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ASSESSMENT OF THE ROBUSTNESS FOR FRAME STRUCTURES BASED ON A PROBABILITY INDEX

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Abstract: In the process of designing buildings and structures, the issue arises of ensuring their mechanical safety under accidental loads. Furthermore, existing buildings and structures are often subjected to technogenic impacts that may lead to complete or partial failure of the structural system. To provide a quantitative characterization of the resistance of load-bearing structures to such factors, the concept of survivability has been introduced in the modern scientific literature. However, the formalization of survivability in the form of specific quantitative indicators remains a pressing and insufficiently explored problem for both steel and reinforced concrete structures. This study proposes a methodology for computing the probabilistic survivability index for frame structural systems. The calculation is based on a modified model of classical reliability theory, which assumes that the failure of the frame system occurs through the formation of a mechanism with a minimum number of plastic hinges. The assessment of structural failure or component degradation is performed on an energy basis using the J-integral. To analyze the scatter of random parameters, statistical modeling based on empirical data is employed. The article provides examples of survivability index calculations. Practical implementation of the proposed method has demonstrated its effectiveness, allowing it to be recommended for evaluating the mechanical safety of steel and reinforced concrete frame structures, including cases with increased responsibility requirements and specific robustness criteria against progressive collapse.

Keywords: robustness, mechanical safety, deformations, steel structures, reinforced concrete structures, emergency situation, progressive collapse, reliability, probability of failure

МЕТОД ОЦЕНКИ ЖИВУЧЕСТИ РАМНЫХ КОНСТРУКЦИЙ НА ОСНОВЕ ВЕРОЯТНОСТНОГО ИНДЕКСА

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Аннотация: В процессе проектирования зданий и сооружений возникает задача обеспечения их механической безопасности при воздействии аварийных нагрузок. Кроме того, существующие здания и сооружения нередко подвергаются техногенным воздействиям, приводящим к полному или частичному разрушению конструктивной системы. Для количественной характеристики сопротивляемости несущих конструкций данным факторам в современной научной литературе введено понятие живучести. Однако формализация живучести в виде конкретных количественных показателей, является актуальной и недостаточно проработанной как для стальных, так и для железобетонных конструкций. В настоящем исследовании предлагается методика вычисления вероятностного индекса живучести для рамных конструктивных систем. В основе расчёта лежит модифицированная модель классической теории надёжности, предполагающая, что разрушение рамной системы происходит посредством формирования механизма с минимальным числом пластических шарниров. Оценка разрушаемости конструкций или компонентов, их составляющих, выполняется на энергетической основе с использованием J-интеграла. Для анализа разброса случайных параметров используется метод статистического моделирования на базе эмпирических данных. В статье приведены примеры расчёта индекса живучести. Практическая реализация предложенного метода показала его эффективность, что позволяет рекомендовать его применение для оценки механической безопасности объектов с каркасами из стали или железобетона, в том числе при наличии повышенных требований по ответственности и особых критериев устойчивости к прогрессирующему разрушению.

Ключевые слова: живучесть, механическая безопасность, деформации, стальные конструкции, железобетонные конструкции, аварийная ситуация, прогрессирующее разрушение, надежность, вероятность отказа

1. INTRODUCTION

Current issues of ensuring the robustness of frame structural systems in buildings and structures represent one of the priority areas in modern construction science. The analysis of recent scientific research has made it possible to systematize the key directions of this subject area:

- assessment of the robustness of structural systems based on the analysis of nodal connections of elements;
- development and improvement of sets of deterministic and probabilistic robustness characteristics, as well as the integration of intelligent predictive models influencing the progression of progressive collapse;
- investigation of progressive collapse processes in structural systems using limit analysis methods and numerical simulation;
- analysis of types of accidental impacts and loads;
- development of structural solutions aimed at effectively preventing the development of progressive collapse;
- development and enhancement of methodologies for robustness assessment and management using search optimization algorithms combined with machine learning and artificial neural networks.

A systematic review of scientific studies covering the above-mentioned directions is presented below, which confirms the necessity of developing a probabilistic methodology for robustness assessment, as proposed in this research.

Progressive collapse and robustness criteria. In [1], the influence of initial localized defects on the ability of steel profiles to maintain their functionality under accidental scenarios is investigated. Numerical simulations and experimental observations have shown that damage concentrated in limited zones leads to accelerated formation of plastic hinges, which in turn reduces the overall robustness of the structure. It was established that an increase in the number of stories in a building, as well as the removal of edge or end columns, significantly increases the system's susceptibility to progressive collapse. An alternative interpretation of the concept of structural robustness based on plastic limit analysis is proposed in [2], which is considered more accurate compared to traditional quasi-static and dynamic evaluation methods. This approach makes it possible to account for the actual reserves of the structure's load-bearing capacity without the need for labor-intensive dynamic calculations.

Methods of load and damage identification as a basis for quantitative evaluation of the real robustness index of structures are also reflected in a number of studies. For example, [3] emphasizes the importance of accurate diagnostics of the actual condition of a structure for subsequent robustness assessment. In [4], an algorithm based on decision-tree methods is proposed, which employs vibration response data in combination with a calibrated finite element model. This approach allows determining the hazard level of damages, evaluating the residual service life, and providing a quantitative measure of robustness under potential accidental scenarios.

The evaluation of the load-bearing capacity of steel elements under localized thermal loads is addressed in [5], where the behavior of structures in fire conditions is studied. Despite the considerable number of studies devoted to the deformational properties and patterns of progressive collapse in frame systems, the evaluation of structural strength as a factor limiting the spread of failures still requires further systematization both within deterministic and probabilistic models.

Accidental Loads and Impacts. Research in the field of robustness assessment of structural systems under accidental loads is focused on developing parameters that quantitatively characterize the intensity of extreme actions, including seismic ones [6]. Such parameters include spectral accelerations and displacements, which make it possible to indirectly assess the stability of the system against progressive collapse. These data demonstrate the multifactorial nature of conditions influencing robustness. The distinctive feature of this approach lies in comparing the parameters of the impact not only by their amplitude characteristics but also according to criteria of efficiency, feasibility, and distance to the epicenter of the accidental action.

In [7], emphasis is placed on the behavior of steel frames under thermal effects caused by fire. A methodology is proposed that involves local evaluation of thermal impact on individual elements of the structure, followed by a generalized verification of the strength of the entire system. Fire scenarios consider both the removal of central (internal) and edge columns, thereby allowing different failure configurations to be analyzed.

