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Estimation of the Reduction Coefficient When Calculating the Seismic Resistance of a Reinforced Concrete Frame Building after a Fire

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Abstract: The consequences of destructive earthquakes show that the problem of analyzing the response of reinforced concrete frames under seismic loads after a fire is relevant. The calculation models used for individual elements and buildings as a whole must take into account the nonlinear properties of concrete and reinforcement. In the spectral calculation method, the nonlinear properties of materials are taken into account by introducing a reduction coefficient to the elastic spectrum. When determining the reduction coefficient, a common deformation criterion is based on the use of the plasticity coefficient. The seismic resistance of a three-span, five-story reinforced concrete frame under four different fire exposure options is considered. The residual strength and stiffness of frame elements after a fire is assessed by performing a thermal engineering calculation in the SOLIDWORKS software for a standard fire. For the central sections of the elements, the highest temperatures were obtained after heating—during the cooling stage. The reduction coefficient is estimated by performing a nonlinear static analysis of reinforced concrete frames in OpenSees and constructing load-bearing capacity curves. Fracture patterns and damage levels in plastic hinges are analyzed. Based on the numerical modeling of reinforced concrete frames after exposure to fire, it was revealed that the most dangerous scenario is the occurrence of a fire on the first floor of the building. Based on the obtained plasticity coefficients, reduction coefficients were determined in the range of 2.62 to 2.44. The influence of fire on the permissible damage coefficient of a reinforced concrete frame is assessed using the coefficient φ_K —the coefficient of additional damage after a fire, which is equal to the ratio of the reduction coefficients for the control and fire-damaged frames. Depending on the percentage of damaged structures on the first floor, the following values were obtained: 50% or less— $\varphi_K = 1.09$; 100%— $\varphi_K = 1.17$. The obtained coefficients are recommended to be used when assessing the seismic resistance of a reinforced concrete frame after a local fire.



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1. Introduction

Requirements for ensuring the safety of buildings and structures lead to the need for continuous improvement of structural design methods not only for special impacts but also for their combinations. In general, they represent the problem of taking into account special impacts for systems that have initial damage. These include problems of seismic resistance or the survivability of buildings against progressive collapse in fire conditions [1,2], environmental influences when taking into account corrosion of reinforcement and concrete, a combination of seismic forces and fire [3], the survivability of building structures under special impacts, other impacts of a technogenic nature, and earthquake resistance in case of damage accumulation [4]. In general, they represent the problem of taking into account

special impacts for systems with initial and acquired damage. Calculation and design of structures under these impacts refers to a special limit state.

In the standards of the Russian Federation [5], the work of structures beyond the limits of elasticity (i.e., with the implementation of the plastic phase of deformation) is determined by introducing the coefficient K_1 when calculating the seismic load, which is called the “permissible damage coefficient” of building structures and other structures. Essentially, the value of the “allowable damage coefficient” is a characteristic of the limiting state of the structure. Coefficient K_1 varies from zero to one and depends on the level of permissible damage caused by load-bearing structures of various types. In Eurocode 8 [6] contains a “behavior coefficient” of the structure, which has a similar physical meaning.

Studies of the seismic response of buildings of various structural systems [7] showed that buildings in frame-braced structures receive a fairly high average level of damage $d = 2.3$. At the same time, the coefficient of variation for buildings with a frame-braced structural system is $v = 0.37$, which indicates significant deviations of the average value from the actual seismic response. If the coefficient of variation exceeds 0.33, then this indicates heterogeneity of information and the need to exclude the largest and smallest values.

The normalized values of the limit state characteristics through the “permissible damage coefficient” K_1 allow large deviations from the average values, which leads to large losses during a design earthquake. In order to reduce these losses, it is necessary to increase the seismic resistance of buildings and structures at the legislative level.

When calculating and designing structures, it is necessary to take into account the technical condition of buildings and structures before the earthquake, as well as preliminary damage arising from various natural and manmade impacts.

This article examines an example of the response of reinforced concrete frames under seismic loads after a fire.

There are two real scenarios for earthquakes after a fire:

(1) In a building located in an earthquake-prone area, a fire first occurs. Then, after some time of operation, an earthquake occurs. The structures will probably be strengthened by the time of the earthquake, and engineering justification for strengthening measures must be carried out by taking into account fire damage.

(2) The probability of the development and spread of a fire after an earthquake increases sharply, which is associated with damage to automatic fire extinguishing systems, failure of underground communications, difficulty of access of fire brigades to the fire site, and a number of other factors [8]. Thus, the structures initially damaged by the earthquake are exposed to fire. After a short time, a structure damaged by an earthquake and fire may be subject to repeated tremors (aftershocks), leading to the complete exhaustion of its bearing capacity.

The solution to this class of problems is currently available using methods of direct integration of the equations of motion in a nonlinear formulation. However, such methods are quite difficult to apply in design practice. This is due, first of all, to a huge number of factors that can vary from task to task—for example, the level of static load, the nature of the reinforcement, various conditions for the spread of fire, etc. One of the possible engineering approaches may be the linear–spectral calculation method, which forms the basis of design standards in most countries.

The linear–spectral method is the basis for Russian design standards in seismic areas [5]. In the European standard Eurocode 8 [6], it is called modal response spectrum analysis. The beginnings of this method were laid out in the works of M. Biot.

The method is based on the concept of a response spectrum (see Figure 1a), which is a function of the maximum responses of a linear oscillator $S_d(T)$ to the influence of the acceleratorogram of a real earthquake from the period of its own oscillations T at a certain fixed value of the damping coefficient ξ [9].

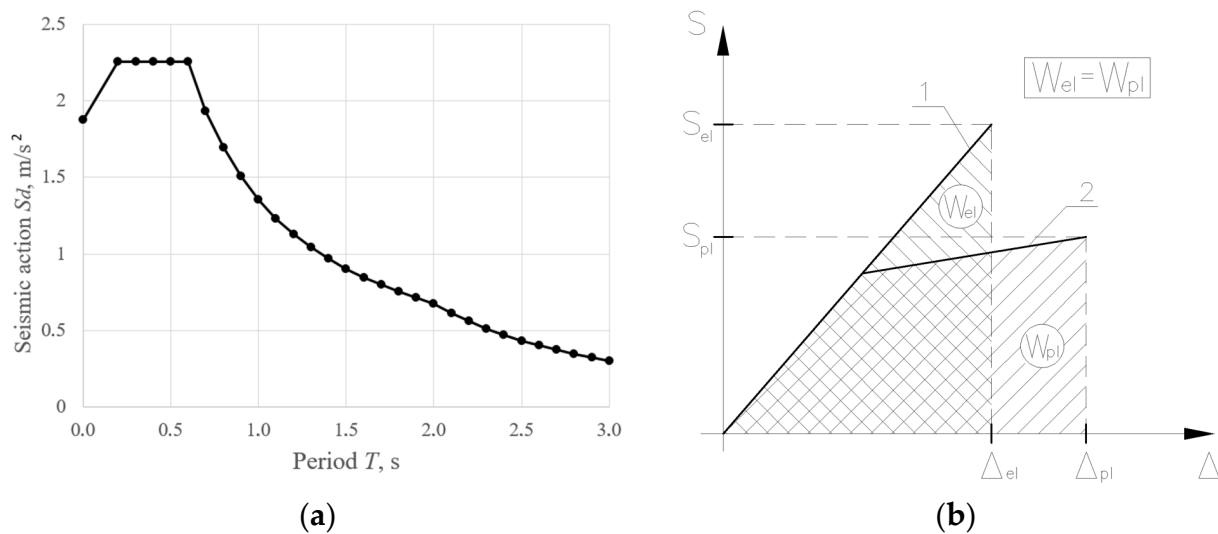


Figure 1. Basic provisions of the spectral technique: (a) calculated response spectrum, (b) scheme for determining the reduction coefficient. 1—linear elastic system; 2—elastic-plastic system.

