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ANALYSIS OF NONLINEAR STATIC-DYNAMIC DEFORMATION OF REINFORCED CONCRETE FRAMES IN OUT-OF-LIMIT STATES *

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Statement of the problem. The article presents a computational model, an algorithm for computational analysis and the results of calculating the nonlinear static-dynamic deformation of reinforced concrete frames in out-of-limit states caused by the sudden removal of one of the supporting structures.

Results. To design a numerical model of the static-dynamic loading mode of a structural system, the LS-DYNA software package was used that makes use of a detailed 3D model implementing an explicit finite element method. During the computational analysis, the physical and mechanical characteristics of the deformation of materials were taken in three variants: those obtained based on the experimental data by G. A. Geniev, the experimental data by N. V. Fedorova and M. D. Medyankin under the static-dynamic uniaxial testing mode of a limited number of standard samples of prisms and according to the Russian standards SP (CII) 385.1325800.2018.

Conclusions. Numerical analysis of the static-dynamic deformation of the reinforced concrete framed system of a multi-storey building has established that the differentiated accounting of the quantitative value of the concrete viscosity modulus and, accordingly, the time and level of static-dynamic loading of the structure allows one to identify the criteria for the special limit state of the elements of reinforced concrete structural systems of buildings and structures in a more rigid manner.

Keywords: reinforced concrete, progressive collapse, survivability, special limit state, nonlinear dynamic analysis, static-dynamic deformation.

Introduction. As problems aimed at studying the problem of survivability of buildings and structures and reducing the number of accidents during the operation of building structures need to be addressed, a number of new associated scientific issues have emerged calling for further examination both for theoretical substantiation of individual provisions and their practical application in the calculation of structures [1, 5, 17]. One of them is to identify the stress-strain of reinforced concrete structural systems immediately following dynamic loading caused by failure of individual elements considering the specific features and time of extra loading of structural system elements under the discussed impact [6, 14, 19].

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While calculating reinforced concrete structures for progressive collapse from such an impact, static or dynamic diagrams of deformation of materials have been employed so far, which does not correspond to the discussed loading modes. Therefore this paper presents the results of the analysis of nonlinear deformation of reinforced concrete frames in out-of-limit states caused by a special emergency impact with the use of the calculated parameters of the experimentally obtained diagrams of the static-dynamic deformation of concrete.

1. Theoretical justification. Back in the 1960s, in the works by Prof. G. A. Geniev, according to experimental studies of low- and middle-class concrete and their analysis, a deformation theory of plasticity of concrete and reinforced concrete in a complex stressed state was developed [3].

The major physical dependence of the version of the deformation theory of concrete plasticity applied can be represented as follows:

$$T = G_0 \left(1 - \frac{\Gamma}{2\Gamma_s} \right) \Gamma, \quad (1)$$

where $T = (1/\sqrt{6}) \cdot \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}$ is the intensity of tangential stresses,

G_0 is the initial shear modulus, $\Gamma = \sqrt{2/3} \cdot \sqrt{(\varepsilon_1 - \varepsilon_2)^2 + (\varepsilon_2 - \varepsilon_3)^2 + (\varepsilon_3 - \varepsilon_1)^2}$ is the intensity of shear deformation, Γ_s is the limited intensity of shear deformation. Due to its versatility, flexibility, and ease of use, this deformation theory has been applied in solving a number of practical problems of calculating complexly stressed reinforced concrete structures [9].

Developing this theory while solving dynamic problems, G. A. Geniev [4] set forth a relatively simple and sufficiently rigorous for engineering calculations rheological mathematical model for identifying the dynamic strength of concrete under the action of external dynamic loads lasting no longer than tenths and hundredths of a second. The model consists of parallel-connected elements A and B. The first element A is described by the deformation theory of concrete plasticity (1). The viscous Newtonian element B causes an increase in the dynamic strength of concrete and is characterized by a constant K which is the modulus of viscous resistance. This model leads to the solution of the nonlinear Riccati equation that enables an analytical relationship to be established between the dynamic action time t_d and the corresponding dynamic strength limit $\varphi_b^d = T^d / T$:

$$t_d = \frac{2}{\omega} \cdot \frac{\text{arcctg} \sqrt{\varphi_b^d - 1}}{\sqrt{\varphi_b^d - 1}}. \quad (2)$$

