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On the construction of the diagram of calculation method of rod reinforced concrete structures under the action of low negative temperatures

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Abstract. As a result of generalization and analysis of the data in the field of experimental studies, the significant influence of low negative temperatures on the development of stresses and strains in reinforced concrete structures from the force effects was justified. It is shown that this influence largely depends on various structural and technological factors: (characteristics of the composition of concrete, its structures, water-cement ratio and of the natural moisture content of concrete). The necessity of taking into account these factors of influence in the calculation of reinforced concrete structures on the strength, deformability and crack resistance at the same time power and temperature effects has been justified. Diagrams of deformation of concrete in the frozen to minus 70 ºC condition are developed. Taking into account these diagrams the development of the diagram method is given, which allows to calculate reinforced concrete core structures under simultaneous influence of power loads, and low climatic (to minus 70 ºC) negative temperatures.

1. Introduction, purpose and objectives of the study

At present, in connection with the plans to expand oil and gas production in the Northern regions of Russia, including in the Arctic shelf, and the construction of appropriate floating terminals, as well as tanks for storage of liquefied natural gases and their transportation afloat in terminals, along with the problem of ensuring the durability requirements to reinforced concrete structures of these structures, the problem of developing new methods of calculation of such structures operated under the simultaneous impact of force loads and significant low negative temperatures becomes urgent.

It is known that the resistance of concrete under the influence of negative temperatures (frost resistance of concrete) is characterized by a certain number of cycles of alternate freezing and thawing, after which the strength of concrete is reduced to a regulated value (see GOST 10060-2012, the first basic method).

However, this characteristic is not enough for a real assessment of the durability of reinforced concrete structures. This requires an assessment of the influence of low negative temperatures, especially climatic (up to minus 70 °C), on the basic regulatory strength and deformation characteristics of concrete required for the calculation and design of concrete and reinforced concrete structures operated in such conditions. The results of the relevant experimental studies are presented in

[1-3]. Especially it is necessary to mention the fundamental monograph of Professor, doctor of technical Sciences V. M. Moskvin in coauthorship with his postgraduates [2], dedicated to the physical and technological basis of concrete and structures from it at the severe weathering.

The results of the above-stated experimental studies of the strength and deformation properties of concrete in frozen state to different temperatures (up to minus 70 °C) are summarized from the standpoint of solving the problem of durability of reinforced concrete structures operated in severe climatic conditions in [2, 3].

In the work [4] it was shown that degradation of properties of such material (conglomerate type) as concrete, is reflected most fully in the diagram method of calculation and its development. The initial basis of this method was the deformation diagrams of concrete and reinforcement under normal conditions of positive temperatures.

In work [5] it is proposed the initial approach to the construction of the diagram method of calculation of reinforced concrete structures under the action of low negative temperatures. However, the results of experimental studies [1, 2, 6] showed that the deformation diagrams of concrete, in addition to low negative temperatures, are largely influenced by the structural-technological characteristics of concrete, and, first of all, such a technological factor as the initial moisture content of concrete at the time of its freezing. It's also necessary to take into account the effect of low temperatures on the coefficients of thermal strains of concrete and reinforcement, and the strain diagram of reinforcement. All these factors are taken into account in the equations of the calculation diagram method presented below.

Taking into account all the above in this paper, based on the analysis of available experimental investigation data [1–3, 6], the following research tasks are set:

1) to perform mathematical processing of the available experimental data on strength and deformation characteristics of heavy concrete to determine the coefficients of change of these characteristics according to the appropriate formulas, depending on the initial moisture of the concrete and its freezing temperature;

2) to perform, using results of processing of experimental data (see task 1), correction of diagrams of deformation of concrete under load of axial compression in the conditions of action of low negative temperatures (to -70 °C) in comparison with the corresponding diagrams of deformation of concrete under the same load in the conditions of action of positive temperatures;

3) to determine the nature of the effect of such temperatures on the change in strength, the initial modulus of elasticity, the relative deformations of concrete in the tops of the diagrams constructed during the test under axial compression loads in the temperature range from + 20 °C to -70 °C;

4) to determine the influence on the characteristics of the constructed diagrams of such structural-technological parameters of concrete, as its water-cement ratio (W/C), and, consequently, the porous structure, as well as the initial moisture (mass ratio of moisture in %/%).

