

РАСЧЕТ И ПРОЕКТИРОВАНИЕ СТРОИТЕЛЬНЫХ КОНСТРУКЦИЙ ANALYSIS AND DESIGN OF BUILDING STRUCTURES

DOI: 10.22363/1815-5235-2025-21-4-321-333

EDN: CGZOGJ

Research article / Научная статья

Stages of Resistance of Reinforced Concrete Frames in Accidental Design Situation

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Received: May 14, 2025

Revised: July 21, 2025

Accepted: July 30, 2025

Abstract. The study addresses the stress-strain state stages of reinforced concrete frames in zones of potential local collapse due to failure of a vertical element, such as a column or pylon. The paper provides initial assumptions about the mechanisms of secondary failure propagation in multi-storey reinforced concrete building frames, depending on the initial local collapse scenario. Based on these assumptions, the paper formulates force and deformation criteria for the stress-strain state stages of reinforced concrete building frames in the zone of potential local collapse. Using energy balance conditions, simplified relations were developed to estimate the ultimate static load for compressive arch and catenary actions of floor slab structures. The calculated force and deformation criteria values were compared with the experimental values. These comparisons demonstrate that the accuracy of the proposed relations is acceptable for engineering calculations.

Keywords: compressive arch action, catenary action, reinforced concrete frame, structure, progressive collapse, robustness

Conflicts of interest. The author declares that there is no conflict of interest.

Funding. The research was carried out with the support of the grant of the Russian Science Foundation No. 24-49-10010, <https://rscf.ru/project/24-49-10010/>

For citation: Savin S.Yu. Stages of resistance of reinforced concrete frames in accidental design situation. *Structural Mechanics of Engineering Constructions and Buildings*. 2025;21(4):321–333. <http://doi.org/10.22363/1815-5235-2025-21-4-321-333>

Стадии напряженно-деформированного состояния железобетонных каркасов зданий в аварийной расчетной ситуации

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Поступила в редакцию: 14 мая 2025 г.

Доработана: 21 июля 2025 г.

Принята к публикации: 30 июля 2025 г.

Аннотация. Исследованы стадии напряженно-деформированного состояния железобетонных рамных конструкций каркасов зданий в зоне возможного локального разрушения при потере несущей способности одного из вертикальных элементов (колонны, пилоны). Приведены исходные предпосылки о механизмах распространения вторичных разрушений в желе-

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зобетонных многоэтажных каркасах зданий в зависимости от сценария начального локального разрушения. На основе этого сформулированы силовые и деформационные критерии стадий напряженно-деформированного состояния железобетонных рамных конструкций каркасов зданий в зоне возможного локального разрушения. С использованием положений энергетического баланса построены упрощенные зависимости для оценки предельной статической нагрузки для арочной и цепной стадий напряженно-деформированного состояния конструкций перекрытий. Приведены результаты сопоставления расчетных значений силовых и деформационных критериев с экспериментальными данными. Продемонстрирована приемлемая для инженерных расчетов точность предложенных в работе зависимостей.

Ключевые слова: арочная стадия, цепная стадия, железобетонная рама, каркас, прогрессирующее обрушение, живучесть

Заявление о конфликте интересов. Автор заявляет об отсутствии конфликта интересов.

Финансирование. Исследование выполнено за счет гранта Российского научного фонда № 24-49-10010, <https://rscf.ru/project/24-49-10010/>

Для цитирования: Савин С.Ю. Стадии напряженно-деформированного состояния железобетонных каркасов зданий в аварийной расчетной ситуации // Строительная механика инженерных конструкций и сооружений. 2025. Т. 21. № 4. С. 321–333. <http://doi.org/10.22363/1815-5235-2025-21-4-321-333>

1. Introduction

Buildings and structures are exposed to physical impact of various nature and intensity during their service life, including accidental situations, i.e., situations not covered by the normal operating conditions of construction objects. Accidental impact occurs much less frequently than climatic or functional loads. However, the failure of one or more load-bearing elements as a result of an accidental impact can cause *damage* that is disproportionate to the initial impact, such as the *collapse* of part of the frame or the entire structure.

One special aspect of designing reinforced concrete building frames taking into account the risk of an accidental situation (failure of a load-bearing element or connection) is the consideration of the arch and catenary action stages of slab structure deformation as allowable states [1–3]. The consideration of such stages of resistance allows to reduce the costs of ensuring mechanical safety, which can be generally limited to the installation of a system of additional restraints and local frame strengthening. In [4], experimental data on the deformation of fragments of reinforced concrete building structures in the form of plane and spatial frames are presented, demonstrating significant excess bearing capacity when resisting according to the catenary or membrane mechanisms. At the same time, [5–7] show that when transitioning to the catenary mechanism of resistance, the corner columns and columns around the perimeter of the building become vulnerable to failure.

