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# RC Frame Behavior at a Column Collapse Scenario Considering Crack Opening Effects

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ABSTRACT

This study examines the influence of the reinforced-concrete frame topology and crack opening effects on the structural behavior under a column removal scenario. For this purpose, a variant of discretization of the finite element model has been proposed, which takes into account additional rotations during the formation of discrete cracks. As a criterion of the bearing capacity of structures, the equilibrium point at the force-displacement curve was adopted. Verification of the proposed method has been performed using experimental data for a U-shaped frame. The paper provides a comparative analysis of the calculation results for 3-story, 5-story and 9-story reinforced-concrete frames using structural elements and the proposed method. It is found that accounting for the discrete nature of cracks had practically no effect on the magnitudes of axial forces in the elements. However, for bending moments, the proposed method showed a decrease compared to the traditional approach. The increase of axial tensile forces in the support sections of the beam above the zone of local failure has been detected for reinforced-concrete frames with 3 - 5 stories, which can lead to an increase in the influence of buckling for the columns to which such a beam is adjacent.

***Keywords:*** Reinforced concrete, Frame, Failure, Crack, Modeling, Finite element method, Moment, Angle of rotation.

## INTRODUCTION

The robustness of structures at initial damages and failures has recently become a more important problem. Thus, Kiakojouri et al. (2020), as well as Fedorova and Savin (2021) identified design and construction errors, collisions of vehicles with load-bearing elements, accumulation of a critical level of environmental damage, seismic impacts, military actions, … etc. as the possible causes of initial failure of structural members. Reducing the risk of disproportionate collapse of structural systems under such events requires the improvement of methods to assess their safety under accidental impacts.

Studies by Faghihmaleki et al. (2017), Vu et al. (2024), Kolchunov and Moskovtseva (2024), Kolchunov et al. (2024), and Savin et al. (2023) indicated that deformation of load-bearing systems when initial local damage or failure occurs has distinct peculiarities. In particular, to reinforced-concrete frames, the violation of concrete matrix continuity due to cracking in reinforced-concrete frames of buildings ought to be considered. Consequently, the joints or cross-sections of elements gain extra flexibility that impacts the redistribution of forces, as well as the deformation of the load-bearing structural elements.

Accidental actions can result in the loss of existing and formation of new interactions and ties within the load-bearing system. For instance, horizontal slabs may transform to a catenary (membrane) system and lose their resistance to horizontal loads (Bažant & Verdure, 2007). Infill walls can contribute to the force resistance of a system and dissipate impact energy (Li et al., 2016).

The catenary (membrane) action of beams and slabs is permitted by design codes to prevent failure propagation. However, it makes inapplicable traditional methods for safety assessment, which establish the correlation between the load-bearing capacity of columns and their effective lengths during normal operation. Due to constraint degradation, the effective length of columns may significantly increase. On the other hand, the large deflections of the beams may cause additional horizontal loads on the columns.

Pham et al. (2022) investigated the catenary action of beams on a model of a two-span reinforced-concrete frame for blast removal of the middle column. Based on the test results, the failure of the upper longitudinal reinforcement in the beam was identified at the beam-corner column joint and the failure of the lower reinforcement in the beam was observed at the column removal point. Crushing of the compressed concrete in the corner column at the beam-column joint was also observed. This was due to the change in the effective design length of the column and the additional lateral impact of the beam after its transition to the catenary action.

An essential factor in guaranteeing the structural integrity of buildings amidst abrupt removal of load-bearing members is identifying the duration for force redistribution *via* alternative paths. The appearance of inertia forces may accompany this process. The collapse of the Alfred Murray Federal Building was analyzed by Tagel-Din and Rahman (2006). The results indicated that the first-floor column failure, leading to progressive collapse, occurred approximately 0.3-0.4 second after the blast wave impact. The feasibility of utilizing the static push-down approach for progressive collapse analysis was evaluated by Marchand and Stevens (2015) and by Xu and Ellingwood (2011). Almazov et al. (2011) reported that the dynamic amplification factor for reinforced-concrete building frames falls within the range of 1.15 to 1.33 for the push-down method.

Kabantsev and Kovalev (2022) employed the plasticity coefficient as the ratio of total and elastic deformation to evaluate the dissipative properties of structures. This coefficient enables a reduction in the design loads on structures inflicted by accidental impacts. An energy-based pull-down approach was previously used by Geniyev (1992) and by Kolchunov et al. (2016) for this purpose. This method supposes that the static equivalent of the dynamic impact is applied at the point of a column removal. Savin et al. (2022) showed that this approach assesses the capacity of a load-bearing structure to withstand dynamic loading without the need to identify the dynamic amplification factor directly. Fedorova et al. (2020) observed that concrete exhibits distinct force resistance characteristics when subjected to dynamic loading resulting from the sudden removal of a structural member in the building frame. Yu et al. (2021) conducted a numerical study on this phenomenon, with the results being in agreement with the experimentally obtained data (Fedorova et al., 2020).

Detailed analyses of the progressive collapse behavior of building fragments and sub-structures are performed using finite element analysis in research software, such as Ansys, LS-Dyna, Abaqus, … etc. Usually, these software packages employ solid finite elements to model concrete and beam type elements for steel rebars. This methodology is utilized in numerous studies, such as Xu and Huang (2024), and Alshaikh et al. (2021). The simulation results were highly consistent with physical experiments. Nonetheless, simulation of structures with a large number of elements is time-consuming and presents a significant drawback. On the other hand, the design practice usually uses the method of structural finite element, which is also suitable for initial analysis in research. This method considers the effects of cracking by adjusting cross-sectional stiffnesses along the structural finite element. However, the method assumes spreading of the cracking effect along the element length, which can cause an inaccurate evaluation of load-bearing system behavior in emergency situations.

Recently, discrete-continuum methods of computational analysis have been developed. According to them, the structural system is divided into elements connected by a system of ties according to the peculiarities of deformation and failure mechanisms of sections in the limit state. Such methods include the applied element method considered by Alanani et al. (2020) for reinforced-concrete structural systems. Kolchunov et al (2024) applied a discrete-continuum semi-analytical method to evaluate the behavior of reinforced-concrete frames with layered beams under column removal scenarios. Kodysh and Mamin (2003) proposed the discrete ties method for analysis of flat slabs. Bondarenko and Kolchunov (2004) proposed design models of resistance for the analysis of the behavior of typical joints and structural members of reinforced-concrete frames. However, these approaches are mainly focused on the assessment of limit states under normal service conditions and do not consider the specificity of loading and deformation modes under accidental actions.