2. Diagrams of concrete deformation taking into account the influence of low negative temperatures and moisture of concrete on the change of its strength, the initial modulus of elasticity and limit deformations under axial compression

The relationships between the relative strains and stresses of concrete under axial compression in the frozen state are recorded as:

$$\varepsilon_{\mathrm{b}} = \frac{\sigma_b}{\mathrm{E}_{bT} \nu_b} \quad , \tag{1}$$

where $\varepsilon_b, \sigma_b, \mathrm{E}_{bT}$ – correspondingly the relative strain, stress and modulus of elasticity of concrete in the frozen state

$$\mathrm{E}_{bT} = \mathrm{E}_b \cdot \beta_{\mathrm{E}} , \tag{2}$$

where E_b – modulus of elasticity of concrete in initial conditions (at t°=20°C and corresponding moisture content W%),

β_E – coefficient of change of the initial modulus of elasticity of concrete in the frozen state,

ν_b – coefficient of variation of the secant modulus ($\nu_b E_{bT}$ – the secant modulus of concrete in the frozen state).

Processing of data of the experimental researches [1- 3] on concretes with various natural moisture content at W/C=0,4 and W/C=0,5 showed that on average at change of moisture in the range W=3,1%-5,1% it is possible to accept:

$$\beta_E = 1 + \left[0{,}025 + 0{,}12\left(\frac{W\% - 3\%}{1\%} \right) \right]\left(\frac{20^\circ C - t^\circ C}{90^\circ C} \right), \tag{3}$$

The following dependencies are also defined for the range of moisture content of concrete from W=3,1% to 5,1% during storage to freezing and testing under load of 28 days in the chamber of normal-humidity curing (in terms of air temperature 20 ºC +/- 2 ºC and its humidity 90% +/- 5%).

Prismatic strength of concrete R_b , which is equal to the tension $\hat{\sigma}_{bT}$ at the top of the frozen concrete diagram, is determined from the dependence:

$$\hat{\sigma}_{bT} = \hat{\sigma}_b \beta_R \tag{4}$$

where $\hat{\sigma}_b$ – stresses at the top of the concrete diagram in the initial test conditions (at t = 20 ° C and corresponding moisture content W%), β_R – the coefficient of influence of low temperatures on the increase of stress at the top of the diagram. The analysis of experimental data [1-3] showed that:

$$\beta_R = 1 + \left[0{,}10 + 0{,}45\left(\frac{W\% - 3\%}{1\%} \right) \right]\left(\frac{20^\circ C - t^\circ C}{90^\circ C} \right); \tag{5}$$

Stresses $\hat{\sigma}_{bT}$ correspond to the increased strain $\hat{\varepsilon}_{bT}$ at the top of the diagram,

$$\hat{\varepsilon}_{bT} = \hat{\varepsilon}_b \cdot \beta_\varepsilon, \tag{6}$$

where $\hat{\varepsilon}_b$ – relative deformations at the top of the concrete diagram at normal conditions.

For heavy concrete:

$$\hat{\varepsilon}_b = 200 \cdot 10^{-5} \cdot \sqrt[5]{R_b/R_0}, \tag{7}$$

where R_0 = 20 MPa.

