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Resistance of Reinforced Concrete Frames to Progressive Collapse at Catenary Action of Beams

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Abstract. The specific properties of deformation and failure of a reinforced concrete frame are investigated under sequential realization of arch and catenary action of beams after removal of the middle row column. Numerical modeling with the use of solid- and beam-type finite element models is performed for the purposes of the study. It was found that at the failure of the column of the second row the beam of the frame above the point of column removal transform to catenary structure, as evidenced by the relative deflection 1/29.8 (179 mm). The compressed concrete at the outer face of the corner column then collapsed, followed by the complete collapse of the frame. It is shown that the results of calculation performed with the use of the frame model based on solid finite elements were visually close to the results of numerical modeling with the use of bar finite element models before the onset of catenary action of the beams. For more correct modeling of reinforced concrete frame structures when catenary action of beams is realized in them. It is advisable to use specific modeling methods, such as accounting for additional rotations of sections at crack formation.

Сопротивление железобетонных рам прогрессирующему обрушению при больших прогибах ригелей

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Ключевые слова: железобетон, рама, ригель, арочный механизм, цепной механизм, аварийное воздействие, метод конечных элементов

Аннотация. Исследуются особенности деформирования и разрушения железобетонной рамы при последовательной реализации арочного и цепного механизма сопротивления ригелей после удаления колонны среднего ряда. Для целей исследования выполнено численное моделирование с использованием объемных и стержневых конечно-элементных моделей.

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Установлено, что при отказе колонны второго ряда ригель рамы над удаленной из расчетной модели колонны переходил к вантовому механизму сопротивления, о чем свидетельствовал относительный прогиб 1/29,8 (179 мм) конструкции над удаленной колонной. Затем сжатый бетон у внешний грани угловой колонны разрушился, после чего происходит полное обрушение рамы. Показано, что результаты расчета, выполненные с использованием модели рамы из объемных конечных элементов, оказались визуально близки к результатам численного моделирования с использованием стержневых конечно-элементных моделей до наступления цепной работы ригелей. Для более корректного моделирования работы железобетонных рамных конструкций при реализации в них цепной работы ригелей целесообразно использовать специальные методы моделирования, такие как учет дополнительных поворотов сечений при образовании трещин.

INTRODUCTION

Since September 11, 2001, structural engineering has a new research direction aimed at preventing the disproportionate collapse of structures. In accident situations associated with the failure of a load-bearing element of the structural system, the slabs (roof) above the zone of initial local failure are the first to be involved in the redistribution of loads [1–3]. Therefore, it is important to analyze the peculiarities of resistance, specified criteria for exceeding the special limit state of such structures, as well as the influence of the deformed state of floor structures on the resistance of vertical load-bearing structures such as columns and pylons.

According to SP 385.1325800.2018 “Protection of buildings and structures against progressive collapse. Design code. Basic statements”, the deflections of bending elements of the structural system for a special limit state should not exceed 1/30 the span, excepting prestressed structures for which the ultimate deflection should not exceed 1/50 of the span.

In the study [4], the maximum relative deflection for series 1 frame (without prestressing the beams) after sudden removal of the center column was 1/16.4 span, while for series 2 frame (with prestressing the beams) it was 1/54.7 span. For the first series frame, the failure of the columns at the nodes adjacent to the beams was observed. For the frame with prestressed beams, the relative deflection did not exceed 1/50 of the span. Despite the significant crack opening in the most stressed section of this beam, the load-bearing capacity (from the position of the special limit state) was not exhausted under the considered special impact.

Al Shaikh et al. [5] investigated the fracture resistance mechanisms. Negative values of the axial force indicate the operation according to the beam scheme before the formation of cracks, and after the formation of a network of cracks it indicates the operation according to the arch scheme. The change in the sign of the axial force with increasing load shows the realization of a catenary (membrane) resistance mechanism of the hanging system type. Thus, in the beam resistance to failure can be distinguished into three phases: elastic work, arch mechanism and chain mechanism. The first phase (i.e., the elastic phase) occurred during the uncracked concrete stage. Then, after cracks were formed in the tensile zone concrete the arch mechanism was activated. The beginning of the catenary stage can be demonstrated by determining the point at which the value of the axial force is zero, i.e., when the compressive axial force drops to zero and begins to transform into tensile force as shown in Fig. 1.

