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RC FRAME RESISTANCE TO PROGRESSIVE COLLAPSE CONSIDERING CRACK OPENING EFFECTS

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Abstract

In this paper, an approach is developed to account for the effect of discrete cracks on the response of reinforced concrete building frames under a column failure scenario. The approach implies the introduction of traditional finite element models of discrete ties that take into account the relationship between moments and rotations, considering the specifics of the performance of materials, sections, and structures under conditions of redistribution of forces as a result of initial local failure in the structural system of a building. Validation of the proposed approach is performed on the experimental data. Also, it is compared with the modeling results of the existing approaches. The effect of discrete cracking on the deformed state of reinforced concrete building frames under the scenario of column failure is established. The discrete 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. The analysis of the diagrams shows that for reinforced concrete frames with 3 and 5 stories, there is an excess of tensile axial forces in the beam over the values according to the traditional calculation method.

Keywords: Crack, Failure, Frame, Finite Element Method, Modelling, Moment, Reinforced Concrete, Rotation

I. Introduction

Since 2001, due to the collapse of the Twin Towers of the World Trade Center, there has been a significant increase in interest in the problem of ensuring the robustness of facilities to accidental effects, including damage or collapse of separate load-bearing elements. Although the engineering community was aware of cases of building collapse as a result of localized failures or damage to separate structures before, such as the cases of Ronan Point [XVIII] and Alfred Murray [XXIV] buildings, the Transvaal Park complex [VI], or Sampoon Shopping Center [VIII], the intensive growth of publications on this problem falls on the 2000s [I].

The main feature of the above cases of building collapses is the low probability of the event that caused the initial collapse or damage to the structural system. However, the consequences of a change in the design scheme of structures as a result of the failure of an element and subsequent redistribution of loads were catastrophic. Kokot S., and Solomos G. [XIII] note that despite the low probability of accidental impact, the risk of unacceptable consequences is significant. In this regard, design situations associated with the occurrence of initial local damage or failure should be considered in the design of structures of increased levels of responsibility.

As a result of an emergency, specific resistance mechanisms of structures could be activated. Thus, the permissible limits of using simplified resistance models could be exceeded. In particular, in an exceptional event, the slab structures are allowed to resist as a catenary system. For beams of reinforced concrete building frames, the change of resistance mechanisms is associated with the formation of cracks. For example, at the formation of cracks in the tensile area of the cross-section, the arch mechanism is activated in the beam. The formation of plastic hinges in three or more sections along the length of the beam leads to the catenary action [IV], [XXVI]. At the catenary action, cracks are formed in the section, as evidenced by the results of experimental studies of the behavior of reinforced concrete frame models. In cross-sections with cracks, there is a sharp change in curvature, which affects the redistribution of forces, deflections of structures, and the realization of resistance mechanisms. In this regard, the models used in assessing the structural robustness in the event of local failure require a more profound justification.

The finite element models with solid finite elements is widely used in numerical studies of the behavior of reinforced concrete structural models at local failures [XX]. When solving nonlinear problems in the calculation of reinforced concrete structures, correct accounting of cracking and deformation of bearing elements requires the use of dense meshes of partitioning into finite elements [XVI]. This leads to a significant increase in the calculation time and makes it practically impossible to use solid finite elements in the design practice with respect to the response analysis of reinforced concrete building frames. In addition, the redistribution of forces along the section depth caused by cracking is not fully taken into account even in this case, unless the finite elements are expanded or finite elements are excluded in sections where the strength criteria have been exceeded.

In studies [XIV], [XV], a model of a coupled structure was used to more correctly account for the effect of cracking on the deformations of structures. The proposed approach is promising, but it has not yet been implemented in any software package. Also, these studies were limited to the serviceability limit state and ultimate limit state, i.e., they did not consider the behavior of structures under exceptional event conditions.

The use of structural finite elements, the most widely used in design practice, has several other disadvantages, including ignoring the failure in inclined sections, and smoothing of the effect of a discrete crack along the length of the element. These disadvantages are not so significant when checking structures at the stage of normal operation and when the limit state is reached. However, in conditions of significant increase of forces in elements, and intensive cracking, this approach leads to inaccuracies, as noted in the studies [XVII], [X].

Recently, discrete-continuum methods of computational analysis have been developed [XXV]. 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 may include the applied element method considered by Alanani et al. [III], and Grunwald et al. [X] for reinforced concrete structural systems. However, in these studies, there is no information on the assignment of stiffness parameters of such tie elements. Therefore, there is a need to substantiate the stiffness parameters taking into account the effects of redistribution of forces during crack formation revealed in [XIV],[XV],[XI].

For flat slabs, Kodysh and Mamin [XII] propose a similar approach, which the authors call as the discrete tie method. For the analysis of the behavior of typical joints and structural members of reinforced concrete frames, Bondarenko and Kolchunov [VII] proposed design models of resistance. However, this approach is mainly focused on the assessment of limit states under normal service conditions and does not consider the specificity of loading and deformation modes under accidental actions. Thus, there is a need for research to justify the application of such an approach to the structural analysis of reinforced concrete structures under accidental impacts.

Because of the research gap in the application and validation of numerical models of deep discretization, the objective of the study is (I) to develop and validate an approach to account for the effect of discrete cracks on the response of reinforced concrete building frames under the column failure scenario; (II) to investigate the effect of the discrete cracks on the behavior of the reinforced concrete multistorey frames under different column removal scenario.