The methodology of local–global analysis is applied in [8], where the robustness of frame structures under thermal heating is assessed. In this approach, a single column is subjected to dynamic loading, while the remaining part of the frame is analyzed under static loading. It is shown that such a method can be universally applied to structures made of various materials and improves the accuracy of evaluating system behavior under accidental conditions.

Influence of Joint Connections on Structural Robustness. The study [9] focuses on analyzing the stability of steel frame buildings with bolted connections depending on their height. Buildings with 5, 10, and 15 stories were considered, and the results showed that taller buildings demonstrated better robustness characteristics. Particular attention was paid to probabilistic approaches, in particular, the construction of fragility curves for various progressive collapse scenarios. In [10], [11], it is emphasized that the robustness of a structure is a determining parameter of its safety, especially in steel frame systems where the key elements are beam-to-column joints. The Monte Carlo method was applied to account for stochastic variations in mechanical parameters, which was used to analyze system vulnerability under accidental impacts. Tornado diagrams were employed to evaluate the sensitivity of the system to variations in parameters such as span length, modulus of elasticity, and yield strength of elements, considering statistical variability of material properties. Studies [12], [13] are dedicated to the behavior of frame joints subjected to bending moments. The authors highlight the importance of forming a ductile failure mechanism of connections to increase the overall robustness and reliability of the system. Parameters promoting joint ductility are proposed, enabling the effect of sequential localization of failures. Publications [14], [15] investigate the behavior of steel frames under sudden column removal and in the presence of flange end-plates. It was found that including end-plates contributes to effective localization of damage, thereby improving robustness. Accident scenario analyses in [16], [17], [18] demonstrated that the strength and design characteristics of joints have a decisive influence on the system’s ability to withstand both instantaneous and progressive failures of columns. Experimental data indicate that flange geometry significantly affects damage localization and thus determines the level of robustness. In [19], a solution is proposed to enhance the ductility of beam-to-column joints by using special slots, which enable a chain-reaction mechanism and thereby increase the overall robustness of the frame.

Deterministic and Probabilistic Indicators of Robustness. Integration of Artificial Intelligence Technologies. In several studies [20], [21], the problem of robustness of structural systems is addressed using a combined approach that integrates traditional engineering calculation methods with modern artificial intelligence tools. Data obtained from the analysis of deformation characteristics of structures under different accidental scenarios are processed using machine learning algorithms. This makes it possible to develop predictive models that assess the influence of structural parameters on overall robustness. Results demonstrate that optimizing geometric configuration has a stronger impact on improving robustness than the use of materials with higher mechanical properties. In [22], [23], the introduction of a structural safety index is proposed, which incorporates uncertainties inherent in design conditions. Such an index allows for informed decisions regarding the need to adjust design parameters, taking into account the probability of progressive collapse development. It was found that the inclusion of uncertainties in calculations can reduce the estimated robustness by up to 30% compared to deterministic assessments. At the same time, the strength of steel is particularly emphasized as a key parameter influencing the final evaluation. Modern approaches to structural health monitoring also include damage detection technologies employing neural network models. For instance, [24] proposes a method of visual image analysis that uses artificial neural networks to determine the degree of corrosion damage in steel elements, enabling more accurate quantitative evaluation of the robustness index. A comprehensive robustness assessment of seismic-resistant structures is presented in [25], where the robustness index is defined as the difference between unity and the sum of both material and non-material losses. These losses are calculated based on failure scenarios obtained through direct dynamic simulation, fragility curve construction [26], and probabilistic failure evaluation. Depending on the obtained robustness index values, a classification of seismic resistance levels of multi-story steel-framed buildings is proposed. In [27], robustness is considered through the lens of strength loss in elements resulting from structural vulnerability. Accidental scenarios in this approach are modeled either as completed events or with assigned probabilities. It is concluded that systems with high ductility demonstrate higher robustness indices. In [28], a multi-criteria methodology for robustness evaluation is proposed, which considers energy, loading, and stiffness parameters, as well as specific failure mechanisms. A system of indices is introduced that comprehensively characterizes the mechanical safety of steel-framed buildings and structures. Here, robustness is defined as the ability of the system to maintain stability in the presence of local damages that could trigger cascading collapse. Publication [29] provides a comparative analysis of approaches to evaluating the robustness of steel structures against progressive collapse. Both a deterministic method, applied in seismic design of buildings [30], and a probabilistic method, based on event-tree construction, are used. Simulation results led to the formulation of a new robustness index that accounts for both the potential of chain failure and the deformational properties of frame joints.

Engineering Solutions Aimed at Preventing Progressive Collapse. In [31], a structural solution is proposed to enhance the robustness of steel frame systems by incorporating an additional roof truss. This truss redistributes internal forces in cases of partial or complete removal of beam or column elements under different accidental scenarios. Results of numerical modeling showed that under accidental loading the system transitions into a plastic state, with deformations forming due to the development of plastic hinges. Optimal stiffness values of truss elements were determined, which provide the highest efficiency under emergency conditions. The influence of diagonal bracing on the robustness of steel frames against progressive collapse is examined in [32]. Two system configurations were considered—with and without bracing. It was found that including diagonal members significantly increased the ultimate load-bearing capacity of the structure—by 129.7% and 45.1% in different cases. These results demonstrate the substantial effect of bracing schemes on robustness. It should be noted, however, that robustness in this work was evaluated solely within a deterministic framework. In [33], high-rise buildings (50 stories) designed according to ASCE 7 and AISC standards were analyzed. Alongside assessing frame robustness, the effect of building plan configuration on resistance to progressive collapse was also studied. It was established that when the central column is removed, the overall plan shape of the building has a decisive influence on system behavior. Conversely, when edge columns are removed, local factors dominate and the plan geometry plays a smaller role. Thus, plan geometry may be considered an additional parameter affecting robustness in certain collapse scenarios. Publication [34] is also devoted to the robustness of tall buildings, while [35] considers a combined scenario in which the initial accidental action is an earthquake, followed by fire. The research focused on four- and eight-story steel frame structures with V-shaped or diagonal bracing. The obtained results highlight the importance of considering combined impact scenarios when evaluating robustness parameters.

Based on the literature review, the scientific relevance of the task addressed in this study—probabilistic robustness assessment of frame structural systems—has been substantiated.