Data processing of experimental studies [1-3] shows the following:

$$\beta_\varepsilon = 1 + \left[0{,}05\left(\frac{W\% - 3\%}{1\%} \right) + 0{,}085\left(\frac{W\% - 3\%}{1\%} \right)^2 \right]\left(\frac{20^\circ C - t^\circ C}{90^\circ C} \right); \tag{8}$$

By defining $\hat{\sigma}_{bT}$, $\hat{\varepsilon}_{bT}$ and E_{bT} we can calculate the ratio of the secant modulus corresponding to the top of the diagrams:

$$\hat{\nu}_{bT} = \frac{\hat{\sigma}_{bT}}{\hat{\varepsilon}_{bT} E_{bT}} \tag{9}$$

The secant module of concrete for intermediate loading levels $\eta = \dfrac{\sigma_b}{\hat{\sigma}_{bT}}$ determined by the dependence [4], taking into account the adjustment η and $\hat{\nu}_{bT}$,

$$\nu_b = \hat{\nu}_{bT} + (1 - \hat{\nu}_{bT})\sqrt{1 - \omega_1 \eta - (1 - \omega_1)\eta^2} \tag{10}$$

were ω_1 – diagram curvature parameter:

for the ascending branch $\nu_0 = 1; \quad \omega_1 = 2 - 2{,}5\hat{\nu}_{\mathrm{bT}}$

(11)

for the descending branch $\nu_0 = 2{,}05\hat{\nu}_b; \quad \omega_1 = 1{,}95\hat{\nu}_{\mathrm{bT}} - 0{,}138$

After determining the ν_b for different stress levels according to the formula (1), the values of the relative deformations of heavy concrete frozen up to various temperatures are determined and the corresponding deformation diagrams are constructed. For Figure 1 and Figure 2 diagrams of concrete deformation at initial moisture content W_1=3,2% and W_2=5,1% tested at different temperatures (from +20 °C to -70 °C) are presented.

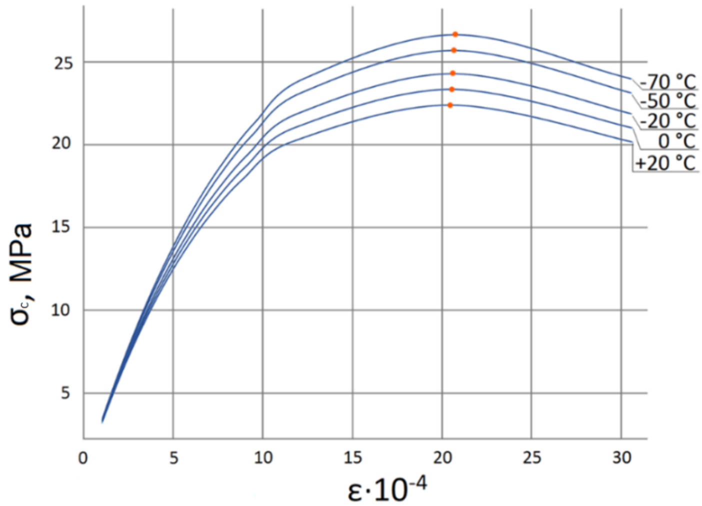

Figure 1. Deformation diagrams for axial compression of heavy concrete with initial moisture content W_1=3,2% in the temperature range from + 20 °C to -70 °C (points on the curves of the diagrams indicate the deformation of concrete at $\sigma = R_b$). *Legend*: σ_c – tension axial compression; ε is the longitudinal deformation of concrete specimens under axial compression.

Under uniaxial tension $(\sigma_0 = \sigma_{bt}, \hat{\sigma}_{bT} = \hat{\sigma}_{btT})$:

$$\hat{\sigma}_{btT} = R_{bt} \cdot \beta_{Rt}, \tag{12}$$

$$\hat{\nu}_{btT} = (0{,}6 + 0{,}15 R_{bt} / \mathrm{R}_{b0}) / \gamma_{bT}, \tag{13}$$

where $R_{b0} = 2.5\ MPa$, β_{Rt} – coefficient of increase of concrete axial tensile strength.