II. Models and Methods

II.i. Design of the RC Frames

The study considers reinforced concrete multi-storey building frames for which finite element models, including structural FE with nonlinear flexible node connections, have been developed to account for additional rotations of the sections during cracking. A situational approach was adopted to simulate the effects on the frame as a result of the accident. This means that the reasons for the initial local failure are not taken into account. It is assumed that the inertial forces are generated in the structural element of a frame during its structural transformation.

The paper presents an analysis of the influence of the topology of the structural system on the redistribution of forces between vertical members after column removal. For this purpose, 3-storey, 5-storey, and 9-storey frames were considered (Figure 1a).

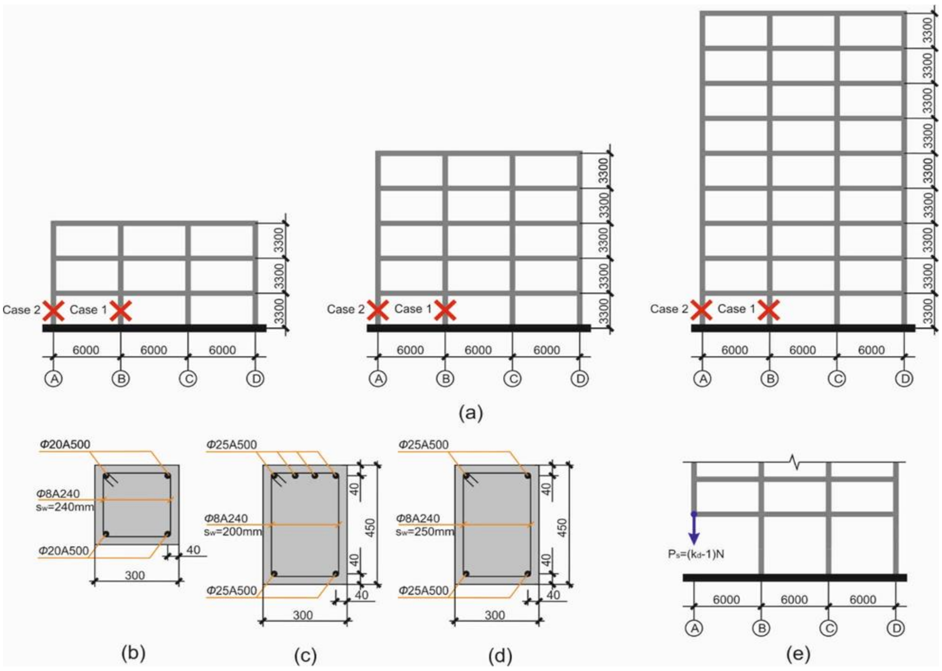


Fig. 1. Reinforced concrete frames (a); cross-section and reinforcement scheme of columns (b); cross-section and reinforcement scheme of beams near columns (c); cross-section and reinforcement scheme of beams at mid-span (d); accidental impact (e)

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 class B30 concrete (compressive strength f_c = 22 MPa, initial modulus of elasticity E_c = 32,500 MPa, strain at pick stress ε_{c0} = 0.002, ultimate strain in compression $\varepsilon_{c,ult}$ = 0.0035). The dimensions of the column sections were assumed to be 300×300 mm (Fig. 1b), and of the beams were 300×450 mm (Fig. 1 c, d). A500 (yield strength of reinforcement f_y = 500 MPa, ultimate strain $\varepsilon_{s,ult}$ = 0.025) was used for longitudinal reinforcement, and A240 (yield strength of shear reinforcement f_{yw} = 170 MPa) for stirrups. These reinforcement parameters were chosen about the structural requirements of SP 63.13330 [XXIII], based on the results of the calculation for the main load combination according to SP 20.13330 [XXI].

II.ii. Accidental Design Situations

Two variants of initial local failures were considered (Fig. 1a):

Case 1: sudden failure of the first-floor column along the A axis;

Case 2: sudden failure of the first-floor column along the B axis.

The accidental design situations were considered separately using a static non-linear analysis. The forces in the secondary design schemes were determined taking into account the dynamic effects on the structural system. For case 1, the forces in the first and second-floor columns were analyzed along the B axis. For Case 2, such analysis was carried out for the first and second-floor columns along axes A and C.

The calculation for the accidental action has been carried out according to SP 385.132580 [XXII] for a special combination of loads, including only the dead and long-term live loads accepted with characteristic values. The physical and mechanical properties of the materials were adopted following SP 63.13330 [XXIII]. Dynamic effects on load-bearing systems were modelled using the energy-based quasi-static method proposed in the study [IX]. This assumes that the reaction of the destroyed column is applied with the opposite sign to the point of it removal.

The value of this force can be determined from equation (1):

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

where F_i is in the end section of the structural member before its material or stability failure, k_d is the dynamic amplification factor (Fig. 1, e)

Based on the numerical study of the resistance of RC frames to progressive collapse carried out by Almazov et al. [V] and the results of tests on a full-scale model of a spatial frame [II], the dynamic increase factor k_d = 1.25 has been adopted. 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.

II.iii. Modeling of Frame Elements with Discrete Cracks

Structural beam-type finite elements were used to model the stress-strain state of the frame. Physical and geometrical nonlinearity (influence of buckling) was taken into account. 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 based on the "moment-rotation angle" diagram (Fig. 2, b).