2. METHODS

The following assumptions are used as the main prerequisites for checking the robustness of a frame structure:

- the condition that the structure remains in a final deformed state without transition into a collapse mechanism after accidental loading;
- the condition of non-fracture of sections after stabilization of the dynamic effect of the accidental action;
- the condition of section strength under peak dynamic loads.

2.1 Geometric Stability After Accidental Loading. Such a state can be mathematically expressed as follows:

$$\left([K]\cdot\{\delta\}=\frac{1}{\alpha}\{P_0\}+\{P_M\}\right)\wedge \wedge\left(\left(cond_\infty[K]\right)^{-1}\leq\lambda\right) \quad (1)$$

where $[K]$ is the global stiffness matrix of the frame finite elements; $\{\delta\}$ is the displacement vector; $\{P_M\}$ is the nodal load vector simulating the application of moments at plastic hinge formation points; $\{P_0\}$ is the external nodal load vector; $\alpha = P_0/P_{\lim}$, $P_{\lim}$ is the limiting load factor at which equilibrium conditions are still satisfied. The parameter λ is presented in Table 1.

Table 1. Determination of parameter λ

Characteristic of the system		
Geometrically stable	Instantly unstable	Geometrically unstable
$\lambda > 10^3$	$10^{-10} < \lambda < 10^{-2}$	$\lambda < 10^{-16}$

Geometric instability of the system under accidental action may arise not only due to the applied loads but also because of member failure. The coefficient λ may be computed through selection or by solving an extremal combinatorial problem. The conditioning number of the stiffness matrix is calculated using the infinity norm:

$$cond_\infty[K]=\left\|[K]\right\|_\infty\times\left\|[K]^{-1}\right\|_\infty, \quad (2)$$

where $\|\bullet\|_\infty$ is the symbol for the infinite norm.

2.2 Condition of Non-Fracture After Dynamic Effect of Accidental Loading. As a rule, building structures use materials that initiate elastoplastic fracture. For most structures, this type of failure begins with the formation of a crack. If the crack propagates, the structure first undergoes local damage and then complete failure. The condition of local crack growth in fracture mechanics is approximately described by the Rice–Cherepanov contour J-integral [36]:

$$J=\int_C\left(Wdy-\sigma_{iy}n_y\frac{\partial U_i}{\partial x}\right)dc\geq J_{cr}, \quad (3)$$

where W is the strain energy density at the crack tip; σ_{iy}, U_i are the equivalent stresses and displacements at contour point C in direction y perpendicular to crack propagation; J_{cr} is the critical value of the integral defining the energy required for crack growth.

2.3 Strength Condition of Structural Members. Practical experience in analysis and design of load-bearing structures has shown that the strength of members under peak dynamic loads can be assessed in quasi-static form. The static equivalent is obtained by multiplying the static load by a dynamic coefficient, which usually does not exceed 2.0. Material strain-rate strengthening should also be considered. In this case, standardized design methods can be applied. For reinforced concrete and steel structures, the main formulas are summarized in Table 2.

Table 2. Strength assessment conditions for members under accidental action

Beams	Columns	Joints
Steel structures [37]		
1. Strength in the span under compression and bending according to formula (106). 2.Overall stability of the element according to formula (69) 3.Shear strength in support sections according to formula (42) for structural design class 1, according to formulas (54)-(55) for classes 2 and 3.	1. Overall stability of the element in accordance with formula (109) 2. Strength check using formula (106) for short columns	1. Welded joints, formulas (175)-(181) 2. Bolted joints, formulas (186)-(188), (190)
Reinforced concrete structures [38]		
$M < M_{ult}$, M is the bending moment resulting from an emergency impact. M_{ult} is the maximum moment absorbed by the cross-section with control deformations in tensioned rebars.	$Ne < M_{ult}$; $N < N_{ult}$, $M_{ult} = R_b bx(h_0 - 0,5x) +$ $+R_{sc}A'_s(h_0 - a')$ e is the eccentricity of the longitudinal force, N, N_{ult} are the forces acting and perceived by the cross-section, respectively.	Condition $l_{an}^{ef} \geq l_{an}$ of actual anchoring reinforcement. Condition $\sigma_s \leq R_s$ of tensile strength of reinforcement σ_s, R_s are the stress and design resistance of reinforcement.

2.3 Reliability Indices for Robustness Assessment

2.3.1 Steel Structures. Two criteria are introduced for reliability indices: the probability of failure due to normal stress exceeding limits (Eq. 4), and due to shear stress (Eq. 5). These failure types are typical for sections of steel frame elements:

$$\beta_\sigma = \frac{|R_y| - |\sigma_{MN}|}{\sqrt{S(\breve{\sigma}_{MN})^2 + S(\breve{R}_y)^2}},$$

$$\breve{\sigma}_{MN} = \breve{N} / A \pm \breve{M}_x y / I_x,\ \breve{N} = f_1(\breve{Q}),\ \breve{M}_x = f_2(\breve{Q}) \quad (4)$$

$$\beta_\tau = \frac{|R_s| - |\tau_x|}{\sqrt{S(\breve{\tau}_x)^2 + S(\breve{R}_s)^2}},$$

$$\breve{\tau}_x = \begin{cases} \breve{Q}_x S / I_x t_w & : 1 \text{ class of SSS} \\ \breve{Q}_x / A_w & : 2, 3 \text{ classes of SSS} \end{cases}, \quad (5)$$

$$\breve{Q}_x = f(\breve{Q})$$

where β_σ, β_τ are reliability indices; σ_{MN}, τ_x are mean values of normal and shear stresses in critical sections; R_y, R_s are design strengths in bending and shear; $S(\cdot)$ are standard deviation of values, $\breve{N}, \breve{M}_x, \breve{Q}_x, \breve{\sigma}_{MN}, \breve{\tau}_x$ are values of variants in samples of internal forces and stresses obtained by statistical modeling or based on data monitoring of the technical condition of structures; A, S, I_x, t_w, A_w, y values are the geometric characteristics of the section under consideration, in particular for an I-beam, this is the area, static moment, central moment of inertia, wall thickness, web area, and the coordinate of the point where the stress is calculated; $\breve{Q}$ is the value of the random variable of the applied load; f, f_1, f_2 are analytical laws that determine the dependence of internal forces on the load. The standard deviation of the random variable σ_{MN} (1) in the presence of longitudinal forces and bending moments in the section is calculated as:

$$S(\sigma_{MN}) = \sqrt{D(\breve{N}/A) + D(\breve{M}_x y / I_x) + 2\,\mathrm{cov}\left((\breve{N}/A), (\breve{M}_x y / I_x)\right)}, \quad (6)$$

$$\mathrm{cov}\left((\breve{N}/A), (\breve{M}_x y / I_x)\right) = \frac{1}{n}\sum_{i=1}^{n}\left((\breve{N}\breve{M}_x y)/(I_x A)\right)_i - \left(\overline{\breve{N}/A}\right)\left(\overline{\breve{M}_x y / I_x}\right), \quad (7)$$

where $D(\cdot)$ is the variance from the argument; $\mathrm{cov}((\cdot),(\cdot))$ is the covariance, $(\bar{\cdot})$ is the mean value for the sample population under consideration, n is the sample size.