Each accelerogram can be assigned a response spectrum, which will have its own frequency composition and maximum response. Therefore, in practice, a unified calculated spectrum is used, which is the envelope of the reaction spectra for a representative sample of real accelerograms with a given type of soil condition. In this case, the seismic impact can be applied both uniformly to all support nodes and differentially [10]. In Russian standards, it is customary to use the spectrum of dynamic coefficients $\beta_i(T)$, which represents the spectrum of reactions for a conditional accelerogram with a maximum amplitude equal to $\ddot{y}_0 = 1$.

Next, the maximum reactions are determined for each found form to a unified impact, which is described by a spectrum of reactions or a spectrum of dynamism coefficients [11].

The reaction of the j -th mass of the system m_k^i is equal to:

$$S_{0ik}^j = m_k^i A \beta_i K_\psi \eta_{ik}^j, \quad (1)$$

where η_{ik}^j —acceleration of the k -th mass according to the i -th form in direction j ;

A —acceleration at the base level, taking into account the magnitude of the earthquake;

K_ψ —coefficient taking into account the dissipative properties of the structure.

The calculated value of the reaction is determined as:

$$S_{ik}^j = K_0 K_1 S_{0ik}^j, \quad (2)$$

where K_0 —coefficient taking into account the purpose and responsibility of the structure.

K_1 —coefficient taking into account the permissible damage to buildings and structures.

The coefficient K_1 is inversely proportional to the reduction coefficient R , which is numerically equal to the ratio of the reaction occurring in a linearly elastic system S_{el} to the reaction obtained under the assumption of elastoplastic behavior of this system S_{pl} , provided that the energy expended to obtain these reactions is equal (Figure 1b).

$$R = \frac{1}{K_1} = \frac{S_{el}}{S_{pl}}. \quad (3)$$

According to the approach adopted in the European standards EN 1998-1, the calculated response spectrum is taken as a seismic effect, which is described by the shape coefficients T_B , T_c , and T_d , as well as the soil condition coefficient S . The response spectrum is determined with a damping coefficient of 5% [12].

The ability of the frame to dissipate the energy of seismic vibrations due to the inelastic work of its elements is taken into account by introducing the behavior coefficient q into the equations of the elastic spectrum, which is equal to the reduction coefficient of the system R .

The numerical value of the reduction coefficient R depends on the type of structural system of the building, the plasticity class, and the regularity of rigidities in height and in the plan. It is worth noting that if a fire occurs in one of the compartments, the initially regular building becomes irregular due to a decrease in the rigidity of the elements damaged by the fire.

At its core, the coefficient R is a characteristic of the limiting state of a structure. There are many approaches to its definition: based on reaction, ultimate plasticity, energy, etc. [13]. The most common is the deformation criterion based on the plasticity coefficient μ , the foundations of which are laid in the work of N. Newmark [14], according to which the reduction coefficient depends on the period of the lowest tone of oscillations of the system T and is determined by the formula:

$$R = \frac{1}{K_1} = \begin{cases} 1, & \text{when } T_1 < 0.1 c \\ \sqrt{2\mu_{tot} - 1}, & \text{when } 0.1 \leq T_1 \leq 0.5 c \\ \mu_{tot}, & \text{when } T_1 > 0.5 c \end{cases} \quad (4)$$

where μ is the plasticity coefficient, which is defined as the ratio of the limiting strains of the elastoplastic and elastic systems.

The reduction coefficient R is assigned as a whole for the entire structure and integrally takes into account the elastic–plastic properties of individual structural elements. In the modern edition of the standards, the coefficient of permissible damage is fixed for a specific structural system of a building (with a steel or reinforced concrete frame, reinforced concrete wall system, etc.). As studies show [15], such an approach to reinforced concrete buildings can lead to significant errors, both in the direction of increasing costs and in the direction of reducing the reliability of the design decisions made.

Therefore, each fire impact scheme will have its own behavior coefficient R , the value of which must be clarified using nonlinear methods, which requires additional research [16].

If a building with a reinforced concrete frame is exposed to seismic loads after a fire, a premature loss of the load-bearing capacity of structural elements and collapse of the entire building may occur. The main danger here is associated with the deterioration of the physical and mechanical characteristics of reinforcement and concrete after fire exposure [17].

According to the European norms EN 1992-1-2:2004 [18], the mechanical characteristics of reinforcement and concrete depend on their heating temperature when exposed to fire. Strength and strain characteristics were determined experimentally without considering creep at heating regimes from 2 to 50 K·min^{−1}.

In addition, the values of mechanical properties of reinforcement and concrete under high-temperature heating are regulated in the American standards ACI 216.1-14 [19]. The values of mechanical properties are given in tabular form.

In Russian standards SP 468.1325800.2019 [20], the mechanical characteristics of materials are specified depending on the heating temperature of a given section using the coefficients $\gamma_{bt(st)}$ and $\beta_{b(s)}$. The design resistances and moduli of elasticity of concrete and reinforcement are multiplied by these coefficients. The values of the coefficients are given taking into account a wide base of experimental studies of control specimens heated to different temperatures [21].

Experimental and numerical model tests were carried out to study the effect of fire on the seismic response of reinforced concrete frames. In [22], seismic loading tests were carried out on precast reinforced concrete columns that were preliminarily subjected to fire. It was found out that by the stage of fire exposure of 30, 60, and 90 min, the columns received a lot of mechanical damage in the form of spalling and delamination, which is associated with an increase in pore pressure. This reduced the initial strength and stiffness of the columns.

Steel-reinforced concrete columns exhibit higher plastic properties after fire due to the steel core [23]. It should be noted that the temperature of the inner core of the section is significantly lower than that of the outer layers. As a result, the mechanical characteristics of rigid profiles change insignificantly and are almost completely restored after cooling down. This explains why rigid reinforcement is highly effective in the cross-section of reinforced concrete columns compared to using only flexible reinforcement.

In [24], it is found that the effect of fire before earthquake affected the bearing capacity, deformability, and damping properties of a reinforced concrete single-story and single-span frame. The mechanism failure changed when the loss of bearing capacity occurred along the column. In the undamaged frame, the failure occurred along the beam.

Similar results were obtained when testing a single cell of a multi-story reinforced concrete frame [25]. The frame designed according to the “strong column–weak beam” concept after a fire attack lost its load-bearing capacity due to shear failure of the column.

Temperature cracks in concrete that occur during fire testing reduced the rate of degradation of element stiffness because brittle failure associated with sudden loss of stiffness was eliminated. A key feature of these tests was also the consideration of floor slabs and beams in a perpendicular direction on the distribution of the temperature field during fire exposure. No static load was transferred at the time of the fire. As in similar tests, degradation of seismic characteristics was recorded.

