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METHOD FOR ANALYZING THE PROBABILITY OF STABILITY FAILURE OF A RC FRAME IN AN ACCIDENT SITUATION

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Abstract: Despite the general principles on reliability for structures have been stated in the international standard ISO 2394, the implementation of these principles in the design and construction practice faces with difficulties. One of the current problems is probabilistic robustness checking for buildings and structures. This study focuses on the development of a probabilistic model to calculate the risk of failure of a reinforced concrete frame due to stability failure of its elements in a scenario of sudden removal of a structural component. The critical forces for the eccentrically compressed structural components of the frame are determined using the displacement method, considering the two-stage static-dynamic loading regime due to the sudden column collapse. The study takes into account the change in stiffness of frame elements due to cracking during loading. In addition, it proposes the probabilistic equigradient-based model for the failure of the frame elements after the sudden removal of a column. The mathematical formulation of this model is that if some value is a critical force associated with some parameters satisfying a given condition, then the extremum of that value can be found using an auxiliary Lagrangian function. To illustrate the application of the model, the paper provides an example of the calculation of the stability failure probability of a reinforced concrete frame structure under the considered loading regime. The calculated critical force values were compared with the experimental data for the reinforced concrete frame tested for sudden column removal.

Keywords: probability, stability, disproportionate collapse, reinforced concrete, frame, accidental action

МЕТОДИКА РАСЧЁТА ВЕРОЯТНОСТИ ПОТЕРИ УСТОЙЧИВОСТИ ЖЕЛЕЗОБЕТОННОЙ РАМЫ ПРИ ЕЕ СТРУКТУРНОЙ ПЕРЕСТРОЙКЕ

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Аннотация: Несмотря на то, что общие принципы надежности конструкций изложены в международном стандарте ISO 2394, реализация этих принципов в практике проектирования и строительства сталкивается с трудностями. Одной из актуальных проблем является вероятностная оценка живучести зданий и сооружений. В настоящей статье предложена вероятностная модель расчета риска отказа железобетонной рамы из-за потери устойчивости ее элементов, вызванной особым воздействием – удалением одной из колонн. Определение критических сил во внецентренно сжатых колоннах рамы при рассматриваемом двухэтапном режиме нагружения рамы статической нагрузкой и динамическом догружении от внезапного особого воздействия выполнено методом перемещений. При этом учтено изменение жесткостей элементов рамы вследствие трещинообразования при нагружении конструкции. Вероятностная модель отказа элементов рамы после внезапного удаления одной из колонн построена с использованием принципа эквиградиентности, математическая формулировка которого состоит в том, что если некоторая величина – критическая сила связана с некоторыми параметрами, удовлетворяющими заданному условию, то экстремум этой величины может быть найден с помощью вспомогательной функции Лагранжа. Приведен пример определения вероятности потери устойчивости конструкции железобетонной рамы при рассмотренном режиме нагружения. Выполнено сопоставление расчетных и опытных значений критических сил при потере устойчивости рамы после внезапного удаления колонны.

Ключевые слова: вероятность, устойчивость, прогрессирующее обрушение, железобетон, каркас, особое воздействие

1. INTRODUCTION

As a result of constantly increasing natural and anthropogenic disasters, as well as significant deterioration of assets, the problem of structural safety of buildings and structures becomes one of the most important in the scientific research direction for structural engineers of many countries. A significant number of experimental and theoretical studies have been carried out to evaluate the structural robustness and effectiveness of solutions for the protection of structural systems against disproportionate collapse in case of accidental actions caused by the exhaustion of the load-bearing capacity of any structural component [1-3]. As a rule, the published researches consider the ultimate strain of the material as a criterion for the exhaustion of the bearing capacity at an accidental design action. The dynamic hardening of materials as a function of strain rate can also be considered according to diagrams and analytical expressions [1,2,3].

Current standards [4-7] require stability checks of slender components of reinforced concrete structures, where the deformation and failure are significantly influenced by geometric nonlinearity. Failure of such components may occur at forces less than the strength of the sections. In ACI 318 [4], a 5% reduction of the capacity of the component by the axial force is set as the ultimate limit state criterion in accidental design situation. EN [5-6] allows a slightly higher, up to 10%, reduction of the capacity of the component with respect to axial force, considering the second order nonlinear effects (buckling). SP63 and SP385 [8,9] also specify the calculation of reinforced concrete elements taking into account physical, geometrical and structural nonlinearity. These standards specify the strength of cross-sections and ultimate strains as capacity criteria for the structural components. In this case, the second-order effects are accounted through the buckling factor applied to the calculated value of the first-order bending moment. However, it should be noted, that the target values of the reliability index for stability failure and material failure of structural components are different. The second problem is the correct consideration of the stiffness of structures, since it significantly affects the failure load of slender components. Meanwhile, stiffness expressions in standards are semi-empirical and based on limited experimental data.