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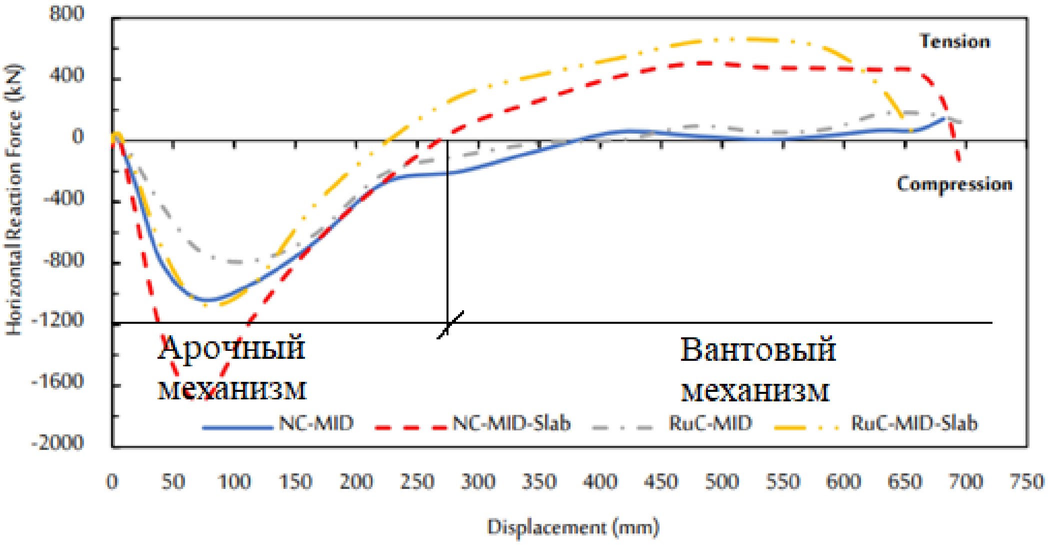

Fig. 1. Relationship between axial force and deflection of the beam [5]
Рис. 1. График зависимости между продольной силой и прогибом перекрытия [5]

For the case of mid-column failure, the catenary mechanism starts when the relative deflection reaches 1/24 (375 mm) in the model without a floor slab and 1/33 (270 mm) in the model with a floor slab [5]. It is observed in [6] that due to the limitation of horizontal displacement by the surrounding undamaged elements, the slabs continue to resist the propagation of failure due to the realization of the arch scheme of structural resistance until certain vertical displacements are achieved, as shown in Fig. 2.

