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Accounting For Non-Linear Work Of Reinforced Concrete In The Algorithms Of Calculation And Design Of Structures

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ABSTRACT

An algorithm is developed for calculating rod systems taking into account the nonlinear work of reinforced concrete. The calculation is based on the nonlinear theory of reinforced concrete.

KEYWORDS

Reinforced concrete, accounting, Algorithms

INTRODUCTION

Real structural materials, as a rule, are deformed non-equilibrium and nonlinearly. The non-equilibrium deformation manifests itself in the fact that when structures are loaded, in addition to elastic instantaneous deformations, so-called non-elastic deformations and, in particular, creep deformations develop.

Strain nonlinearity is the absence of a proportional relationship between stresses and strains. This applies to both creep deformations and elastic instantaneous deformations. The concrete and reinforcing steel that make up reinforced concrete also obey this rule. The external signs of relationships between stresses and strains in concrete and reinforcing steel are identical. This makes it possible, when necessary, to use for them phenomenological equations of deformations that are uniform in mathematical notation. However, the manifestation of non-equilibrium and non-linearity of deformation of concrete and

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reinforcing steel is different. Therefore, for reinforcing steel, simplified deformation equations are usually used.

The equations of state of concretes (1) reflect the fact of nonlinearity of both creep deformations and elastic-instantaneous deformations and, although it is built on a phenovenological basis, from the point of view it corresponds to the molecular-kinetic theory of solids.

$$\varepsilon(t,t_0) = \varepsilon(t,t_0) + \frac{S_M\left[\frac{\sigma(t)}{R(t)}\right]}{E_M^o(t)} - \int_{t_o}^{t} S_n\left[\frac{\sigma(\tau)}{R(\tau)}\right]\frac{\partial}{\partial\tau}C*(t,\tau)d\tau \qquad (1)$$

One of the possible ways of further development of the equations of the mechanical state of concretes may be to reject the empirical estimate of the deformation parameters found in equation (1) and their appointment based on the theory of dislocation, thermodynamics and molecular kinetic theory.

In time and as the loads increased, the deformation properties of concrete and reinforcement, when working together, caused a redistribution of stresses between them, reduce the stiffness of sections up to the appearance of plastic hinges and a change in the static structure scheme, increase deflections and cause a redistribution of forces in statically indeterminate systems, affect the vibration mode , structural stability, etc.

Despite the high level of the modern theory of reinforced concrete, achieved thanks to the works of Soviet and foreign construction scientists, it requires further development, especially in connection with the need to take into account the nonequilibrium and nonlinearity of deformation.

Direct application of equation (2)

$$\int_{to}^{t}\frac{1}{E_M^o(t)}\frac{dS_n}{d\tau}d\tau = \frac{1}{E_M^o(t)}\int_{to}^{t}\frac{dS_M}{d\tau}d\tau = \frac{1}{E_M^o(t)}S\left[\frac{\sigma(\tau)}{R(\tau)}\right] \qquad (2)$$

for describing the stress-strain state of bodies, the materials of which are deformed nonlinearly and in nonequilibrium, generally meet certain mathematical difficulties, leading research to insufficiently studied systems of nonlinear differential integro-differential equations in partial derivatives.

These difficulties are further aggravated for reinforced concrete structures, which are characterized by internal static indeterminacy, redistribution in time of efforts between concrete reinforcement, the development of inherent stresses from shrinkage or swelling deformation, age-related changes in mechanical properties, cracking and uneven participation in the work of the stretched zone of concrete, etc. P.

When engineering calculation of concrete and reinforced concrete structures, the practical expediency of differential accounting for all of the above factors is impossible. Here, of course, an integral assessment of the deformative properties of reinforced concrete is more appropriate.

In nonlinear structural mechanics, ideal nonlinearly deforming systems are considered that do not have internal static uncertainty and materials with different properties under tension and compression [1, 2]. At the same time, in most cases, to overcome even some of the difficulties mentioned, one or another formal mathematical linearization technique is used, and the change in the physical essence of the problem being solved often remains unclear (for example, without sufficient justification, rod systems are replaced by systems of transverse dimensionless rigid threads).The foundations of the modern theory of reinforced concrete were developed in the works of scientists N.Kh. Harutyunyan, A.A. Gvozdev, A.F. Loleita, Ya. V. Stolyarov et al. The problem of concrete deformation under creep conditions is discussed in the works of N.Kh. Harutyunyan, P.I. Vasilieva, I.A. Kiisa, M.I. Manukyan, I.I. Ulitsky, S.E. Freifeld, V.M. Bondarenko, S.V. Alexandrovsky, I.E. Prokopovich and others.