Studies [1; 5] have provided experimental data on the boundaries of the deformation stages in damaged frames of building structures. Based on experimentally proven assumptions, [8–11] provide analytical dependencies for evaluating the resistance of structures according to the arch and catenary mechanisms. However, it should be noted that such studies have a number of limitations. For example, experimental studies [9; 12] consider two-span beam structures over local collapse, restrained from horizontal displacements, which does not allow to take into account the influence of the bearing capacity and deformability of vertical elements (columns, pylons) on the mechanisms of secondary failure propagation in a structural system damaged as a result of an accidental impact. In order to overcome this limitation, studies [1; 5] consider single-storey fragments of reinforced concrete building frames. However, these results cannot be directly extrapolated to multi-storey frame structures, in which, in most cases, it is impossible to consider the behavior of isolated elements, since their load-bearing capacity and deformability depend on the topology of the structural system and kinematically possible failure mechanisms. Another limitation is the simplified application of concentrated loads and the approximation of the curvature diagram along the length of the two-span girder above a distant vertical element.

Thus, despite the presence of a significant number of publications with the results of experimental and numerical studies on the issues of arch and catenary actions of reinforced concrete frame structures, there are no formalized force and deformation criteria for transitioning into these stages, taking into account the

interaction of elements in the frames of multi-storey buildings, as well as the bearing capacity and deformability of vertical elements, the presence of a system of additional restraints, various scenarios of initial local collapse. In this regard, the purpose of this study was to formulate and substantiate the strength and deformation criteria for the stages of resistance of reinforced concrete multi-storey building frames in an accidental situation in case of failure of one of the elements of the structural system.

2. Models and Methods

2.1. Initial Assumptions

To analyze the stages of resistance of reinforced concrete multi-storey building frames in an accidental situation, a zone of possible local collapse is identified [13] — an area bounded by one span in each direction from the element being removed, taking into account the superimposed restraints simplistically modeling the interaction with the rest of the structural system. Limiting the analyzed area allows to obtain an analytical solution to the problem, while it is consistent with the results of experimental and numerical studies, as well as the requirements of design standards for limiting secondary failure.

Depending on the scenario (location) of the initial local failure, various mechanisms of secondary failure are possible in the structural system of a building damaged by accidental impact. Based on the analysis of the cases of collapse of buildings, structures and their parts as a result of failure of a bearing element, as well as data from experimental and numerical studies, the classification of secondary failure mechanisms was performed (Table 1).

Table 1

Characteristic mechanisms of secondary failure in damaged reinforced concrete multi-storey building frames

Structural elements of the building frame	Characteristics of the failure mechanism	Scenarios of initial local failure		
		Removal of a corner column	Removal of the column of the edge or middle row, in which the zone of possible local failure includes the contour of the building	Removal of an edge or middle row column, in which the area of possible local failure does not include the contour of the building
Structural elements of floors (beams, crossbars)	Fracture in normal cross-sections during the bending stage of operation	Fedorova N.V., Korenkov P.A. [14], Iliushchenko T.A. et al. [15], Adam J. et al. [4]	No data	No data
	Fracture along normal cross-sections at the stage of compressive arch action	No data	Fedorova N.V., Korenkov P.A. [14], Iliushchenko T.A. et al. [15]	Pham A.T. et al. [1; 5]
	Fracture from rebar breakage at the stage of catenary action	No data	Pham A.T. et al. [1; 5], Fedorova N.V. et al. [16; 17]	Pham A.T. et al. [1; 5]
	Fracture along inclined and spatial sections during combined resistance of elements	Kolchunov V.I., Moskovtseva V.S. [18]	Kolchunov V.I., Moskovtseva V.S. [18]	No data
Vertical elements (columns, pylons)	Fracture along normal cross-sections	Savin S.Yu. et al. [19]	Pham A.T. et al. [1; 5]	Pham A.T. et al. [1; 5]
	Fracture along the supporting inclined sections	Kolchunov V.I. et al. [6]	Kolchunov V.I. et al. [6]	No data
	Fracture along inclined sections in the anchoring zone of floor slab reinforcement in tension	No data	Choi H., Kim J. [20], Kolchunov V.I., Moskovtseva V.S. [18]	No data