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 (2):

$$C_i = 2\frac{M_{ult} - M_{crc}}{\Delta\varphi_{crc,0}}, \tag{2}$$

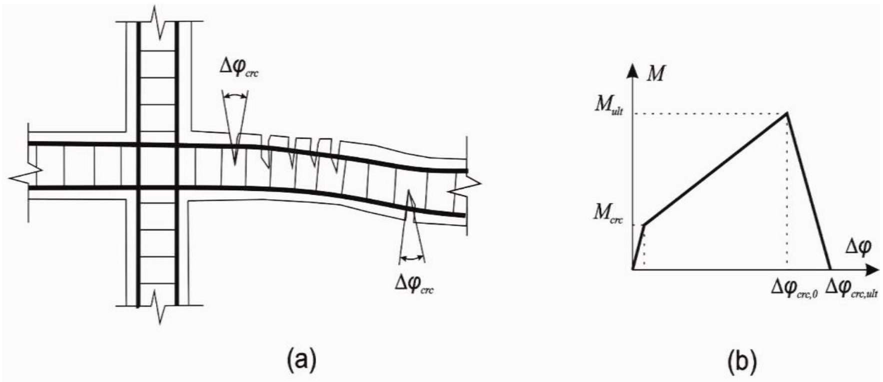


Fig. 2. Schematic diagram of a characteristic fragment of a reinforced concrete frame at the pre-crack stage, the material behavior is assumed to be close to elastic and the angles of rotation are almost identical to the curvatures in the sections

Where M_{ult} is the ultimate bending moment perceived by the cross-section of the element, determined from the equation (3):

$$M_{ult} = \left(N \cdot e_0 \cdot \eta\right)_{ult} = f_c \cdot \left[b \cdot x_{calc} \cdot \left(h_0 - \frac{x_{calc}}{2}\right)\right] + \left(f_y \cdot A_s' - \frac{N}{2}\right) \cdot \left(h_0 - a'\right), \tag{3}$$

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 [XXIII]; 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 (4):

$$M_{crc} = f_{ct} \cdot \gamma \cdot W_{red} \pm N \cdot e_x, \tag{4}$$

where f_{ct} is the characteristic tensile strength of concrete; γ is the coefficient, which accounts for plastic strain and is assumed for a rectangular cross-section as γ = 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 the formula (5):

$$\Delta\varphi_{crc,0} = \frac{a_{crc,0}}{h_{crc}}, \tag{5}$$

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 expressions (6)

$$a_{crc,0} = \varphi_1 \cdot \varphi_2 \cdot \varphi_3 \cdot \psi_s \cdot \frac{f_y}{E_s} \cdot l_s;$$
$$h_{crc} = h_0 - \frac{x_{calc}}{\alpha} \cdot \left(1 - \frac{\varepsilon_{ct,ult}}{\varepsilon_{c,ult}}\right). \tag{6}$$

Where φ_1, φ_2, φ_3 are the factors, that account for the duration of load action, longitudinal reinforcement profile, and type of stress-strain state of the element; ψ_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_s is the base spacing between cracks; $\varepsilon_{ct,ult}$, $\varepsilon_{c,ult}$ are the ultimate strains for concrete in tension and compression respectively; α is the transition factor from the design compressed depth x_{calc} to the actual one x_{fact}, determined by formula (7):

$$\alpha = \frac{x_{calc}}{x_{fact}}. \tag{7}$$

The nonlinear semi-rigid rotation restraints 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 following SP 63.13330 [XXIII] by formula (8):

$$10 \cdot d_s \le l_s = 0{,}5 \cdot \frac{A_{ct}}{A_s} \cdot d_s \le 40 \cdot d_s, \tag{8}$$

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.

Figure 2, b shows a "Moment vs. Additional Angle of Rotation" diagram to determine the stiffness of the element coupling.

Having determined the stiffness parameters, the problem is further solved using the finite element method in the form of the displacement method, taking into account the nonlinear elastoplastic restraints at the ends of the beam elements. The basic system of the displacement method is shown in Fig. 3, a. It includes characteristic fragments: (1) the beam-column interface (Fig. 3, b), in which the beams and columns are modeled by beam elements and the beam-column interface is modeled by a plane element; (2) the beam-element interface belonging to a structural element, e.g., a beam (Fig. 3, c).

