through finite element modeling. The progressive collapse resistance mechanisms in a beam–column assembly and the performance and effectiveness of each mechanism in preventing progressive collapse were investigated. According to the results, the four resistance mechanisms, i.e., the flexural action, compressive arch action, plastic hinge formation, and catenary action, were studied. The results of this study can help better our understanding of the performance of RC beam–column assembly during the progressive collapse.

2 Finite Element Modeling

Today, as a result of progress in computer technology, more complicated problems can be investigated numerically. Numerical analyses require proportionate laboratory data as the majority of these models should be validated and calibrated to produce a precise response. It is specifically applied to RC structures, because of their composite nature and complex behavior under multi-axial loading. The current study used results from Yu and Tan's (2013) study to model progressive collapse mechanism in an RC beam–column assembly and validate the finite element model. This RC assembly included two beams and three columns and was a part of a five-story RC building with a moment-resisting frame system. This beam–column assembly was selected from the outer frame of the building, and its mid-column was removed to perform progressive collapse simulation. The structure was designed in accordance with ACI 318 (2005) with seismic and non-seismic detailing. In the finite element modeling, the specimen was selected with seismic detailing. Dimensional specifications and details of the reinforcement in this specimen are presented in Fig. 1. The geometric properties of the model and mechanical characteristics of the materials were selected exactly similar to the laboratory specimen. The solid and beam elements were used for modeling concrete assemblages and longitudinal rebars, respectively. The solid is a 3D element with eight nodes, which uses reduced integration method for finding the integral and is able to consider large nonlinear deformations. The reduced integration scheme for these elements is based on a single point uniform strain formulation where the strains are obtained as average strain over the element

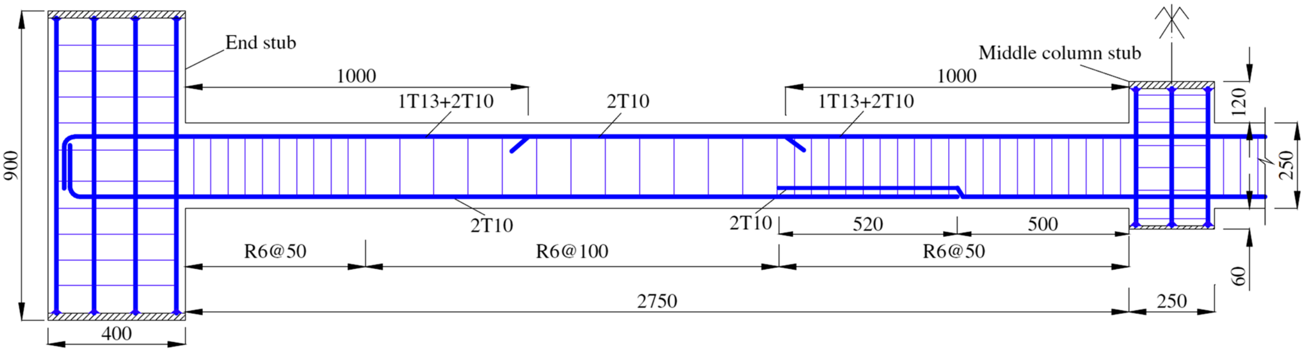


Fig. 1 Details of reinforcing beam–column assembly of laboratory specimen (Yu and Tan 2013)

volume. The beam is a two-node three-dimensional element capable of considering axial, flexural, and torsional deformations along the element. The rebars were embedded in the concrete.

To simplify the boundary conditions and concentrate on structural mechanisms in the beams and middle joint, the end columns were replaced with two enlarged columns. This design condition can also provide a suitable anchorage for longitudinal rebars in the affected beams. Meshing was carried out using the regular meshing technique. Sensitivity study was conducted to determine the optimal size of meshes. The mesh size was selected in a way that the analysis results became relatively independent of the meshing results. In addition, the analysis speed was considered in the mesh size determination. Accordingly, the mesh size is selected as 50 mm. In this beam–column assembly, the applied load on the middle joint is in the form of vertical displacement. The extent of the vertical displacement in the middle joint continues to increase until the frame reaches its maximum vertical load-bearing capacity. Model was analyzed using dynamic explicit analysis. Figure 2 presents the investigated RC beam–column assembly, along with support conditions. Table 1 summarizes the concrete and reinforcing steel material properties used in finite element modeling. Figure 3 presents an image of the model created with finite element software, along with loading and boundary conditions.