Regarding the identified research gap, the objective of the study is to investigate the influence of the topology of the reinforced-concrete frames and crack opening effects on the structural behavior under a column removal scenario. In order to achieve the given objective of the study, a variant of discretization of the finite element model has been proposed, which takes into account additional rotations during the formation of discrete cracks. As a criterion of the bearing capacity of structures, the equilibrium point at the force-displacement curve was adopted. Verification of the proposed method has been performed by comparison with experimental data for a U-shaped frame. The paper provides a comparative analysis of the calculation results for 3-story, 5-story and 9-story reinforced-concrete frames using structural elements and the proposed method.

## MODELS AND METHODS

### Design of the RC Frames under Investigation

The study analyzes the influence of stories' number, location of initial failure, and cracking effects on the redistribution of forces in reinforced-concrete frames at a column removal scenario. For this purpose, 3-story, 5-story and 9-story frames, as shown in Fig. 1 (a), were considered.

The frame spans were assumed to be the same with a length of 6 m in the axes, and the floor height was assumed to be the same for all frame design variants and was set at 3.3 m. The spacing of the vertical load-bearing structures in the longitudinal and transverse directions is the same. The frame structures are made of concrete B30. Mechanical properties of concrete are adopted in accordance with Russian design code SP 63.13330 (2018), which provides design strength and modulus of elasticity for prism specimens. Thus, compressive strength is $f_c$ = 22 MPa, initial modulus of elasticity is $E_c$ = 32,500 MPa, strain at pick stress is $\varepsilon_{c0}$ = 0.002, and ultimate strain in compression $\varepsilon_{c,ult}$ = 0.0035. The reinforcement scheme of the column sections is presented in Fig. 1 (b). Fig. 1 (c) provides reinforcement scheme of the beam cross-sections adjusted to column, while Fig. 1 (d) gives the same at the middle of the span. A500 steel with yield strength $f_y$ = 500 MPa and ultimate strain $\varepsilon_{s,ult}$ = 0.025 was used for longitudinal reinforcement. A240 steel with yield strength at shear $f_{yw}$= 170 MPa was adopted for stirrups. The reinforcement parameters were chosen with regard to the structural requirements of SP 63.13330 (2018), based on the results of the calculation for the main load combination according to SP 20.13330 (2016), which includes live loads on slabs of 1.5 kN/m$^2$, snow load with the standard value of 1.5 kN/m$^2$, and wind load for wind region I, type A.

As accidental design situations, two variants of initial local failures presented in Fig. 1 (a) were considered: sudden failure of the first-floor column along the A axis (case 1) and the first-floor column along the B axis (case 2). The accidental design situations were considered independently of each other using the secondary design scheme of the frame shown in Fig. 1(e) and the nonlinear static method for analysis.

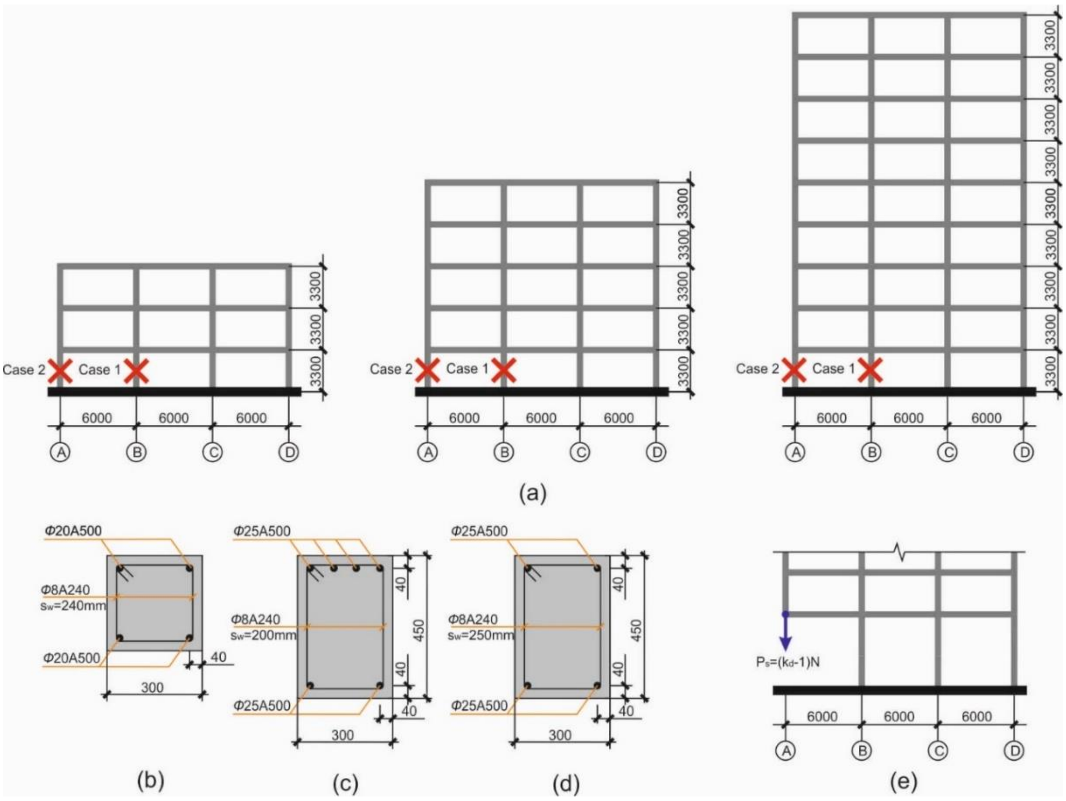



**Figure (1): (a) Reinforced-concrete frames; (b) Cross-section and reinforcement scheme of columns; (c) Cross-section and reinforcement scheme of beams near columns; (d) Cross-section and reinforcement scheme of beams at mid-span; (e) Accidental impact**

**Model of Structural Element with Discrete Cracks**

It is assumed that after crack formation, the change in ductility (stiffness) at the structural finite element coupling nodes of the computational model is taken into account due to the additional rotation of the sections in the tensile zones along the crack boundaries, as shown in Fig. 2 (a). Such rotation is considered on the basis of the "moment - rotation angle" diagram presented in Fig. 2 (b).