Source: made by S.Yu. Savin

Three characteristic scenarios of initial local failure are identified here, depending on the degree of influence on the possibility of achieving specific stages of resistance of damaged building frames (arch, catenary). Thus, when a corner column (pylon) is removed in case of accident, there is no thrust in the slab structure above the local failure due to the absence of horizontal restraints, which makes it impossible to develop compressive arch action and subsequently transition to resistance according to the catenary mechanism. When removing the column of the edge or middle row, in which the zone of possible local failure includes the contour of the building, the deformability of the vertical elements of the edge rows has a significant impact on the possibility and parameters of the arch and catenary resistance mechanisms. In addition, in this scenario of initial failure, vertical elements along the contour of the building are vulnerable to fracture along normal or inclined sections. The third scenario for removing the column of the edge or middle row, in which the zone of possible local failure does not include the contour of the building, almost always allows for the compressive arch action of the slabs, and with appropriate reinforcement and anchoring parameters, the transition to the catenary action after the fracture of concrete in the support zone and in the middle of the span. At the same time, there is a decrease in the risk of failure of the vertical elements, which in this case have excess bearing capacity according to the criteria of the accidental limit state.

2.2. Force and Deformation Criteria for Multi-Level Resistance

A reinforced concrete frame structure in the area of possible local failure is considered (Figure 1). At this step, the cases of collapse associated with failure of elements under combined resistance, such as combined action of bending moment and torque, shear forces, which require independent detailed analysis, are not considered.

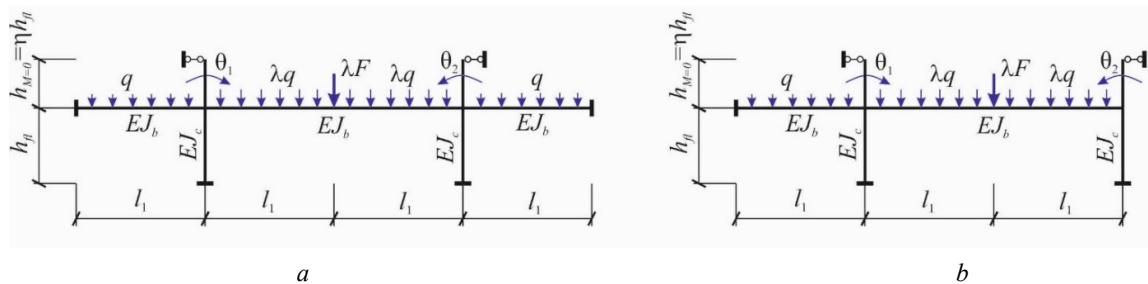


Figure 1. Reinforced concrete frame structure in the zone of potential local collapse:
a — fully braced; b — partially braced

Source: made by S.Yu. Savin

Taking into account possible mechanisms of secondary failure discussed in Table 1, multi-level resistance of the structural system of a building can be represented as a piecewise linear diagram of the relationship between the generalized load P and the displacement of the structural node above the local failure, as shown in Figure 2. The characteristic points of this diagram represent the force and deformation criteria for changing the stages of resistance and exhaustion of the bearing capacity of the analyzed area of possible local failure.

Then, from the energy balance condition [21–23], the maximum static load that can be resisted by the load-bearing system in the arch and catenary action stages of resistance can be determined from the following expressions:

- for the compressive arch action stage:

$$P_{s,u,CAA} = \frac{1}{2z_3} [P_1 z_1 + (P_1 + P_2)(z_2 - z_1) + (P_2 + P_3)(z_3 - z_2)], \quad (1)$$

- for the catenary action stage:

$$P_{s,u,CA} = \frac{1}{2z_4} \left[P_1 z_1 + (P_1 + P_2)(z_2 - z_1) + (P_2 + P_3)(z_3 - z_2) + (P_3 + P_4)(z_4 - z_3) \right]. \quad (2)$$

Upon removal of a corner column $P_2 = P_3 = P_4 = 0$.

The characteristic points of the diagram in Figure 2 are discussed further using assumptions based on the results of experimental studies and kinematic analysis.