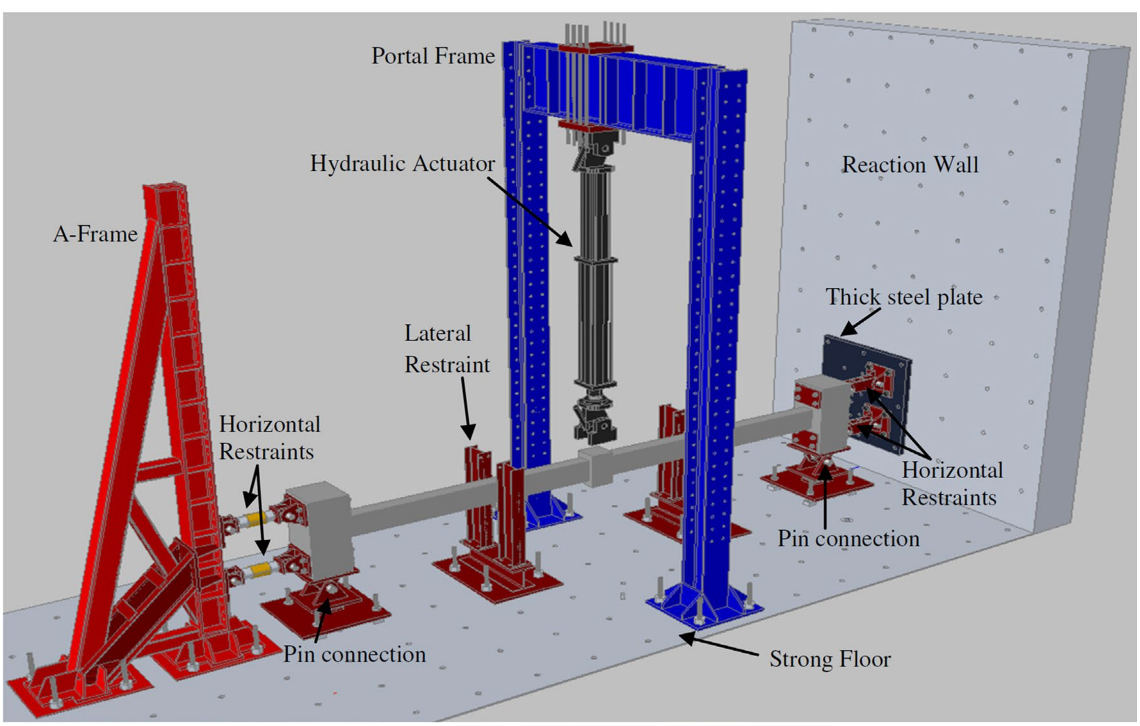


Fig. 2 RC beam–column assembly tested by Yu and Tan (Yu and Tan 2013)

Table 1 Material properties for computer input

Tested items		Nominal diameter (mm)	Elastic modulus (MPa)	Yield strength (MPa)	Ultimate strength (MPa)
Longitudinal reinforcing bars	T10	10	182,611	511	731
	T13	13	185,763	527	640
Stirrups	R6	6	178,500	310	422
Concrete (150 mm × 300 mm)			Compressive strength: 31.2 MPa		
			Splitting tensile strength: 3.2 MPa		
			Initial modulus of elasticity: 27,663 MPa		