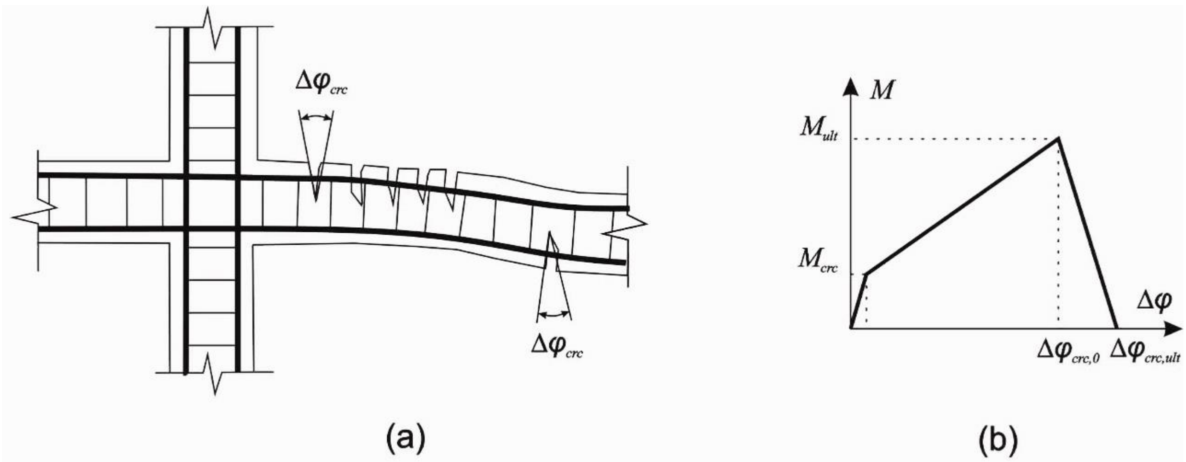



**Figure (2): (a) Schematic diagram of a characteristic fragment of a reinforced-concrete frame and (b) The diagram "moment vs. additional angle of rotation"**

At the pre-crack stage, the material behavior is assumed to be close to elastic and the angles of rotation are almost identical to the curvatures in the sections.

The stiffness $C_i$ of the beam element interface node after crack formation and before reaching the ultimate bending moment can be obtained from Equation (1):

$$C_i = 2\frac{M_{ult} - M_{crc}}{\Delta\varphi_{crc,0}}, \quad (1)$$

where $M_{ult}$ is the ultimate bending moment perceived by the cross-section of the element, determined from Equation (2):

$$M_{ult} = (N \cdot e_0 \cdot \eta)_{ult} = f_c \cdot \left[ b \cdot x_{calc} \cdot \left( h_0 - \frac{x_{calc}}{2} \right) \right] + \left( f_y \cdot A_s' - \frac{N}{2} \right) \cdot (h_0 - a'), \quad (2)$$

where $N$ is an axial force applied to a member; $e_0$ is the calculated eccentricity of the axial force; $\eta = (1 - N/N_{cr})$ is the coefficient that takes into account the effect of buckling on the ultimate limit state of an eccentrically compressed member at the nominal critical force $N_{cr}$, determined according to SP 63.13330 (2018); $f_c$ is the characteristic compressive strength of concrete; $f_y$ is the characteristic yield strength of the longitudinal reinforcement; $b$ is the cross-sectional width; $x_{calc}$ is the depth of section in compression assuming rectangular stress distribution; $h_0$ is the effective cross-section depth; $A_s'$ is the reinforcement area at the most compressed face of the section.

The cracking moment in a reinforced-concrete element $M_{crc}$ is determined from Equation (3):

$$M_{crc} = f_{ct} \cdot \gamma \cdot W_{red} \pm N \cdot e_x, \quad (3)$$

where $f_{ct}$ is the characteristic tensile strength of concrete; $\gamma$ is the coefficient, which accounts for plastic strain and is assumed for a rectangular cross section as $\gamma = 1.3$; $W_{red}$ is the geometrical moment of resistance of the reduced section; $e_x = W_{red}/A_{red}$ is the distance from the point of axial force application to the core point farthest from the face of the element to be inspected for cracks.

Crack opening angle increment $\Delta\varphi_{crc,0}$ is determined by Equation (4):

$$\Delta\varphi_{crc,0} = \frac{a_{crc,0}}{h_{crc}}, \quad (4)$$

where crack opening width $a_{crc,0}$ and crack depth $h_{crc}$ at the ultimate value of bending moment $M = M_{ult}$ are approximately determined from the expressions in Equation (5):

$$a_{crc,0} = \varphi_1 \cdot \varphi_2 \cdot \varphi_3 \cdot \psi_s \cdot \frac{f_y}{E_s} \cdot l_{crc};$$
$$h_{crc} = h_0 - \frac{x_{calc}}{\alpha} \cdot \left( 1 - \frac{\varepsilon_{ct,ult}}{\varepsilon_{c,ult}} \right). \quad (5)$$

where $\varphi_1$, $\varphi_2$, $\varphi_3$ are the factors which account for the duration of load action, longitudinal reinforcement profile and type of stress-strain state of the element; $\psi_s$ is the factor that accounts for uneven distribution of reinforcement strains in tensile concrete between cracks; $E_s$ is the modulus of elasticity of reinforcing steel; $l_{crc}$ is the base spacing between cracks; $\varepsilon_{ct,ult}$, $\varepsilon_{c,ult}$ are the ultimate strains for concrete in tension and compression, respectively; $\alpha$ is the transition factor from the design compressed depth $x_{calc}$ to the actual one $x_{fact}$, determined by Equation (6):

$$\alpha = \frac{x_{calc}}{x_{fact}}. \quad (6)$$

The non-linear elastically pliable fixations from rotation can be pre-defined in intermediate sections of structural elements based on the analysis of the cross-sectional stress-strain state and load application schemes. The spacing can be preliminarily assumed to be equal to a quarter of the basic distance between cracks, determined in accordance with SP 63.13330 (2018) by Equation (7):

$$10 \cdot d_s \le l_{crc} = 0,5 \cdot \frac{A_{ct}}{A_s} \cdot d_s \le 40 \cdot d_s, \quad (7)$$

where $A_{ct}$, $A_s$ are the areas of the tensile zone of concrete and tensile longitudinal reinforcement; $d_s$ is the diameter of the longitudinal reinforcement bars.