Thus, in addition to the robustness checks for the strength of structural components at accidental actions [10-15], the robustness check for stability failure criteria should be performed for frames with slender columns [16,17]. In the studies of buckling of elastoplastic structural components [17,18] and in the works [20,21] on buckling of reinforced concrete elements, the extremum of force – displacement diagram (null tangent stiffness) was adopted as the stability criterion. The work [20] points out that the calculation of reinforced concrete eccentrically compressed elements should consider both of physical and geometric nonlinearity. The authors provided specific recommendations for determining the stiffnesses of such elements in the presence of cracks in them. However, such studies have usually been limited to structural elements and do not take into account the phenomenon for framed structures, where structural components may experience unloading at buckling due to the reaction of the frame. At the same time, the supporting effect of the frame is of great importance as it provides load transfer in the hazardous event.

In this context, the energy-based criterion proposed by A.V. Aleksandrov and V.I. Travush [19] is promising to identify the most dangerous components of the structural system from the point of view of stability failure. According to this criterion, the shear force and bending moment in the end section of the structural element, which loses stability, perform the largest negative work. In a similar formulation, the problem of stability of reinforced concrete frames was solved in [22,23], where the stiffness parameters and the free length of the columns in the structures were varied.

There are relatively few studies that have applied the stability failure criterion to investigate the structural robustness of frames. For example, the studies [16,24] provide design models, stability failure criteria and algorithms for the stability analysis of reinforced concrete frames in column removal scenarios. In the study [25], an analytical model of an eccentrically compressed supper element was proposed to investigate the stability failure of W-shaped steel columns under accidental impact. In [26], the results of an experimental study of different types of stability loss of cold-deformed composite steel columns are presented. During the tests, all experimental specimens with cross-sectional imperfections exhausted the load-bearing capacity due to buckling.

Currently, there are also very few publications on the probabilistic robustness analysis of structural systems. We can mention the works of G.A. Geniev [27,28], who solved the reliability problem of reinforced concrete multi-span beams and obtained the minimum probability of their failure under an accidental action for given initial conditions.

As paper [29] noted, the reliability analysis requires probabilistic definitions of loads and strength of a structure. However, the standards and publications in this field do not contain such clear definitions for accidental design situations, which should be considered at robustness check. There is a lack of target reliability (or risk) values, partial factors or global resistance factor calibrated for conditions of accidental design situation. The fundamental postulate adopted in establishing acceptable (tolerable) risk is that the risks associated with the failure of structural systems of buildings designed and erected in accordance with current best practice are considered sufficiently small and acceptable to society. On the other hand, current best practices are reflected in standards [30,31] and structural design codes [32]. The correct application of these practices allows to create the safe and reliable structural systems. Taking into account the fact that the approaches for consequence assessment given in [29] have high level uncertainties, the modeling of collapse consequences in most cases is simplified by considering only human casualties [10,34]. Thus, in accordance with the model [34], the expected number of potential casualties N_j is calculated depending on the expected collapse area.

This study aimed at development of a probabilistic model for calculating the stability failure of a reinforced concrete frame at sudden removal of a column. Critical parameters of the frame structure are determined using an energy-based approach; meanwhile the probability of stability failure of the frame structural components under such an accidental action is determined using the equigradient method.

2. MODELS AND METHODS

The study examines probability of stability failure of the two-span scaled reinforced con-crete frame previously tested for sudden loss of the column in the paper [39] The frame is loaded with concentrated forces eccentrically applied at the upper joints of the structure. The columns of the second floor of the frame are of relatively large slenderness ratio that makes it vulnerable to stability failure. The frame dimensions, reinforcement scheme, and cross-sections are shown in Figure 1

The loading of the test frame involved two stages as shown in figure 2. At the first stage, constant static concentrated forces P_1 = 4.7 kN, P_2 = 26.2 kN, P_3 = 19.5 kN were applied with an eccentricity e = 10 mm to the upper joints of the frame that corresponds to normal operation conditions. The second stage involved an accidental action due to the sudden removal of the outermost column of the first floor simulating initial localized failure.