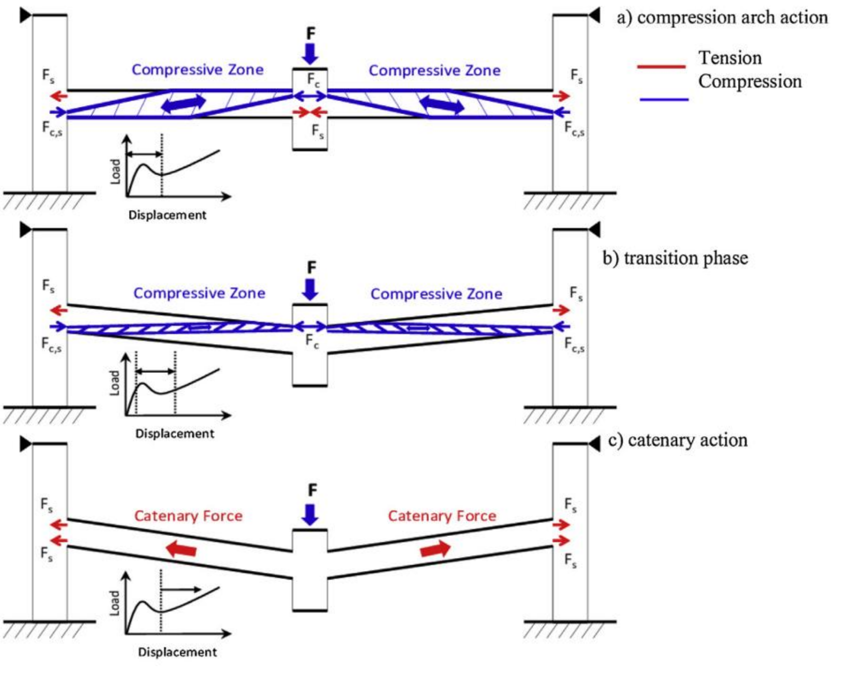

Fig. 2. Arch and catenary resistance mechanism [6]
Рис. 2. Арочный и вантовый механизм сопротивления [6]

The influence of horizontal constraints is reduced when the beam or slab has a larger deflection value compared to the cross-sectional depth. Since compressed arch structure is the primary resistance mechanism, many experimental and analytical studies have been conducted to effectively evaluate its role. Based on experimental tests [7], the compressed arch structure is the primary resistance mechanism at low deformations and quantitatively increases the specimen resistance by about 30–150 %. The increase in resistance of compressed arch structure is observed in elements with large cross-sectional depth-to-span ratio and low percentage of reinforcement.

At large deflection, the compressive axial force in the beam or slab turns into a tensile force, which indicates the beginning of the next stage, the catenary mechanism [6]. The catenary mechanism is mainly determined by the magnitude of tensile forces in the longitudinal reinforcement under the action of vertical load. As shown in Fig. 2, the tensile force in a beam or slab has a vertical component due to the large deflection of the beam or slab. This vertical component helps to resist the increasing loads on the structure after column removal. The catenary mechanism is activated at large displacements because the magnitude of the vertical force is directly related to the angle of rotation of the beam. Based on the allowable relative deflections found in the experiments and established in the standards, the ratio between the vertical force component and the horizontal force component in beams and slabs operating on the catenary scheme is proportional to the relative deflection.

The slabs provide additional resistance for arch or catenary actions. Gouverneur et al. [8] investigated the load and deformation response of the one-way slabs in which catenary action is observed. The effect of reinforced concrete slabs can be studied from two perspectives. On the one hand, the slabs act as compressed/stretched membranes. On the other hand, the presence of slabs changes the bearing capacity of beams because their flexural and torsional stiffness would be enhanced. This is known as the flange effect, where the effective width of the flange is determined by the span of the beam and the relative thickness of the slab. Relevant studies generally assume that slabs have a positive effect on the ultimate load carrying capacity of structures. However, the degree of influence obtained in different tests varies widely due to the simultaneous influence of parameters such as beam section depth, beam span, location of the column to be removed, slab thickness, and the seismic design. In addition, slabs have also been found to affect the performance of reinforced concrete frame structures such as load resistance mechanisms [9–11], failure modes [12, 13] and load redistribution [14].

Pham et al. [15] studied the model of a two-span reinforced concrete frame for the sudden removal of the middle row column with beams passing to resistance by the type of a catenary system. According to the test results, the rupture of the upper reinforcement in the support sections of the outermost columns and the lower reinforcement in the support sections of the middle column were observed. The failure of the concrete of the compressed zone in the outermost column at the height of the beam-column joint was also observed. This failure was caused by the change in the effective design length of the column and additional lateral action on the column from the beam after its transition to the catenary action.

Thus, this study is aimed at investigating the features of deformation and failure of a reinforced concrete frame in the case of sequential realization of arch and catenary mechanism of beam resistance after removal of the middle row column.

METHOD

To identify the features of deformation and fracture, numerical modeling of a reinforced concrete frame was performed. The design parameters of this frame were adopted according to the study of Weijian Yi et al. [16]. The general view of the frame is shown in Fig. 3. Weijian Yi et al. evaluated the robustness of the frame against progressive collapse at the scenario of a middle column removal. In order to evaluate the possibility of realizing the mechanism of exhaustion of bearing capacity due to stability failure, the removal of the second-row column was considered in this study, as shown in Fig. 3.

Columns and beams are made of concrete class B35. Column and beam reinforcement is made of 4 Ø12 bars of class HRB400 according to Chinese standard (equivalent to A400). The columns have a cross-section of 200 × 200 mm and the beams have a cross-section of 100 × 200 mm. The load P_1 has a value of 500 kN and P_2 = 100 kN.