**Research Method**

For the purposes of the study, the Lira-CAD software package was used, which allows to perform static and dynamic structural analysis using the finite element method. The calculation for the accidental action caused by the sudden failure of a load-bearing element has been carried out according to SP 385.132580 (2018) for a special combination of loads, including only the fatal and long-term live loads accepted with characteristic values. The physical and mechanical properties of the materials were taken from SP 63.13330 (2018). Dynamic effects on load-bearing systems were modelled using the quasi-static method proposed in the study conducted by Geniyev (1999). This assumes that the reaction of the removed column is applied with the opposite sign to the joint of the frame above the collapsed column, as shown in Fig. 1 (e).

The value of such a force can be determined from Equation (8):

$$P = (k_d - 1) \cdot F_i. \tag{8}$$

where $F_i$ is the force in the end section of the structural member before its failure; $k_d$ is the dynamic amplification factor.

Based on the numerical study of the resistance of RC frames to progressive collapse carried out by Almazov et al. (2011) and the results of tests on a full-scale physical model of a spatial frame (Adam et al., 2020), we adopted $k_d$ = 1.25. This corresponds to a dynamic effect in the form of 0.25 of the reaction of the removed element at the joint of the frame.

For the quasi-static formulation of the problem adopted in the study, the constitutive equation of the finite element method has the form:

$$[K]\{u\} = \sum\{F\}, \tag{9}$$

where $\{u\}$ is the displacement vector in the nodes of the finite element model; $[K]$ is the stiffness matrix; $\sum\{F\}$ is the vector of external loads in the nodes of the finite element model.

Fig. 3 illustrates a discretization method for modeling the reinforced-concrete frame. In this way, the problem is solved taking into account the physical and geometrical non-linearity by the sequential loading method in the following order.

1. It is assumed that the finite element mesh spacing is not larger than the theoretical distance $l_{crc}$ between adjacent cracks determined from Equation (7).
2. In order to take into account the discrete nature of cracking in the models of reinforced-concrete frames, the finite elements are expanded at the nodes and special zero-length finite elements of non-linear ties are introduced between the nodes of adjacent finite elements. These ties take into account the moment-rotation relationship, the parameters of which are determined according to Equations (2), (3), and (4).

It is assumed that before cracking, the interface nodes of the beam elements behave as absolutely rigid. After crack formation, the change of flexibility (stiffness) in the nodes is taken into account by additional rotation of the cross-sections. To illustrate the proposed method, a flow chart is shown in Figure (4).

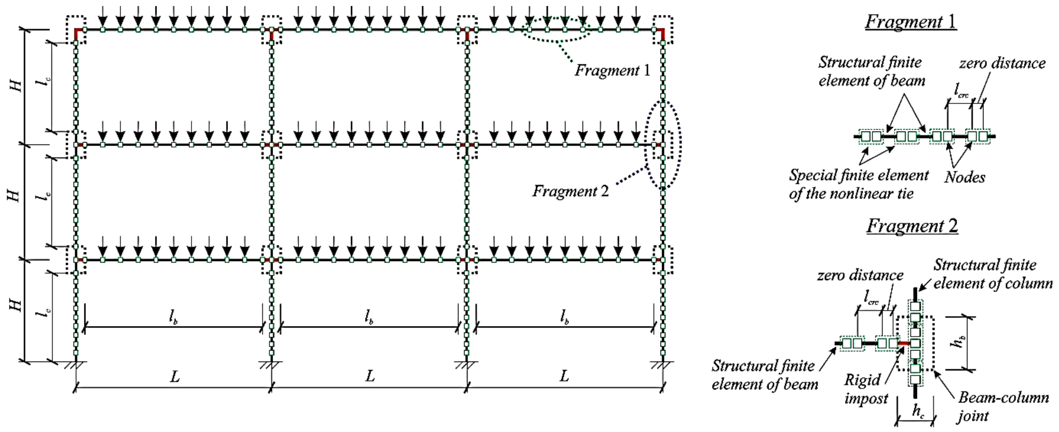



**Figure (3): Discretization in modeling of the reinforced-concrete frame**

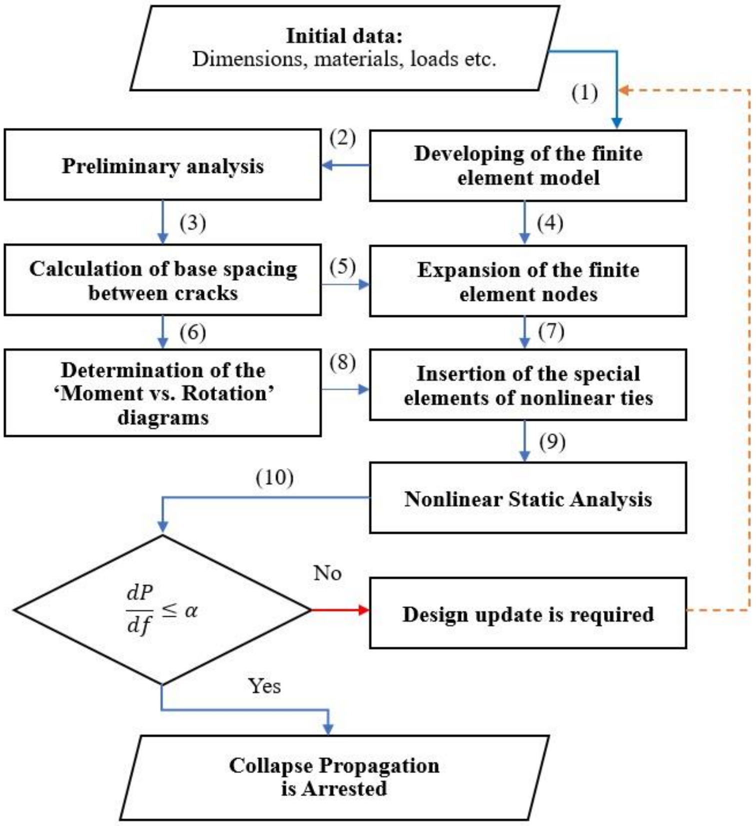



**Figure (4): Flowchart of the proposed method of discretization**

**Criteria of Bearing Capacity and Stability of Structural Elements**

As a criterion, the values of 1/30 for the ultimate relative deflection at the point of column removal, the ultimate strain in the compressed concrete $\varepsilon_{c,ult}$ = 0.0035, and tensile steel reinforcement $\varepsilon_{s,ult}$ = 0.025 are used to evaluate the capacity of the beams.