When beam elements of the structural system fail due to stability conditions, the reliability index takes the form:

$$\beta_s = \frac{|R_y| - |\sigma_{Mx}|}{\sqrt{S(\breve{\sigma}_M)^2 + S(\breve{R}_y)^2}}, \quad \breve{\sigma}_{Mx} = \frac{\breve{M}_x}{\varphi_b W_{cx}}, \quad (8)$$

where σ_M is conditional normal stress in the beam, taking into account the longitudinal force; W_{cx}, φ_b are values determined by regulatory documents during stability analysis, in particular according to [37].

For columns loaded as a result of emergency impact by longitudinal force and bending moment, the stability reliability index is:

$$\beta_{st} = \frac{|R_y| - |\sigma_N|}{\sqrt{S(\breve{\sigma}_N)^2 + S(\breve{R}_y)^2}}, \quad \breve{\sigma}_N = \frac{\breve{N}}{\varphi_e A}, \quad (9)$$

where σ_N is conditional normal stress in the column, taking into account the longitudinal force; φ_e is stability coefficient under compression with bending, determined depending on the values of relative eccentricity η and conditional flexibility $\bar{\lambda}$ and relative eccentricity, reduced to the cross-section shape by a coefficient.

2.3.2 Reinforced Concrete Structures

The specific features of reinforced concrete failure imply two principal failure mechanisms. The first corresponds to section failure due to the formation of a plastic hinge. The second occurs when concrete is excluded from load-bearing, resulting in a chain-type mechanism.

For the first failure mechanism, the reliability index β_R of beams is calculated as follows:

$$\beta_R = \frac{|M_{ult}| - |M_{\max}|}{\sqrt{S(M_{ult})^2 + S(M_{\max})^2}}, \quad (10)$$

where M_{ult} is the bending moment in the section that can be resisted by concrete and reinforcement; $M_{\max}$ is the maximum moment in this section under accidental loading; $S(M_{ult})$, $S(M_{\max})$ are the standard deviations of the random variables M_{ult}, $M_{\max}$.

For reinforced concrete columns, the reliability index β_C under accidental action is determined as:

$$\beta_C = \frac{\left(|N_{ult}| - |N_{\max}|\right) \cdot e}{\sqrt{S(eN_{ult})^2 + S(eN_{\max})^2}} \quad (11)$$

where N_{ult}, $N_{\max}$ are the maximum axial forces carried by concrete and reinforcement in the section and the axial force under accidental loading; respectively; e is the eccentricity of axial force; $S(eN_{ult})$, $S(eN_{\max})$ are the standard deviations.

In the absence of experimental data, the standard deviations can be generated under the assumption of a normal distribution of the random variables. In the case of the chain-type failure mechanism, columns subjected to additional accidental loads generally do not fail. Therefore, the reliability indices may only be calculated for beams. In this case, rupture of reinforcement and its slip in the intact part (loss of anchorage) may occur. For reinforcement rupture, the reliability index can be expressed as (12), and for anchorage loss as (13):

$$\beta_{RV1} = \frac{|\varepsilon_u| - |\varepsilon_{\max}|}{\sqrt{S(\varepsilon_u)^2 + S(\varepsilon_{\max})^2}}, \quad (12)$$

$$\beta_{RV2} = \frac{l_{an} - l_{an}^{ef}}{\sqrt{S(l_{an})^2 + S(l_{an}^{ef})^2}}, \quad (13)$$

where ε_u is the rupture strain of reinforcement; $\varepsilon_{\max}$ is the tensile strain of reinforcement under accidental load; l_{an} is the design anchorage length determined according to standards; l_{an}^{ef} is the anchorage length of reinforcement after formation of the kinematic chain under accidental loading; $S(\varepsilon_u)$, $S(\varepsilon_{\max})$ $S(l_{an})$, $S(l_{an}^{ef})$ denotes the standard deviations of these random variables.

2.4 Quantitative Assessment of Robustness

Robustness of structural systems in the works of both domestic and international researchers has most often been evaluated using deterministic indicators. These were associated with the analysis of transition into a geometrically unstable system under accidental actions. For frame structural systems, robustness can be assessed using a probabilistic indicator. Its physical meaning lies in the fact that the system does not fail either under normal service conditions or under accidental scenarios, that is:

$$W_R = P_{no} P_{dam}^E, \quad (14)$$

where P_{no} is the probability of no-failure of the system under normal operation, and P_{dam}^E is the probability of no-failure of the system under an accident or different accidental scenarios. In conventional design, this probability is typically within $P_{no} = 0.95 \div 1$; in optimal design aimed at minimizing costs without consideration of robustness, it is $0.98 \div 1$. For a fully safe system in the idealized case, $W_R = 1$, but in such a case the cost of the structure may become unacceptably high.