The frame has pinned restraints at the joints as shown in Figure 2 that corresponds to the experimental specimen. The stiffnesses and lengths of the beams and columns of the frame are also assumed to be the same as in the specimen tested in the study [37] in order to evaluate the developed approach comparing the calculation results with the experimental data.

The probability P_i of the stability failure of the frame components at the scenario of the sudden

removal of the outermost column of the first floor was evaluated in the following way.
Firstly, the critical force parameters have been determined for each column of the primary design scheme using the displacement method. The design scheme with accepted unknown displacements (rotation angles) is shown in Figure 3.

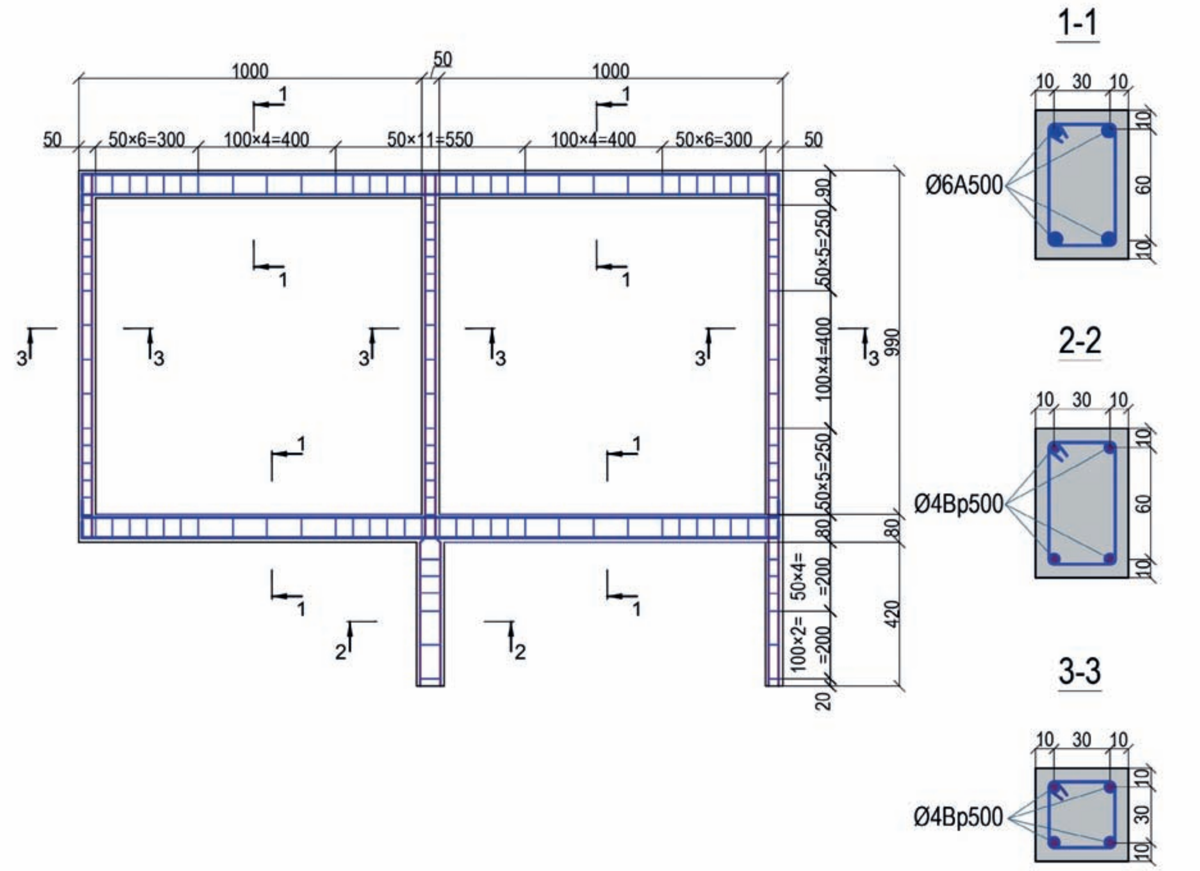


Figure 1. Design of the reinforced concrete frame and its reinforcement scheme

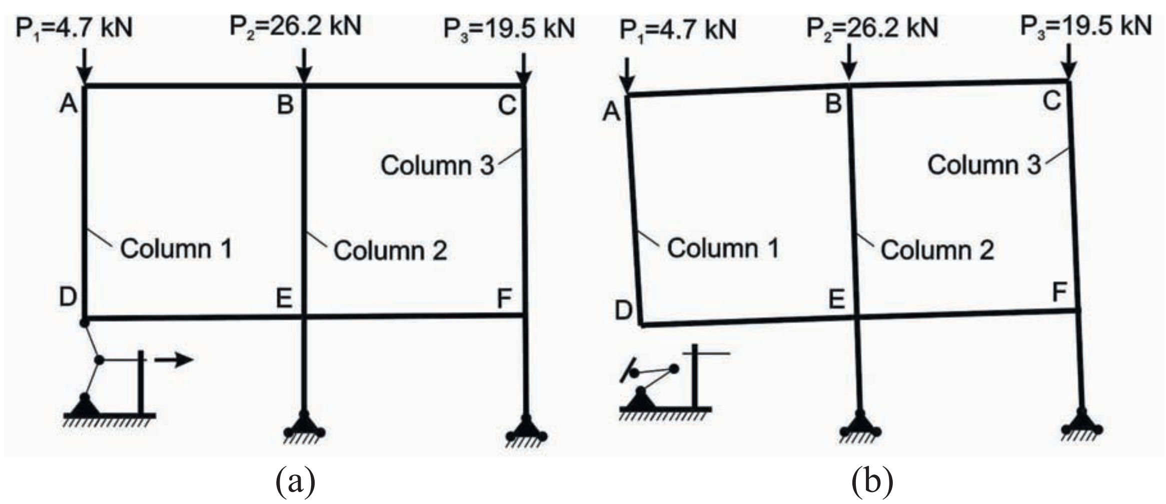


Figure 2. Design scheme of the reinforced concrete frame at normal operation (a) and at sudden loss of the column (b)

According to the accepted geometrical dimensions of the frame and reinforcement scheme presented in Figure 1, the reduced linear stiffnesses of the structural components $i_n = B_{red,n}/l_n$ have been calculated to obtain the expressions for the force parameters $\nu_n = l\cdot(P_i/B_{red,i})^{0.5}$, where $B_{red,n}$ and l_n are the cross-section stiffness and component length (or height), P_i is the axial force acting in the i^{th} structural component.

During two-stage loading of the reinforced concrete frame, the stiffnesses of the column cross-sections change along the height due to cracking. As it is known, the accuracy of calculation of the stiffnesses significantly effect on the values of critical forces for compressed-bent components. Therefore, to achieve the correct result, the study employs a modified version of the stiffness calculation model, which accounts for deformation effect of F. Thomas – Vl.I. Kolchunov [35] at determination of the mean strains in steel rebars in the blocks between cracks. According to this model, the stiffness of an eccentrically compressed element with cracks is determined from the expression:

$$B_{red,i} = \frac{E_s \cdot A_s \cdot z \cdot h_0}{1.25 + \dfrac{\psi_b \cdot \mu \cdot \alpha}{\nu \cdot \xi}}, \tag{1}$$

where E_s, A_s are the elasticity modulus and cross-sectional area of tensile steel rebars; z is the leverage of the internal force pair in concrete and reinforcement; h_0 is the effective cross-section depth of the structural component; ξ is the relative compressive depth of a component cross-section; ν is the ratio of the secant modulus of concrete to the initial one; ψ_b is the factor accounting for irregularity of tensile concrete strains in the block between cracks; $\mu\cdot\alpha$ is the reinforcement ratio reduced with regard to concrete.

The design scheme of the frame according to the displacement method is shown in Figure 3. The stability equations for the frame are adopted according to the displacement method. The unknown parameters of these equations are the rotation angles of the frame joints Z_1, Z_2, Z_3, Z_4, Z_5, Z_6, the horizontal floor displacements Z_7, Z_8, and vertical displacement Z_9 of the joint above the column to be removed for modelling accidental action. The vertical displacement Z_9 is assumed to be zero before accidental action appears.

Then, the joint reactions r_{in} at unit displacements (rotations) should be obtained to construct the matrix of coefficients r for unknown displacements (rotations) matrix Z:

$$r \times Z = 0, \tag{2}$$

where

$$r = \begin{pmatrix} r_{11} & \dots & r_{1i} \\ \dots & \dots & \dots \\ r_{i1} & \dots & r_{ii} \end{pmatrix}. \tag{3}$$

In the general case of a robustness check of a structure, unknown vertical displacements are assigned at all axes where a hypothetical initial localized failure of a column is possible. Thus, the matrix of reactions of unit displacements (rotations) retains its form for any initial local failure scenario, which is modeled by equating the stiffness of the removed element to zero.