In order to evaluate the failure mechanism of columns at the ultimate limit state, a moment-based criterion is used, as discussed for separate elements in Mahdi et al. (2023). The essence of the approach is as follows. If the first-order moment caused by transverse bending is greater than the second-order moment when the point of ultimate equilibrium is reached on the force-displacement curve, then such failure occurs in the material. In the opposite case, failure occurs due to loss of stability.

A change in mechanism can occur when the beams above the column removal point lose their ability to redistribute loads through the arch mechanism and become a catenary system. The amount of deflection of the beams above the column removal point is one of the criteria to evaluate the change from arch mechanism to catenary. Therefore, a correct assessment of the deformed state of reinforced-concrete frames, taking into account the formation of cracks and their significant opening, is very important to predict the potential resistance mechanisms of the beams and, consequently, the possible threats to the column structures.

As for the case of material failure and stability failure, an energy-based criterion is used. This involves comparing the increments of potential strain energy in adjacent steps of the calculation.

The moment of stability failure is corresponding to the change of sign of the strain energy increment, as in Equation (10):

$$\left|1-\frac{U_{i+1}}{U_i}\right| \leq a, \tag{10}$$

where $U_{i+1}$, $U_i$ is the strain energy for $(i+1)^{th}$, $i^{th}$ loading stages; $a$ is a given small number.

Equation (10) can also be presented in another form, as shown in Equation (11):

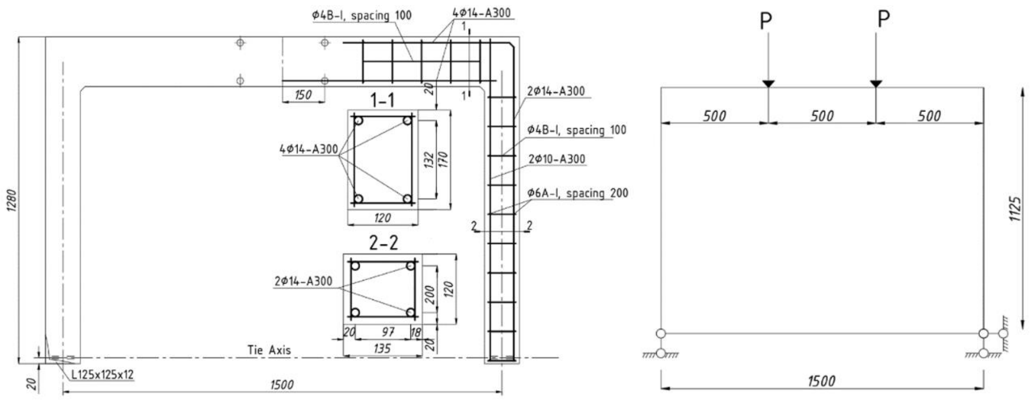
$$\left|\frac{dP}{df}\right| \le a, \quad (11)$$

where $dP$ is the increment of axial force; $df$ is the increment of deflection.

## RESULTS AND ANALYSIS

### Validation of the Proposed Method

To validate the adequacy of the proposed model to the character of deformation of reinforced-concrete frame elements during cracking, we have used experimental data for reinforced-concrete U-shaped frames with tie (Bondarenko & Kolchunov, 2004). The reinforcement scheme of the test frame is shown in Fig. 5 (a), and Fig. 5 (b) presents the design scheme. B35 concrete with $f_c$ = 25.5 MPa, $f_{ct}$ = 1.95 MPa, $E_c$ = 34500 MPa, $\varepsilon_{c,ult}$ = 0.0035 was utilized for fabrication of the test frame. The frame reinforcement is made of A300 steel bars with $f_y$ = 300 MPa, $E_s$ = 200000 MPa, and $\varepsilon_{s,ult}$ = 0.025.




(a) (b)

**Figure (5): (a) U-shaped frame: reinforcement scheme and (b) Calculation scheme**

Fig. 6 shows the bending moments in the frame considering traditional models of the finite element method - 1, ductility in structural nodes - 2, processing of experimental data – 3, and by the method mentioned above – 4.

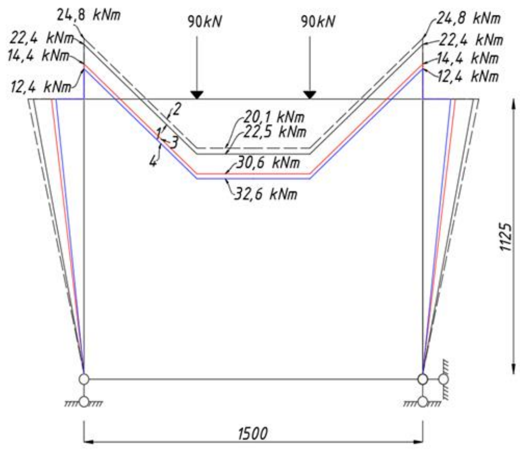



**Figure (6): Diagrams of bending moments in U-shaped reinforced-concrete frame: 1 - For rigid connections; 2 - For pliable connections; 3 - According to experimental data; 4- According to the proposed method**

The bending moments calculated by the proposed method almost completely coincided with the experimental data. As a result of cracking, there was a redistribution of bending moments in the beam of the frame. According to the proposed method, the moment in the structural joints is 12.4 kN·m and the moment in the span is 32.6 kN·m. By finite element method (FEM) the moment in the structural joints and in the span is 24.8 kN·m and 20.1 kN·m, respectively. Compared with the proposed method, the moments calculated by FEM are decreased by 50% at joints and increased in the span by 62.2%. This was also reflected in the stress-strain state of the frame struts, which also showed a decrease in bending moments in the upper support sections.