Finite element modeling of reinforced concrete frames under seismic impact after a fire was also carried out. Thus, in [26], the sensitivity of criteria for assessing seismic resistance (forms and frequencies of vibrations, displacements, forces) to different positions of fire exposure was studied using the example of a three-span, five-story reinforced concrete spatial frame. Sixteen options for the position of the fire source were considered. The simulation results were compared with analytical calculations using a cantilever scheme.

The results of the spatial calculation of a 12-story reinforced concrete frame under conditions of a combination of fire and seismic impacts, which were obtained in [27], are interesting. Fire exposure was taken into account in one of the spans on the second floor of the building. An increase in the natural vibration frequencies of the building and the horizontal displacement of the coating slab was obtained. This study examines only one fire pattern.

More schemes are considered in [28]. Five fire scenarios are proposed for fire exposure of variable duration on different floors.

One can also note the results of numerical modeling of a six-story frame building when fire exposure occurs on the first floor [29]. Good agreement was obtained with data from full-scale, low-cycle tests of beams and columns isolated from the frame.

The impact of fire leads to degradation of the seismic resistance parameters of elements [30]. The degradation of rigidity is more pronounced in the ascending section of the hysteresis diagram, since at this stage concrete damaged by fire takes an active part in the work.

In addition, studies were carried out on the actual operation of a reinforced concrete frame assembly under low-cycle loading conditions after a standard fire [31].

In order to perform the analysis of a fire-damaged reinforced concrete frame, it is necessary to take into account the nonlinear performance. In the spectral method, this is done by introducing the reduction factor R . The coefficients specified in the norms do not allow the actual reserve of the plastic work of the building after a local fire to be estimated.

At present, the values of the reduction factor and K_1 coefficient for reinforced concrete frame buildings taking into account the impact of local fire have not been evaluated. This study proposes a procedure for calculating the values of these coefficients obtained by numerical nonlinear static analysis (pushover analysis) of reinforced concrete frames at different positions and fire intensities.

In addition, the question of designating the most dangerous place for a fire to occur remains unresolved. Herein, we talk about the problem of determining the position of the

fire impact (span, floor, etc.) that causes an extreme value of some factors (for example, force) in a given section of the structure.

Enumerating all possible options for the position of the fire source will lead to inappropriate computational costs. The task may also be complicated if seismic isolators are used in the building [32,33]. Thus, clarifying the values of the reduction coefficient for reinforced concrete frames damaged as a result of a local fire is an urgent task.

2. Models and Methods

2.1. Calculation Scheme

As the structure under study, we consider the transverse frame of a five-story, three-bay building with a reinforced concrete frame (Figure 2) with a floor height of 3.50 m and a span length of 6 m. The pitch of the transverse frames is 6 m. The total height of the building is 17.75 m.

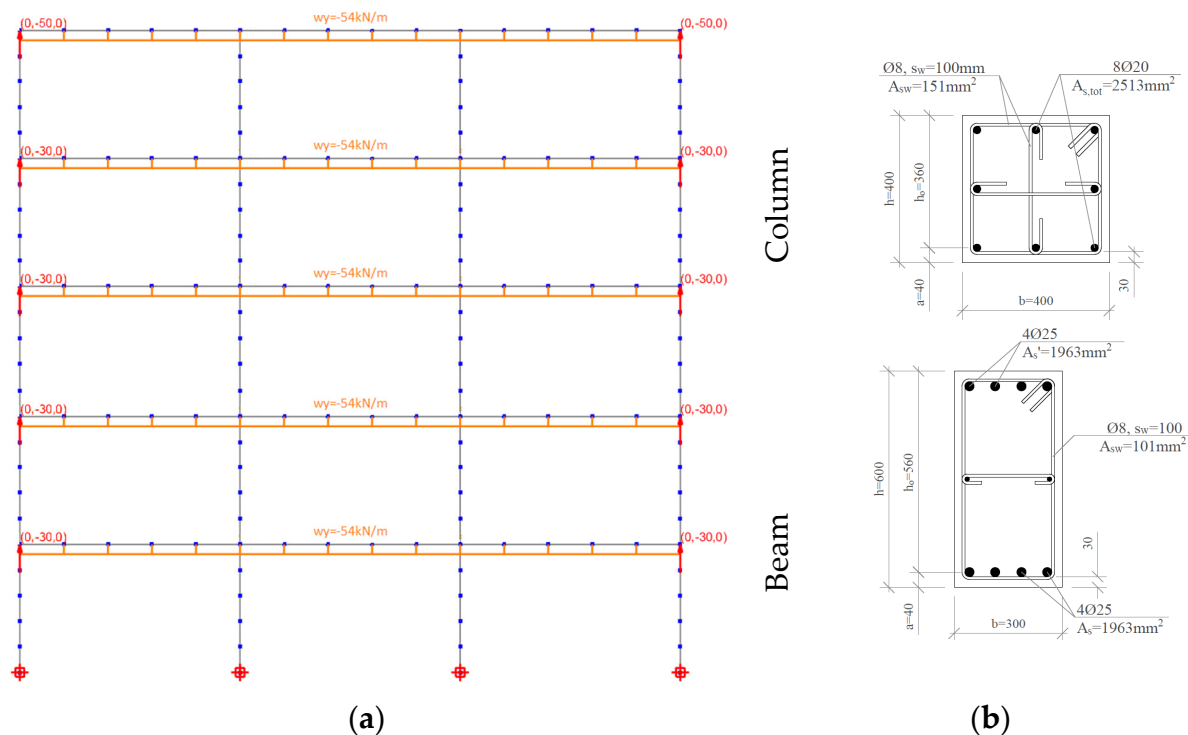


Figure 2. Calculation model of the frame: (a) finite element model in the OpenSees PC, (b) cross-sections of frame elements.

Modeling and calculation of the reinforced concrete frames are performed in an OpenSees v3.5.0 PC. Since the original program has only a text interface, the graphical environment OpenSees Navigator v2.5.8 is used to build a calculation scheme and analyze the results.

The structure of the reinforced concrete floor forms a hard disk that is not deformable in its plane. The movements of the model nodes within each floor are combined. The movements of the nodes are connected only in the direction of the OX axis. The movements in the OZ direction and rotations relative to the OY axis for all nodes are independent. Within frame nodes, the stiffness of the ends of columns and beams is increased to more accurately account for the dimensions of the deformable part of the elements.

Reinforced concrete columns have a square cross-section measuring 400×400 mm. The longitudinal reinforcement of 8 rods is $\text{Ø}20$, $A_{s,\text{tot}} = 2513 \text{ mm}^2$. The transverse reinforcement from bent stirrups is $\text{Ø}8$ with step $s_w = 100$ mm, $A_{sw} = 151 \text{ mm}^2$. The transverse reinforcement coefficient by volume $\rho_s = 0.00378$. The beams have a rectangular cross-section with dimensions of 300×500 (h)mm. The upper and lower longitudinal reinforcement

is symmetrical from $4\varnothing 25$, $A_s = A_s' = 1963 \text{ mm}^2$. The transverse reinforcement from bent stirrups is $\varnothing 8$ with step $s_w = 100 \text{ mm}$, $A_{sw} = 101 \text{ mm}^2$. The protective layer of concrete is 30 mm. The reinforcement class is A500.

The sections are divided into several layers: a protective layer and 6 layers of a concrete core, limited by transverse stirrups. For a column, the protective layer has the mechanical characteristics of ordinary concrete, and the core has the mechanical characteristics of confined concrete.