However, the above purely mathematical difficulties, especially for structures with an inhomogeneous stress state, have not yet made it possible to create a calculation method that takes into account the nonlinearity of deformation of reinforced concrete. Therefore, it is necessary to conduct further research and development of the theory of reinforced concrete in order to create a unified method for calculating reinforced concrete structures, taking into account their basic properties and, especially, non-linearity and non-equilibrium deformation.

In the proposed work, the calculation of a prestressed reinforced concrete truss taking into account the nonlinear creep and shrinkage of concrete is based on the methods of displacement and the integral modulus of deformations. The main system of the displacement method is formed by the imposition of links on the nodes of the truss that prevent the rotation of nodes and rods (Fig. 1), and the angular displacements of the nodes and rods of the truss are taken as the main unknowns.

The equations of the displacement method are homogeneous and have the following form:

$$\sum_{i=1}^{TC}\left(\frac{\delta_{bb}^{i}}{\delta_{aa}^{i}\delta_{bb}^{i}\delta_{ab}^{2i}}\varphi_{ia}+\frac{\delta_{ab}^{i}}{\delta_{aa}^{i}\delta_{bb}^{i}\delta_{ab}^{2i}}\varphi_{ib}-\frac{\delta_{bb}^{i}+\delta_{ab}^{i}}{\delta_{aa}^{i}\delta_{bb}^{i}\delta_{ab}^{2i}}\varphi_{iab}\right)=0 \qquad (3)$$

where TS is the number of rods in the considered node of the truss;

φ and ψ - angular displacements of nodes and rods;

δii, δjj- displacements from single unknowns.

Formula (3) is introduced as follows:

Figure 2 shows a given scheme of a truss element.

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In the main system (Fig. 3), it is considered that the elements of the trusses work on eccentric compression and tension.

Internal forces appear in the elements of the truss from the acting single unknowns (Fig. 4).

We compose the canonical equations of the method of forces

$$\left.\begin{array}{c} \delta_{aa}M_a + \delta_{ab}M_b + \delta_{aN}N_{ab} = \varphi_a + \psi_{ab} \\ \delta_{ba}M_a + \delta_{bb}M_b + \delta_{bN}N_{ab} = \varphi_b + \psi_{ab} \\ \delta_{Na}M_a + \delta_{Nb}M_b + \delta_{N}N_{ab} = \Delta \end{array}\right] \qquad (4)$$

Mohr's Law Moves

$$\delta_{ij} = \int_{i-1}^{i} \frac{M_i \mathrm{M}_i ds}{E_i J_i} + \mu \int_{i-1}^{i} \frac{Q_i Q_i dS}{G_i F_i} + \int_{i-1}^{i} \frac{N_i \mathrm{N}_i ds}{E_i F_i}$$