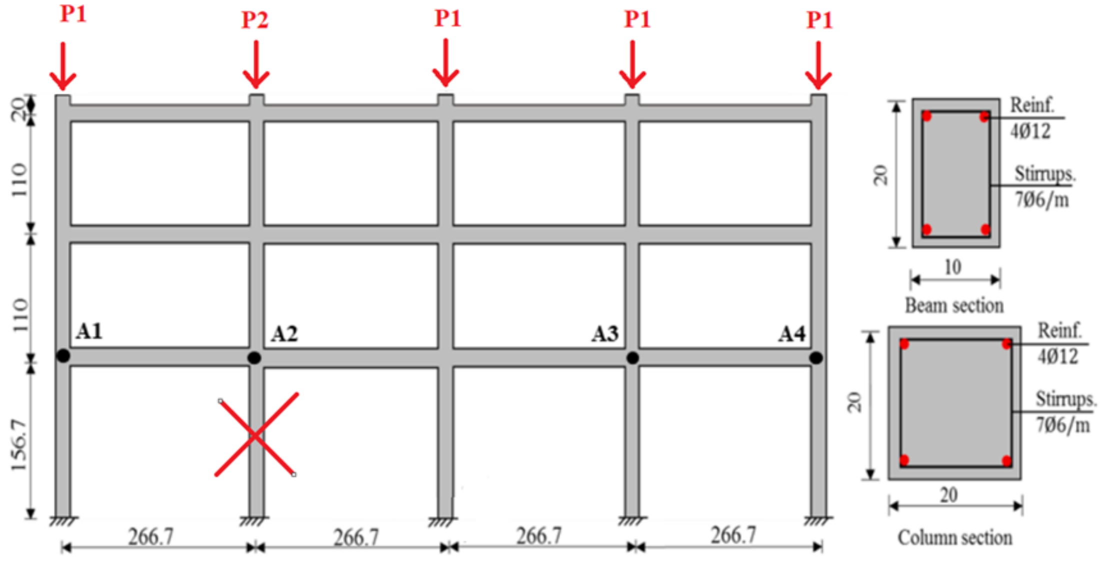

Fig. 3. Dimensions and reinforcement scheme of reinforced concrete frame sections (cm)
Рис. 3. Размеры и схема армирования сечений железобетонной рамы (см)

Based on diagram in Fig. 3, a finite element model was developed. In this model, the concrete is modeled by 8-node solid finite elements and the longitudinal and transverse reinforcement is modeled by beam-type elements. Perfect bonding of reinforcement with concrete at all stages of deformation was assumed. Material properties were modeled using idealized deformation diagrams. The calculation was performed in two stages. Stage one involves calculation of the frame according to the primary design scheme (before the accidental action) for concentrated loads P_1 = 500 kN applied to the heads of the columns of the upper floor of the frame. Stage 2 involves removal of the column of the second row (see Fig. 3). The general view of the finite element model is presented in Fig. 4.

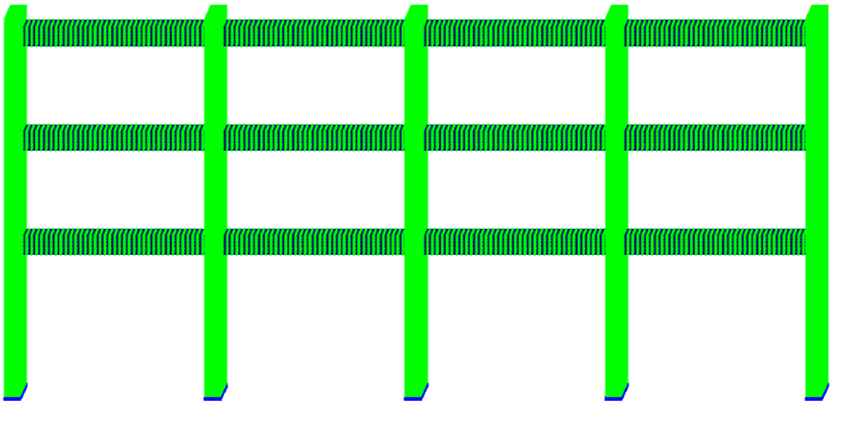

Fig. 4. General view of the finite element model of reinforced concrete frame in LIRA-CAD software
Рис. 4. Общий вид конечно-элементной модели железобетонного рама в ПК ЛИРА-САПР