The agreement between calculation results and experimental data allows us to apply the developed method to the assessment of the influence of the topology on the robustness of reinforced-concrete frames of multi-story buildings at a column removal scenario.

### Parametric Study of the Effect of Reinforced-concrete Frame Topology on the Redistribution of Forces under Accidental Action

This part of the study provides the results of calculations of reinforced-concrete frames using the proposed method and the traditional one. Fig. 7 shows the changes in axial forces and bending moments in the upper section of the first-floor columns at a removal of the middle column. Fig. 8 provides axial forces and bending moments in the lower section of the second-floor columns at a removal of the middle column. A comparison of the calculation results of the traditional approach and the proposed method is presented in Table 1.

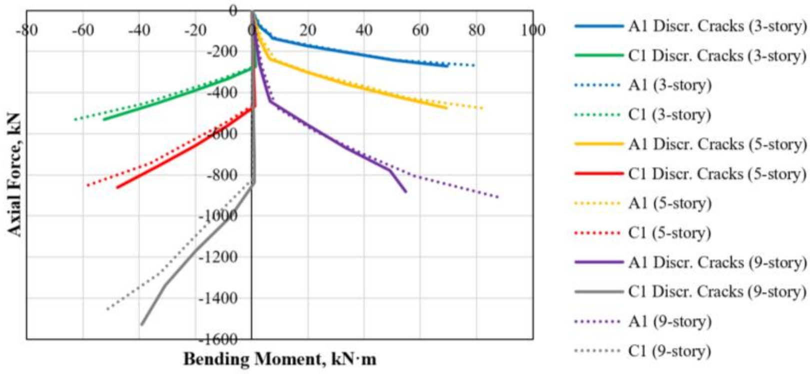



**Figure (7): Axial forces and bending moments in the upper section of the first-floor columns at a removal of the middle column (Case 1)**

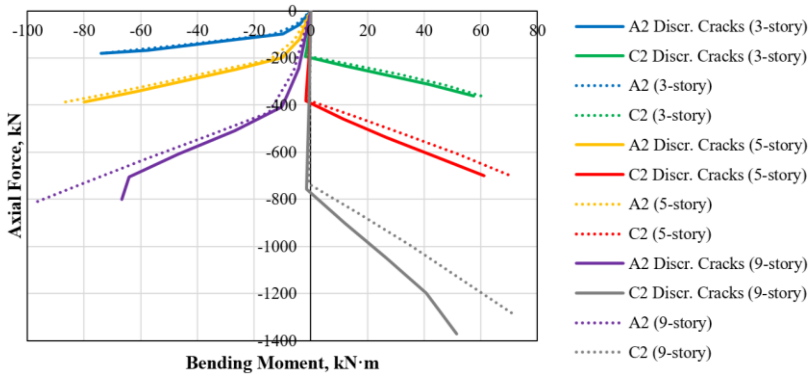



**Figure (8): Axial forces and bending moments in the lower section of the second-floor columns at a removal of the middle column (Case 1)**

**Table 1. Comparison of the calculation results of the traditional approach and the proposed method for Case 1**

| Frame Type | A1 | A1 Discr Crc | Δ, % | A2 | A2 Discr Crc | Δ, % | C1 | C1 Discr Crc | Δ, % | C2 | C2 Discr Crc | Δ, % |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Bending Moment, kN·m** | | | | | | | | | | | | |
| **3-storey** | 80 | 69.4 | 15.3 | -73.6 | -74 | -0.5 | -63.8 | -52.5 | 21.5 | 61.6 | 57.6 | 6.9 |
| **5-storey** | 83.2 | 69.2 | 20,2 | -86.7 | -79.9 | 8.5 | -59.9 | -47.7 | 25.6 | 69.9 | 61.2 | 14.2 |
| **9-storey** | 88.6 | 54.8 | 61.5 | -97.9 | -66.8 | 46.6 | -51.3 | -39.1 | 31.2 | 72.4 | 51.4 | 40.9 |
| **Axial force, kN** | | | | | | | | | | | | |
| **3-storey** | -269 | -272 | -1.1 | -179 | -181 | -1.1 | -534 | -530 | 0.8 | -365 | -361 | 1.1 |
| **5-storey** | -478 | -475 | 0.6 | -386 | -388 | -0.5 | -858 | -860 | -0.2 | -697 | -700 | -0.4 |
| **9-storey** | -910 | -883 | 3.1 | -815 | -799 | 2.0 | -1454 | -1530 | -5.0 | -1294 | -1370 | -5.5 |

The consideration of the discrete character of cracks practically did not affect the values of axial forces in the elements. However, for bending moments within the proposed method, a decrease was observed in comparison with the traditional approach. Moreover, the difference exceeded 60% for the upper supporting part of the column along axis A.

The calculation results for the second accidental design situation (case 2) are presented in Fig. 9, which shows the axial forces and bending moments in the upper sections of the columns of the first floor and the lower supporting sections of the second-floor columns along axis B. The comparison of the calculation results of the traditional approach and the proposed method is shown in Table 2.

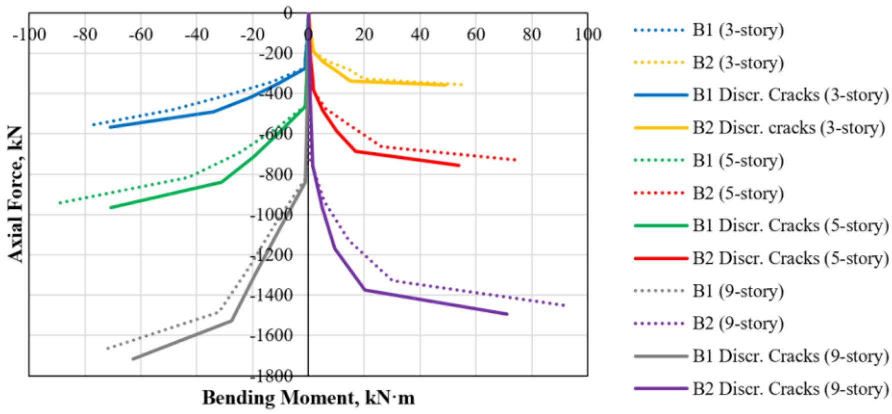



**Figure (9): Axial forces and bending moments in the upper sections of the first-floor columns and in the lower sections of the second-floor columns at a removal of the middle column (case 2)**