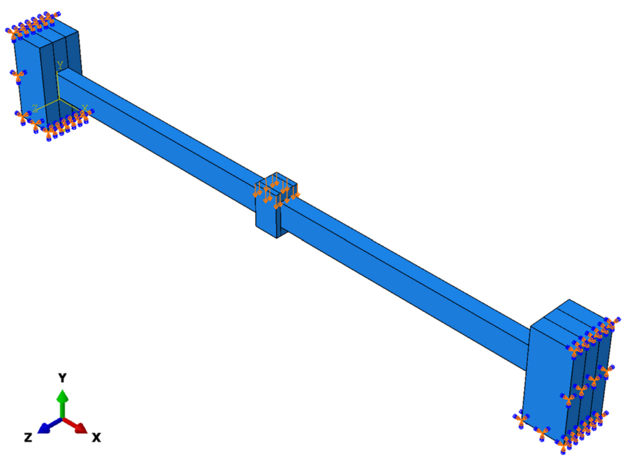


Fig. 3 Finite element model with details of loading and boundary conditions

3 Modeling Results

By applying linear incremental loading, the middle joint is shifted 600 mm downward. The application of a large displacement until the ultimate rupture point allows for investigating all progressive collapse resistance mechanisms. Figure 4 presents the beam–column assembly deformation in the finite element model. Comparison of laboratory and numerical results showed a good agreement between the refraction mechanism and place of plastic hinge formation. The laboratory and numerical results pertinent to applied load changes and horizontal reaction force to the middle joint displacement are presented in Figs. 5 and 6. Negative values in Fig. 6 present axial compressive stress, and positive values represent axial tensile force. Regarding the difference between real and modeling conditions, such as complete homogeneity and symmetry of geometry and materials, in finite element models, and that all details cannot be introduced to laboratory modeling, the difference in two diagrams is acceptable. As a result, model validation is ensured.