Dependence (2) includes a parameter ω whose value is identified experimentally for a particular class of concrete — from a limited number of uniaxial dynamic tests. It should be noted that according to G. A. Geniev, the average value of this parameter of the experimental data available at that time for heavy low-grade concretes under dynamic loading was approximately taken equal to $\omega = \pi \cdot 10^2 = 314 \text{ c}^{-1}$. However, for the above problems of calculating structures for progressive collapse, this parameter of concrete deformation does not completely reflect the mode in question: static loading of structures with an operational load followed by dynamic extra loading by an emergency effect. For such a loading mode of high-strength concretes and particularly high-strength concretes, there are no experimental data for identifying the viscosity modulus under dynamic as well as static-dynamic loading conditions.

At the Research Scientific University Moscow State University of Civil Engineering under the supervision of Prof. N. V. Fedorova a new method of testing prototypes of prisms made of concrete of different strengths under such loading conditions was developed and tests of static-dynamic deformation of ordinary and high-strength concrete were performed [7]. The values of the viscous resistance modulus of concrete are experimentally identified $K(\eta)$ as well as a parameter at different levels of static loading of concrete. Based on the obtained experimental data, an analytical design model was designed for identifying the dynamic strength of concrete depending on the level of its static loading $\eta = T^{st} / T_s^{st}$, $\eta \in [0, 1]$ and time of extra dynamic loading [8]:

$$t_d = \frac{2}{\omega} \cdot \frac{\text{arccctg} \sqrt{\frac{\phi_b^{st-d} - \eta}{1 - \eta} - 1}}{\sqrt{\frac{\phi_b^{st-d} - \eta}{1 - \eta} - 1}}. \quad (3)$$

The developed computational model is based on a working hypothesis about the affinity-like behavior of concrete deformation diagrams under static and dynamic loading. The calculation results for the suggested version of the deformation model are in good agreement with the results of experimental studies of prototypes of concrete prisms [8].

The dynamic calculation of buildings and structures for progressive collapse, described in some research papers [2, 18], is considered the most challenging but also the most accurate method for assessing the stress-strain of a structure under the static-dynamic mode of its loading. The major difficulties of such a calculation are associated with the rapid deformation and failure of reinforced concrete structures. The authors make use of software packages that implement the finite element method using an explicit integration scheme for the equation of

dynamics. However, such programs use dynamic methods that do not allow problems under static loading of the structure to be solved. The disadvantage of the explicit integration scheme while modeling the first stage of static loading of the process under study is that the «slow» static load is perceived by the system as a shock, which does not correspond to the physics of the above process.

To eliminate this disadvantage as well as undesirable vibrations of the investigated structural system under the influence of a static operating load, [2] describes the mode of static-dynamic loading of the structural system using the LS-DYNA software package. In this mode, during a sufficiently long period of time from $t = 0$ to t_1 , constant and temporary long-term loads were applied and gradually increased to their specified values. Then, the structural system was kept under this load for $t = t_2 - t_1$ to stabilize the stress state in it under the applied loads. This modeling approach is fairly simple and straightforward, but there are downsides associated with the volume of the calculation and the limited resource of the computer due to the need for a significant investment of time for integrating the equations as well as due to the impossibility of ensuring the quasi-static state of the structural system at the first stage of its static loading. On top of that, the use in [2] of a constant value of the coefficient of dynamic increase in the strength of materials over the entire range of two-stage static-dynamic loading leads to a rather rough estimate of the stress-strain of the structure under the discussed two-stage loading mode.