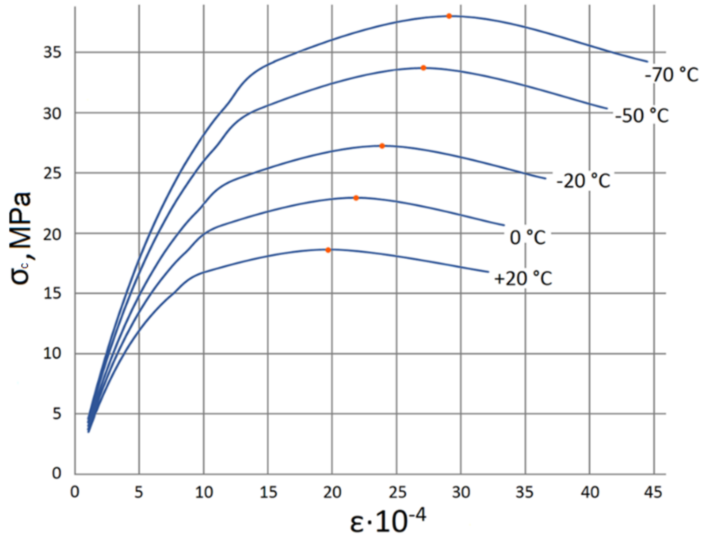

Figure 2. Deformation diagrams for axial compression of concrete with initial moisture content W_2=5,1% at the temperature range from + 20 °C to -70 °C (points on the curves of the diagrams indicate the deformation of concrete at $\sigma = R_b$). *Legend*: σ_c – tension axial compression; ε is the longitudinal deformation of concrete specimens under axial compression.

According to experimental data [6] it can be taken

$$\beta_{\mathrm{Rt}} = 1 + \left[0{,}15 + 0{,}5\left(\frac{\mathrm{W\% - 3\%}}{1\%}\right)\right]\left(\frac{20^\circ\mathrm{C} - \mathrm{t}^\circ C}{90^\circ\mathrm{C}}\right) \tag{14}$$

$$\gamma_{\mathrm{bT}} = 1 + 0{,}1\left(\frac{\mathrm{W\% - 3\%}}{1\%}\right)\left(\frac{20^\circ\mathrm{C} - \mathrm{t}^\circ C}{90^\circ\mathrm{C}}\right) \tag{15}$$

The axial tension parameters ν_{bt}, ω_1 are calculated from the dependencies (10), (11) when replaced with $\hat{\nu}_{bT}$ on $\hat{\nu}_{btT}$, ν_b on ν_{bt}.

3. Diagrams of deformation of reinforcement in conditions of low negative temperatures

Deformation diagrams of the reinforcement are constructed in accordance with the dependencies given in [4] with additional consideration of the influence of low negative temperatures. Low negative temperatures affect the yield strength $\sigma_{0,2}$ and the elastic modules of the reinforcement E_s, which in conditions of low temperatures are designated $\sigma_{0,2T}$ and E_{sT}. According to the data presented in [6], it is possible to accept

$$\mathrm{E_{sT}} = \mathrm{E}_s\left[1 + 0.19\left(\frac{20^\circ C - t^\circ C}{210^\circ C}\right)\right]; \tag{16}$$

$$\mathrm{E_{sT}} = \mathrm{E}_s\left[1 + 0.19\left(\frac{20^\circ C - t^\circ C}{210^\circ C}\right)\right]; \tag{17}$$