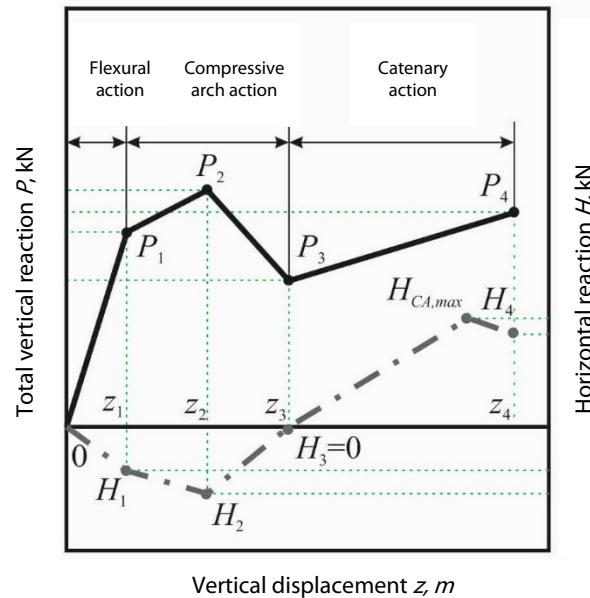


Figure 2. Schematic deformation diagram of the reinforced concrete frame in the zone of potential local collapse

Source: made by S.Yu. Savin

2.3. Flexural Stage of Resistance of Slabs

The ultimate load and deflection during the flexural stage of resistance are determined from the conditions:

$$P_1 = P(M_{ult}), \quad z_1 = \int_0^{l_1} \left(\frac{1}{r} \right) \left(\frac{l_1}{2} - x \right) dx. \quad (3)$$

Here, according to the design code approach¹, in the margin of stiffness, the curvature can be determined from a conditional elastic calculation:

$$\left(\frac{1}{r} \right) = \frac{\varepsilon_{sm} - \varepsilon_{bm}}{h_0} = \frac{(M - 0.8M_{crc})}{E_s A_s z_s h_0} + \frac{0.9M}{E_{b,red} A_b z_s h_0}, \quad (4)$$

where z_s , A_b is the moment arm of the inner couple of forces and the area of compressed concrete, determined from the calculation of the ultimate forces for the flexural stage of resistance of the element.

¹ SP 63.13330.2018 Concrete and reinforced concrete structures. General provisions. <https://docs.cntd.ru/document/554403082>

The ultimate bending moment in the normal section of the element during the transition to the compressive arch action stage of resistance in terms of the generalized load P is expressed as:

$$M_{ult} = \min(|M_{sup,ult}|, |M_{spn,ult}|), \quad (5)$$

where

$$\begin{aligned} -M_{sup,ult} &= \frac{F_{ult}l}{8} + \frac{q_{ult}l^2}{12} = P_1 \frac{l_1}{4} \frac{1 + \frac{2}{3}\beta}{1 + \beta}, \\ M_{spn,ult} &= \frac{F_{ult}l}{8} + \frac{q_{ult}l^2}{24} = P_1 \frac{l_1}{4} \frac{1 + \frac{1}{3}\beta}{1 + \beta}. \end{aligned} \quad (6)$$

The following notation is adopted in (6):

F is the concentrated force applied to the node of a two-span girder over local failure in the structural system of a building;

q is the uniformly distributed load on the two-span girder.

The relationship between loads F , q and the generalized load P :

$$P = (F + ql) = F(1 + \beta) = ql \frac{1 + \beta}{\beta}, \quad \beta = \frac{ql}{F}, \quad l = 2l_1. \quad (7)$$

In quasi-static analysis, simultaneous consideration of loads F and q , which is inherent in the pull-down and push-down approaches, respectively, allows to partially eliminate the disadvantages of these approaches, discussed in more detail in [24].

2.4. Compressive Arch Action Stage of Resistance of Slabs

The maximum load carried at the compressive arch action stage in a two-span girder with equal spans and the corresponding deflection are determined taking into account the physical constraints from the following conditions:

$$\begin{aligned} \frac{\partial M_{ult}}{\partial z} &= 0 \Rightarrow z_2 = z_{ult}; \\ P_2 &= P_{ult} = \frac{4M_{ult}}{l_1}, \end{aligned} \quad (8)$$

where l is the length of one span (according to the initial model) in the two-span girder; $M_{ult} = M_{ult}(z)$ is the ultimate bending moment resisted by the support section of the girder at the compressive arch action stage, calculated as for an eccentrically loaded element, taking into account the acting thrust:

$$N = \Delta L \frac{C_1 C_2}{C_1 + C_2} = \left[(l_1^2 + 2(h_0 - a')z - z^2)^{\frac{1}{2}} - l_1 \right] \frac{C_1 C_2}{C_1 + C_2}. \quad (9)$$

Here, C_1 , C_2 are the reactions due to unit horizontal displacement of the left and right support sections of the arbitrary arch.