Considering the above new scientific achievements of domestic and foreign scientists in the field of concrete and reinforced concrete mechanics, it appears essential to make use of refined experimental parameters of concrete deformation under the considered static-dynamic loading regime in the problems of calculating buildings and structures for progressive collapse. Thus the goal of this work is to analyze nonlinear static-dynamic deformation of reinforced concrete frames in extreme states by means of actual experimental diagrams of static-dynamic concrete deformation. As an example, the results of a numerical study of a three-dimensional frame of a monolithic reinforced concrete building, subjected to a special effect in the form of a sudden removal of one of the load-bearing columns of the first floor considering the specific features of static-dynamic deformation of concrete are presented. To eliminate the noted downsides of the design analysis of structures under the discussed two-stage loading mode, this paper presents a new more universal technique for modeling the process of static-dynamic loading of a structure using the dynamic relaxation method and two-stage design models.

2. Designing a computational model. To implement the suggested technique, the frame of a reinforced concrete monolithic 30-storey residential building is discussed which has a cruciform shape in plan with overall dimensions of 27.0×48.0 m (Fig. 1a). The height of the floor is 3 m. The height of the building is 90 m. The columns of the frame have a square section of 750×750 mm. For the perception of horizontal loads, monolithic reinforced concrete diaphragms of stiffness with a thickness of 200 mm are used. The floor slab, 150 mm thick, is reinforced with a cross system of stiffeners with a cross-sectional area of 150×300 mm located at the bottom of the slab. External walls and internal partitions are made of lightweight materials and supported by floor slabs. The safety factor in the design scheme does not include building elements that are not load-bearing during normal operation. The building rests on a rigid, non-deformable foundation. While calculating for progressive collapse, a variant of the secondary design scheme was considered (in the terminology of the Russian norm SP (CII) 385.1325800.2018) for scenarios of special impact: the inner column B2 on the first floor was removed from the design model (Fig.1b).

Fig. 1. Construction scheme (a) and the plan of the first floor (b) of reinforced concrete building frame

Numerical studies of the operation of structures with local damage were performed in a non-linear dynamic formulation using the LS-Dyna software complex having numerical stability and a wide variety of constitutive models. The maximum size of the model elements $1 \text{ m} \times 1 \text{ m}$ was chosen by the authors due to a compromise between the optimal calculation accuracy and the number of computational operations. The constructed model of the building in the LS-Prepost preprocessor, consisting of 52.200 finite elements, is shown in Fig. 2.

Fig. 2. General view of the finite element model of the building frame

Reinforced concrete columns and girders are modeled by means of BEAM161 bar elements with the Hughes-Liu formulation. Floor slabs and longitudinal stiffening diaphragms are modeled by means of 4-node shell elements (SHELL163) [16]. To address the problem of static-dynamic deformation of reinforced concrete frames in out-of-limit states, an explicit finite element method was employed with the possibility of constructing a Lagrangian mesh. Such a model has a sufficient accuracy at lower computational costs than a volumetric element model and can be beneficially used for modeling a multi-storey skeleton building.

3. Considered loads, influences and mode of stress. Depending on the static-dynamic loading regime of the structural system, the calculation for progressive collapse is performed in stages. At the initial stage, the stress-strain of the building frame structures under normal operating conditions is identified for the primary design scheme. At the same time, in compliance with SP (CII) 286.1325800.2016, the following design loads are accepted: constant loads from the dead weight of structures (25 KN/m^3) and the total temporary long-term load (8 KN/m^2). The reliability factor for load and responsibility is taken equal to one. During $t = 0$ to $t_1 = 0.5$ seconds, constant and temporary continuous loads $P(t)$ were applied and increased to operational values P_0 using the * LOAD_BODY and * LOAD_SHELL_SET cards (Fig. 3a). The sloped section of the load curve is introduced for ensuring the stability of the integration process and provides for a step-by-step application of the load from zero to the specified design values. The internal force $R(t)$ acting in the removed column also increased from zero to the full value of R_0 over the same period of time. At the same time, in order to exclude possible undesirable dynamic fluctuations due to a sharp application of the operational load in an explicit setting, the dynamic relaxation method is used (* CONTROL_DYNAMIC_RELAXATION map). The essence of this method is that at each integration step, a decrease in the nodal velocities

of finite elements of the structure is ensured by multiplying by a certain coefficient below one (the DRFCTR coefficient and by default it was taken equal to 0.995). Hence at the initial stage, a «pseudo-static» design calculation is conducted.