The relationship ratio between the stresses and deformations of the reinforcement, as in (1), is assumed to be:

$$\varepsilon_{\mathrm{s}} = \frac{\sigma_s}{\mathrm{E}_s \nu_s}, \tag{18}$$

where on the linear section of the diagram $\nu_s = 1$, on the nonlinear section ν_s is determined by the modified formula (11), where $\nu_b, \hat{\nu}_b, \eta$ are replaced, respectively, by the values $\nu_s, \hat{\nu}_s, \eta$ that are determined depending on the type of nonlinear sections of the deformation diagrams of the reinforcement according to the recommendations [4]. The dependence (19) is used to describe deformations of an element in sections or parts of sections where there are no cracks. At the same time the condition of compatibility of deformations of reinforcement and concrete ($\varepsilon_s = \varepsilon_b$) is observed. In the part of the section with a crack, this condition is violated and the average deformation of the reinforcement ε_s is introduced in the area between the cracks, which are determined by the dependence

$$\varepsilon_s = \frac{\sigma_s \psi_s}{E_s \nu_s} = \frac{\sigma_s}{E_s \nu_{sm}}, \tag{19}$$

where $\nu_{sm} = \nu_s / \psi_s$, ψ_s - the coefficient of V. I. Murashov, taking into account the influence of stretched concrete between cracks on the average deformation of reinforcement, which can be determined by the formula [4].

4. Changes in the coefficients of thermal strains of concrete

According to experimental studies [1, 2], the coefficient of thermal deformation (α_{bt°) also largely depends on the initial moisture content of concrete under the influence of negative temperatures. So, for concrete with natural moisture content after aging in the mode of normal-humidity hardening in the temperature range from + 20 °C to 0 °C , the average value $\alpha_{bt} \approx 0{,}9 \cdot 10^{-5}$ (°C)$^{-1}$, when the temperature changes from 0 °C to – 40 °C, then $\alpha_{bt} \approx 1{,}13 \cdot 10^{-5}$ (°C)$^{-1}$. When the temperature changes from -40 °C to -70 °C, then $\alpha_{bt} \approx 0{,}9 \cdot 10^{-5}$ (°C)$^{-1}$. The coefficient of temperature deformation of the value in the temperature range from +20 °C to -70 °C according to [1, 2] remains constant and equal $\alpha_{st0} \approx 1 \cdot 10^{-5}$ (°C)$^{-1}$.

5. Construction of physical relations of the diagram model taking into account the influence of low negative temperatures

The above relationships between stresses and deformations of concrete and reinforcement (diagrams of their deformation), as well as the coefficients of thermal deformations, serve as the basis for the construction of physical relations to the calculation of reinforced concrete structures operated under conditions of low negative temperatures [4, 5].

Let's consider the construction of such relations in relation to the calculation of reinforced concrete structures.

In Figure 3, the calculation scheme of the normal section of the reinforced concrete element exposed to the action of two moments is presented: M_x (in the plane ZOY) and M_y (in the plane ZOX) and the normal force N acting along the axis Z. The position of the point 0 at the origin x, y, z can in principle be arbitrary, but it is rational to place it in the center of gravity of the section, which is determined in the elastic stage of deformation of the element.

In the derivation of physical relations tensile stresses and forces are taken for positive and compressive for negative. Freezing temperatures of concrete and reinforcement are also taken as negative. In the part of the section with cracks, the concrete is turned off from work and all efforts are transferred to the reinforcement. In this area the free rebar diagram $\sigma_s - \varepsilon_x$ is replaced by a diagram $\sigma_s - \varepsilon_{sm}$, where σ_s - the stress of the rebar in the crack, ε_{sm} is the average relative strain on the participation between the cracks. Average deformations ε_{sm} are determined by the method of V. I. Murashev. The change in the relative deformations of concrete and reinforcement ε_e(*e=b, bt, s, sm*) along the height of the section of the reinforced concrete element follows the hypothesis of flat sections [4].

Numerical integration is used to derive physical relations. The cross section of the concrete element (Figure 3) is divided into *i* elementary sections of concrete with areas A_{bi} and coordinates of their centers of gravity Z_{bxi}, Z_{byi} (i - number of concrete section). Reinforcing bars falling into the section with a crack, fixed by area of each rod A_{sj} and the coordinates of its center of gravity Z_{sxj}, Z_{syj}. Accordingly, for rods outside the field with a crack the same parameters of the rods are denoted by A_{sk}, Z_{sxk} , Z_{syk} . It is necessary to consider signs of coordinates in the accepted system x, y, z (Figure 3). Designations are introduced: σ_{bi}, ε_{bi} – the stresses and relative deformations of the concrete in the elements i (at the parts that are cracked $\sigma_{bi} = 0$); σ_{si}, ε_{sj} – the stresses in the rods of rebar pieces j-x in the cracks and their average deformation, σ_{sk}, σ_{sk} - the stress in the rods of rebar's k-x, located in the part section without cracks.