At the same time, the following conditions must be satisfied to ensure the bearing capacity of vertical elements at the flexural and compressive arch action stages of the slab structures:

$$\begin{cases} M_c = 4\mu i_b \theta_2 \leq M_{c,ult}(N_c); \\ Q_{cn} = \frac{6\mu i_b \theta_2}{h_{fl}} \leq Q_{cn,ult}; \\ Q_{cv} = N_b \leq Q_{cv,ult}. \end{cases} \quad (10)$$

The following notation is adopted in (10):

N_b is the thrust force as a result of girder deformation, determined by formula (9); M_c is the bending moment in the upper support section of the lower storey column; Q_{cn} is the shear force in the upper support section of the lower storey column; Q_{cv} is the shear force in the lower support section of the upper storey column.

The angle of deflection of the corresponding node of the girder to column connection θ_i is determined by the following formula for a fully braced frame (see Figure 1, *a*):

$$\begin{aligned} \theta &= \frac{\eta q l_1^2 (6\lambda - \beta + 10\beta\lambda)}{12\beta i_b (5\eta + 3\mu + 4\eta\mu)}, \\ i_b &= \frac{EI_b}{l_1}; \quad \mu = \frac{EI_b}{l_1} \cdot \frac{h_{fl}}{EI_c}; \quad \eta = \frac{h_{M=0}}{h_{fl}}. \end{aligned} \quad (11)$$

For a partially braced frame (see Figure 1, *b*):

$$\begin{aligned} \theta_1 &= \frac{\eta q l_1^2 (18\eta\lambda - 3\beta\mu - 2\beta\eta + 18\lambda\mu + 30\beta\eta\lambda - 4\beta\eta\mu + 30\beta\lambda\mu + 24\eta\lambda\mu + 40\beta\eta\lambda\mu)}{12\beta i_b (16\eta^2\mu^2 + 32\eta^2\mu + 11\eta^2 + 24\eta\mu^2 + 9\mu^2)}, \\ \theta_2 &= \frac{\eta q l_1^2 (42\eta\lambda - \beta\eta + 18\lambda\mu + 70\beta\eta\lambda + 30\beta\lambda\mu + 24\eta\lambda\mu + 40\beta\eta\lambda\mu)}{12\beta i_b (16\eta^2\mu^2 + 32\eta^2\mu + 11\eta^2 + 24\eta\mu^2 + 9\mu^2)}. \end{aligned} \quad (12)$$

At the moment of transition to the catenary action stage, by assuming the first-order approximation $z_3 = (h_0 - a')$, the following is obtained:

$$P_3 = (h_0 - a') \left(\frac{\sum_{i=1}^n \sigma_{si} A_{si}}{l_1 - \Delta_{x1}} + \frac{\sum_{j=1}^m \sigma_{sj} A_{sj}}{l_2 - \Delta_{x2}} \right). \quad (13)$$

Here, σ_{s1} , σ_{s2} can be determined according to²:

$$\sigma_{si, \max} = 2.39 \sqrt{\frac{\tau_{b, \max} E_s}{d_s} \Delta L_{CAA}^{1.4}} \leq R_s. \quad (14)$$

Where $\tau_{b, \max}$ is the maximum bond stress, corresponding to elastic resistance of the reinforcement; d_s is the diameter of longitudinal rebar; ΔL_{CAA} is the maximum elongation of the girder edge upon transitioning from the compressive arch action to the catenary action, determined by the formula:

$$\Delta L_{CAA} = \sqrt{l_1^2 + (h_0 - a')^2} - l_1. \quad (15)$$

² FIB Model Code 2010. CEB and FIP, 2011.

For longitudinal rebars located at distance z_{si} from the center of gravity of the compressed longitudinal reinforcement in the section, ΔL_{si} is obtained as:

$$\Delta L_{si} = \frac{\Delta L_{CAA}}{h_0 - a'} z_{si}. \quad (16)$$

By substituting ΔL_{si} instead of ΔL_{CAA} into formula (14), stress σ_{si} can be determined in all longitudinal rebars in the section.