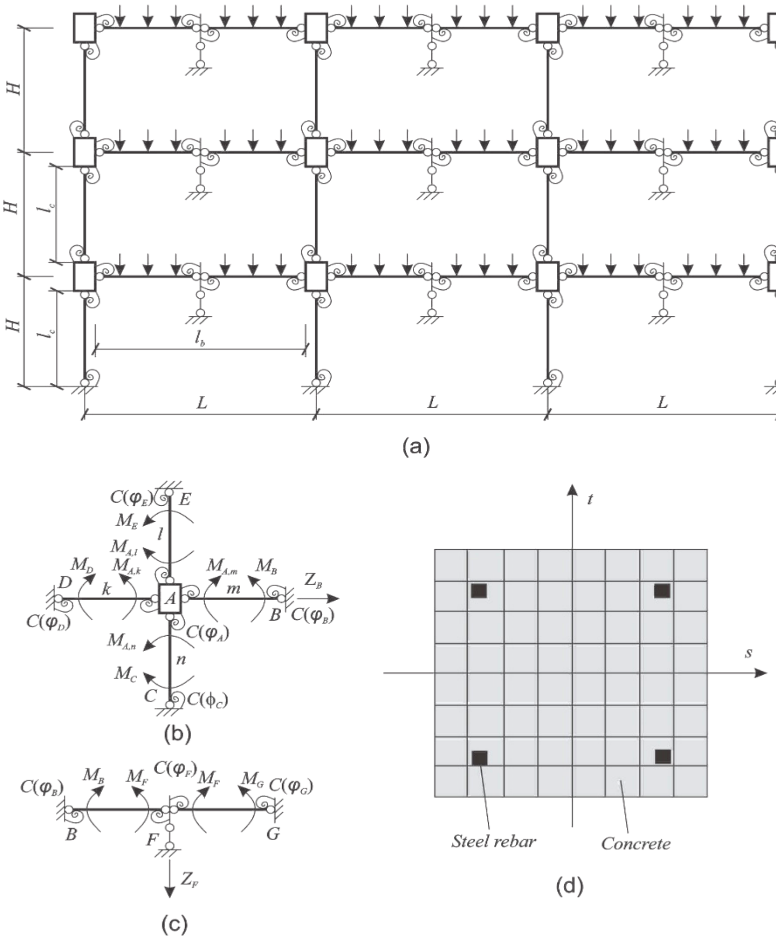


Fig. 3. Calculation scheme of the displacement method for the stability analysis of reinforced concrete frames under accidental action: general view of the scheme (a), scheme of the column and beam interface joint (b), scheme for the most dangerous normal section along the beam length (c), structural element (d)

The equation with respect to the unknown displacements and rotation angles Z of the basic system shown in Fig. 3, has the form (9):

$$Z = r^{-1} \times R, \tag{9}$$

where r is the stiffness matrix; Z, R are the matrices of displacements and forces, respectively.

The energy criterion is used as a criterion for stability failure or, in a broader sense, for transition to transient deformation of a structural system. The increment of the strain potential energy due to external influence is considered. The moment of stability failure corresponds to the change of sign of the strain energy increment:

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

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:

$$\left|\frac{dP}{df}\right| \leq a, \tag{11}$$

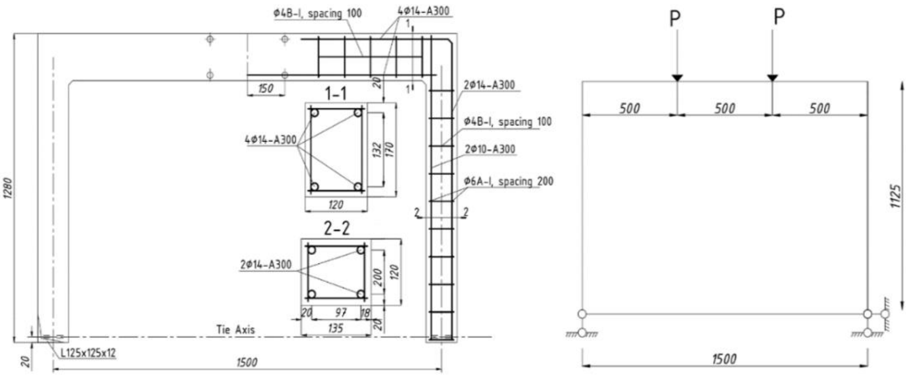
where dP is the increment of axial force, and df is the increment of deflection.

This criterion is applicable both in central compression and eccentric compression. It also allows evaluation of the robustness of the load-bearing system as a whole.

II.iv. Parameters of the Experimental Frame Used for Model Verification

To verify 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 [VII]. The general view and reinforcement scheme of the frame are shown in Figure 4. The material of the test frame is B35 concrete (f_c = 25.5 MPa, f_{ct} = 1.95 MPa, E_c = 34500 MPa, $\varepsilon_{c,ult}$ = 0.0035). The frame reinforcement is made of A300 steel bars (f_y = 300 MPa, E_s = 200000 MPa, $\varepsilon_{s,ult}$ = 0.025). The assumed material deformation diagrams are shown in Figure 5.

The frame was modeled by universal physically and geometrically nonlinear beam-type structural finite elements. All nodes of the initial design model were expanded with only linear displacement coupling. As a result, a computational scheme was obtained consisting of unconnected structural elements coupled only by linear displacements and deforming independently under rotation. Special zero-length finite elements were introduced between the nodes of adjacent structural elements to simulate the rotation of sections during the opening of discrete cracks according to expressions (3), (5).



(a) (b)

Fig. 4. U-shaped frame: reinforcement scheme (a), calculation scheme (b)

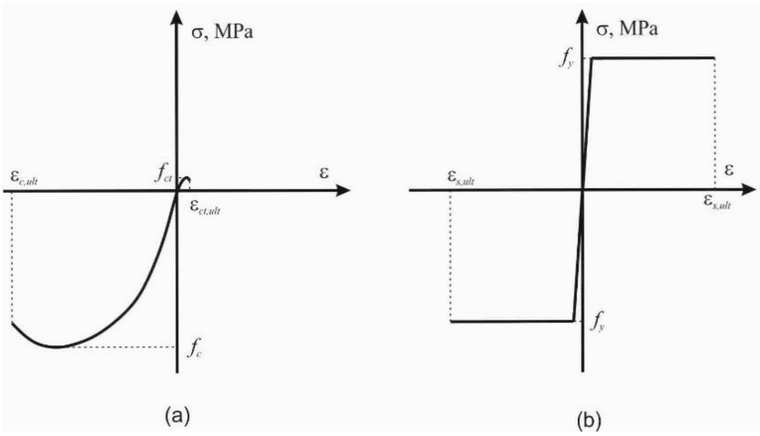


Fig. 5. Stress-strain diagrams for concrete (a) and steel reinforcement (b)

III. Results and Discussion

Comparison of Numerical Modeling Results and Experimental Data

For the experimental reinforced concrete frame, the static analysis was performed for two 90 kN concentrated forces applied as shown in Figure 4, b. Figure 6 shows the results of the calculation of 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 discussed above.