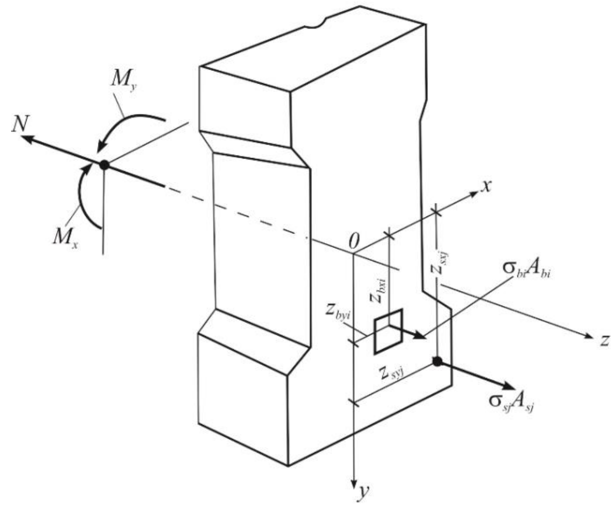

Figure 3. A calculation scheme of normal section of reinforced concrete element.

In the derivation of the physical relations the following equations are used:
– equations of equilibrium of external forces and internal forces in the normal section of the element

$$\begin{aligned} M_x &= \sum_i \sigma_{bi} A_{bi} z_{bxi} + \sum_k \sigma_{sk} A_{sk} z_{sxk} + \sum_j \sigma_{sj} A_{sj} z_{sxj}; \\ M_y &= \sum_i \sigma_{bi} A_{bi} z_{byi} + \sum_k \sigma_{sk} A_{sk} z_{syk} + \sum_j \sigma_{sj} A_{sj} z_{syj}; \\ N &= \sum_i \sigma_{bi} A_{bi} + \sum_k \sigma_{sk} A_{sk} + \sum_j \sigma_{sj} A_{sj}, \end{aligned} \tag{20}$$

– equations determining the distribution of deformations along the cross section of the element, which follow from the hypothesis of plane sections [4]

$$\begin{aligned} \varepsilon_{\mathrm{i}} &= \varepsilon_0 + \frac{1}{r_x} z_{bxi} + \frac{1}{r_y} z_{byi} \\ \varepsilon_{\mathrm{k}} &= \varepsilon_0 + \frac{1}{r_x} z_{sxk} + \frac{1}{r_y} z_{syk} \\ \varepsilon_{\mathrm{j}} &= \varepsilon_0 + \frac{1}{r_x} z_{sxj} + \frac{1}{r_y} z_{syj} \end{aligned} \tag{21}$$

justified the above relationships between stresses and strains of concrete and reinforcement in the form of diagrams of their deformation under the action of low temperatures; $1/r_x$, $1/r_y$ are the curvatures of the element, ε_0 is the relative strain at the axis level z.