Then the system was kept under the described load for the time $t = t_2 - t_1$ to achieve the criterion of convergence of the calculation on dynamic relaxation when the amount of kinetic energy remaining in the system at the final moment of calculation (t_2) is less than 0.001 (Fig. 3a). Therefore the stress-strain is achieved in the elements of the structural state of the column shutdown, approximately its static loading.

Fig. 3. Loading mode of the operational load $P(t)$ and projected impact $R(t)$ in the original (a) and secondary (b) calculation scheme

At the second stage of the analysis, the stress-strain of the structures is identified based on the secondary design scheme obtained from the original one by excluding the load-bearing structural element. In this case, the results of the calculation based on the original design scheme using the dynamic relaxation method are the initial conditions for the secondary design scheme. To simulate the instantaneous removal of the column of the first floor B2, the force $R(t)$ acting in it, which was previously identified by means of the calculation based on the original design scheme, is forced to zero during a certain short time interval t_r , called the column failure time (Fig. 3b). Through the course of the dynamic calculations, the column failure time was taken as $t_r < 0.01T$, where T is the time corresponding to the first natural vibration frequency of the damaged structure. This approach is implemented using the *LOAD_REMOVE_PART map. The service load in the secondary design was kept constant for the time period $t_4 > t_r$. The numerical values of t_1 , t_2 , t_r , t_4 are presented in the table in Fig. 1b. Hence in the second stage of modeling, the external load acting on the structure and the time-varying forces in the removable structural column are considered.

4. Physical and mechanical characteristics and deformation diagrams of materials. In the computational analysis, the physical and mechanical characteristics of the deformation of materials were taken in three versions: those obtained according to the experimental data by

G. A. Geniev, according to the experimental data by N. V. Fedorova and M. D. Medyankin according to the Russian standards SP (CII) 385.1325800.2018. All the load-bearing structural elements of the calculated building frame are designed from B70 concrete and reinforced with A500 class working reinforcement. In the structures of the girders, symmetric reinforcement is adopted to protect the building from progressive collapse following the removal of the load-bearing column. The material model *MAT_CONCRETE_EC2 was employed which was developed specifically to describe the behavior of reinforced concrete structures using beam and shell FE. The strength characteristics of materials are taken equal to their standard values. The input parameters of static deformation of materials required for quasi-static analysis at the first stage of loading according to the primary design scheme are presented in Table 1.

Table 1

Input parameters of static deformation of materials for pseudostatic analysis according to the original design scheme

		Denoted according to SP (CII) 63	Denoted in LS Dyna	Input values in LS Dyna
Concrete V70	Density	$\rho(t/mm^3)$	RO	$2.5 \cdot 10^{-9}$
	Compressive strength design value	$R_{b,ser}$ (MPa)	FC	50
	Calculated tensile strength	$R_{bt,ser}$ (MPa)	FT	3
	Ultimate tensile strain	ε_{bt2}	ECUTEN	0.00015
	Initial elasticity modulus of concrete	E_b (MPa)	ET36	$4.1 \cdot 10^4$
	Poisson's ratio	$\nu_{b,p}$	PRT36	0.2
	Specific compression strain	ε_{b0}	EC1_6	-0.002
		ε_{b2}	ECUT36, ECSP69	-0.0035
Reinforcement A500	Elasticity modulus	E_s (MPa)	YMREINF	$2 \cdot 10^5$
	Poisson's ratio	$\nu_{s,p}$	PRRINF	0.3
	Calculated tensile strength	$R_{s,ser}$ (MPa)	SUREINF	500
	Specific deformation	ε_{s2}	Via *CURVE maps	0.033
	Reinforcement percentage along hte local axis X	$\mu_{s,x}$	FRACRX	Columns: 0.672 %; Crossbars, floor slabs and stiffening diaphragms: 0.704 %
	Reinforcement percentage along hte local axis Y	$\mu_{s,y}$	FRACRY	Columns: 0.672 %; Crossbars, floor slabs and stiffening diaphragms: 0.704 %

A diagram of the static deformation of concrete was constructed according to the input parameters (Fig. 4a). The behavior of reinforcing steel is presented using a piecewise linear model of the Prantl type (Fig. 4b).