Section modeling is performed using the Fiber Section tool—the section is divided into individual rectangular fibers and layers of reinforcing bars. Each fiber is assigned the characteristics of its corresponding material.

The transition from the stress diagram in the section to the generalized internal forces is determined using the procedure of numerical integration of stresses over a normal section. In this case, the hypothesis of plane sections is considered valid.

2.2. Characteristics of Reinforcement and Concrete

The mechanical characteristics of the materials correspond to class B25 concrete and A500 reinforcement. For reinforcement and unconfined concrete, bilinear deformation diagrams are adopted, and for confined concrete, a trilinear diagram is adopted (Figure 3). The following characteristics are specified:

- (1) Unconfined concrete: $R_b = 14.5 \text{ MPa}$, $E_b = 9667 \text{ MPa}$, $E_b' = 7037 \text{ MPa}$, $\varepsilon_{b1} = 0.0015$, $\varepsilon_{b2} = 0.0035$;
- (2) Limited concrete: $R_{b,tr} = 18.91 \text{ MPa}$, $E_{b,tr}' = 3933 \text{ MPa}$, $\varepsilon_{b3,tr} = 0.00504$;
- (3) Reinforcement: $R_s = 435 \text{ MPa}$, $E_s = 200,000 \text{ MPa}$, $\varepsilon_{b0} = 0.002175$, $\varepsilon_{b2} = 0.025$.

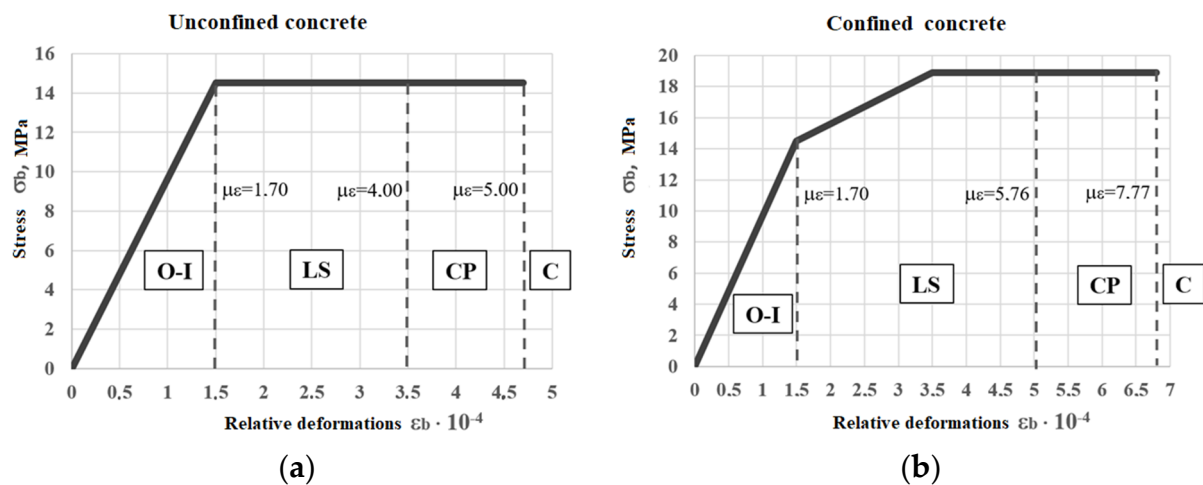


Figure 3. Deformation diagrams of unconfined (a) and confined concrete (b) and levels of permissible damage (where the damage levels are: O-I—operational and immediate occupancy; LS—life safety; CP—collapse prevention; C—collapse).

In the program, the mechanical characteristics of concrete and reinforcement are modeled using the hysteretic material. In addition to the above mechanical and deformation parameters, to determine concrete models, the following are set: coefficients $p_\varepsilon = 0.7$ and $p_\sigma = 0.8$ to take into account the pinching effect; coefficient $\beta = 0.375$, which characterizes the decrease in initial stiffness with an increase in the proportion of plastic deformations; low-cycle stiffness degradation coefficient $D_1 = 0.08$. The mechanical characteristics of reinforcement and concrete within each layer are constant and are adjusted depending on the maximum heating temperature of a given layer by introducing coefficients $\gamma_{bt(st)}$, $\beta_{b(s)}$, and θ_b .

2.3. Loads, Fire Effects, and Parameters of Thermal Engineering Calculations

Vertical permanent and temporary loads are applied to the beams. Constant loads are the dead weight of the structural elements of the building, the weight of the floor and covering structure, and the weight of wall fences.

The dead weight of structures is taken into account automatically in the OpenSees PC. The specific gravity of reinforced concrete structures is assumed to be 25 kN/m^3 .

The total uniformly distributed load on the beams is assumed to be 54 kN/m . The weight of the wall fences is specified in the form of concentrated forces on each floor: 30 kN for the interfloor floor; 50 kN per coating.

The different positions of fire sources are analyzed both along the height of the frame and in the plan. The considered options for fire effects are presented in Figure 4. In addition, an option was considered in the absence of fire exposure FNF. In general, there are five calculation options for the reinforced concrete frame under study.