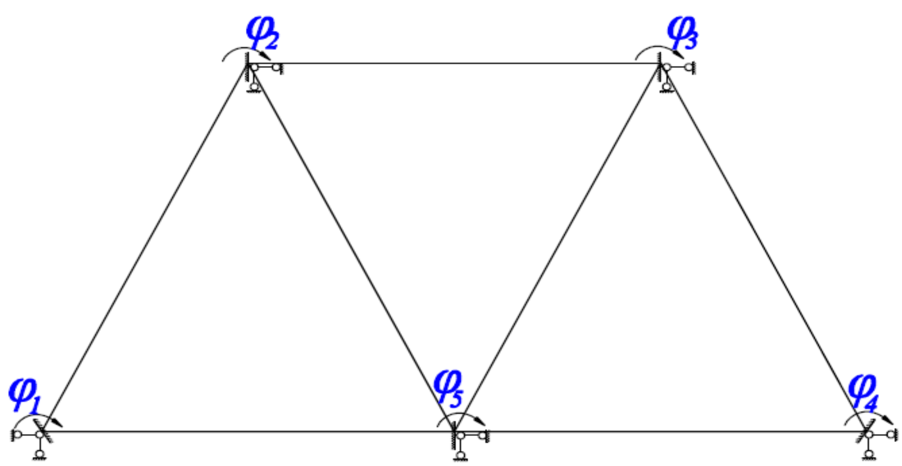

Fig 1. Design scheme of the farm

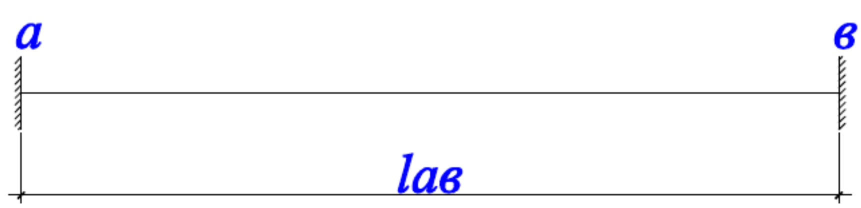

Fig 2. Specified element diagram

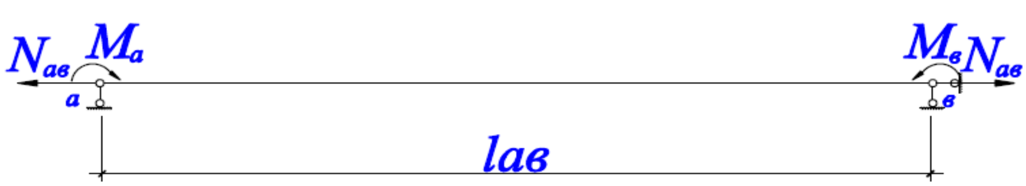

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Fig 3. Basic element diagram