By assuming that the determinant of the matrix r equals zero, the solution of the characteristic equation in the Maple software gives the critical values of the force parameters ν_n.

Using these values and assuming that $Z_9 = 0$, we can calculate the critical forces $P_{cr,\mathrm{I},i}$ for each column of the primary design scheme of the frame. Similarly, the critical forces $P_{cr,\mathrm{II},i}$ can be calculated for the secondary design scheme, when the outermost column of the first floor is removed and $Z_9 \neq 0$. In this calculation, the dynamic effects in the structure at sudden column loss are replaced by the equivalent static force [36] applied in the joint above the removed component.

The sudden loss of a column usually results in a decrease of the critical force value for the modified structural system. In this case, the topology of the structural system may be the tool that allow ensuring the stability of the components under such an accidental action.

The considered approach for determining the critical force in physically and structurally nonlinear reinforced concrete frames allows us to perform a probabilistic assessment of the risk of frame stability failure under an accidental design situation, such as a hypothetical column loss.

To solve the problem of calculating the stability failure probability for structural components of the frame, we use the direct method of probabilistic analysis and the equigradient principle of G.A. Geniev [27]. The mathematical formulation of this principle is as follows. If a function P depends on parameters whose sum is constant, i.e., $q_1 + q_2 + ... + q_n = C = \text{const}$, then the extremum of the function P on the given set of values q_i satisfies the equality:

$$\frac{\partial P}{\partial q_1} = \frac{\partial P}{\partial q_2} = ... = \frac{\partial P}{\partial q_n}. \tag{4}$$

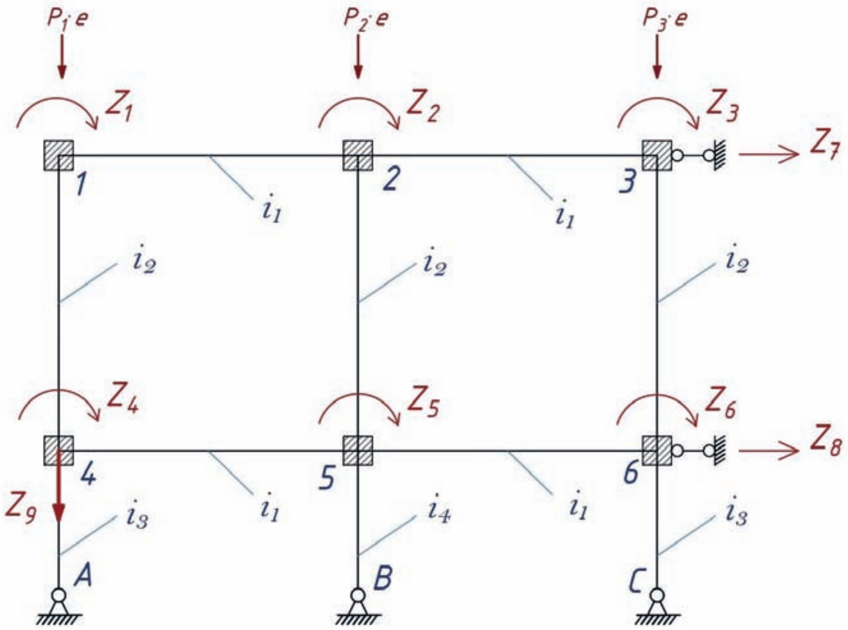


Figure 3. Design scheme of the displacement method for the primary design scheme of the frame

Assume that there is a relation between some parameters affecting on the stability of a structural element such as:

$$D = D\left(P, q_1, q_2, ..., q_n\right) = 0, \tag{5}$$

where $P = P_{cr}$ is the critical force related with the parameters q_1, q_2, ..., q_n, which satisfy the following condition:

$$q_1 + q_2 + ... + q_n = C = \text{const}. \tag{6}$$

Thus, the extremum of the function $P = P(q_1, q_2, ..., q_n)$ fulfilling condition (6) can be found using the auxiliary Lagrangian function:

$$L = P\left(q_1, q_2, ..., q_n\right) = \lambda \cdot \left(q_1 + q_2 + ... + q_n\right), \tag{7}$$

where λ is the Lagrange multiplier.