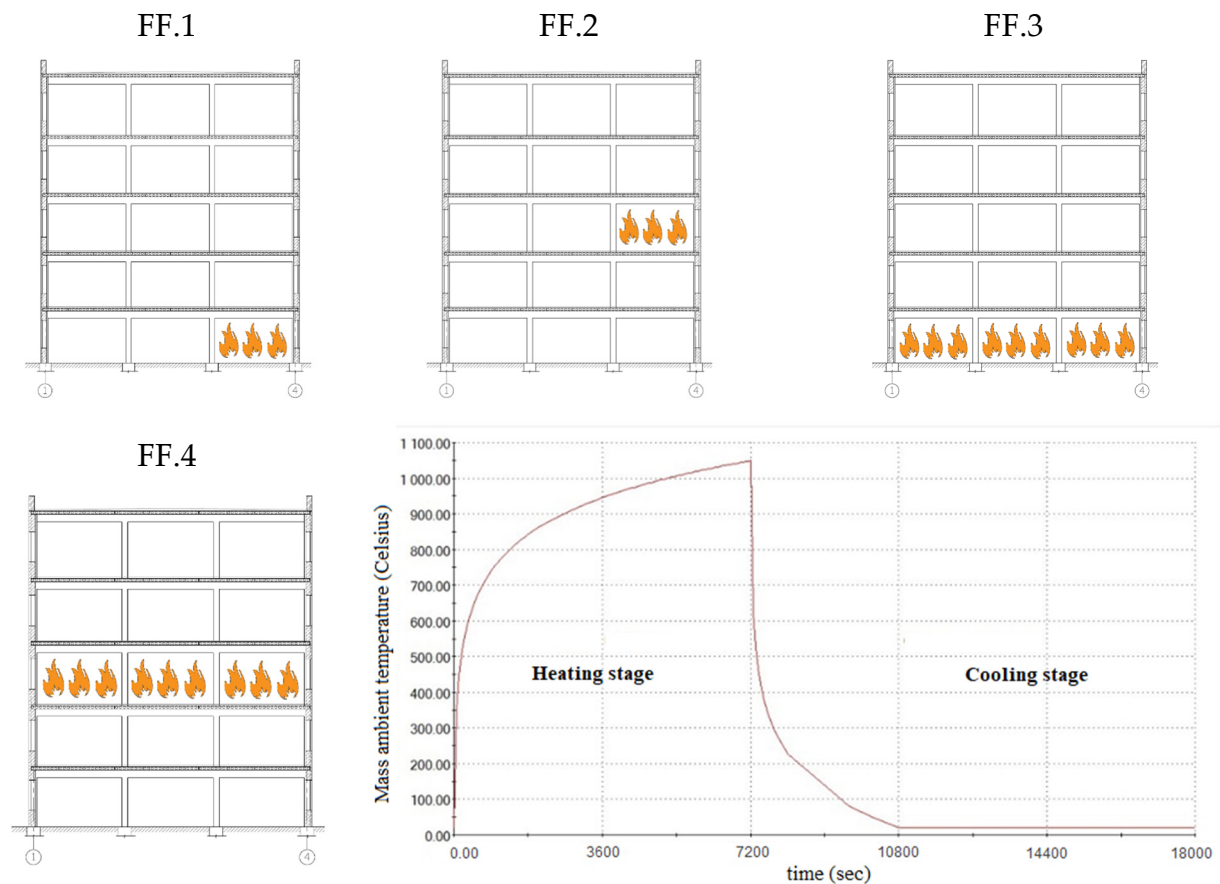


Figure 4. Fire exposure options and standard temperature curve taking into account the cooling stage.

It is accepted that only two columns and a beam above the fire are damaged by fire. It is assumed that the middle columns are damaged by fire evenly on four sides, and the outer columns and floor beams on three sides. The temperature fields in the sections of reinforced concrete elements correspond to the impact of a standard fire according to ISO 834 [34] with a duration of 120 min.

A preliminary thermal calculation is carried out in order to obtain temperature fields in the sections of frame elements after exposure to a standard fire lasting 120 min. Modeling is performed in the SOLIDWORKS PC 2023 SP4.0. The calculation was performed in a three-dimensional formulation using volumetric finite elements of the tetrahedron type, and the maximum side size of the finite element is 30 mm.

The thermal characteristics of the materials are assumed to be the same for all samples depending on the temperature: for reinforcement, $\lambda = 0.58 - 0.0048t$ and $\lambda = 0.48 - 0.00063t$;

for concrete, $\lambda = 1.14 - 0.00055t$ and $\lambda = 0.71 - 0.00083t$. The density values of the concrete and reinforcement are considered independent of the temperature and equal to $\rho_b = 2400 \text{ kg/m}^3$ and $\rho_s = 7850 \text{ kg/m}^3$, respectively.

The following thermal loads are taken: the initial temperature of the element and the temperature of a standard fire. The initial temperature is uniform throughout the frame and amounts to 11.4°C . Fire exposure is taken according to the standard temperature curve according to ISO-834 (see Figure 4). The thermal analysis also takes into account the cooling stage; it is assumed that cooling to the initial temperature occurs in 1 h. The fire effect is applied to the element according to the convective mechanism, and the convective heat transfer coefficient $h = 25 \text{ W/(m}^2\cdot\text{K)}$, which corresponds to natural convection at low air speed.

2.4. Nonlinear Static Analysis

To determine the actual level of system ductility reserves at a given level of permissible damage, a nonlinear static method can be used [35]. The method is based on constructing a load-bearing capacity curve of the system in the axes “horizontal force–displacement of the coating slab” under quasi-static lateral loading with identification of zones of formation of plastic hinges. The load-bearing capacity curve is constructed by numerical modeling, taking into account the spatial layout of the building, geometric, and physical nonlinearity.

After constructing the curve, the actual value of the system’s plasticity coefficient μ_{tot} is estimated:

$$\mu_{tot} = \frac{\Delta_{LS}}{\Delta_y}, \quad (5)$$

where Δ_{LS} —the maximum deformation of the building corresponding to the selected level of permissible damage. In this case, the life safety (LS) level is adopted.

Δ_y —conditional deformation corresponding to the transition to the elastic–plastic section of the bilinear diagram (see Figure 5a).

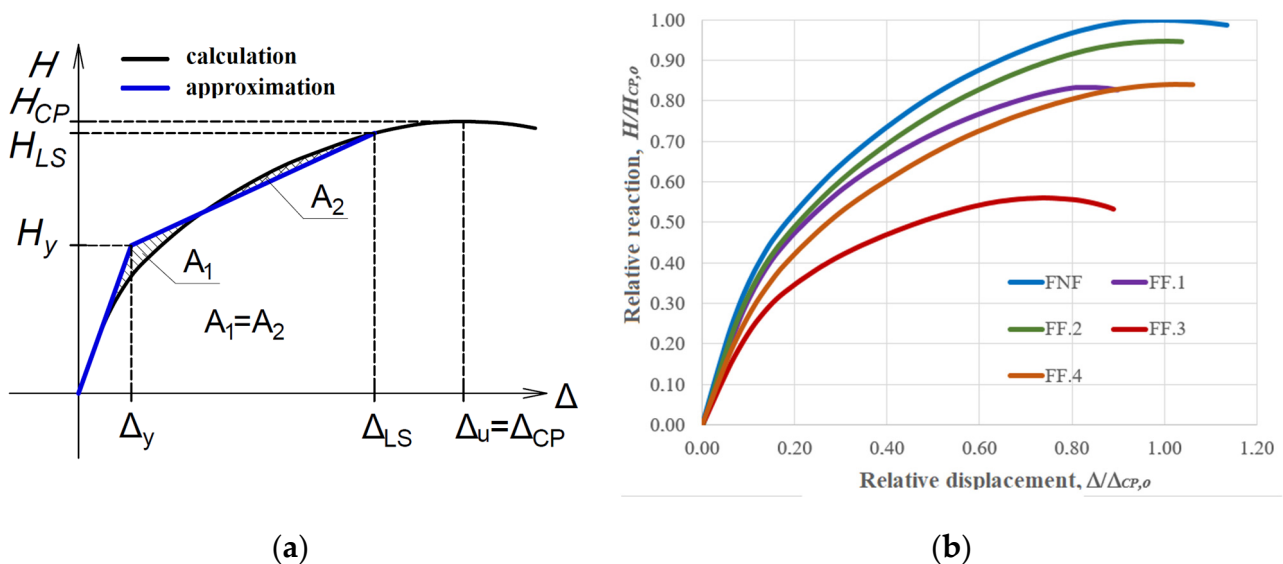


Figure 5. Bearing capacity curves based on the results of numerical calculations in the OpenSEES PC: (a) the rule of bilinear approximation of the bearing capacity curve; (b) bearing capacity curves in relative units.

Next, the reduction coefficient of the system R is determined. In this study, we will use N. Newmark’s deformation criterion (see Formula (4)).

Nonlinear analysis of reinforced concrete frames consists of several successive stages. First, a nonlinear static calculation of the frame is performed for vertical loads applied to

the floors. The law of application of loads is linear static. The number of loading steps is 100. The solution algorithm corresponds to Newton's method, and integration is carried out with force control.

Next, the shapes and frequencies of natural vibrations of the frame are determined via modal analysis. The analysis is performed taking into account the deformations obtained from the static analysis. The number of vibration modes taken into account is five.

Then nonlinear static analysis is performed. A displacement is applied to the reinforced concrete frame at the level of the covering slab. The displacement increases until the load-bearing capacity is completely lost. Deformations of the ground of the base are not taken into account [36]. For columns, the longitudinal bending effect (P- Δ effect) is taken into account.