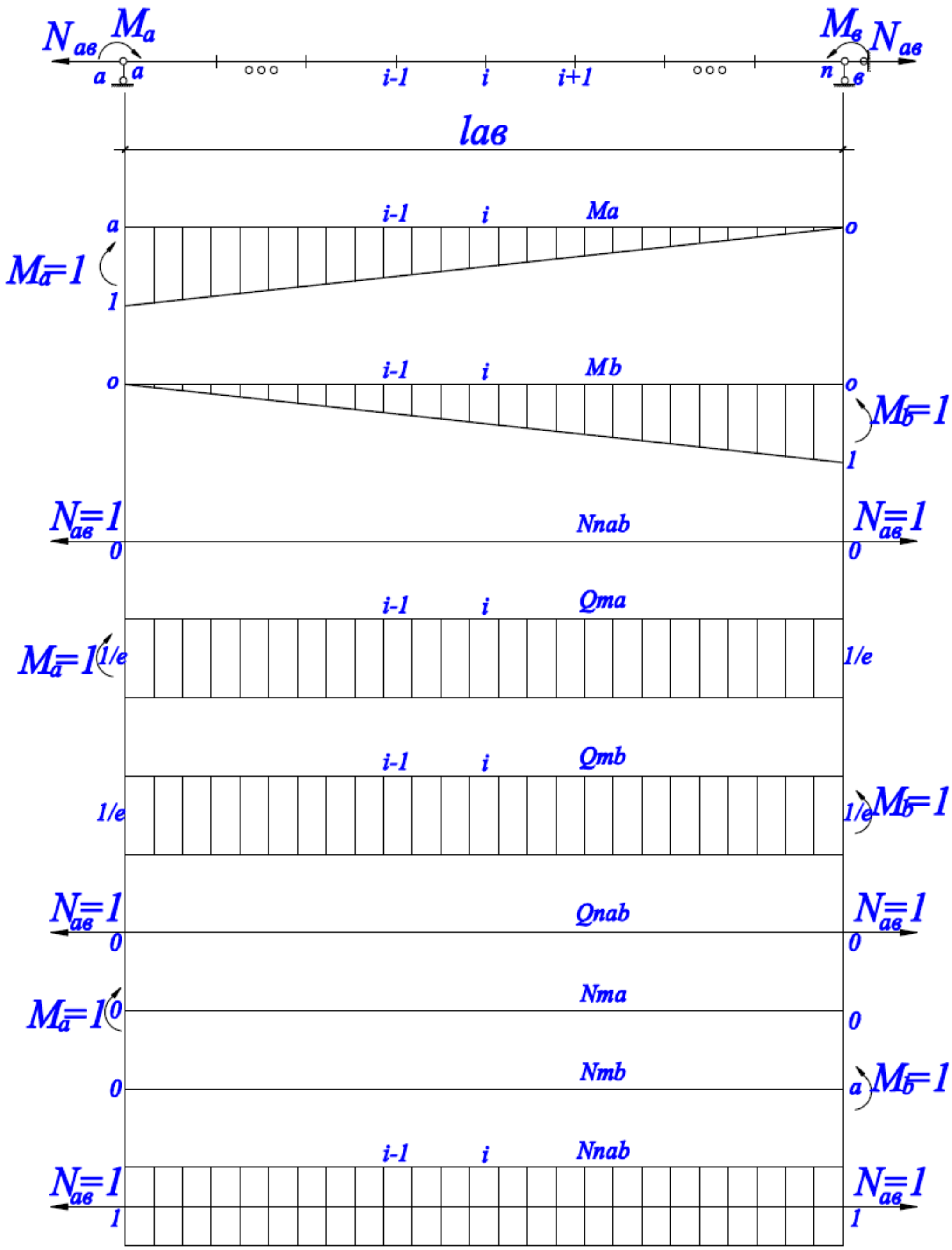


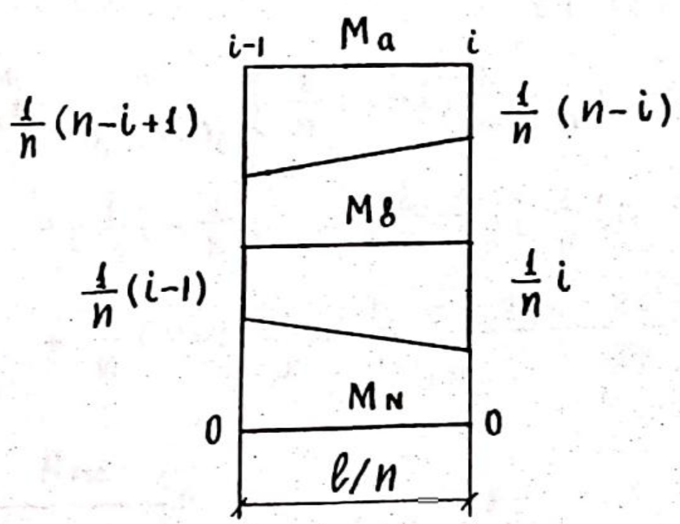
Fig. 4. Diagrams of internal forces from unit forces.

$$\delta_{ij} = \delta_{ij}^{M} + \delta_{ij}^{Q} + \delta_{ij}^{N} \qquad (5)$$

Considering the formula (5)

$$\left.\begin{aligned} \delta_{aa} &= \delta_{aa}^{M} + \delta_{aa}^{Q} + \delta_{aa}^{N} \\ \delta_{\text{вв}} &= \delta_{\text{вв}}^{M} + \delta_{\text{вв}}^{a} + \delta_{\text{вв}}^{N} \\ \delta_{N} &= \delta_{N}^{M} + \delta_{N}^{Q} + \delta_{N}^{N} \end{aligned}\right] \qquad (a)$$

$$\left.\begin{aligned} \delta_{a\text{в}} = \delta_{\text{в}a} &= \delta_{a\text{в}}^{M} + \delta_{a\text{в}}^{Q} + \delta_{a\text{в}}^{N} \\ \delta_{Na} = \delta_{aN} &= \delta_{aN}^{M} + \delta_{aN}^{Q} + \delta_{aN}^{N} \\ \delta_{N\text{в}} = \delta_{\text{в}N} &= \delta_{\text{в}N}^{M} + \delta_{\text{в}N}^{Q} + \delta_{\text{в}N}^{N} \end{aligned}\right] \qquad (b)$$



We define displacements:

$$\delta_{aa}^{M} = \frac{2}{D_{i-1}+D_i}\left\{\frac{1}{n}(n-i)\frac{l}{n}\left[\frac{1}{n}(n-i+1)+\frac{1}{n}(n-i)\right]\frac{1}{2} + \left[\frac{1}{n}(n-i+1)-\frac{1}{n}(n-i)\right]\frac{l}{n}\frac{1}{2}\left\{\frac{1}{n}(n-i)+\left[\frac{1}{n}(n-i+1)-\frac{1}{n}(n-i)\right]\frac{2}{3}\right\}\right\}$$
$$= \frac{l}{n^3}\sum_{i=1}^{n}\frac{2(n-i)^2+2(n-i)+2/3}{D_{i-1}+D_i};$$

$$\delta_{\text{вв}}^{M} = \frac{2}{D_{i-1}+D_i}\left\{\frac{1}{n}(i-1)\frac{l}{n}\left[\frac{1}{n}i+\frac{1}{n}(i-1)\right]\frac{1}{2} + \left[\frac{1}{n}i-\frac{1}{n}(i-1)\right]\frac{l}{n}\frac{1}{2}\left\{\frac{1}{n}(i-1)+\left[\frac{1}{n}i-\frac{1}{n}(i-1)\right]\frac{2}{3}\right\}\right\}$$
$$= \frac{l}{n^3}\sum_{i=1}^{n}\frac{2i^2+2i+2/3}{D_{i-1}+D_i};$$