Fig. 4. State diagrams of materials under quasi-static loading according to the primary design scheme of:
a — concrete; b — reinforcement

While calculating, the limiting values of maximum deformations in compressed concrete were taken as a criterion for the transcendental state of reinforced concrete elements of a structural system $\epsilon_{b2} = -0.0035$ and in stretched reinforcement $\epsilon_{s2} = 0.033$. If this criterion was not met in a specific element of the structural system, it was excluded from work.

The analytical dependence (3), according to which the coefficient of increase in the static-dynamic ultimate strength of concrete is calculated for the calculation according to the secondary design scheme, includes the following parameters:

The analytical dependence (3), according to which the coefficient of increase in the static-dynamic ultimate strength of concrete φ_b^{st-d} is calculated for the calculation according to the secondary design scheme, includes the following parameters:

- (η) is the the relative level of static loading of reinforced concrete elements of the structural system identified by comparing the operational load on the structure with the ultimate load that the structure can accept (the load-bearing capacity of the structure). In the present study, the relative level of static loading (η) is taken equal 0.3;
- t_d is the time of dynamic reloading, i.e. the time when during the transition of a constructively nonlinear system from n -times statically indeterminate to $(n-1)$ -times statically indeterminate, an increment of stresses and strains takes place in concrete and reinforcement of elements from a sudden redistribution of power flows. This parameter can be identified in a simplified way using the well-known equation of motion of a substructure in the form of a reinforced concrete beam with one degree of freedom which simulates the zone of local failure [11, 12]:

$$u(t) = \begin{cases} (u_{st})_0 \left[\frac{t}{t_r} - \frac{1-2\zeta^2}{t_r \omega_D} e^{-\zeta \omega_n t} \sin \omega_D t - \frac{2\zeta}{t_r \omega_n} (1 - e^{-\zeta \omega_n t} \cos \omega_D t) \right] & \text{at } 0 \leq t \leq t_r, \\ (u_{st})_0 \left\{ \begin{aligned} &1 + \frac{2\zeta}{t_r \omega_n} [e^{-\zeta \omega_n t} \cos \omega_D t - e^{-\zeta \omega_n (t-t_r)} \cos \omega_D (t-t_r)] \\ &-\frac{1-2\zeta^2}{t_r \omega_D} [e^{-\zeta \omega_n t} \sin \omega_D t - e^{-\zeta \omega_n (t-t_r)} \sin \omega_D (t-t_r)] \end{aligned} \right\} & \text{at } t > t_r, \end{cases} \quad (4)$$

where t_r is the time of failure of the column; $\omega_D = \omega_n \sqrt{1-\zeta^2}$ is the frequency of eigenfrequencies of a substructure considering the energy dissipation; $(u_{st})_0 = P_0 / (m\omega_n^2)$ is a static deflection of the substructure; $\zeta = c / (2m\omega_n)$ is the damping coefficient.

Analyzing the equation (4), it is easy to see that the maximum time of dynamic extra loading t_d is always longer than t_r . Therefore for identifying this time let us look at the speed of the discussed substructure only for the case $t > t_r$:

$$\dot{u}(t) = \frac{d}{dt} [(u_{st})_0 \{ 1 + \frac{2\zeta}{t_r \omega_n} [e^{-\zeta \omega_n t} \cos \omega_D t - e^{-\zeta \omega_n (t-t_r)} \cos \omega_D (t-t_r)] - \frac{1-2\zeta^2}{t_r \omega_D} [e^{-\zeta \omega_n t} \sin \omega_D t - e^{-\zeta \omega_n (t-t_r)} \sin \omega_D (t-t_r)] \}] \quad (5)$$

The dynamic deflection of the substructure reaches its maximum value when its speed drops down to zero.

$$\dot{u}(t) = 0. \quad (6)$$

Based on the solution of equation (6), a formula is obtained for identifying the time of dynamic extra loading of the substructure considering the time of failure of the column:

$$t_d = \frac{t_r}{2} + \frac{T}{2}, \quad (7)$$

where $T = 2\pi / \omega_n$ is the oscillation period of the substructure which is identified using the method described in [13]. The results of calculations according to the above formula are shown in Table 2.