The bending moments in the frame according to the results of calculation using the proposed method almost completely coincided with the experimentally determined values. As a result of cracking, there was a redistribution of bending moments in the beam of the frame. The moments in the structural joints decreased by 50 % and increased in the span by 37.5% compared to the calculation using the traditional approach of the structural finite element method. 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.

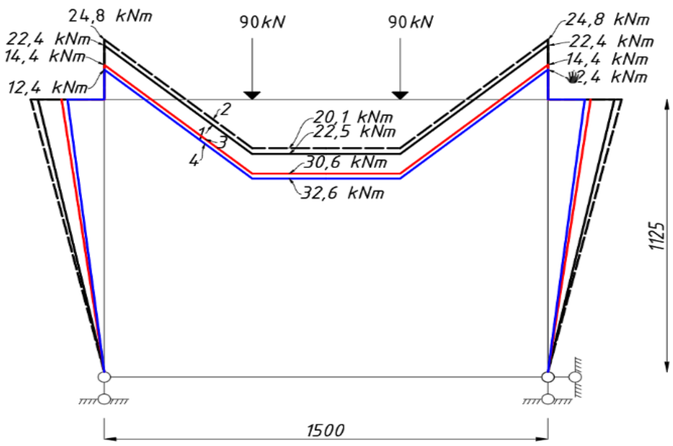


Fig. 6. Diagrams of bending moments in a U-shaped reinforced concrete frame: 1 - for rigid connections; 2 - for pliable connections; 3 - according to experimental data; 4 - according to the proposed method

The obtained agreement of the results allows us to apply the developed method to the assessment of the influence of the topology of the structural system on the robustness of reinforced concrete frames of multi-storey buildings under accidental actions, taking into account the violation of the concrete continuity due to the formation of cracks.

Parametric Study of the Effect of Reinforced Concrete Frame Topology on the Redistribution of Forces under Accidental Action

In this section, the results of calculations for reinforced concrete frame structures are presented, taking into account the developed method and the traditional method of calculation using structural elements. Figures 7, 8, and 9 show plots of changes in axial forces and bending moments in the upper cross-sections of the columns of the first floor and the lower supporting cross-sections of the columns of the second floor along axes A and C when the column of the first floor along axis B is removed. These columns were selected for analysis because they are adjacent to the zone of initial local failure and are subjected to the greatest loads as a result of the restructuring of the structural system (redistribution of loads along alternative paths).

The increase in the storey of the building practically did not affect the change of bending moments in the column sections. However, it led to an increase in longitudinal forces in the elements. Comparison of the calculation results of the traditional approach and the proposed method are shown in Figures 10, 11, and 12.