For a frame structural system with *m* beams, *n* columns, and *k* joints, the probabilistic indicator of robustness can be computed as:

$$W_R = \prod_{i=1}^{m_R}\left(P_{no,i}\left(1-P_{dam,i}\right)\right)\times \prod_{j=1}^{m_c}\left(P_{no,j}\left(1-P_{dam,j}\right)\right)\times \prod_{k=1}^{m_U}\left(P_{no,k}\left(1-P_{dam,k}\right)\right). \qquad (15)$$

Here $P_{no,i}$, $P_{no,j}$, $P_{no,k}$ are the probabilities of no-failure of beams, columns, and joints of the frame in the absence of accidents under normal service; $P_{dam,i}$ is the probability of accidental failure of column *i*; $P_{dam,j}$ is the probability of accidental failure of beam *j*; is the same for joint *k*. These probabilities are determined as:

$$P_{dam} = 1-(0{,}5+\Phi(\beta)) \qquad (16)$$

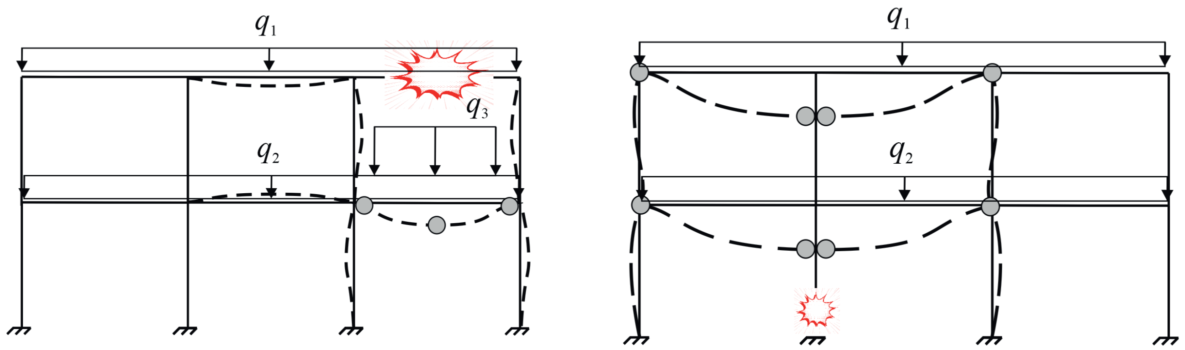
where $\Phi(\beta)$ is the Laplace function, β is the reliability index (Eqs. (4), (5), (8)–(13)) calculated for each structural element or joint. If the condition of no-failure for one or more groups of elements or joints is not satisfied, robustness of the system is considered not ensured. In this case, it can be written as:

$$\left.\begin{array}{l}\left(\prod_{i=1}^{m_R}(w_i)\right)\vee \\ \vee\left(\prod_{j=1}^{m_c}(w_j)\right)\vee \\ \vee\left(\prod_{k=1}^{m_U}(w_k)\right)\end{array}\right| \begin{array}{l} P_{dam}^{\max} = 0{,}5; \\ \Phi(\beta)=0; \quad =\left[0 \div 0{,}6\right] \\ \beta = 0. \end{array} \qquad (17)$$

In particular, if all beams and joints of the frame remain functional but columns fail, the system is not considered robust

$$W_R = 0{,}95\cdot 0{,}6\cdot 0{,}95 \approx 0{,}55 < 0{,}6.$$

Under different accidental scenarios, element failures may be multiple and realized according to a sequential scheme (Fig. 1a) or a parallel scheme (Fig. 1b).



a) *b)*

Figure 1. Possible failure schemes: (a) failure of a beam; (b) failure of a central column; shaded circles – zones of possible plastic hinge formation

For example, the probability of failure for a system of two beams failing one after another in sequence 1–2 can be determined on Fig. 1a as: $P_{dam} = P_{dam1} + P_{dam1}\cdot P_{dam2}$. If condition (1) is not satisfied, or according to (2) the system is geometrically unstable, then robustness is not ensured $W_R = 0$.

3. RESULTS AND DISCUSSION

A steel frame made of structural steel with a yield strength of $R_{yn} = 375$ MPa is considered. The scheme is shown in Figure 2. Each element is assigned a number: beams and columns are designated by digits. The cross-sections of members were selected with a safety margin of load-bearing capacity. It is assumed that this reserve of strength may be mobilized under accidental conditions. Potential accidental scenarios are indicated with crosses. These situations may be accompanied by the formation of a system of plastic hinges. It is assumed that such a mechanism ensures localization of failure and prevents further progression.

The design of the structure was carried out with account of standard service loads. The calculation included the following factors:

- self-weight (dead load) of reinforced concrete slab and steel elements $q = 40$ kN/m;
- wind load including mean and pulsating components, applied from both directions;
- snow load for Region III (Moscow) $s = 13,2$ kN/m;
- additional snow accumulation $ds = 39,6$ kN/m;
- live load on floor slabs $v = 9$ kN/m;
- live load on the roof $v_0 = 3$ kN/m.

Standard deviations of random load values, corresponding stresses, and strength of S375 structural steel were determined using statistical modeling. To obtain the required standard deviations, a base random sample of values uniformly distributed between 0 and 1 was generated. Sample values are given in Table 3.