RESULTS AND ANALYSIS

Fig. 5 shows the deformed state of the reinforced concrete frame model at the calculation iteration preceding the collapse of the structure. The deflection of the beam over the column removal point was 179 mm, which is close to the depth of its cross-section, which is equal to 200 mm. This indicates that the beam is in a transient state during the change from arch action to catenary action. Fig. 6 shows the patterns of the collapsed finite elements of the model.

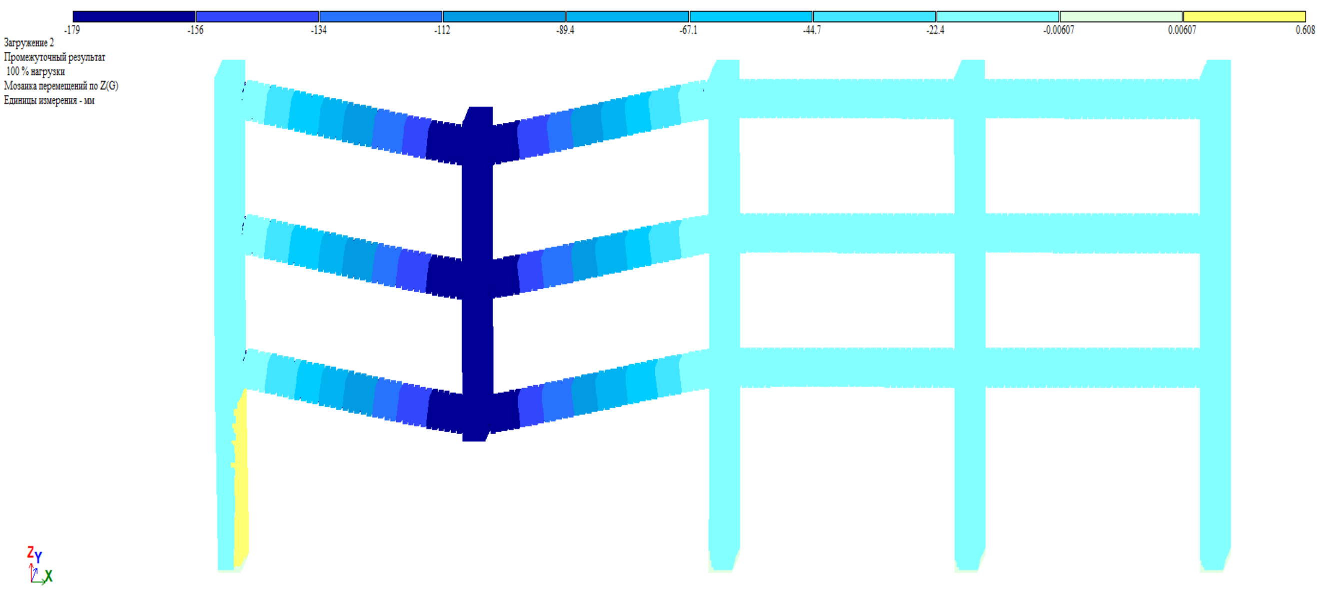

Fig. 5. Deformed state of the reinforced concrete frame at the calculation iteration preceding the structural collapse
Рис. 5. Деформированное состояние железобетонной рамы на итерации расчета, предшествующей обрушению конструкции