3. Results

3.1. Thermal Analysis Results

Based on the calculation results in the SOLIDWORKS software, temperature fields were obtained in the sections of the elements in Figure 6. The maximum temperature on the surface of the elements at the end of the heating stage was 1056 °C.

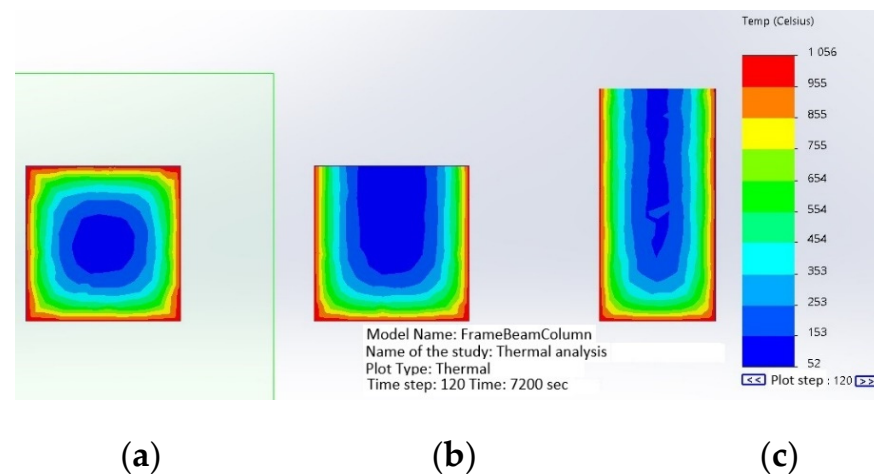


Figure 6. Distribution of temperature fields across sections at the end of the heating stage for (a) columns with 4-sided fire exposure; (b) the same, with 3-sided fire exposure; and (c) beams with 3-sided fire exposure.

After 120 min of standard fire for all elements, the temperature increase in the center of the sections continued. This is explained by heat transfer from the more heated peripheral regions of the section to the less heated inner regions. The temperature increase in the cooling phase for the extreme and corner regions of the cross-section was insignificant in relation to the central regions. The envelopes of the temperature fields at each point under consideration, taking into account the cooling stage, were used in further calculations.

3.2. Modal Analysis Results

Based on the results of modal analysis, the first five vibration modes of the reinforced concrete frames were determined at different positions and intensity of fire exposure. The results of the modal analysis are shown in Figure 7 and Table 1. All modes were bending.

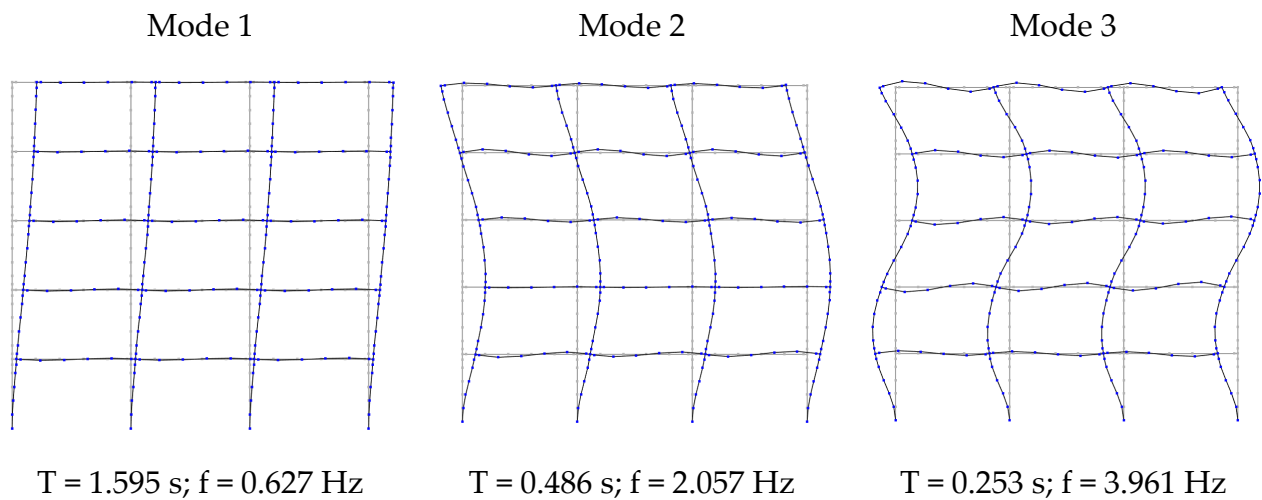


Figure 7. Vibration modes of reinforced concrete frame FNF.

Table 1. Modal analysis results.

Parameter	Mode Number	Frame Brand				
		FNF	FF.1	FF.2	FF.3	FF.4
Period T , s	1	1.595	1.671	1.649	1.773	1.706
	2	0.486	0.505	0.499	0.524	0.510
	3	0.253	0.260	0.259	0.266	0.265
	4	0.156	0.160	0.158	0.164	0.160
	5	0.113	0.114	0.116	0.112	0.119
Frequency f , Hz	1	0.627	0.598	0.607	0.564	0.586
	2	2.057	1.981	2.001	1.910	1.960
	3	3.961	3.847	3.857	3.758	3.769
	4	6.405	6.245	6.321	6.108	6.243
	5	8.907	8.785	8.647	8.674	8.399

The period of the first mode of oscillation did not exceed 2 s. According to the classification given in [37], the system can be classified as a frame of medium flexibility.

When a fire occurred in the outermost cell of the first floor (frame FF.1), an increase in oscillation periods was observed, which indicates an increase in the flexibility of the system. The longest periods were obtained for the frame in which the fire occurred in all cells of the first floor (FF.3). The periods of vibration of frames in which a fire occurred on the third floor also increased, however, less intensely than during a fire on the first floor. The degree of damage to structures also affected the value of the period of natural oscillations.

3.3. Results of Nonlinear Static Analysis and Estimation of the Reduction Coefficient

According to the calculation results, the largest bending moments occurred in sections of columns and beams adjacent to the frame nodes. The values of the bending moments in the beams indicate that the maximum load-bearing capacity was reached in the extreme sections; that is, plastic hinges were formed in the sections. Moreover, the formation of plastic hinges in the beams was observed both for the control frame and for the FF.3 frame.

As a result of damage caused by the fire on the first floor, there was an additional increase in bending moments in the columns and beams of the reinforced concrete frame, which were not affected by the fire. The process of the adaptability of the frames damaged by local fire to the external impact was observed.

Figure 8 shows diagrams of the formation of plastic hinges in the reinforced concrete frames. The hinges were grouped depending on the damage levels achieved by unconfined or confined concrete in the section. The following damage levels were accepted: minor damage (OI), moderate damage (LS), severe damage (CP), complete destruction (C).