The extremum condition for this function is as follows:

$$\frac{\partial L}{\partial q_i} = 0. \tag{8}$$

Thus, we obtain the Lagrange multiplier:

$$-\lambda = \frac{\partial P}{\partial q_1} = \frac{\partial P}{\partial q_2} = \ldots = \frac{\partial P}{\partial q_n}. \qquad (9)$$

The extremum of the critical force function P satisfies to the condition (4).

From this follows the conclusion: if some value of P depends on the parameters q_1, q_2, ..., q_n, which are related to each other by condition (6), then for the occurrence of the extremum of the function P it is sufficient to fulfill the criterion (4).

Let P_{s0i} is the initial load of the i^{th} structural element (i = 1, n) and $q_{0i} = v_{0i}$ is the value of the critical force parameter for the given frame before the column removal.

For this frame, the probability of non-destruction of the whole system P_{s0} can be found as follows:

$$P_{s0} = \prod_{i=1}^{n} P_{s0i}. \qquad (10)$$

The sum of the values of critical force parameters before the column removal equals:

$$Q_0 = \sum_{i=1}^{n} v_{0i} = const. \qquad (11)$$

The probability of non-destruction of i^{th} element of the frame, according to the method of direct mathematical analysis can be determined using the following relationship:

$$P_{s0i} = 1 - P_{f0i}, \qquad (12)$$

where P_{f0i} is the probability of the i^{th} structural element stability failure.

The probability of the i^{th} element stability failure can be determined from the expression (13):

$$P_{f0i} = 0.5 - \Phi(U), \qquad (13)$$

where $\Phi(U)$ is the Gauss probability integral:

$$\Phi(U) = \frac{1}{\sqrt{2 \cdot \pi}} \cdot \int_0^U e^{-\frac{x^2}{2}} dx, \qquad (14)$$

For

$$U = \frac{\overline{R} - \overline{\Omega}}{\sqrt{s_R^2 + s_0^2}},$$

where $\overline{R}, \overline{\Omega}$ are mean values of critical load (capacity) and applied load; s_R and s_Ω are the standard deviations of the mean values, respectively.

The relationship between the probability P_{si} of non-destruction of the i^{th} element and the parameter v_i for a system of series-connected elements can be represented in the form:

$$P_{s0i} = 1 - a_i^{-\frac{v_i}{v_{0i}}}, \qquad (15)$$

where

$$a_i = \left(1 - P_{0i}\right)^{-1} \qquad (16)$$

According to the equigradient principle of G.A. Geniev, the solution of the problem is reduced to determining the conventional extremum of function (11) satisfying to condition (15) and constraint (12) at the number of compressed elements in the secondary design scheme of the frame n = 5:

$$f = \prod_{i=1}^{5} \left(1 - a_i^{-\frac{v_i}{v_{0i}}}\right) + \lambda \cdot \left(Q_0 - \sum_{i=1}^{5} v_i\right). \qquad (17)$$

According to condition (4), it can be written:

$$\begin{cases} a_1^{-\frac{v_1}{v_{01}}} \cdot \dfrac{\ln a_1}{v_{01}} - \lambda = 0; \\ \quad \ldots \\ a_5^{-\frac{v_5}{v_{05}}} \cdot \dfrac{\ln a_5}{v_{05}} - \lambda = 0. \end{cases} \qquad (18)$$

By dividing expressions (18) to $P_{si} = 1 - a_i^{-\frac{v_i}{v_{0i}}}$, we obtain:

$$\frac{\ln a_1}{v_{01}} \cdot \frac{1-P_{s1}}{P_{s1}} = \ldots = \frac{\ln a_5}{v_{05}} \cdot \frac{1-P_{s5}}{P_{s5}} = A. \quad (19)$$

Using equality (19), the following relationships can be written:

$$\begin{aligned} &\frac{P_{f1}}{P_{s1}} : \frac{P_{f2}}{P_{s2}} : \frac{P_{f3}}{P_{s3}} : \frac{P_{f4}}{P_{s4}} : \frac{P_{f5}}{P_{s5}} = \\ &= \frac{\ln a_1}{v_{01}} : \frac{\ln a_2}{v_{02}} : \frac{\ln a_3}{v_{03}} : \frac{\ln a_4}{v_{04}} : \frac{\ln a_5}{v_{05}}. \end{aligned} \quad (20)$$

Thus, the expression for the critical values of v_i is as follows:

$$v_i = \frac{1}{\gamma_i} \cdot \ln\left(1 + \frac{\gamma_i}{A}\right), \quad (21)$$

where $\gamma_i = \dfrac{\ln a_i}{v_{0i}}$.