**Table 2. Comparison of the calculation results of the traditional approach and the proposed method for case 2**

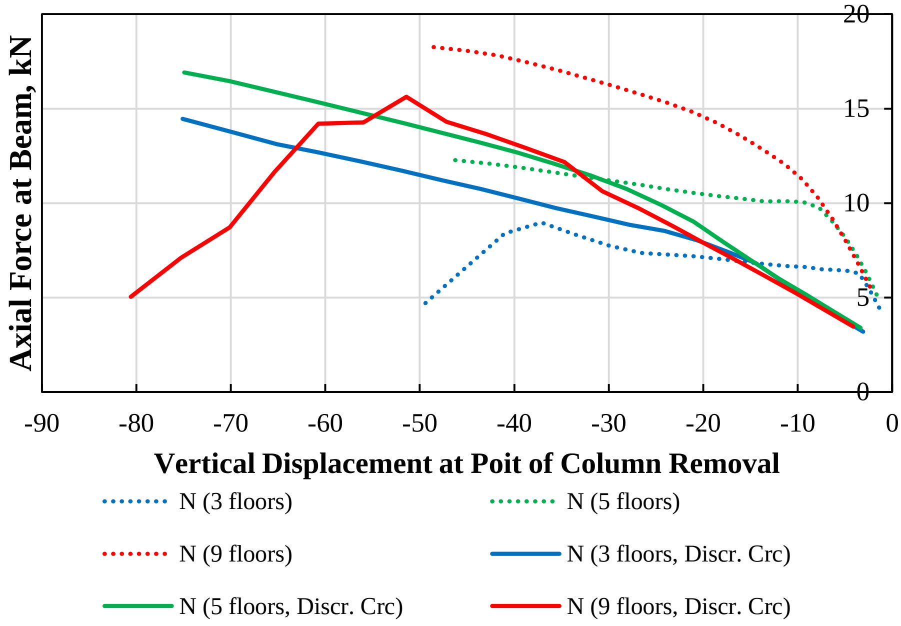
| Frame Type | B1 | B1 Discr Crc | Δ, % | B2 | B2 Discr Crc | Δ, % |
|---|---|---|---|---|---|---|
| **Bending Moment, kN·m** | | | | | | |
| **3-storey frame** | -77.9 | -70.9 | 9.9 | 56.7 | 49.1 | 15.5 |
| **5-storey frame** | -90.3 | -70.6 | 27.9 | 74.6 | 53.7 | 38.9 |
| **9-storey frame** | -73.4 | -62.7 | 17.1 | 91.6 | 71.2 | 28.7 |
| **Axial force, kN** | | | | | | |
| **3-storey frame** | -557 | -567 | -1.8 | -357 | -359 | -0.6 |
| **5-storey frame** | -994 | -966 | 2.9 | -731 | -756 | -3.3 |
| **9-storey frame** | -1674 | -1717 | -2.5 | -1449 | -1494 | -3.0 |

The consideration of the discrete nature of the cracks had practically no effect on the magnitudes of the axial forces in the elements, as in the previous case. For bending moments within the proposed method, an ambiguous influence of the floor was observed compared to the traditional approach. For a 5-storey frame, a reduction of moments up to 40% was observed. However, for 3-storey and 9-storey frames, this reduction remained at 10-15%. This can be explained for the 3-storey frame by the lower values of loads on the elements and for the 9-storey frame by the superior ability to redistribute loads.

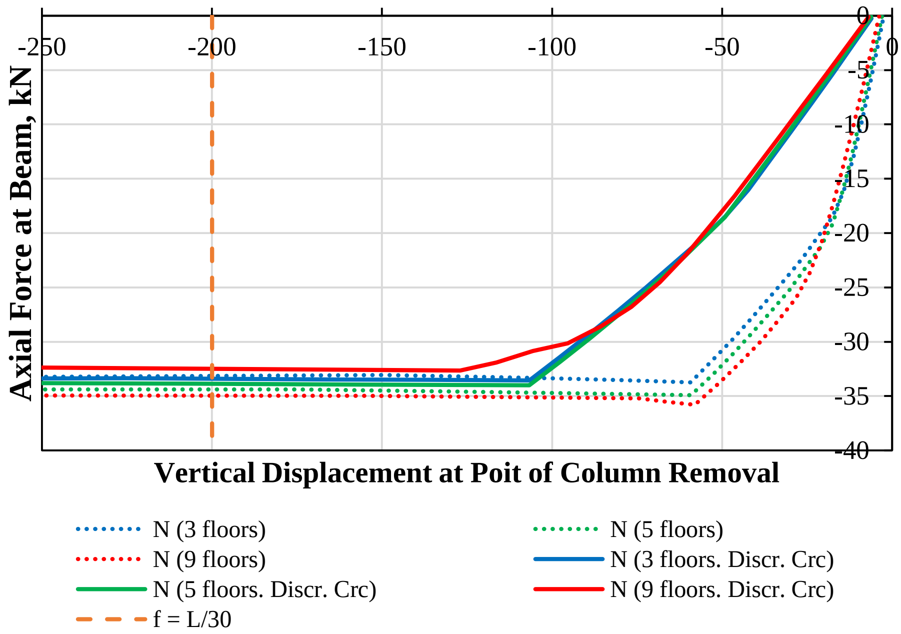
## DISCUSSION

### Relationships between Axial Forces and Vertical Displacement in Beams at a Point of Column Removal

Fig. 10 shows the results of comparing the axial forces in the beams at a point of column removal as a function of floor and modeling method.