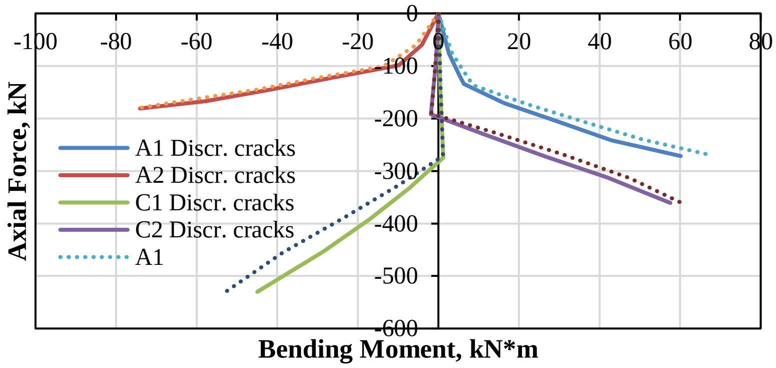


Fig. 7. Axial forces and bending moments in the 3- storey frame for Case 1

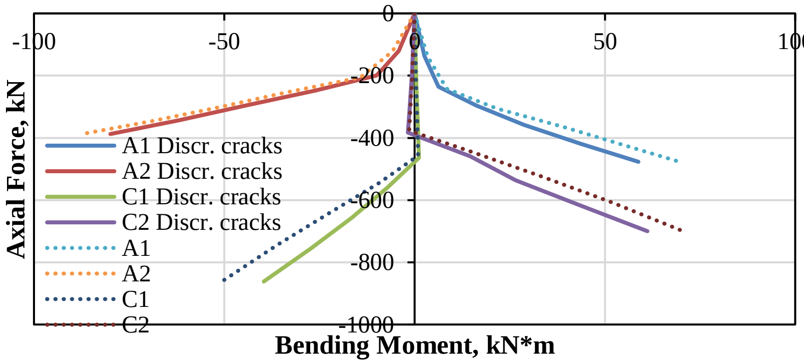


Fig. 8. Axial forces and bending moments in the 5- storey frame for Case 1

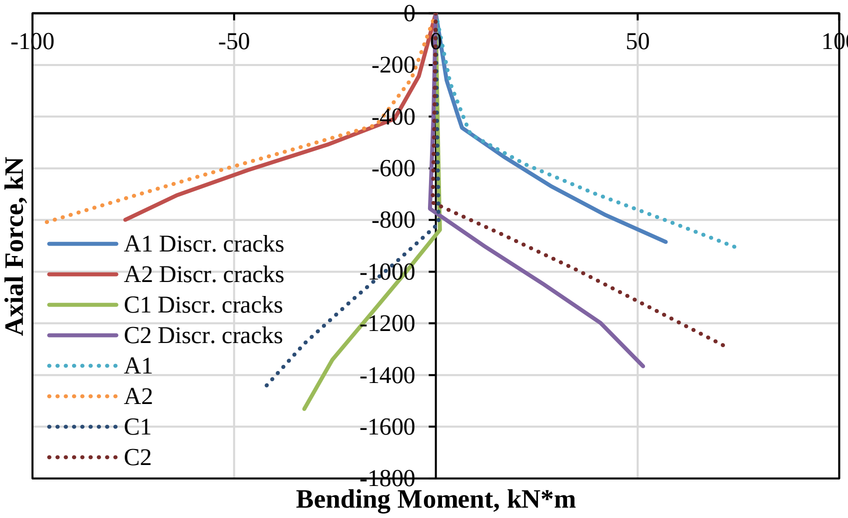


Fig. 9. Axial forces and bending moments in the 9- storey frame for Case 1

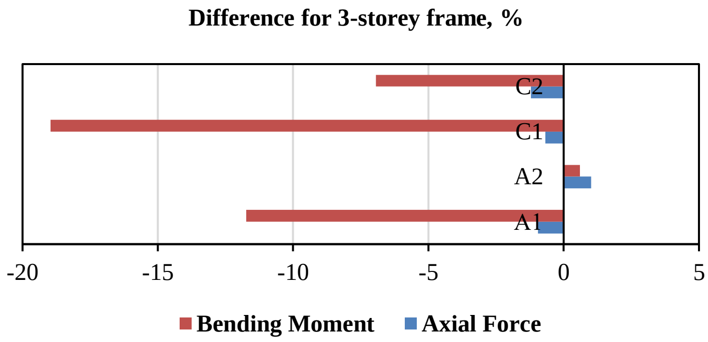


Fig. 10. Changes of axial forces and bending moments in columns of the 3- storey frame for Case 1

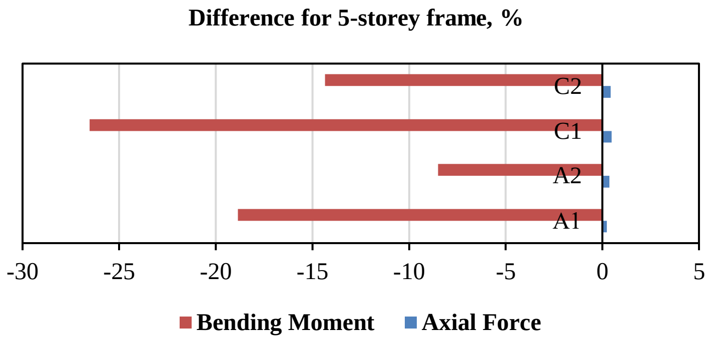


Fig. 11. Changes of axial forces and bending moments in columns of the 5- storey frame for Case 1

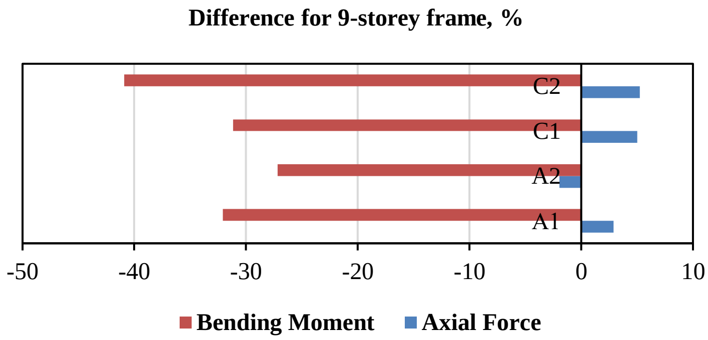


Fig. 12. Changes of axial forces and bending moments in columns of the 9- storey frame for Case 1

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 the A axis.

The calculation results for the second design situation (Case 2), assuming the removal of the corner column of the first floor, are presented in Figures 13, 14, and 15, which show the plots of changes of axial forces and bending moments in the upper sections of the columns of the first floor and the lower supporting sections of the columns of the second floor along the axis B. As in the previous case, the selection of these columns for consideration was due to their proximity to the zone of initial local failure and the greatest influence of the accidental action on them.