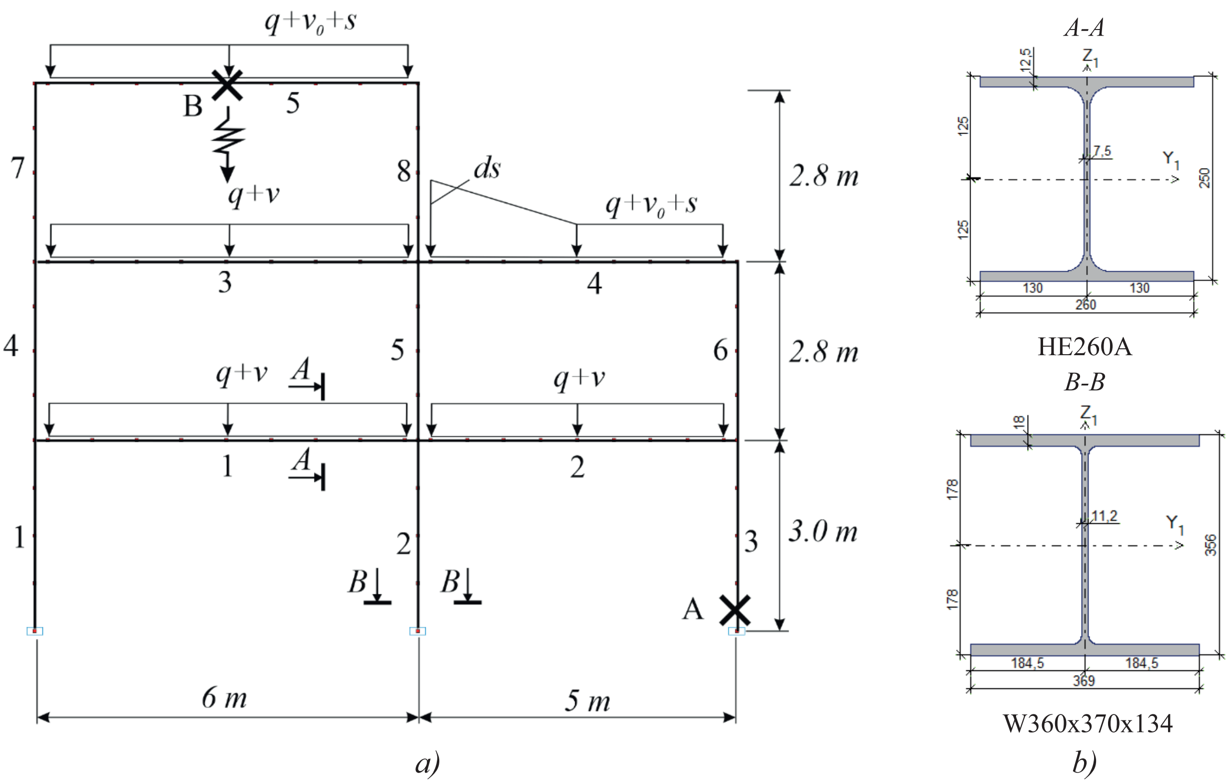


Figure 2. Calculation scheme of the frame (a): 1–8 – column numbers, 1–5 – beam numbers; cross-sections of columns and beams (b)

Table 3. Sample data for determining standard deviations under the normal distribution law

ω_{1-10}	ω_{11-20}	ω_{21-30}	ω_{31-40}	ω_{41-50}	ω_{51-60}	ω_{61-70}	ω_{71-80}	ω_{81-90}	ω_{91-100}
0,980	0,112	0,733	0,266	0,212	0,140	0,740	0,847	0,233	0,368
0,070	0,370	0,017	0,540	0,028	0,757	0,336	0,611	0,812	0,355
0,115	0,209	0,591	0,708	0,896	0,438	0,885	0,981	0,961	0,189
0,069	0,896	0,250	0,715	0,926	0,567	0,458	0,187	0,917	0,904
0,938	0,463	0,396	0,623	0,753	0,674	0,010	0,701	0,093	0,989
0,329	0,969	0,398	0,627	0,914	0,486	0,471	0,836	0,759	0,500
0,079	0,127	0,843	0,105	0,245	0,025	0,742	0,050	0,565	0,629
0,810	0,441	0,831	0,752	0,327	0,022	0,293	0,574	0,419	0,003
0,196	0,935	0,960	0,149	0,981	0,667	0,525	0,066	0,297	0,016
0,958	0,166	0,248	0,937	0,705	0,453	0,368	0,052	0,431	0,761

Experimental data on the coefficient of variation of steel strength under uniaxial tension were used as initial input. Statistical variants of stress σ_{MN} and strength R_y values were calculated as: $\sigma_{MN,i} = \omega_i \overline{\sigma}_{MN}$, $R_{y,i} = \omega_i \overline{R}_y$, $\overline{\sigma}_{MN}, \overline{R}_y$ are the sample means for the normal stress and the design strength. Thus, using the data of synthetic statistics based on the normal distribution (or processing experimental data when available), the reliability indices can be calculated according to formulas (4) and (5).

3.1 Normal Service Conditions. For normal service conditions of structural elements, which from the perspective of structural mechanics are considered long members, normal stresses act as the determining factor in strength evaluation. However, in critical sections of beams, the influence of shear forces additionally manifests itself. Therefore, using only normal stresses for assessing load-bearing capacity may lead to an incomplete characterization of the stress state. To improve the accuracy of analysis, it is advisable to apply equivalent stresses according to the von Mises criterion. This approach makes it possible to account for the combined action of normal and shear stresses. The distribution of equivalent stresses is shown in Figure 3a. For comparison, Figure 3b presents a frame optimized according to the criterion of minimum material consumption. These data were taken as the initial input for calculating the reliability indices, the values of which are given in Table 4.

Analysis of the data presented in Table 4 shows that optimization of the structure according to the criterion of minimum material consumption leads to an economically efficient design solution. However, this significantly reduces the level of structural reliability. The reduction reaches several orders of magnitude. Under such conditions, the probability of failure of load-bearing elements increases, and even factors arising from the statistical nature of the mechanical properties of steel or variability of applied loads may trigger failure. For quantitative evaluation, calculations of the probabilities of no-failure of columns and beams were performed. The results of these calculations are presented in Table 5.

Table 4. For calculating the probability of element failure under normal service conditions

№ of element	Original structure, $\frac{\sigma_e / \varphi}{R_y}$, MPa	Optimal structure, $\frac{\sigma_{e,opt} / \varphi_{opt}}{R_y}$, MPa	$\frac{\beta_R}{\beta_{Ropt}}$	$\frac{\Phi(\beta_R)}{\Phi(\beta_{Ropt})}$	$\frac{P_{dam,i}}{P_{dam,i,opt}}$
Columns					
1	56/337,5	119/300	$\frac{\beta_R > 5}{4,76}$ 3	$\frac{0,5}{0,499997}$	$\frac{0}{3 \cdot 10^{-6}}$
2	57/337,5	130/320	$\frac{\beta_R > 5}{\beta_{Ropt} > 5}$	$\frac{0,5}{0,5}$	$\frac{0}{0}$
3	28/337,5	83/280	Same	Same	Same
4	61/345,0	128/321	-//-	-//-	Same
5	53/345,0	94/342	-//-	-//-	Same
6	67/345,0	205/299	$\frac{\beta_R > 5}{2,47}$	$\frac{0,5}{0,4933}$	$\frac{0}{6,7 \cdot 10^{-3}}$
7	84/345,0	193/321	$\frac{\beta_R > 5}{3,36}$	$\frac{0,5}{0,49955}$	$\frac{0}{4,5 \cdot 10^{-4}}$
8	86/345,0	196/342	$\frac{\beta_R > 5}{3,84}$	$\frac{0,5}{0,499922}$	$\frac{0}{8,8 \cdot 10^{-5}}$
Beams					
1	192/375	289/375	$\frac{4,81}{2,26}$	$\frac{0,499997}{0,4881}$	$\frac{3 \cdot 10^{-6}}{1,19 \cdot 10^{-2}}$
2	163/375	260/375	$\frac{\beta_R > 5}{3,02}$	$\frac{0,5}{0,49863}$	$\frac{0}{1,37 \cdot 10^{-3}}$
3	204/375	313/375	$\frac{4,5}{1,63}$	$\frac{0,499997}{0,4484}$	$\frac{3 \cdot 10^{-6}}{5,16 \cdot 10^{-2}}$
4	225/375	368/375	$\frac{3,9}{0,18}$	$\frac{0,499948}{0,0714}$	$\frac{5,2 \cdot 10^{-5}}{0,4286}$
5	224/375	313/375	$\frac{4,0}{1,63}$	$\frac{0,499968}{0,4484}$	$\frac{3,2 \cdot 10^{-5}}{5,16 \cdot 10^{-2}}$