The value of the parameter A is derived taking into account relations (9), (20) and (21):

$$\begin{aligned} &\sum_{i=1}^{5} v_i = \sum_{i=1}^{5} \ln\left[\left(1 + \frac{\gamma_i}{A}\right)^{\frac{1}{\gamma_i}}\right] = \\ &= \ln \prod_{i=1}^{5} \left[\left(1 + \frac{\gamma_i}{A}\right)^{\frac{1}{\gamma_i}}\right] = Q_0. \end{aligned} \quad (22)$$

Accounting for the properties of logarithms, the value of the parameter A will be equal:

$$A = \exp\left[\frac{\ln\left(\prod_{i=1}^{n} \gamma_i^{\beta_i}\right) - Q_0}{\sum_{i=1}^{n} \beta_i}\right], \quad (23)$$

where $\beta_i = \dfrac{1}{\gamma_i}$.

Thus, if the critical values of v_i are obtained by formula (21), then we can find the extremal values of the function P, i.e., the critical forces. Thus, the non-destruction probability for the frame can be found using expression (12) for each structural component and then calculating by formula (10) for the frame. The obtained results should be compared with the target values of non-destruction probability for specified responsibility and operation condition of a facility. If the non-destruction probability is less than the target one the modification of the system should be carried out. Such a modification may be performed using the equigradient approach considered above. In this case, the non-destruction probability will be the target function, while the condition (6) for the total load or for the sum of the force parameters v_i remains relevant. The flow-chart of the stability failure (or non-destruction) probability is shown in figure 4.