Dependence (3) includes another important parameter associated with the time of dynamic action which is essential for the coefficient of increase in static-dynamic strength — a parameter that characterizes the ductile resistance of concrete. Its significance is definitely experimentally and G. A. Geniev [4] took it for heavy concrete of low strength (about B15) to be constant and equal to $\omega = \pi \cdot 10^2 = 314 \text{ c}^{-1}$. The coefficient of the increase in the dynamic strength of concrete ϕ_b^{st-d} obtained using formula (3) is 1.0018. According to the revisited

experimental data by N. V. Fedorova and M. D. Medyankin for class B70 concretes, the value is much less and amounted to $= 22 \text{ sec}^{-1}$. Accordingly, the value of the coefficient of increasing the dynamic strength of concrete φ_b^{st-d} is 1.19. It should be noted in the Russian standards SP (CII) 385.1325800.2018, for all modes of static-dynamic loading $\varphi_b^{st-d} = 1.15$ is accepted, which causes underestimation of the value of the dynamic strength of concrete of structures in a specific range of dynamic reloading time and to an even greater one with a decrease in the time of special impact. In the U.S. standards, the value of this coefficient while calculating buildings for progressive destruction for all modes of static-dynamic loading is also assumed to be constant but equal to $\varphi_b^{st-d} = 1.25$ [15, 20].

In order to examine the influence of the parameter characterizing the ductile resistance of concrete and, accordingly, the coefficient of increase in its static-dynamic strength in more detail, three variants of the dynamic analysis of the stress-strain state of the structure were performed according to the secondary design scheme using different values φ_b^{st-d} : 1 is according to G. A. Geniev, 2 is according to N. V. Fedorova and M. D. Medyankin, and 3 is according to the Russian norms of SP (CII) 385.1325800.2018. In this case, the coefficient of increasing the dynamic strength of reinforcement for all calculation options was taken equal to 1.2.

Table 2

Input parameters of diagrams of static-dynamic deformation of materials for dynamic analysis
of a structural system according to a secondary design scheme

		Level of static loading	Time of dynamic extra loading	Parameter characterizing the viscous resistance of concrete	Coefficient of the increase in the static and dynamic strength
		η	t_d (sec)	ω (sec^{-1})	φ_b^{st-d}
Concrete V70	According to the experimental data by G. A. Geniev	0.3	0.19	314	1.0018
	According to the experimental data by N. V. Fedorova and M. D. Medyankin			22	1.19
	According to the Russian standards SP (CII) 385.1325800.2018	—	—	-	1.15
Reinforcement A500					1.2

5. Results of numerical studies and their analysis. The analysis of the results of a numerical study of the deformation of the frame elements of a multi-storey building under the considered loads enabled us to conclude the following. At the first stage of loading the frame with

a stage-by-stage distributed load of 8 kn/m^2 , the maximum displacement of the monolithic slab of the upper floor slab was -24.22 mm or $1/310$ of the span (Fig. 5a).

Fig. 5. Mosaic of vertical displacements in the floor disks (a) and the graph of the longitudinal force and time in the B2 column at the first stage of loading

As already noted, the quasi-static application of this load was achieved by means of the method of dynamic relaxation. The effectiveness of this method can be seen from Fig. 5b where static loading is shown in black, and the graph of quasi-static loading of a column using the dynamic relaxation method is shown in red. Comparing these graphs, it can be seen that the use of the dynamic relaxation method provides at the first stage of loading the building frame close to a static increase in the force in the column. It increases linearly from zero and stabilizes up to 18.500 MN thus not causing undesired vibrations to the entire system.