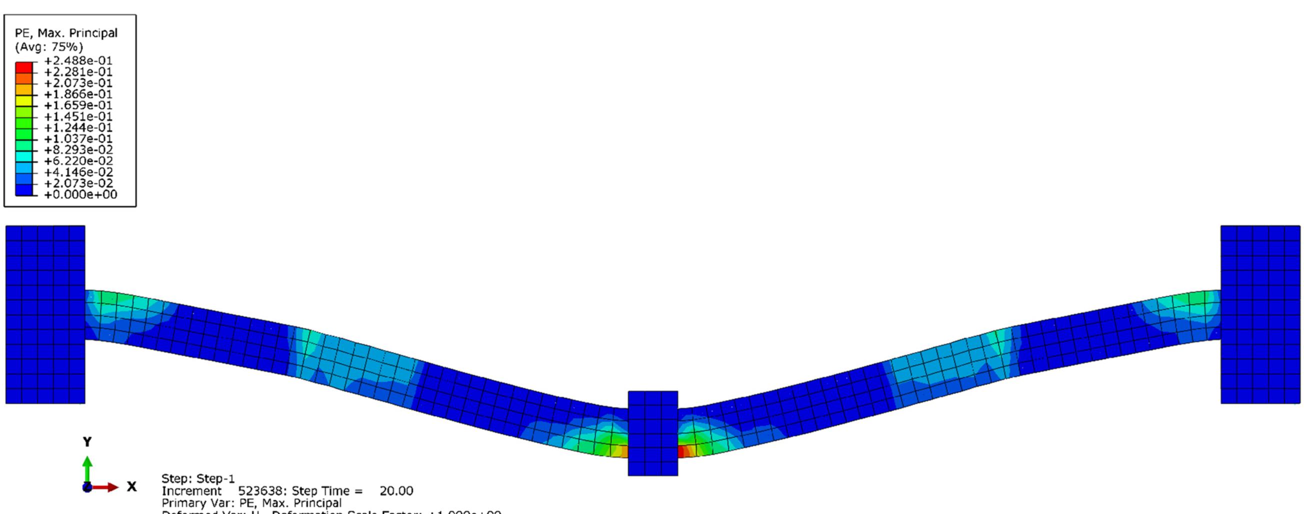


Fig. 4 Deformed image of frame in finite element model

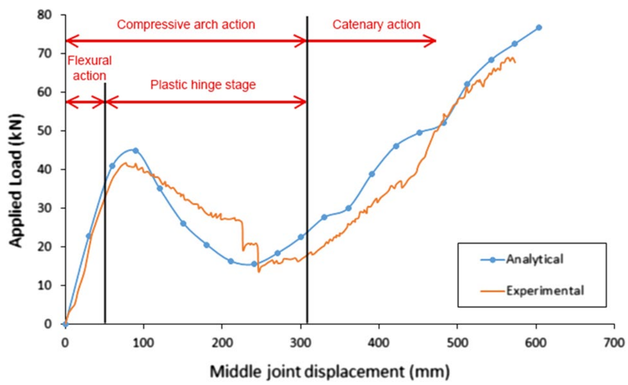


Fig. 5 Progressive collapse mechanisms in applied force–vertical displacement of middle joint curve

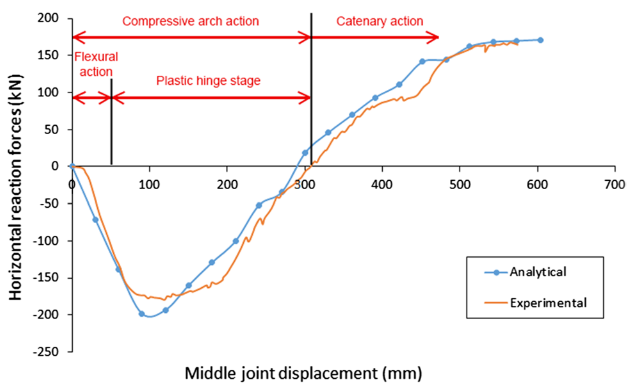


Fig. 6 Progressive collapse mechanisms in horizontal reaction force–vertical displacement of middle joint curve

4 Resistance Mechanisms

Based on the literature review, the resistance mechanisms of the RC beam–column assembly subjected to progressive collapse is characterized with four stages. The classification of structural mechanisms for the modeled specimen, including the flexural action, compressive arch action, plastic hinge formation, and catenary action, is presented in Figs. 5 and 6. These mechanisms were investigated one-by-one.

4.1 Flexural Action

In this stage, the flexural action is the dominant behavior affecting the beam–column assembly and a relatively slight axial compressive stress is generated in the beams. This phenomenon has been observed in the initial displacement of each beam–column assembly. According to Yu and Tan (2013), the flexural action develops from the beginning of loading until the onset of cracking and plastic hinge formation in the middle joint and the beam end. Based on the plastic hinge formation mechanisms and a nominal capacity of flexural action, the flexural action capacity is calculated regardless of axial compressive stress (Yu and Tan 2010). According to Figs. 5 and 6, the axial compressive stress is created even at slight middle joint displacements. As a result, the real flexural capacity of the beam is greater than the nominal flexural capacity, which can be attributed to the compressive arch action caused by the axial compressive action. The nominal moment capacity of RC beam is obtained using equilibrium of forces in the segments of the beam before application of any strength reduction factor.

According to Fig. 5, the structural resistance increases at the beginning of loading when the displacement increases. In the initial displacements, the flexural action mechanism has the main role with respect to the beam–column assembly resistance; and since there is still little axial force in the beam, the compressive arch action has a slight effect. On the other hand, the effect of compressive arch action increases and the flexural action reduces with increasing the displacement and axial force with time. As a result, a precise boundary for the flexural action capacity cannot be considered. Nevertheless, if it is considered the onset of cracking at the beam ends as a criterion, it can be said that the maximum flexural action has been achieved when the displacement reached 50 mm and resistance reached 37 kN (Fig. 5).