(a)



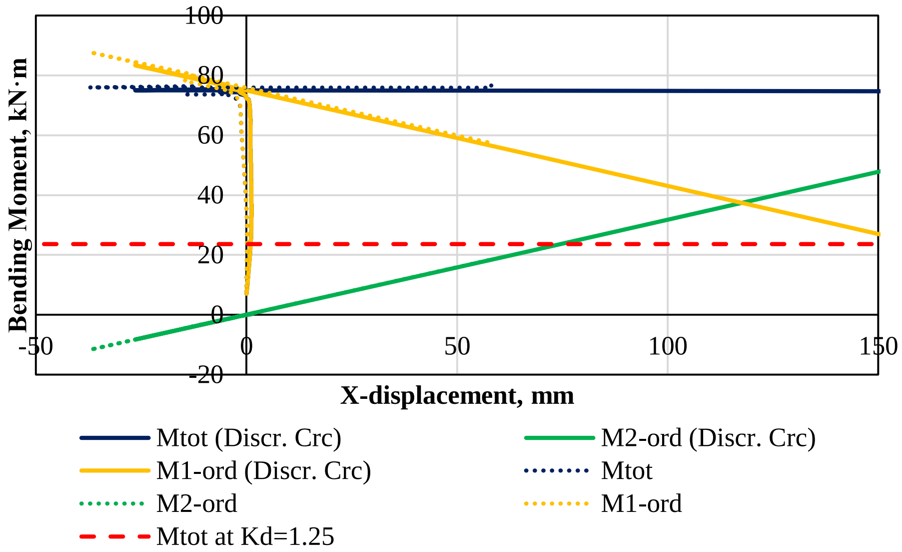

(b)

**Figure (10): Axial forces *vs.* vertical displacement in beams at point of column removal: (a) For Case 1, (b) For Case 2**

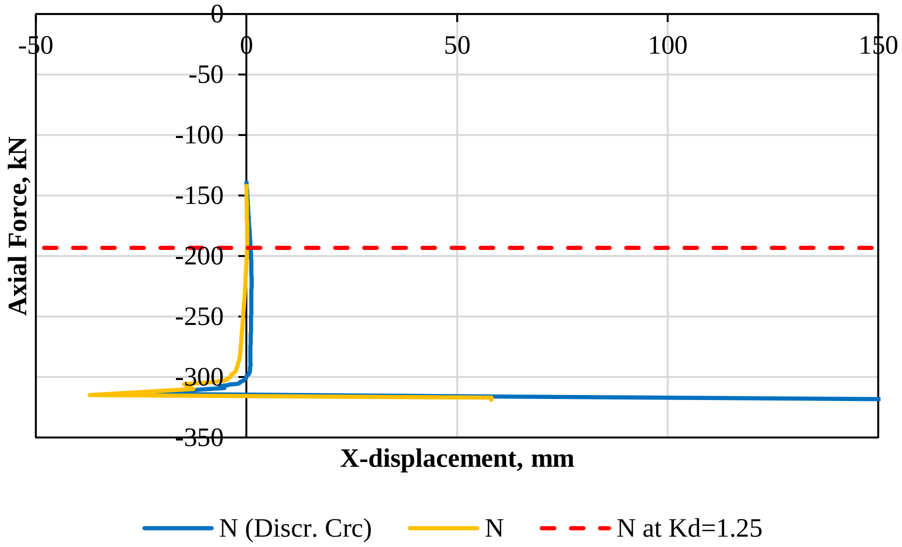
The vertical displacement in beams at point of column removal shows that for reinforced-concrete frames with 3 and 5 stories, there is an excess of tensile axial forces in the beam supports over the values according to the traditional calculation method. For the 9-story frame, a decrease in the magnitude of the longitudinal tensile force in the beam above the zone of initial localized failure was observed compared to the traditional calculation.

**Evaluation of the Load Capacity and Stability of the Column in Quasi-static Analysis**

Fig. 11 (a) shows the moment *vs.* horizontal displacement graph for the top of the column A1. The values of the total moment, as well as the values of the first-order moment from transverse bending and the second-order moment from buckling are given. The red dashed line indicates the value of bending moments in the three-story frame at the dynamic increase factor $k_d$ = 1.25.




(a)




(b)

**Figure (11): Analysis of a hypothetical mechanism of column failure: (a) Bending moment *vs.* X-displacement curves, (b) Axial Force *vs.* X-displacement curves**

For column A1, the first-order moments initially exceeded the second order moments. This indicates a possible material failure mechanism. However, as the axial force in the column and the tensile force in the adjacent beam increased, the horizontal displacement of the top of the column occurred. Initially, this displacement was outward due to the arch action of the beam. Then, abruptly, there was a transition to a catenary action, and the displacements of the column changed directions and increased rapidly. At the same time, the force-displacement graph became straight, i.e., the value of $dP/df$ dropped sharply, as shown in Fig. 11 (b). This behavior of the column can be considered as a loss of stability that corresponds to the intersection of the first- and second- order moment lines in Fig. 11 (a).

## CONCLUSIONS

This paper presents the study of the influence of the reinforced-concrete frame topology and crack opening effects on the structural behavior under a column removal scenario. A variant of discretization of the finite element model was proposed on the basis of accounting for additional rotations during the formation of discrete cracks. As a criterion of the bearing capacity of structures, the equilibrium point at the force-displacement curve was adopted. Verification of the proposed method has been performed using experimental data for a U-shaped frame. The paper provides a comparative analysis of the calculation results for 3-story, 5-story and 9-story reinforced-concrete frames using structural elements and the proposed method. The following conclusions can be formulated on the basis of the results of the performed research:

- A method of discretization of the calculation model has been developed to account for additional rotations of cross-sections of structural elements when cracks are formed in them. The proposed method was verified using experimental data for a test reinforced-concrete frame. The results demonstrate that the use of the discretization method allows for a more accurate representation of the redistribution of forces in the frame during crack formation than the traditional approach.
- It has been demonstrated that the formation of discrete cracks at the operational stage leads to a reduction in the moments in the columns and to an increase in the deflections of the beams. In the event of an accidental design action, the growth of deflections in the beams at the point of column removal results in the catenary action of these beams occurring at an earlier stage than predicted by the traditional modeling method, which does not take discretization into account.
- The study revealed that the most perilous accidental action is the removal of the corner column. In this instance, the damage spreads beyond the initial failure area. The removal of the middle column and the catenary action of the beams result in the ultimate state of the columns being due to a stability failure. Nevertheless, this phenomenon does not occur if the arch action in the beams above the point of column removal provides load transfer through alternative paths.

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