If $\sigma_s > R_s$ as a result of evaluating formula (14), the stress in the reinforcement in tension is determined by formula

$$\sigma_{si, \max} = R_s + \frac{R_{su} - R_s}{\varepsilon_{su}} (\varepsilon_{s, \max} - \varepsilon_{s2}),$$

$$\varepsilon_{s, \max} = \left\{ \frac{8\tau_{bm, pl} \varepsilon_{su}}{d_s (R_{su} - R_s)} \left[\Delta L_{CAA} - 0.288 \left(\frac{d_s R_s^2}{\tau_{b, \max} E_s} \right)^{0.714} - 2d_s \frac{R_s}{E_s} \right] + \varepsilon_{s2}^2 \right\}^{0.5}, \quad (17)$$

where $\tau_{bm, pl} = 0.27\tau_{b, \max}$ is the average bond stress in the plastic region of the rebar; ε_{su} is the ultimate strain of the longitudinal rebar.

The adjusted value of deflection upon the transition from the compressive arch action stage to the catenary action is determined from expression

$$z_3 = z_{30} + \theta l_1, \quad \theta = \frac{\theta_1 + \theta_2}{2}. \quad (18)$$

2.5. Catenary Action Stage of Resistance of Slabs

If the condition of $\sigma_s < R_{su}$ is satisfied for the deformed structure in the transition state (P_3, z_3), the maximum deflection z_4 at the moment of rupture of the most stretched reinforcing bars in one of the support sections at the catenary action stage of resistance can be found by assuming $\sigma_s = R_{su}$:

$$z_4 = \sqrt{(l_1 + \Delta L_{CA})^2 - (l_1 - \Delta_x)^2},$$

$$\Delta L_{CA} = \frac{R_{su} - R_s}{\tau_{bm, pl}} \frac{d_s}{4} \frac{\varepsilon_{su}}{2} + \frac{R_s}{E_s} 2d_s + 0.288 \left(\frac{d_s R_s^2}{\tau_{b, \max} E_s} \right)^{0.714}. \quad (19)$$

The ultimate load P_4 is calculated by formulas (13), substituting z_4 instead of $(h_0 - a')$. At the same time, for the normal sections of the edge columns at the point of connection with the girder in partially braced frame structures, the following condition must be satisfied:

$$M_c \eta_{\text{crit}} \leq M_{c, \text{ult}}(N), \quad \eta_{\text{crit}} = \frac{1}{1 - \frac{\sum P}{\sum P_{\text{crit}}}}. \quad (20)$$

Here, by following the approach considered in ACI 318³, the coefficient of buckling upon relative horizontal displacement of the slabs of one floor is determined taking into account the action of all the

³ ACI 318-19. Building Code Requirements for Structural Concrete. An ACI Standard. Reported by ACI Committee 318.

columns of the floor in question on one side of the local failure in the structural system, where $\sum P$, $\sum P_{crit}$ are the sums of the effective longitudinal forces and critical forces in these columns. In the case when the analyzed zone of possible local failure includes the columns of the edge row or the corner column, the signs of the sum in (20) disappear, and the formula is modified to the known form for the analysis of individual elements.

3. Results and Discussion

3.1. Comparison with Results of Studies by A.T. Pham and K.H. Tan

To verify the proposed strength and deformation criteria for the stages of resistance of reinforced concrete frame structures of buildings, a comparison was made with the results of experimental studies by A.T. Pham and K.H. Tan [1], who tested several substructures of reinforced concrete frames at different levels of gravitational load applied to the node at the midspan and the rate of its application. The general view of the frame structure, its dimensions and loading scheme are shown in Figure 3, *a*. Mechanical characteristics of the structural materials: yield strength of longitudinal reinforcement is at least 535 MPa, temporary rupture strength is at least 615 MPa; the average cylinder strength of concrete under uniaxial compression is 35 MPa. The calculation results in the form of a $P - z$ diagram are shown in Figure 3, *b*.