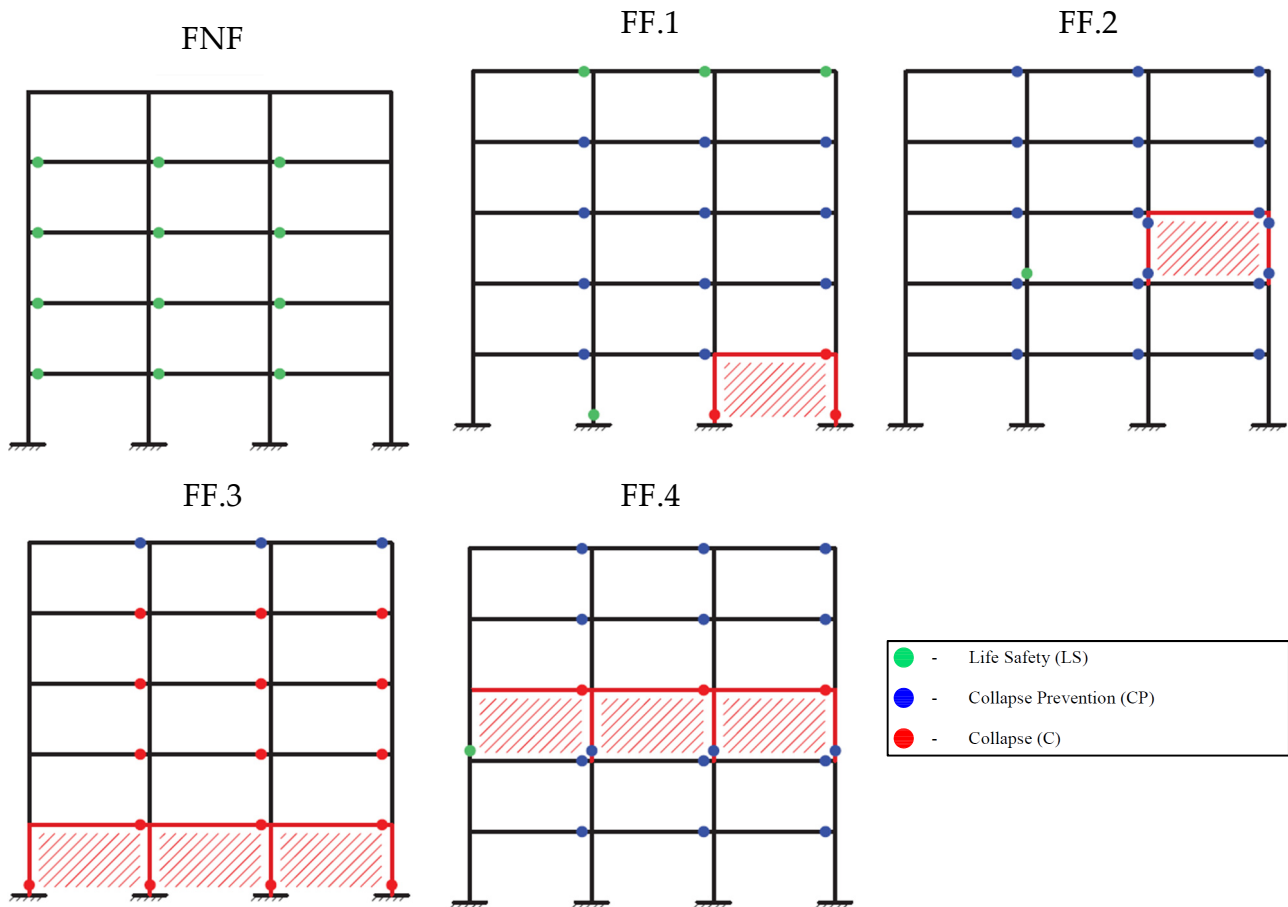


Figure 8. Failure patterns of reinforced concrete frames and damage levels in plastic hinges.

Load-bearing capacity curves for the frames under consideration, constructed from the results of numerical nonlinear static analysis, are given in Figure 5b. Horizontal thrust H and displacement Δ were normalized to the maximum values of thrust and displacement $H_{LS,o}$ and $\Delta_{LS,o}$, obtained by calculating the control frame for damage level LS.

In all cases, fire exposure led to a decrease in the maximum horizontal reaction and frame deformation. The greatest reduction in load-bearing capacity and ductility was observed for the FF.3 frame. The local fire had the least impact on the load-bearing capacity curve for frame FF.2.

The ultimate ductility of frames FF.2 and FF.4 decreased slightly, which indicates a high reserve of ductility of the system in the event of a local fire on the middle floors. However, the reduction in load-bearing capacity for these frames turned out to be more noticeable.

Table 2 shows the results of calculating the reduction coefficients R for reinforced concrete frames depending on the position of the fire exposure. The R coefficients were assessed using two criteria according to Formula (4) with $R = (2\mu_{tot} - 1)^{1/2}$ and $R = \mu_{tot}$. In the second case, larger values of the R coefficients were obtained; that is, a higher level of plasticity was allowed. When assessed using the first expression, the values of the obtained coefficients turned out to be quite close to the standard values: $R = 2.85$ or $K_1 = 0.35$.

Table 2. Results of calculations of the reduction coefficient for reinforced concrete frames.

Frame	Period T_1 , s	H_{CP} , kN	H_{LS} , kN	Δ_{CP} , mm	Δ_{LS} , mm	Δ_y , mm	μ_{tot}	R from (4)	φ_K
FNF	1.595	162.2	158.9	303	255	44	5.923	$\frac{3.293}{5.923}$	1
FF.1	1.671	135.3	131.6	291	245	48	5.087	$\frac{3.029}{5.087}$	$\frac{1.09}{1.17}$
FF.2	1.649	153.6	149.9	302	252	48	5.185	$\frac{3.061}{5.185}$	$\frac{1.08}{1.14}$
FF.3	1.773	90.7	88.4	224	186	42	4.444	$\frac{2.809}{4.444}$	$\frac{1.17}{1.33}$
FF.4	1.706	136.4	129.9	302	251	50	5.064	$\frac{3.021}{5.064}$	$\frac{1.09}{1.17}$

Note: The values above the line are given taking into account Formula (4) at $R = (2\mu_{tot} - 1)^{1/2}$ and the values below the line at $R = \mu_{tot}$.

4. Discussion

4.1. The Influence of a Local Fire on the Destruction Patterns of Reinforced Concrete Frames

Damage caused by fire has a significant impact on the failure patterns of reinforced concrete frames. The formation of plastic hinges in the FNF control frame occurred in accordance with the classical “strong column/weak beam” concept; that is, the zones of plastic deformation were concentrated in the extreme sections of the beams. The columns operated in the elastic region. Thus, the geometric immutability of the frame as a whole was not violated. The level of damage did not exceed LS.

For frames damaged by fire, the formation of plastic hinges in the columns was recorded. In the case of frame FF.1, only the columns located in the damaged cell and one neighboring column entered the plastic stage of operation. In this case, the consequences of the fire were localized. However, the elements in the damaged cell entered the stage of complete destruction—collapse.

For the FF.2 frame, columns located outside the fire impact zone were included in the plastic work. Here, the adaptation of the structural system to external influences was manifested. Moreover, for the FF.2 frame, the damage levels in all elements were within acceptable limits, and the geometric immutability of the system was not violated. The greatest damage was recorded in the FF.3 frame. Moreover, significant levels of damage were recorded for all elements exposed to fire, as well as for most beams.

In the case of a fire on the third floor, a more effective redistribution of forces occurred in all cells (frame FF.4). In all columns, including those exposed to fire, the level of damage did not exceed the permissible level (CP). However, the beams at the source of the fire went into a state of complete destruction.