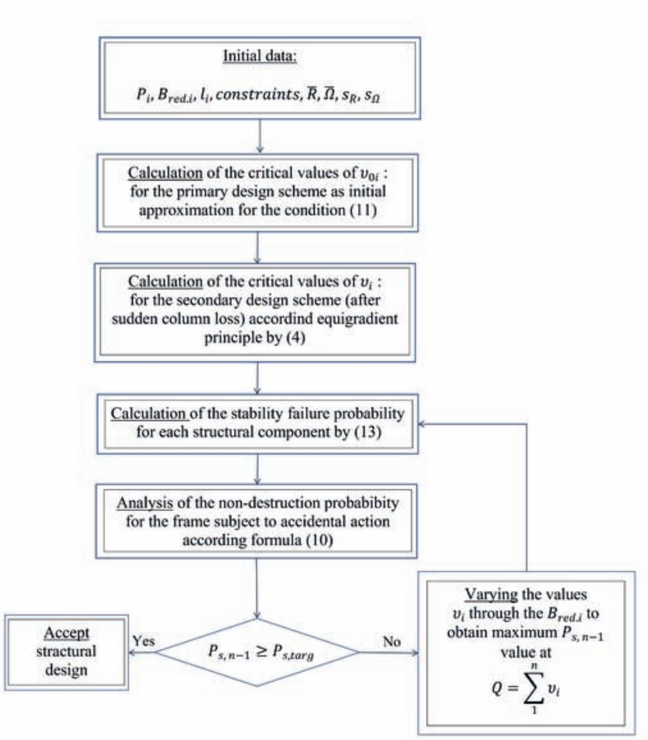


Figure 4. Flow-chart of the method for analysis of stability failure probability

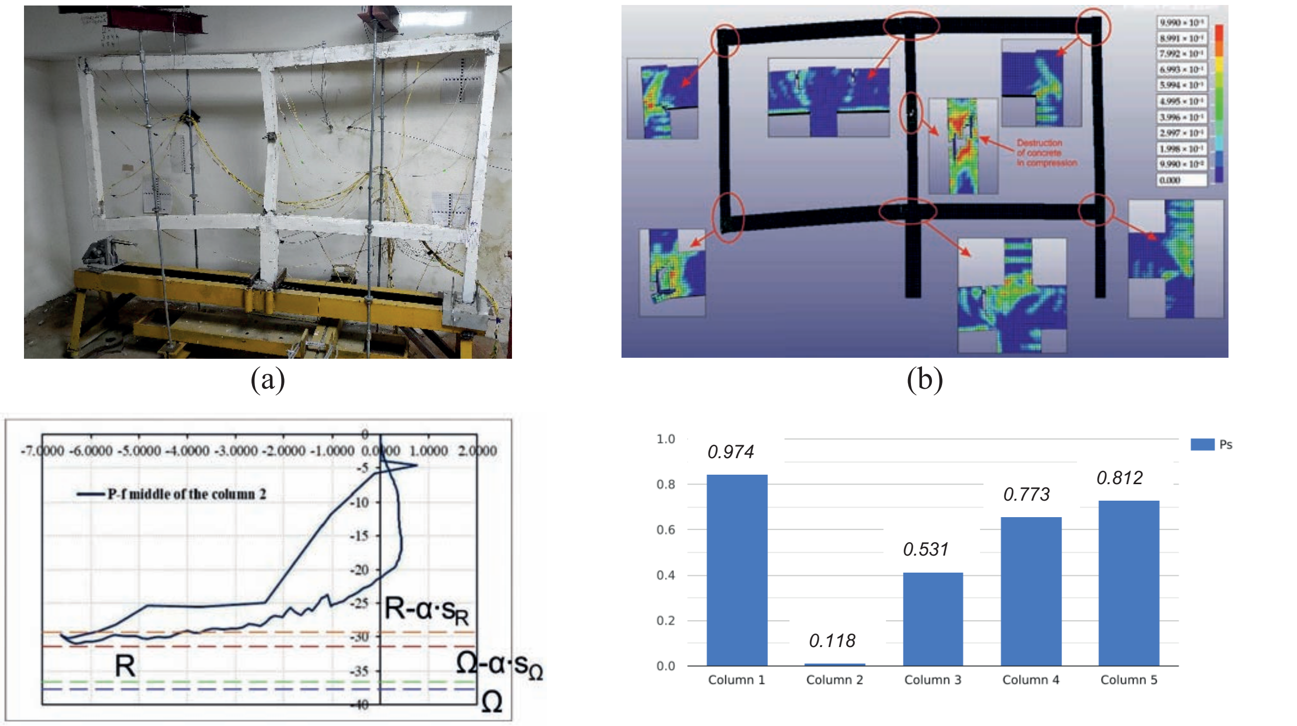
3. RESULTS

Using the method in the previous section of this paper, the stability failure probabilities for the columns of the experimental reinforced concrete frame were calculated for the outermost column removal scenario. The initial data used in the analysis and the results are summarized in Table 1. The probability of non-destruction of the reinforced concrete frame under the accident scenario considered in the study was as follows: $P_{s0} = \prod_{i=1}^{n} P_{s0i} = 0.6936$ for mean values of the critical force $R = P_{cr} = 37.2$kN and mean value of the axial force in the most vulnerable column $\Omega = N = 37.3$ kN.

The obtained results are in agreement with the data of experimental studies and numerical modeling in LS-Dyna, as shown in Figure 5, which demonstrates the stability failure of column No. 2.

Table 1. Initial data and results of calculation of the probability of loss of stability of the experimental reinforced concrete frame elements

Column No.	Mean value of capacity (critical force) R, kN	Mean value of load Ω, kN	Standard deviation of capacity s_R, kN	Standard deviation of load s_Ω, kN	Probability of stability failure P_{fi}	Probability of non-destruction P_{si}
1	17.58	4.8	5.022	7.656	0.026	0.974
2	37.15	37.3	3.797	6.878	0.882	0.118
3	26.74	20.7	1.284	2.343	0.469	0.531
4	36.48	27,2	3.430	2.361	0.227	0.773
5	26.91	19.6	0.849	1.037	0.188	0.812



(a) (b) (c) (d)

Figure 5. Results of the investigation of reinforced concrete frame stability under accidental impact: general view of the frame after test (a), results of modeling in LS-Dyna (b), force-displacement diagram for column No. 2 (c), distribution of probability of non-destruction for the columns (d)

4. CONCLUSIONS

1. A calculation model based on the equigradient principle is proposed to estimate the critical load and the probability of stability failure of reinforced concrete building frames in an accident situation due to the sudden loss of a column.
2. It has been shown that the results of analysis based on the proposed approach agree with the experimental data and the results of numerical modeling. The difference between the calculated mean value of the critical load and the value established experimentally is 5.9 %.
3. The equigradient method considered in the study can be applied to optimize structural design of building frames according to the criterion of minimizing the probability of stability failure at a fixed consumption of materials for the structural system.

5. ACKNOWLEDGEMENTS

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