4.2 Compressive Arch Action

The structural resistance in the compressive arch stage, which is fundamentally due to the axial compressive stress of the beam, results in an increase in the corresponding ultimate capacity of flexural action as compared to the normal state. To create axial compressive stress, the beam should be restrained by the adjacent elements and out-of-plane failure of the lateral columns should be prevented. In this state, the beam enlargement, which is due to the crack distribution throughout the beam, results in an axial compressive stress on the beam. In this stage, the compressive stress increases in the upper part of the beam near the middle joint, and in the lower part near the outer joints. Due to the applied tensile stress, cracks start spreading in the lower parts of the beams adjacent to the middle joint, and in the upper part of the beams adjacent to the outer joints. This phenomenon results in reduced gross concrete section and increased stress in the rebars. Figure 7 presents the schematic of the compressive arch action in a two-span beam–column assembly (Abbasnia and Nav 2016).

In a laboratory study by Su et al., they investigated 12 specimens of RC concrete beams to evaluate their progressive collapse resistance. In this study, the effects of the ratio

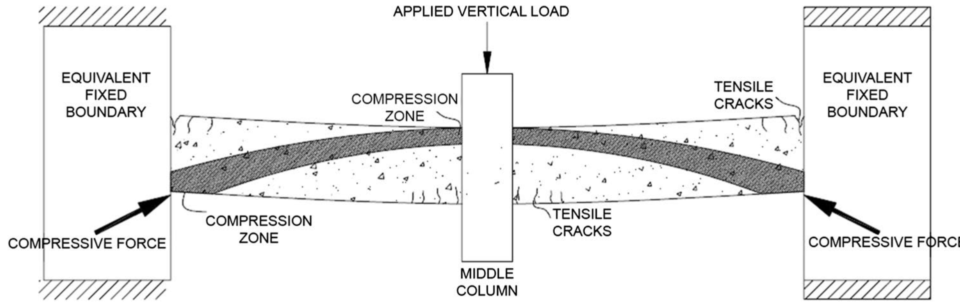


Fig. 7 Schematic of compressive arch action in RC beam–column assembly (Abbasnia and Nav 2016)

of flexural action, ratio of span to beam depth, and loading speed on arch action and vertical load-bearing capacity were investigated. The tested specimen showed that the load-bearing capacity under the arch action increased by 50–160% to when the axial resistance of the beams was not considered (Su et al. 2009).

As compared to the capacity of flexural action, the compressive arch action increases the load-bearing capacity of the element that the maximum capacity of compressive arch action (41kN) was achieved when the displacement reached 80 mm. However, the axial compressive stress reduced and the resistance curve showed a downward trend with increasing middle joint displacement and spread of cracks, thereby reducing the width of the compressive arch stress. This course continues until the axial compressive stress reaches the point of zero and the compressive arch action terminates when the displacement reached 310 mm. In the majority of previous studies, the onset of the compressive arch action is considered after the flexural action capacity. Since (even slight) axial compressive action may form in small deformations, we set the beginning point of loading as the onset of compressive arch action in our classification (Figs. 5, 6).

4.3 Plastic Hinge Formation

In this stage, the yielding of the rebars starts in the cracked areas and a greater stress is applied to the concrete under the compressive stress, resulting in concrete crushing. With an increase in crack depth in the tensile areas, the neutral axis is largely dislocated and due to the plastic hinge formation, the section can no longer resist a greater load. In other words, the onset of plastic hinge formation, which is right after the termination of the flexural action and the onset of cracking, is a transition stage from the compressive arch action to catenary action. Accordingly, the arch action continues even with the plastic hinge formation; however, it gradually disappears with increasing displacement. In this stage, the compressive stress created by the arch action in the lateral columns gradually reduces with reducing the compressive force in the beams. With the yielding of the lower beams and concrete entering the plastic area near the middle joint, the condition is set for plastic hinge formation (Su et al.2009).

The upper threshold of the axial compressive stress created in the beam is considered up to 10% of the gross axial compressive capacity of the concrete beam section (Eq. 1) (Yu and Tan 2013).

$$N = 0.1 f_c A_g \tag{1}$$

where f_c is the compressive strength of concrete and A_g is the beam section.