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$$\delta_N^M = 0;$$

$$\frac{1}{{}_{i-1}F_{i-1} + G_i F_i};$$

$$\frac{1}{{}_{i-1}F_{i-1} + G_i F_i};$$

$$= 0;$$

$$= 0;$$

$$= 0;$$

$$\frac{1}{{}_{-1}F_{i-1} + E_i F_i};$$

$$\delta_{a\text{в}}^M = \frac{2}{D_{i-1} + D_i}\left\{\frac{1}{n}(i-1)\frac{1}{n}\left[\frac{1}{n}(n-i+1)\right]\frac{1}{2} + \left[\frac{1}{n}i - \frac{1}{n}(i-1)\right]\frac{1}{n}\frac{1}{2}\left\{\left[\frac{1}{n}(n-i+1) - \frac{1}{n}(n-i)\right]\frac{2}{3} + \frac{1}{n}(n-i)\right\}\right\} = \frac{l}{n^3}\sum_{i=1}^{n}\frac{2ni - 2i^2 - n + 2i - 2/3}{D_{i-1} + D_i};$$

$$\delta_{aN}^M = 0; \qquad \delta_{\text{в}N}^M = 0;$$

$$\delta_{a\text{в}}^Q = \mu\frac{2}{nl}\sum_{i=1}^{n}\frac{1}{G_{i-1}F_{i-1} + G_i F_i};$$

$$\delta_{aN}^Q = 0;$$

$$\delta_{\text{в}N}^Q = 0;$$

$$\delta_{a\text{в}}^N = 0;$$

$$\delta_{aN}^N = 0;$$

$$\delta_{\text{в}N}^N = 0;$$

Substituting these found displacement values into the formula (a) and (b), we find:

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$$\delta_{aa} = \frac{l}{n^3}\sum_{i=1}^{n}\frac{2(n-i)^2+2(n-i)+2/3}{D_{i-1}+D_i} + \mu\frac{2}{nl}\sum_{i=1}^{n}\frac{1}{G_{i-1}F_{i-1}+G_iF_i}$$

$$\delta_{\text{вв}} = \frac{l}{n^3}\sum_{i=1}^{n}\frac{2i^2-2i+2/3}{D_{i-1}+D_i} + \mu\frac{2}{nl}\sum_{i=1}^{n}\frac{1}{G_{i-1}F_{i-1}+G_iF_i}$$

$$\delta_n = \frac{2l}{n}\sum_{i=1}^{n}\frac{1}{E_{i-1}F_{i-1}+E_iF_i} \qquad ;\ (a)$$

$$\delta_{a\text{в}} = \delta_{\text{в}a} = \frac{l}{n^3}\sum_{i=1}^{n}\frac{2ni-2i^2-n+2i-\frac{2}{3}}{D_{i-1}+D_i} + \mu\frac{2}{nl}\sum_{i=1}^{n}\frac{1}{G_{i-1}F_{i-1}+G_iF_i}$$

$$\delta_{Na} = \delta_{aN} = 0$$

$$\delta_{N\text{в}} = \delta_{\text{в}N} = 0 \qquad ;\ (b)$$

If we substitute (a) and (b) into formula (2.), We obtain

$$\delta_{aa}M_a + \delta_{a\text{в}}M_{\text{в}} = \varphi_a + \psi_{a\text{в}}$$

$$\delta_{\text{в}a}M_a + \delta_{\text{вв}}M_{\text{в}} = \varphi_{\text{в}}\psi_{a\text{в}}$$

$$\delta_N N_{a\text{в}} = \Delta$$

or from here:

$$N_{a\text{в}} = \frac{\Delta}{\delta_N};$$

$$\left.\begin{aligned}\delta_{aa}M_a + \delta_{a\text{в}}M_{\text{в}} = \varphi_a + \psi_{a\text{в}}\\ \delta_{\text{в}a}M_a + \delta_{\text{вв}}M_{\text{в}} = \varphi_{\text{в}} + \psi_{a\text{в}}\end{aligned}\right] \qquad (5)$$

By solving equations (5), we obtain the end bending moments:

$$M_a = \frac{\delta_{\text{вв}}\varphi_a + \delta_{a\text{в}}\varphi_{\text{в}} - (\varphi_{\text{вв}} + \varphi_{a\text{в}})\psi_{a\text{в}}}{\delta_{aa}\delta_{\text{вв}} - \delta_{a\text{в}}^2};$$

$$M_{\text{в}} = \frac{\delta_{a\text{в}}\varphi_a + \delta_{aa}\varphi_{\text{в}} - (\varphi_{a\text{в}} + \varphi_{aa})\psi_{a\text{в}}}{\delta_{aa}\delta_{\text{вв}} - \delta_{a\text{в}}^2}.$$

The bending moment in any section of the bar is determined by the formula:

$$M_i = M_a - \frac{M_a + M_{\text{в}}}{n}i,$$

where i is the serial number of the section within the span;

i = 0,1,2.... n (the usual n is taken constant for all rods, regardless of the rod length).