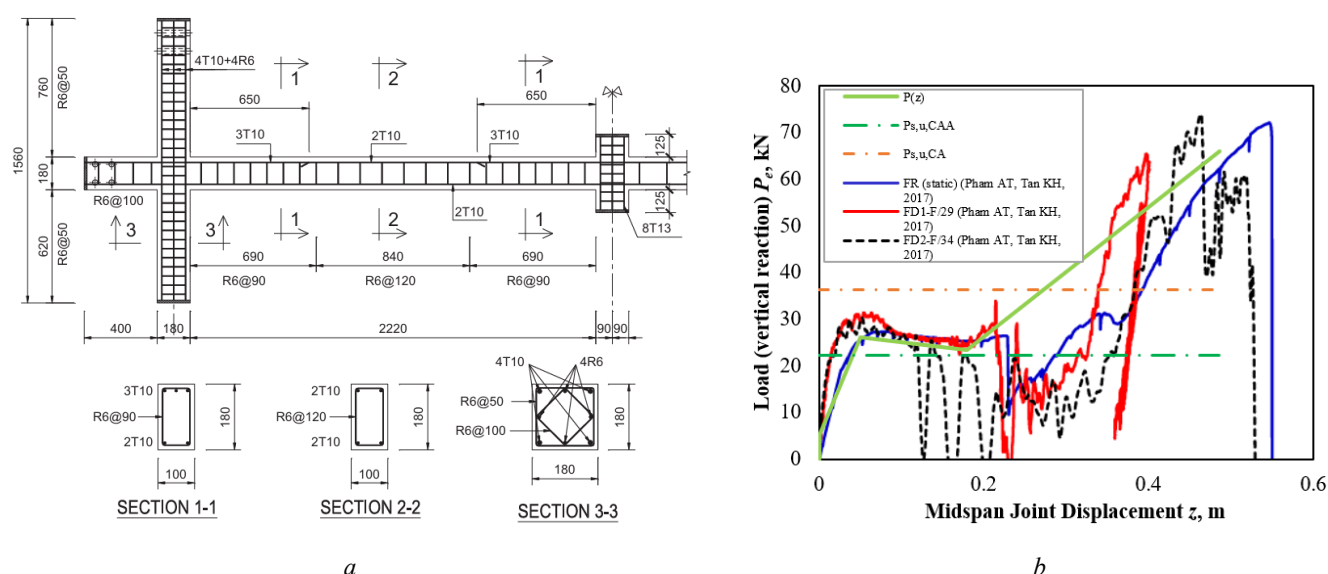


Figure 3. Comparison with experiments by Pham and Tan [1]:

a — dimensions and reinforcement layout of the RC frames;

b — calculated and experimental Load (vertical reaction) vs. Midspan Joint Displacement curves

Source: *a* — made by A.T. Pham, K.H. Tan [1]; *b* — made by S.Yu. Savin

As seen from the graphs, the curves constructed according to the relationships proposed in this study quite well reproduce the experimental diagrams at the characteristic points of transition of the resistance stages. However, due to the absence of intermediate points in the diagram at the catenary action stage, there is a more significant gap between the experimental curves and the analytical line passing through characteristic points (P_3, z_3) , (P_4, z_4) . The result of such a discrepancy may be an overestimation of the ultimate static load according to the energy balance method, which is resisted at the catenary action stage of the slabs above local failure. In this regard, an approximation by a tangent function can be used to refine the region of the diagram corresponding to the catenary mechanism of deformation.

3.2. Comparison with Results of Experimental Studies of Multi-Storey Frames

To verify the initial assumptions about the strength parameters of the elements of a reinforced concrete structural system in the zone of possible local collapse as a result of an accident at different stages, taking into account the design features, initial stress-strain state due to static operational load and specific resistance mechanisms, the results of experimental studies of reinforced concrete frame structures subjected to two-step static-dynamic loading from an accident event are analyzed.

A semi-precast reinforced concrete frame is considered, the test results of which are given in [6]. The frame structure is shown in Figure 4, *a*.

The height of the precast section made of grade B35 concrete is $h_2 = 70$ (mm); the height of the cast-in-situ section made of grade B50 concrete is $h_1 = 30$ (mm); the distance between the longitudinal axes of the two components is $\omega = 50$ mm. The longitudinal reinforcement of compound girders is made of grade B500 reinforcement with a diameter of 4 (mm). The transverse reinforcement of the girders is made of wire reinforcement with a diameter of 2 mm and characteristics similar to grade A240 reinforcement.

The experimental frames were loaded with a static load transmitted through a system of arms. The load was applied in the form of concentrated forces in the thirds of the girder spans. The values of the concentrated loads applied to the frame girders were: $P_1 = 3.2$ kN, $P_2 = 2.64$ kN, $P_3 = 2.04$ kN. Here, the highest load value corresponded to the upper floor, and the lowest to the lower floor of the frame.