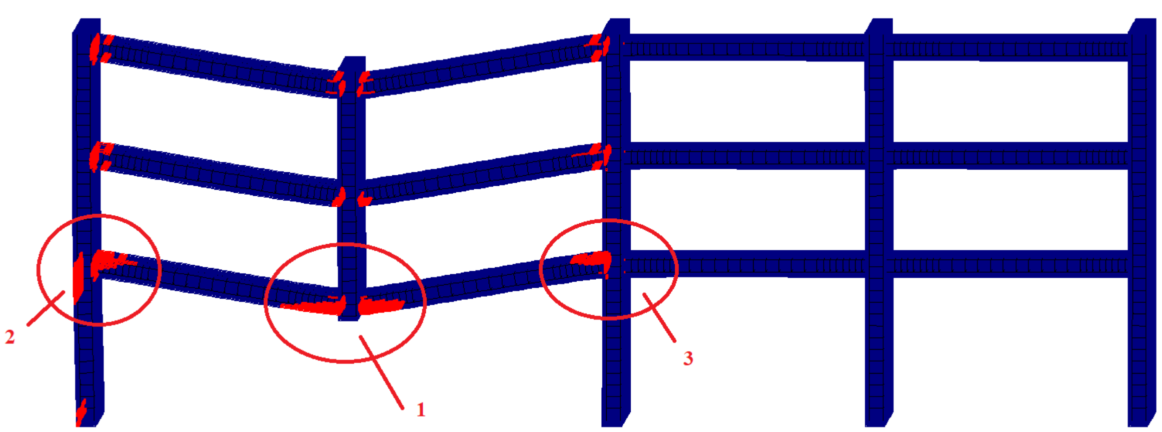

Fig. 6. Fracture patterns of reinforced concrete frame model
Рис. 6. Общий вид разрушений в модели железобетонной рамы

At the failure of the second-row column (assumed as initial local failure), the beam of the frame above the point of column removed transitioned to a catenary resistance mechanism, as indicated by the relative deflection of 1/29.8 (179 mm) of the structure. According to Weijian Yi et al. [16], when the relative deflection reaches a value of 1/38.1, the cable-stayed mechanism begins. According to the study of [17], this mechanism is activated when the relative deflection reaches 1/33.4. Or according to the experiment in [5] this value should be 1/25. Thus, in this study, the transition to catenary action is conservatively estimated on the basis of the results of previous studies as 1/30.

A plastic hinge was formed at a joint 1 (see Fig. 6). The tensile stress in the bottom reinforcement of the beam continued to increase as the concrete in the tensile layer fractured. When the tensile stress reaches its maximum value and the bottom reinforcement is completely destroyed, it leads to the failure of the beam in the support sections above the removed column.

In joint 2 (see Fig. 6), when a plastic hinge is formed, the beam transmits to the columns an axial force, which can be absorbed in the outermost column only due to its own flexural stiffness. As a result, the concrete of the compressed zone was crushed at the beam-column joint. The upper reinforcing bars of the beam also fractured as a result of reaching their yield strength.

At joint 3 of the frames (see Fig. 6), failure of the top reinforcement was also observed. However, the column cross-section was not destroyed because the horizontal displacement of the column was limited by the beam in the third span.

The results obtained are in agreement with the collapse observed in the studies of Pham et al. [15], presented in Fig. 7.

Fig. 7. General view of the failure of reinforced concrete frame elements in the study by Pham et al. [15]
Рис. 7. Общий вид разрушения элементов железобетонной рамы в исследовании Pham и др. [15]

Numerical modeling of the deformation of the reinforced concrete frame was also performed in accordance with the approach proposed in [18] using a structural finite element model, including special finite elements (FE No. 295 in accordance with Lira-CAD) for modeling rotations in cross-sections with cracks. The deformed state of the frame is presented in Fig. 8. Fig. 9 shows the general view of failures in the structural finite element model based on the results of numerical modeling.

Simulation results for solid and beam-type finite element models showed that after the removal of the middle row column, plastic hinges were formed in the beams above the column removal point. Then plastic hinges (and further physical hinges) were formed in the beam-column interfaces. As a result, the beams on either side of the point above the removed column are transformed into a catenary mechanism.