Columns needed to have less acceptable plasticity to prevent the development of plastic hinges in them and the transition of the system to a geometrically variable state. However, for beams, the formation of plastic hinges was acceptable within the accepted damage threshold.

For columns damaged by fire, the plastic work reserve was reduced. However, the occurrence of plastic hinges was acceptable in some cases—for example, if the fire affected locally only one cell of the frame.

In addition, as can be seen from the diagrams in Figure 8, the development of plastic deformations in the columns contributed to the redistribution of forces in the system.

For example, the FF.2 frame, even when plastic hinges occurred in the column, the overall geometric invariability was maintained, and the level of damage in all hinges remained within acceptable limits (CP stage).

4.2. Taking into Account Additional Damage from Fire When Reducing the Elastic Spectrum

The above circumstances justify the use of a differentiated reduction coefficient R for different structural elements. It is necessary to classify the elements by ultimate ductility. Then, it is necessary to assign the values of differentiated coefficients R at given levels of permissible damage.

For columns, the level of damage at the LS stage should be limited and the value of the plasticity coefficient should be taken as $\mu_\epsilon \leq 3.0 (R = 2.24)$. For beams, a higher level of damage can be allowed by assigning a plasticity coefficient of $\mu_\epsilon \leq 4.0 (R = 2.65)$ while allowing severe damage of $\mu_\epsilon \leq 5.0 (R = 3.0)$. Moreover, in any case, the geometric immutability of the system should not be violated.

However, it is worth noting that according to the current standards of most countries, the reduction coefficient R is the same for all load-bearing elements of the system. Thus, in order to comply with the standard methodology, it is necessary to evaluate the value of the single (integral) coefficient R and the influence on its value of different positions and the intensity of fire exposure (see Table 2).

The influence of fire on the coefficient of permissible damage to a reinforced concrete frame is assessed using the coefficient φ_K —the coefficient of additional damage after a fire:

$$\varphi_K = \frac{R}{R^t} \text{ or } \varphi_K = \frac{K_1^t}{K_1} \quad (6)$$

where R^t and K_1^t —the reduction coefficient and the coefficient of permissible damage for the frame after fire exposure, respectively.

According to the calculation results, the highest value of the coefficient φ_K was obtained for frame FF.3 when all three cells of the first floor were exposed to fire. The fire in the outermost cell of the third floor (frame FF.2) had the least impact on the allowable damage to the frame. It is worth noting that the coefficient φ_K significantly depends on the criterion for assessing the reduction coefficient.

When calculating a reinforced concrete frame at the design stage using the spectral method, it is recommended to take into account damage from a possible fire during the operation stage using the worst of the resulting options—a fire on the first floor in all cells (frame FF.3). The presence of several fire compartments at one floor level should be taken into account, and the dynamic nature of the fire and the spatial distribution of the temperature fields should be considered [38,39]. In this case, only part of the structures will be damaged, and the overall ductility of the building will be reduced to a lesser extent.

Taking into account the above, depending on the percentage of damaged vertical load-bearing structures (columns), it is recommended to use the following coefficients φ_K : in case of fire damage to all structures at the first-floor level— $\varphi_K = 1.17$; if 50% of structures are damaged $\varphi_K = 1.09$. In this case, the percentage of damaged structures is taken according to the size of the fire compartment. It is necessary to divide the reduction coefficient R by the values of φ_K or multiply the coefficient K_1 for a frame undamaged by fire.

For these values of the parameter φ_K , the reduction coefficient R should be taken in the range of 2.62 to 2.44 with the corresponding percentage of damaged vertical structures of the first floor (from 50% to 100%). If the percentage of damaged structures is less than 50%, it is recommended that the R coefficient be no more than 2.62. The base coefficient is taken to be $R = 2.85$ or $K_1 = 0.35$.

The application of the obtained values of the reduction factor is valid for buildings of low and medium floor area (1–6 floors) with a reinforced concrete frame without diaphragms. In the presence of diaphragms, i.e., for buildings with a ligament frame, additional research is required.

The proposed coefficients can be used only for preliminary assessment of the seismic resistance of a building damaged by fire using the linear–spectral method and should be additionally specified by calculation in each specific case.

4.3. Recommendations for Increasing the Seismic Resistance of Reinforced Concrete Frame Taking into Account Possible Fire Damage

The seismic resistance of the reinforced concrete frame was most affected by the fire on the first floor in all three cells. In this regard, at the design stage it is advisable to increase the fire resistance of vertical load-bearing structures of the lower floors. This is achieved primarily by constructive measures: increasing the thickness of the protective layer of concrete, the use of heat-resistant concrete, or the use of concrete on lime aggregate.

To increase the fire resistance of reinforced concrete columns, the use of rolled steel profiles as rigid reinforcement looks promising [23]. The effect of high-temperature heating on the profile is limited due to the presence of a larger protective layer of concrete than that of flexible reinforcement.

Increasing the seismic resistance of the reinforced concrete frame as a whole can be achieved by reserving the bearing capacity of the elements. In this case, a local fire in any compartment of the building will lead to redistribution of forces from the damaged elements to the elements not affected by the fire impact. Reserving the load-bearing capacity will increase the adaptability of the building in the event of an earthquake.

In this case, the fire should be limited in a localized part of the building—the fire compartment. It is recommended to additionally take into account the rule that the size of the fire compartment should be determined based on the condition that no more than 50% of the vertical load-bearing structures fall into it. If the floor area of the building is large, the percentage of structures should be reduced.

5. Conclusions

1. A study was carried out of the resistance parameters of reinforced concrete frames to horizontal loads at different positions of fire exposure using the example of a three-span, five-story reinforced concrete frame: Fire exposure reduced the rigidity of the frames in the horizontal direction, the forces in the elements of the frame exposed to fire were redistributed under the influence of lateral load, and fire influenced the pattern of frame destruction.

For fire-damaged frames, the frequency of the first vibration tone decreased by up to 10% (for frame FF.3) relative to the control frame. The maximum displacement amplitude increased most intensively with fire in all three cells, namely, by 33% for FF.3 and by 21% for FF.4. For the above frames, an asymmetry of vibrations was also observed.

2. Based on the numerical modeling of reinforced concrete frames after exposure to fire, it was revealed that the most dangerous scenario was the occurrence of a fire on the first floor of the building. The number of damaged vertical load-bearing elements was also significant. To ensure the geometric immutability of the frame under seismic influence, the columns needed to have a lower permissible plasticity to prevent the development of plastic hinges in them. However, for the beams, the formation of plastic hinges was acceptable within the accepted damage threshold.

The ultimate horizontal response for frames damaged by fire decreased by 16.6% for FF.1, 5.3% for FF.2, 44.1% for FF.3, and 15.9% for FF.4. The ultimate elastic–plastic deformations relative to the control frame decreased by 4% for FF.1 and 26.1% for FF.3. However, for FF.2 and FF.4, the ultimate strains did not change significantly.

3. The values of the coefficient for taking into account additional damage from fire φ_K were assessed by analyzing the load-bearing capacity curves obtained from the results of numerical calculations. These coefficients were obtained from consideration of the most unfavorable place for a fire to occur—on the first floor of the building. Depending on the percentage of damaged structures, the following values were obtained: 50% or less— $\varphi_K = 1.09$; 100%— $\varphi_K = 1.17$, which corresponds to reduction coefficients R equal to 2.62 and 2.44, respectively.

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