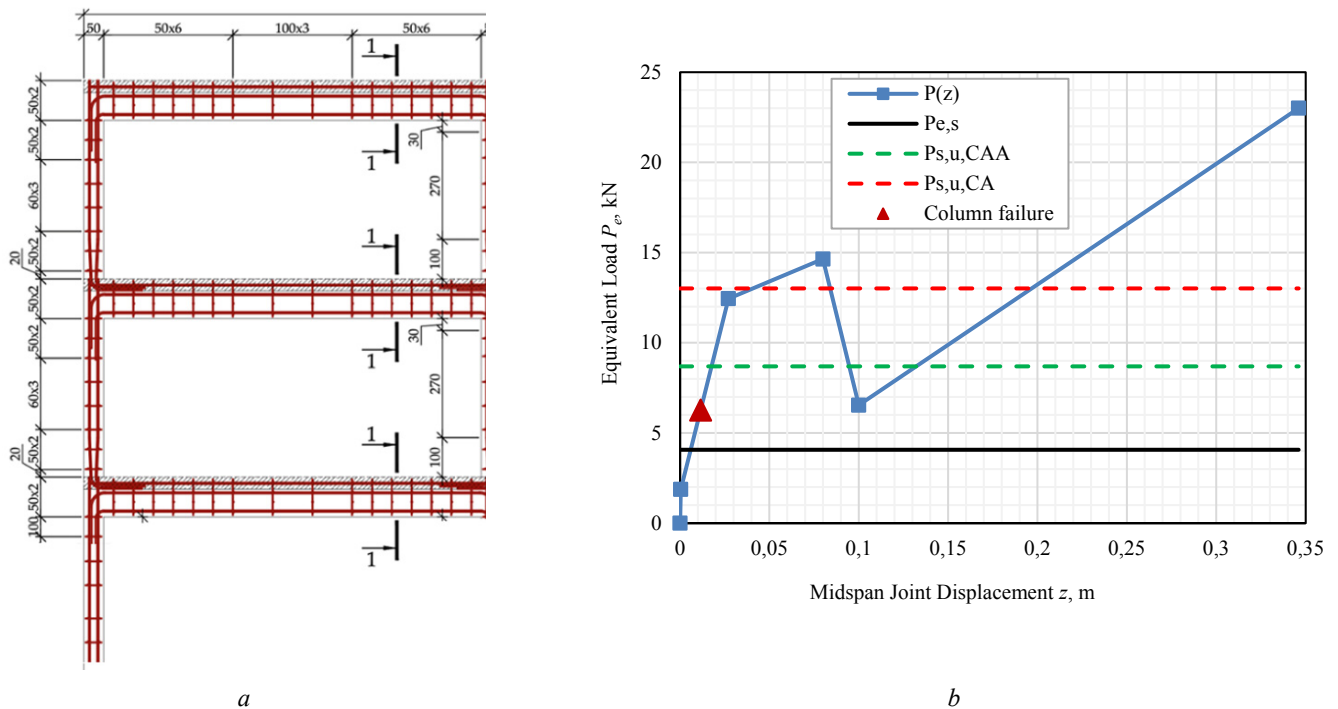


Figure 4. Comparison with experiments by [6]:

a — dimensions and reinforcement layout of the RC frames;

b — calculated and experimental Load (vertical reaction) vs. Midspan Joint Displacement curves

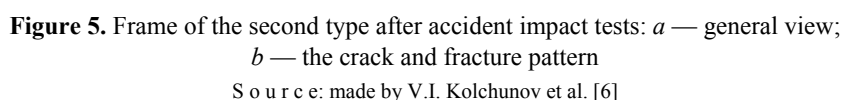
Source: made by S.Yu. Savin

According to the methodology, the physical model of the reinforced concrete frame is tested in two steps. At the first step, gravitational load is applied incrementally (in 10 increments) to the nodes of the experimental model of the structure using a system of arms.

At the second step, the intermediate support, imitating the column of the middle row, is instantly removed. When modeling the accidental impact on the physical model, the column turns into an

Figure 4, *b* shows the calculated load-displacement diagrams of an equivalent model of a two-span

The general view of the deformed state of the frame as a result of an accidental impact is shown in



S o u r c e: made by V.I. Kolchunov et al. [6]

1. The force and deformation criteria of the stages of resistance of reinforced concrete frame structures of buildings in the zone of possible local collapse due to sudden failure of a vertical element (column, pylon) are formulated.

2. Using the obtained criteria and the energy balance conditions, simplified relationships were constructed to estimate the ultimate static load for the arch and catenary action stages of resistance of the slab structures.

3. The comparison of the calculation results based on the relationships obtained in the study for the force and deformation criteria of the resistance stages with the data of experimental studies confirmed their reliability, and also demonstrated the need to take into account the joint deformation of slab structures and vertical load-bearing elements for correct evaluation of the deformed state of building frames in case of an accident.

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