At the second stage of loading using the secondary design scheme, the process of removing the B2 column was simulated by means of reducing the force in this column from 18500 MN to zero during a short time interval t_r (see Fig. 3). Analyzing the general picture of deformation and destruction of the structural scheme of the building frame under a specific design load and a special impact caused by the removal of the specified column, it can be seen that while using the viscosity modulus calculated from the data of G. A. Geniev at the stage of dynamic loading [4], there is an intense change in force flows in the structural system and a considerable zone of local failure (Fig. 6a).

Fig. 6. General view of failure of the structure calculated based on the experimental data by G. A. Geniev (a), Russian standards (SP (CII) 385.1325800.2018) (b) and based on the experimental data by N. V. Fedorova and M. D. Medyankin (c)

Deformation of the building frame while using the viscosity modulus obtained in the experiments by N. V. Fedorova and M. D. Medyankin [7, 8] occurred less intensively and in the removal zone of the B2 column, local fracture was not observed (Fig. 6b). A similar pattern of the deformation of the building frame was obtained using the coefficients of increasing the dynamic strength of concrete and reinforcement recommended by the Russian standards for calculating a specific special impact. It should be noted that the viscosity coefficient of concrete, corresponding to the normalized value of the dynamic strength of concrete $\omega = 26 \text{ sec}^{-1}$ has a slightly greater value than that the one obtained in the experiments by N. V. Fedorova and M. D. Medyankin (see Table 2).

The analysis of the graphs of changes in the deflection of the floor over the removable column depending on the time of dynamic reloading of the structural system t_d for all the three options (Fig. 7) showed the following. While calculating the option using the viscosity modulus according to the data by G. A. Geniev, the deflection of the floor over the removable column

continuously increases and at $t = 1$ sec was 1174 mm or the relative deflection $f/l = 1/13$. In this case, the stabilization of displacement in the area of the removable column does not take place.

Fig. 7. Graphs of the change in the deflection of a point above the removed column, depending on time, calculated based on the experimental data by G. A. Geniev (1), Russian standards (SP (CII) 385.1325800.2018) (2) and the experimental data by N. V. Fedorova and M. D. Medyankin (3)

The deflection of the slab over the removable column while calculating based on the experimental data by N. V. Fedorova and M. D. Medyankin is considerably less than the one calculated based on the data by G. A. Geniev. On top of that, the structural restructuring of the construction system causes a sudden increase in the deformations in reinforcement and concrete, reaching a peak value after about $t_d = 0.19$ sec (dynamic extra loading time) from the moment the column was removed. The maximum deflection of the girder over the removable column in this version was 386 mm or $1/39$ of the span, which does not exceed the criterion of a special limiting state according to Russian standards ($1/30$ of the span) and according to the recommendations provided in foreign literature [10]. Therefore the calculation using the refined viscosity modulus for concrete of various classes makes it possible to calculate the criterion parameters of the structural system in a more rigorous manner.

Conclusions. A computational model, a method for identifying the parameters of the diagrams of the static-dynamic deformation of concrete and reinforcement under a special impact, an algorithm of computational analysis and the results of a nonlinear calculation of the static-dynamic deformation of reinforced concrete frames in out-of-limit states caused by the sudden removal of one of the load-bearing structures are provided.

The results of numerical and experimental studies of frame-bar structural systems of reinforced concrete frames of multi-storey buildings under their two-stage loading — with a static load to a specific design level and extra loading with a dynamic load caused by a sudden structural restructuring of the structural system when one of the supporting structures is removed from the system showed the need for more rigorous consideration of the physical and

mechanical characteristics and deformation diagrams of materials under such loading conditions of structures. The calculated values of such parameters such as the level and mode of loading of the structure, the coefficient of increase in the static-dynamic strength of materials, the time of failure of the removable structural element have a significant (under the studied loading modes up to 19 %) impact on the nonlinear static-dynamic deformation and on their survivability in transcendental states respectively.

The research results can be instrumental in numerical justification of the design of the protection of structural systems of reinforced concrete frames of multi-storey buildings from progressive collapse. By using them, it is not only economical, but also more reliable design solutions that can be obtained.

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