Stiffnesses of sections of truss elements are determined by the formulas:

$$D = EJ = E^{ин}\frac{вh^3}{12} + E_a F_a \frac{E_a}{E_в^{вр}}\left(\frac{h}{2} - a\right)^2 + E_a^` F_a^` \frac{E_a^`}{E_в^{вр}}\left(\frac{h}{2} - a^`\right)^2;$$

$$EF = E^{ин}вh + E_a F_a \frac{E_a}{E_\delta^{вр}} + E_a^` F_a^` \frac{E_a^`}{E_в^{вр}};$$

$$GF = 0{,}4EF = 0{,}4\left(E^{ин}вh + E_a F_a \frac{E_a}{E_\delta^{вр}} + E_a^` F_a^` \frac{E_a^`}{E_\delta^{вр}}\right).$$

If the truss rods are compressed, then the concrete is designed for compression. By determining the longitudinal forces and bending moments in the section of the truss rods, we calculate the stress in the section

$$\sigma_\delta = \frac{N}{\varphi\left(\frac{E_a}{E_\delta^{вр}}F_a + \frac{E_a^`}{E_\delta^{вр}}F_a^` + F_{\delta.пр}\right)} \pm \frac{M}{\frac{D_a}{E_\delta^{вр}} + J_{\delta.пр}} \cdot Z,$$

where φ is the buckling factor:

$$F_{\delta.пр} = F_a \frac{E_a}{E_\delta^{вр}} + F_a \frac{E_a^`}{E_\delta^{вр}} + вh;$$

$$D_a = E_a F_a \left(\frac{h}{2} - a\right)^2 + E_a^` F_a^` \left(\frac{h}{2} - a^`\right)^2;$$

$$J_{\delta.пр} = \frac{вh^3}{12} + \frac{E_a}{E_\delta^{вр}} F_a \left(\frac{h}{2} - a\right)^2 + \frac{E_a^`}{E_\delta^{вр}} F_a^` \left(\frac{h}{2} - a^`\right)^2.$$

Included in these formulas $E_\delta^{вр}$ and $E^{ин}$ in are defined as follows [1]:

$$E_\delta^{вр} = \left\{\frac{\varepsilon_у}{\sigma_\delta} + \frac{\left[1 + \eta_k\left(\frac{\sigma_\delta}{R_{пр}}\right)^{m_k}\right]}{E_M} - \frac{1}{\sigma_\delta}\sum_{j=1}^{i}\left[1 + \eta_П\left(\frac{\sigma_\delta^{ср}}{R_{пр}}\right)^{m_п}\right]\left[C^*(t_i, t_j) - C^*(t_i, t_j)\right]\right\}^{-1};$$

$$E^{ин} = \Phi E_\delta^{вр},$$

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Where $$\varepsilon_y = \eta_4 \alpha_y,$$

here

$$\alpha_\gamma = \eta_1 \eta_2 \eta_4 \alpha_\gamma^c;$$

$$\eta_2 = 1.926 - 0.738 lg в;$$

$$\eta_1, \eta_4, \alpha_\gamma^c - \text{по } [1],$$

в - the smallest section size;