Table 5. For calculating the probability P_{no}

Original structure		Optimal structure		Probability of the frame nonfailure	Risk at a loss of 1 million rubles
Columns $\prod_{i=1}^{8} P_{no,i}$	Beams $\prod_{i=1}^{5} P_{no,i}$	Columns $P_{no,c} = \prod_{i=1}^{8} P_{no,i}$	Beams $P_{no,b} = \prod_{i=1}^{5} P_{no,i}$		
1	$0,999997 \times$ $\times 0,999997 \times$ $\times \cdot 0,999948 \times$ $\times 0,999968 =$ $= 0,99991$	$0,999997 \cdot 0,9933 \times$ $\times 0,99955 \times 0,999912 =$ $= 0,99276$	$0,9881 \cdot 0,99863 \cdot 0,9484 \times$ $\times 0,5714 \cdot 0,9484 =$ $= 0,50714$	$\frac{0,99991}{0,5034}$	$\frac{90}{496600}$

Table 5 shows that the design optimized solely according to the criterion of minimizing material consumption is characterized by a significantly higher level of risk. In this case, the probability of material losses exceeds the corresponding indicator of the initial design by several orders of magnitude. At the same time, failure of the load-bearing system does not necessarily lead to its complete collapse. The most probable scenario is damage to the support joint of the roof beam in the area of increased snow accumulation. Based on the analysis of the data in Tables 4 and 5, it is evident that designs optimized only in terms of material efficiency do not possess robustness.

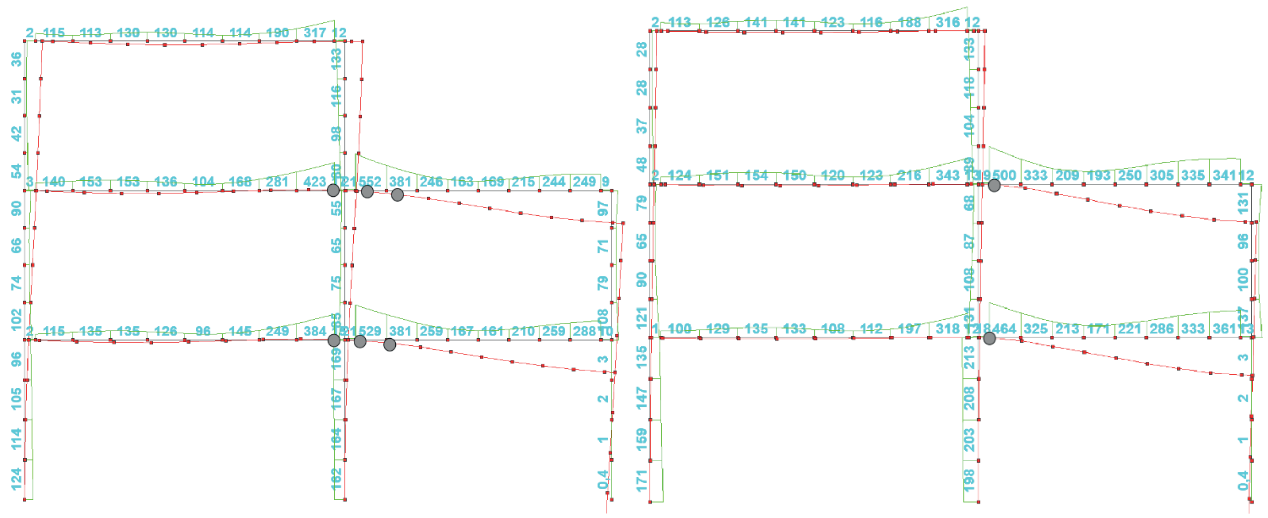
3.3 Accidental Scenarios. Dynamic effects arising from support removal are modeled in a quasi-static formulation, using the well-known formula of G.A. Geniev. In this study, only the initial design is analyzed, without optimization. It is assumed that in all variants of the frame system, joints function without failure. The probabilistic assessment of the robustness of the frame is carried out step by step:

a) The probability of no-failure of the system under normal operation is determined. It is computed as the product of the probabilities of no-failure of individual elements. In conventional design, this probability is close to unity (see Table 5).

b) The probability of no-failure of beams and columns is calculated under the condition of removing one column. In this case, the frame elements follow a parallel failure scheme.

c) The probability of no-failure of beams is determined under the condition of removing one beam while all columns remain intact. In this case, the beams of the frame follow a sequential failure scheme.

3.3.1 Accidental Scenario A. The results of the linear quasi-static analysis are presented in Figure 4a. Analysis of the stiffness matrix of the system showed that it is geometrically stable. Verification of equilibrium conditions made it possible to construct a calculation scheme with a minimum number of plastic hinges. The obtained data are used to form the initial parameters presented in Table 6.



a) b)

Figure 4. Diagrams of equivalent stresses (MPa) of the frame: (a) linear analysis with plastic hinges; (b) analysis considering physical and geometric nonlinearity

Table 6. Calculation of failure probability for structural elements

№ of element	$\frac{\sigma_e / \varphi}{R_y}$, MPa	β_R	$\Phi(\beta_b)$	$P_{dam,i}$
Columns				
1	124/337,5	$\beta > 5$	0,5	0
2	169/337,5	4,43	0,499997	$3,0 \cdot 10^{-6}$
3	-	-	-	-
4	102/345,0	$\beta > 5$	0,5	0
5	85/345,0	То же	То же	То же
6	108/345,0	-//-	-//-	-//-
7	54/345,0	-//-	-//-	-//-
8	133/345,0	-//-	-//-	-//-
Beams				
1	249/375	3,31	0,49945	0,00055
2	288/375	2,28	0,4887	0,0113
3	281/375	2,47	0,4933	0,0067
4	246/375	3,39	0,4996	0,0004
5	317/375	1,52	0,4357	0,0643