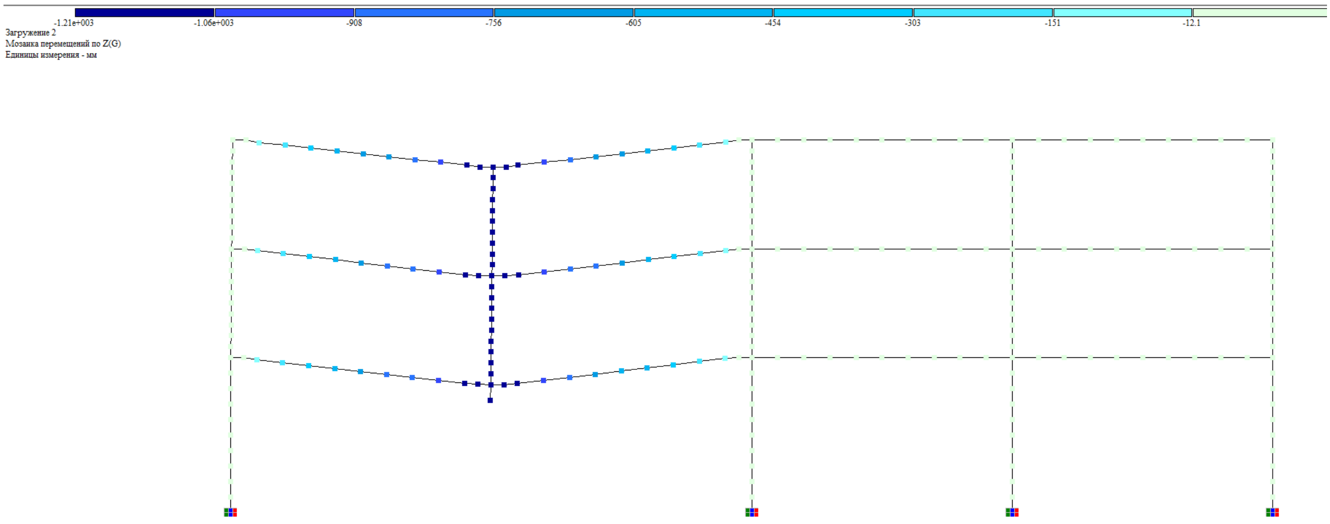

Fig. 8. Deformed state of the structural finite element model of the reinforced concrete frame
Рис. 8. Деформированное состояние стержневой модели железобетонной рамы

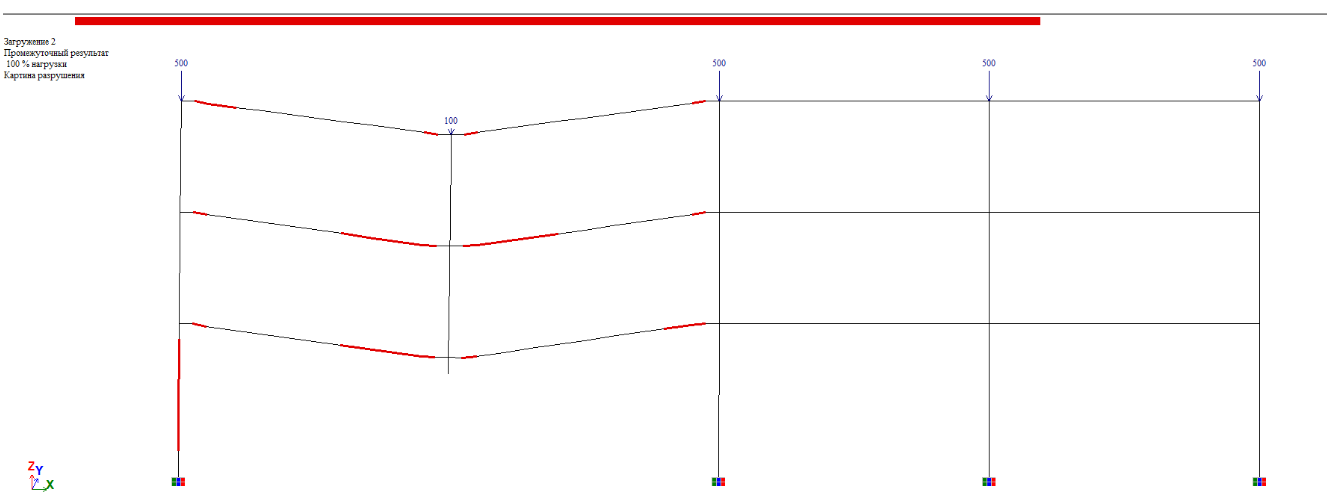

Fig. 9. Fracture pattern of the bar finite element model of the frame
Рис. 9. Картина разрушения стержневой модели рамы