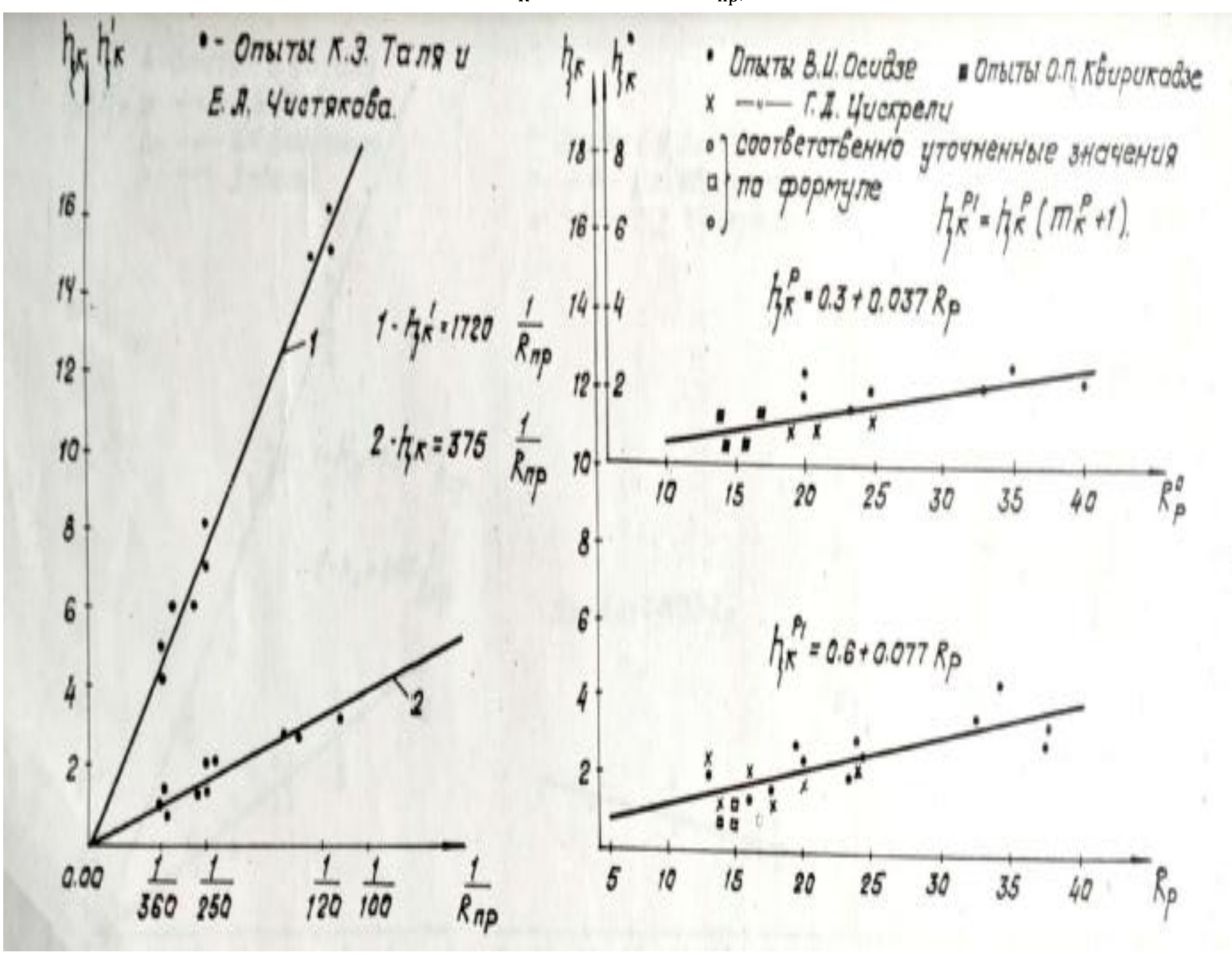
$\eta_к, m_k, \eta_п, m_п$ – are the parameters of deformation nonlinearity;

a) for axial compression:

elastic-instantaneous deformations (Fig. 5 a; 7.) [1]

$$\eta_к = \frac{375}{R_{пр}};$$

$$m_k = 5.7 - 0.005 R_{пр};$$



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Ras 5. Graphs of dependences of nonlinearity parameters:

a-dependence of hп and hп1 on 1 / Rпр,;

b-dependence of hп and hп1 on Pp;

creep deformation (Fig. 6; 7) [1]