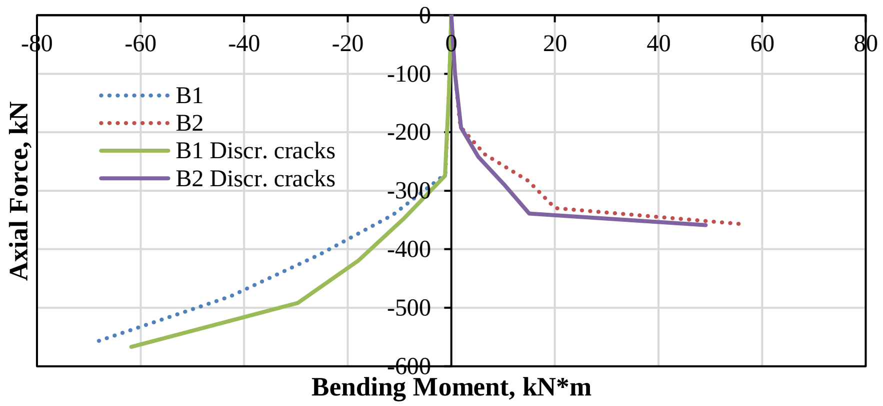


Fig. 13. Axial forces and bending moments in the 3- storey frame for Case 2

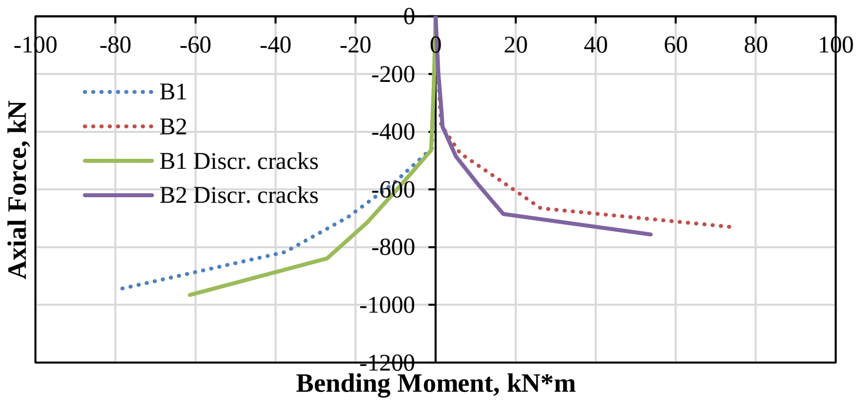


Fig. 14. Axial forces and bending moments in the 5- storey frame for Case 2

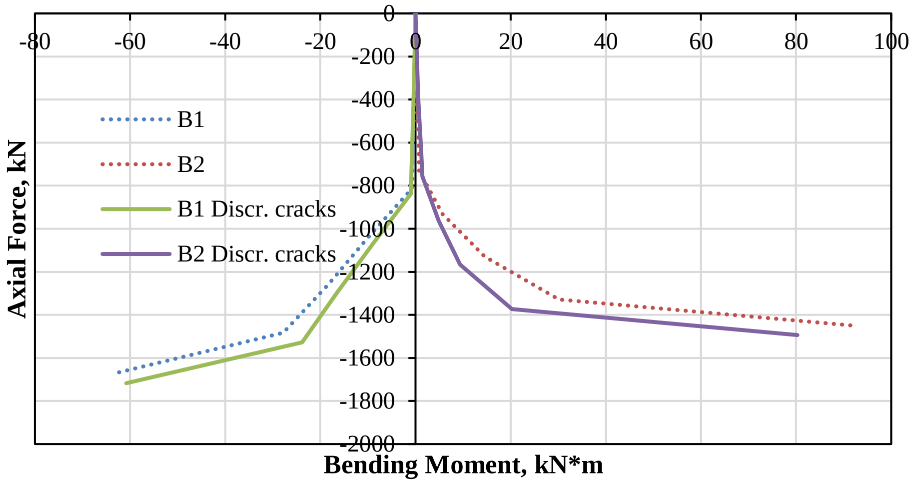


Fig. 15. Axial forces and bending moments in the 9- storey frame for Case 2

For the considered accidental action, the increase of the building floor had a more noticeable effect on the change of the bending moments in the column cross-sections than when the column of the middle row (Case 1) was removed. It was observed that the bending moments increased as the floor's number increased. The axial forces in these members also increased. The comparison of the calculation results of the traditional approach and the proposed method is shown in Figures 16, 17, and 18.

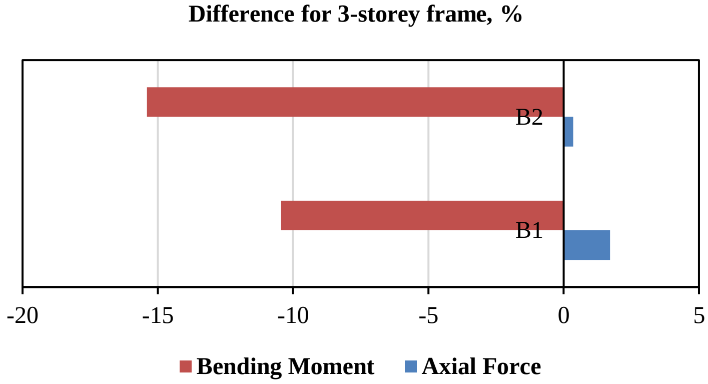


Fig. 16. Changes of axial forces and bending moments in columns of the 3-storey frame for Case 2

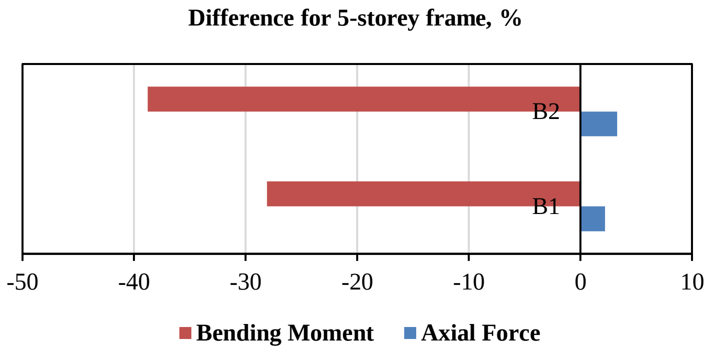


Fig. 17. Changes of axial forces and bending moments in columns of the 5-storey frame for Case 2

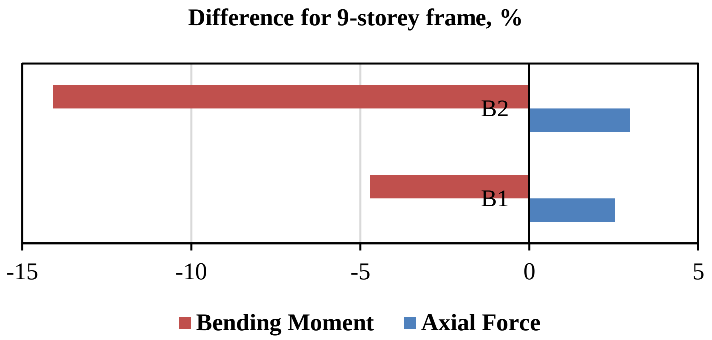


Fig. 18. Changes of axial forces and bending moments in columns of the 9-storey frame for Case 2

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.