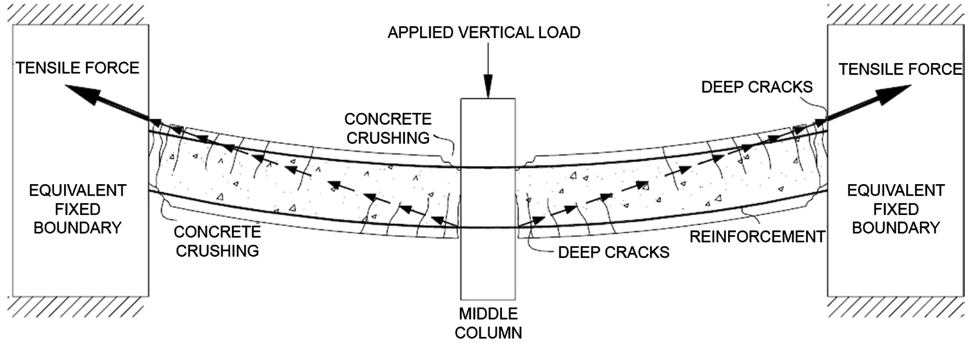
4.4 Catenary Action

In this stage, cracks spread throughout the whole concrete section and tensile force starts acting on the stressed rebars. Since the concrete cannot carry the tensile force, the load transition is typically done by rebars and the role of concrete can be ignored. The beams under tensile force pull the lateral columns inward and increase load-bearing capacity through catenary behavior. To form catenary action, adequate rotational ductile joints are needed to sustain large deformations. In this state, the beam–column assembly behaves like a cable. Figure 8 presents the schematic of the catenary action of the beam–column assembly (Abbasnia and Nav 2016). According to the finite element modeling results, the resistance has an upward trend in this stage; however, this trend terminates when the displacement reaches 600 mm and resistance reaches 78 kN, resulting in rebar fracture.

5 Conclusions

In this study, a micro-model was developed by ABAQUS to perform a nonlinear static analysis on the behavior of RC beam–column assembly subjected to progressive collapse. The good agreement between the modeling and laboratory results indicates acceptable modeling accuracy. Then, we investigated the mechanisms of RC beam–column assembly and the performance and effect of each mechanism with respect to the structural behavior. The progressive collapse resistance mechanisms against the RC beam–column assembly have four stages: flexural action, compressive arch action, plastic hinge formation, and

Fig. 8 Schematic of catenary action in RC beam–column assembly (Abbasnia and Nav 2016)



catenary action. In the flexural performance stage, the flexural behavior is the dominant behavior of the frame action, and a relatively low axial compressive stress acts on the beams. The compressive arch action is formed following the flexural failure of the beams. This action is typically due to the beam's axial compressive stress that causes an increase in corresponding ultimate flexural resistance relative to the normal state. In the stage of plastic hinge formation, the rebars in the cracked areas start yielding. As a result, a greater stress acts on concrete in the compressive areas that causes concrete crushing. With an increase in crack depth in the tensile areas, the neutral axis is largely dislocated and due to the formation of plastic hinge, the section can no longer resist a greater load. In other words, the plastic hinge formation stage begins with a shift from the compressive arch action to catenary action. In the catenary action stage, cracking occurs throughout the concrete section. As a result, the compressed rebars undergo a tensile stress. In this condition, load transfer is typically carried out by the rebars and the role of concrete is negligible. The beams under tensile force pull the lateral columns inward and increase load-bearing capacity through catenary behavior, resulting in rebar fracture.

According to previous studies, the effects of arch action should be considered for more precise determination of the frame vulnerability subjected to progressive collapse. The performance of arch action can be improved by designing deeper beams if other elements, such as lateral beams, columns, and slab, provide adequate lateral strength. Regarding the effect of the catenary action on increased ultimate load bearing of the specimen, structural performance against progressive collapse can be improved by considering this parameter in the design. Finally, it should be noted that some important parameters affecting progressive collapse such as the effect of the floor slab, and beams and columns in the third dimension were not considered in the current study. Considering the effects of these parameters on structural stiffness and catenary action, they affect the structural response. As a result, these factors can be regarded as a basis for future studies.

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