$$\eta_{п} = \frac{450}{R_{пр}};$$

$$\mathrm{m}_{п} = 5 - 0.007\mathrm{R}_{пр};$$

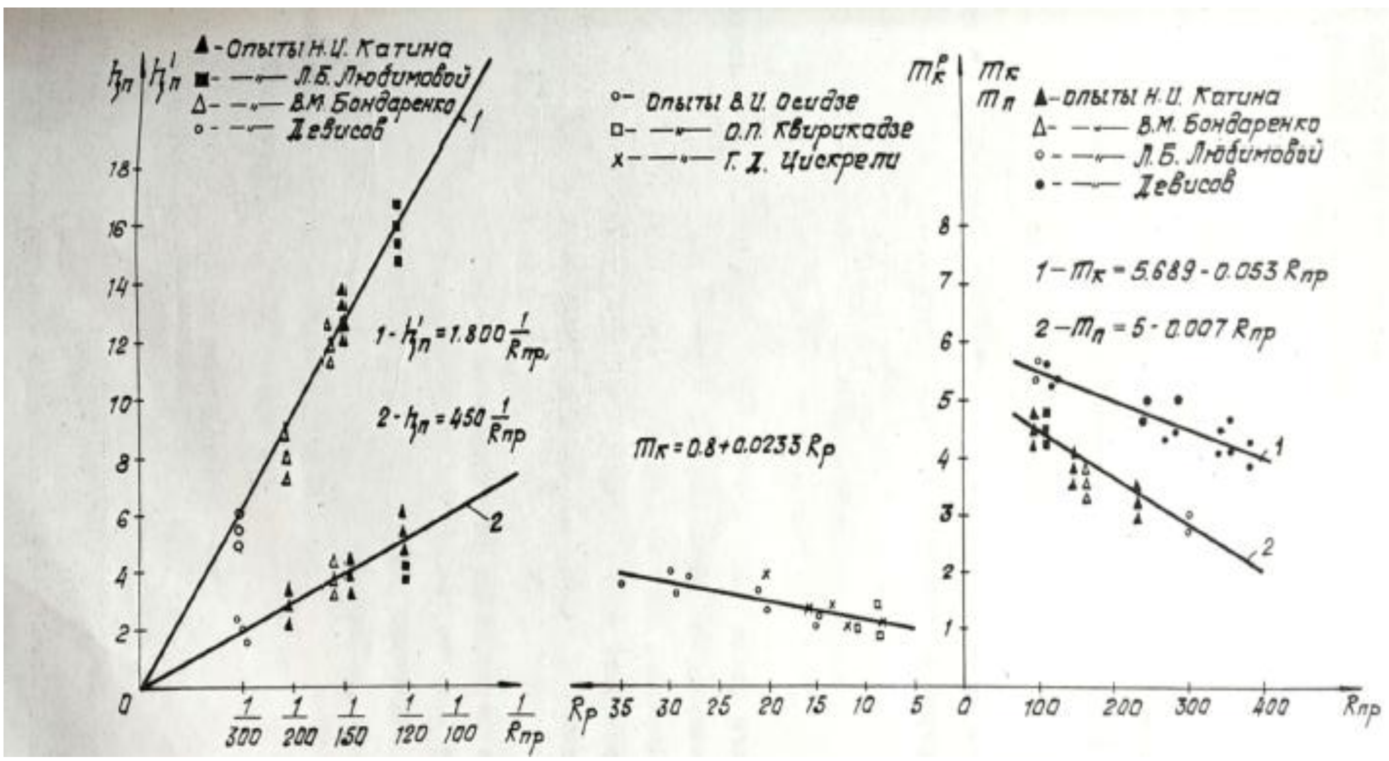

Fig 6. Dependence of parameters nonlinearity of deformations popzuchesmi $h_п$ и $h_{п1} + \frac{1}{R пр}$,

Fig 7. Dependence of parameters by nonlinearity strains M_k, $M_п$, M_c, at prism concrete strength

b) for osevogo rastyajeniya:

uprugo-mgnovenne deformatsii (рис.5., б и 7) [1]

$$\eta_k^P = 0.3 + 0.037R_p; \quad m_k^P = 0.8 + 0.023R_p;$$

Creep deformation

$$\eta_{\text{п}}^{\text{р}} \approx 1.5; \;\; m_{\text{п}}^{\text{р}} \approx 1.0,$$

where $R_{пр}$ -is the limit of the prismatic strength of concrete under axial compression;

R_p - ultimate strength of concrete under axial tension;

$C^*(t_i, t_j)$- measure of creep;

$$\Phi = \frac{2(1+m) + n_\sigma}{1 + 2m + 2n_\sigma},$$

W

where n_σ- is the normal stress nonlinearity parameter $(0 \le n_\sigma \le 1)$ which is determined by the formula

$$n_\sigma = 1 - (1 - f_0)\left(\frac{\sigma}{R_{\text{пр}}}\right)^{m_\sigma},$$

$$f_0 = \frac{E_R^K}{E_O^k}(0 < f_0 \ll 1).$$

Here f_0- is the nonlinearity parameter of the relationship between stresses and strains under uniaxial loading of the sample.

Determined by the ratio of two tangent deformation moduli for a given loading mode;

E_R^k - tangent modulus of deformation at the moment of failure;

$E_0^k = E_0$ - initial deformation modulus;

m_σ- parameter reflecting the rate of increase in the curvature of the diagram of normal stresses as the level of the inhomogeneous stress state increases.

When the truss rods work in tension, all the forces arising in the section are taken up by the reinforcement.

The stresses in the reinforcement are:

$$\sigma_s = \frac{N\left(e_0 + \frac{h}{2} - a^`\right)}{F_a(h_0 - a^`)}.$$

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