Changes in the Axial Force in the Beam

Figures 19, 20, and 21 show the results of comparing the axial forces in the beams above the zone of initial localized failure as a function of floor and modeling method.

The analysis of the diagrams shows that for reinforced concrete frames with 3 and 5 storeys there is an excess of tensile axial forces in the beam supports over the values according to the traditional calculation method. This has a negative effect on the resistance of the first and second storey columns adjacent to the beam by increasing the influence of buckling. For the 9 storey 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. This indicates that the beams and columns of the floors above form a Vierendel truss above the zone of initial localized failure.

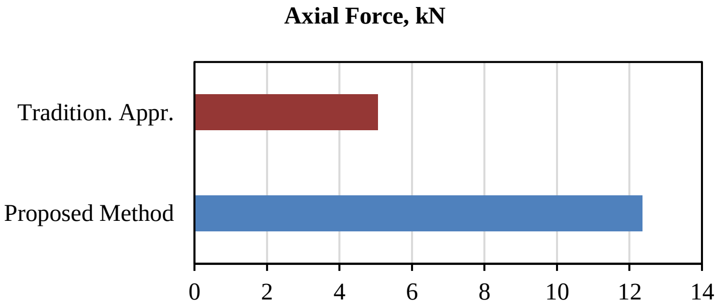


Fig. 19. Axial forces in beams above the zone of initial localized failure for the 3-storey frame

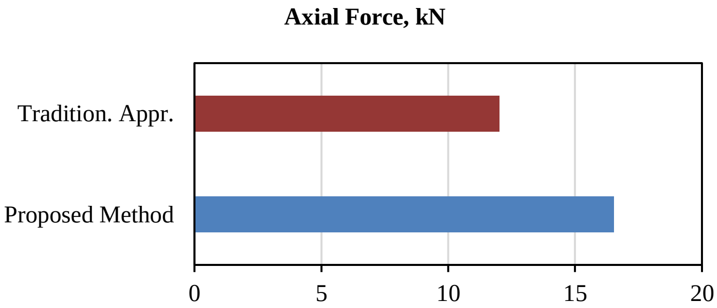


Fig. 20. Axial forces in beams above the zone of initial localized failure for the 5-storey frame

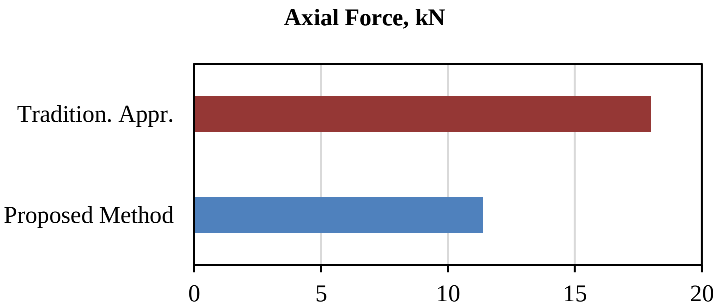


Fig. 21. Axial forces in beams above the zone of initial localized failure for the 9-storey frame

IV. Conclusions

The effect of discrete cracks on the behavior of the reinforced concrete multistorey frames under a column removal scenario is revealed. The following conclusions have been drawn:

I. An approach to take into account the effect of discrete cracks on the response of reinforced concrete building frames under the column failure scenario has been developed. The approach implies the introduction of discrete tie elements into traditional finite element models that take account of the relationship between moments and rotations, considering the specifics of the performance of materials, sections, and structures under the conditions of redistribution of forces as a result of initial local failure in the structure.

II. Validation of the proposed approach is performed on the experimental data. Also, it is compared with the modeling results of the existing approaches. The bending moments in the frame according to the results of calculation using the proposed method almost completely coincided with the experimentally determined values. As a result of cracking, there was a redistribution of bending moments in the beam of the frame. The moments in the structural joints decreased by 148% and increased in the span by 37.5% compared to the calculation using the traditional approach of the structural finite element method. 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.

III. The effect of discrete cracking on the deformed state of reinforced concrete building frames under the scenario of column failure has been established. The increase in the storey of the building practically did not affect the change of bending moments in the column sections. However, it led to an increase in longitudinal forces in the elements. 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 the A axis.

IV. The analysis of the diagrams shows that for reinforced concrete frames with 3 and 5 storeys there is an excess of tensile axial forces in the beam supports over the values according to the traditional calculation method. This has a negative effect on the resistance of the first and second storey columns adjacent to the beam by increasing the influence of buckling.

The proposed approach can be used in analyzing the structural safety of buildings under accidental impacts associated with failure scenarios of a load-bearing element. It can be recommended for implementation in the national code to reduce the risk of unacceptable consequences of localized failures or damage in load-bearing systems of buildings.

Prospects for further research are related to the justification and implementation of multidimensional interaction surfaces and the mutual influence of forces on the behavior of tie elements under operational and accident loads in the discrete modeling approach.

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Conflicts of Interest

The authors declared that there is no conflict of interest.

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