In contrast to the results of the beam-type model, the solid model shows that at the beam-column joints, only the lower tensile zone of the beam collapses at the point where the column is removed. At the connection point of the beam and the neighboring column, the failure occurred only in the upper layer. Further destruction occurs similarly in the upper floors, spreading from bottom to top. Then the compressed concrete at the outer face of the corner column crushed, as shown in Fig. 6, 9, followed by the complete collapse of the frame.

Fig. 10 plots the deflection of the columns at the joint 1 under the applied load P_2. These results are obtained using solid and beam-type finite element models, including those with special elements modeling additional rotations at crack opening.

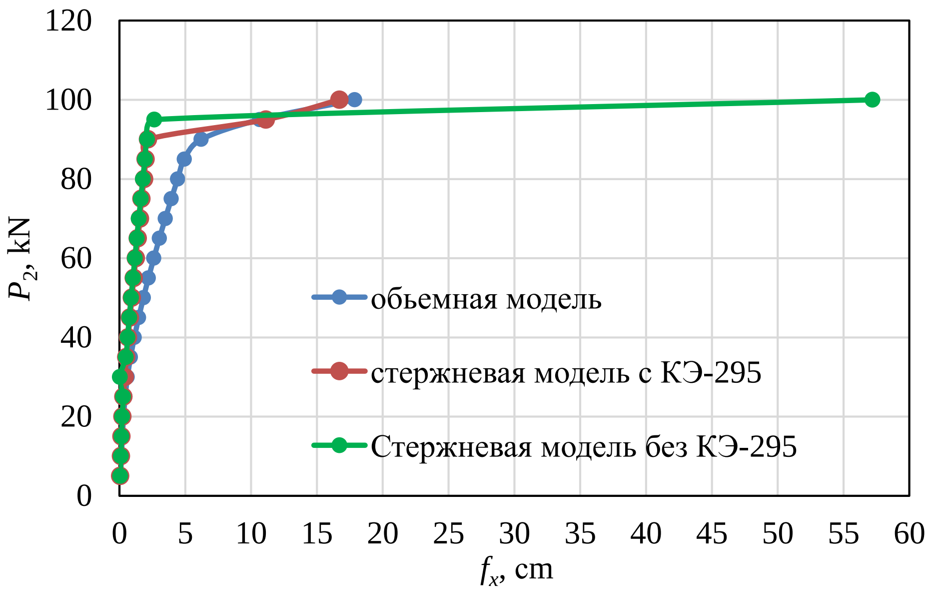

Fig. 10. Load – lateral displacement diagrams of the upper section of the first-floor corner column under accidental design situation

Рис. 10. Графики зависимости «нагрузка – поперечное смещение» верхнего сечения угловой колонны первого этажа при аварийной расчетной ситуации

Fig. 10 demonstrates that the results of calculation performed using the frame model made of solid finite elements are visually close to the results of numerical modeling using bar finite element models before the onset of catenary action of beams. For more correct simulation of reinforced concrete frame structures in case of catenary action of beams, it is recommended to use special modeling methods, such as accounting for additional rotations of cross-sections during cracking [18].

CONCLUSIONS

The article examines the peculiarities of deformation and failure of reinforced concrete frame at sequential realization of arch and catenary mechanism of beams resistance after removal of the column of the middle row. Based on the conducted research the following conclusions have been drawn:

1. It was found that at the failure of the column of the second row the beam of the frame above the point of column removal transform to catenary structure, as evidenced by the relative deflection 1/29.8 (179 mm). The compressed concrete at the outer face of the corner column then collapsed, followed by the complete collapse of the frame.

2. It is shown that the results of calculation performed with the use of the frame model based on solid finite elements were visually close to the results of numerical modeling with the use of bar finite element models before the onset of catenary action of the beams. For more correct modeling of rein-forced concrete frame structures when catenary action of beams is realized in them. It is advisable to use specific modeling methods, such as accounting for additional rotations of sections at crack formation.

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