Based on the data in Tables 5 and 6, it is determined:

$$W_R \approx 0,999997 \cdot (1-0,00055) \cdot 1 \cdot (1-0,0113) \cdot 0,999997 \cdot (1-0,0067) \cdot 0,999948 \cdot (1-0,0004) \times$$
$$\times 0,999968 \cdot (1-0,0643) \cdot 1 \cdot \left(1-3 \cdot 10^{6}\right) \approx 0,9179.$$

The robustness condition is satisfied: $W_R \approx 0,9179 > 0,6$. Nonlinear analysis also confirmed that the system retains robustness. During deformation, two plastic hinges are formed in the beams. The maximum deflection reaches about 49 cm. With a story height of 280 cm, such a deflection does not hinder evacuation of people and/or removal of equipment. It should be emphasized that in this case robustness is ensured by a structural solution providing a partial cantilever suspension effect realized by the beams. However, with an increase in applied loading this solution may prove insufficient. In such situations, the installation of diagonal elements is required to redistribute forces and enhance the system's resistance to progressive collapse.

3.3.2 Accidental Scenario B. The condition of physical fracture of the beam is examined. First, condition (3) is verified. If it is false, then at a certain maximum value of kinetic energy and under the actual stress–strain state, the beam is capable of sustaining the accidental impact with the formation of plastic hinges. In this case, a kinematic chain is formed, and the element continues to function as part of the frame according to the principle of a cable system. If condition (3) is true, a crack is initiated in the beam. Its propagation leads to rupture of the structural element. For definiteness, in this practical calculation the presence of a micro-defect in the support joint of the roof beam is assumed. An approximate evaluation of the Rice integral in this particular case, considering the I-section profile and the condition of lateral bending, is carried out using the formula obtained by transforming expression (3).

$$J = 2\sum_{i=0} \int^{\Delta_i} \left(\frac{\Delta P_i}{2hb_i} + \frac{\Delta\sigma_{s0i} R_u h}{b_i\, R_s} \right) d_{\Delta_i}\,, \qquad (17)$$

$$\sigma_{s0} = \sqrt{\left(\frac{\sigma_z + \sigma_y}{2}\right)^2 + 3\left(\frac{\sigma_z - \sigma_y}{2}\right)^2 + 3\tau_{zy}^2}\,. \qquad (18)$$

where $\Delta\sigma_{s0i}$ is the increment of equivalent stresses at each integration step, ΔP_i is the increment of accidental load, h, b_i are the section width and the height from the crack mouth to the opposite edge of the section, respectively; R_u, R_s are the temporary and design strengths of steel; Δ_i is the increment of structural deflection corresponding to the value ΔP_i. It was established that under vertical dynamic loading applied to the roof beam (Fig. 2), the magnitude of which after conversion to a static equivalent with a coefficient of 2 amounts to $P_D \approx 120$ kN, an uncontrollable crack growth occurs, leading to rupture of the section. In this case, part of the load is transferred to the beam of the lower story. To describe such a failure scenario, the computational model may be modified by introducing contact elements. However, since these modeling aspects are beyond the scope of the present study, they are not considered here. For simplification, it is assumed that approximately 5/6 of the static equivalent of the accidental load, i.e., about 100 kN, is transferred to the underlying beam through the support node. The modified scheme for analyzing the probabilistic robustness indicator is shown in Figure 7. The corresponding values of the reliability index are presented in Table 7. Even the linear quasi-static calculation (Fig. 7a) demonstrates that rupture of the beam causes a significant additional bending moment in the left column of the two adjacent stories. Nonlinear analysis confirms that redistribution of bending forces also occurs in the first-story beam, where a plastic hinge forms in the support section (Fig. 7b). Taking into account the sequential failure of beam 3 and beam 1, we obtain:

$$W_R = \prod_{i=1}^{8}\left(P_{no,i}\left(1 - P_{dam,i}\right)\right)\left[P_{no,1}\left(1 - P_{dam,1}P_{dam,3}\right)\right] \times$$

$$\times \prod_{j=2}^{5}\left(P_{no,j}\left(1 - P_{dam,j}\right)\right) = 0,6318 > 0,55.$$

In this accidental scenario, the system possesses robustness, which is confirmed by the results of nonlinear analysis.

The right-hand side of the robustness criterion ($W_R > 0,6$) for the load-bearing system can be refined depending on the type of structural scheme and the adopted serviceability criteria. However, the performed calculations showed that for reinforced concrete and steel frame systems of buildings and structures subjected to service loads, this condition is fulfilled.

The question of applicability of preliminary quasi-static assessment for predicting robustness remains debatable. Nevertheless, comparison of results from geometric instability evaluation with plastic hinges and multiple analyses considering physical, geometric, and structural nonlinearities revealed no discrepancies between the forecast and subsequent calculations.

Future research perspectives are associated with extending the developed provisions to other types of frame systems, including high-rise and unique structures. At the same time, it is reasonable to consider not only the design stage aimed at preventing accidental states but also the service period, during which damage accumulation is possible. Such damages may include defects caused by overloading, changes in planning solutions, as well as environmental impacts such as corrosion processes.

As potential accidental scenarios, alongside column removal, standard fire exposures and various degradation patterns of elements due to corrosion may also be considered.

CONCLUSION

1. A methodology for probabilistic robustness assessment of frame systems in buildings and structures has been developed. The approach is based on an indicator that accounts for the probability of maintaining the functionality of part of the structural elements while allowing for failure of individual members in the zone of accidental action localization. Through calculations on steel and reinforced concrete frames, an integral threshold value of the proposed robustness criterion $W_R > 0,6$ was obtained, at which the robustness of frame systems is ensured under typical accidental scenarios.
2. The applicability of the methodology has been demonstrated using different schemes of accidental action localization in a steel frame with I-shaped profiles. This confirms its suitability for analyzing both newly designed and retrofitted steel systems.
3. The method allows evaluation of the effectiveness of various strengthening measures for the load-bearing system and for increasing its resistance to progressive collapse. High effectiveness of diagonal bracing in frames has been established, which is consistent with other research findings. At the same time, the limited efficiency of merely increasing cross-sectional sizes for localizing progressive collapse without forming a kinematic chain has been demonstrated.

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