The tensions σ_{bi} on the sections of concrete i, the strains in the reinforcing bars σ_{sk} in areas of concrete with no cracks and tension σ_{sj} in the rods in sections of the cracked element are determined by the dependencies:

$$\left.\begin{aligned} \sigma_{\mathrm{bi}} &= \left(\varepsilon_i - \varepsilon_{bi}^0\right)\mathrm{E}_{b\mathrm{T}}\nu_{bi}; \\ \sigma_{\mathrm{sk}} &= \left(\varepsilon_k - \varepsilon_{\mathrm{sk}}^0\right)\mathrm{E}_{sT}\nu_{sk}; \\ \sigma_{\mathrm{sj}} &= \left(\varepsilon_j - \varepsilon_{sj}^0\right)\mathrm{E}_{sT}\nu_{smj} \end{aligned}\right\}, \tag{22}$$

where ε_i, ε_k, ε_j – the total strain, $\varepsilon^{\circ}_{bi}, \varepsilon^{\circ}_{sk}, \varepsilon^{\circ}_{sj}$ is the relative deformation from the action of low temperatures,

$$\left.\begin{aligned} \varepsilon_{bi}^0 &= \alpha_{bi}\Delta t_i; \\ \varepsilon_{sk}^0 &= \alpha_s \Delta t_k; \\ \varepsilon_{sj}^0 &= \alpha_s \Delta t_j, \end{aligned}\right\}. \tag{23}$$

Δt_i, Δt_k, Δt_j - the increment of temperature at areas i, k, j of relatively normal temperature t = +20 °C.

Subsequently, depending on (23), the expressions (22), (24) for $\varepsilon_i, \varepsilon_k, \varepsilon_j$ $\varepsilon^{\circ}_{bi}, \varepsilon^{\circ}_{sk}, \varepsilon^{\circ}_{sj}$ and the values thus obtained σ_{bi}, σ_{sk}, σ_{sj} are entered in (21). As a result, we come to the general physical relations of the form:

$$\begin{Bmatrix} \mathrm{M_x} \\ \mathrm{M_y} \\ \mathrm{N} \end{Bmatrix} = \begin{bmatrix} \mathrm{D_{11}D_{12}D_{13}} \\ \mathrm{D_{12}D_{22}D_{23}} \\ \mathrm{D_{13}D_{23}D_{33}} \end{bmatrix} \begin{Bmatrix} \dfrac{1}{\mathrm{r_x}} \\ \dfrac{1}{\mathrm{r_y}} \\ \varepsilon_0 \end{Bmatrix} - \begin{Bmatrix} \mathrm{M_{tx}^0} \\ \mathrm{M_{ty}^0} \\ \mathrm{N_t^0} \end{Bmatrix}, \tag{24}$$

where M^0_{tx}, M^0_{ty}, N^0_{ty} – the conditional (equivalent) the efforts caused by temperature deformations ε^0_{bi} - in concrete, ε^0_{sj}, ε^0_{si} – in the reinforcement, defined by the formulas:

$$\left.\begin{aligned} M_x^0 &= \sum_i \varepsilon_{bi}^0 A_{bi} z_{bxi} E_{bT}\nu_{bi} + \sum_k \varepsilon_{sk}^0 A_{sk} z_{sxk} E_{\mathrm{sT}k}\nu_{sk} + \sum_j \varepsilon_{sj}^0 A_{sj} z_{sxj} E_{sTj}\nu_{smj}; \\ M_y^0 &= \sum_i \varepsilon_{bi}^0 A_{bi} z_{byi} E_{bT}\nu_{bi} + \sum_k \varepsilon_{sk}^0 A_{sk} z_{syk} E_{\mathrm{sT}k}\nu_{sk} + \sum_j \varepsilon_{sj}^0 A_{sj} z_{syj} E_{sTj}\nu_{smj}; \\ N_y^0 &= \sum_i \varepsilon_{bi}^0 A_{bi} E_{b\mathrm{T}}\nu_{bi} + \sum_k \varepsilon_{sk}^0 A_{sk} E_{sTk}\nu_{sk} + \sum_j \varepsilon_{sj}^0 A_{sj} E_{sTj}\nu_{smj}, \end{aligned}\right\}; \tag{25}$$

D_{ij} – stiffness of the element, which is calculated by the formulas:

$$\left.\begin{aligned}
D_{11} &= \sum_i A_{bi} z_{bxi}^2 E_{bT} \nu_{bi} + \sum_k A_{sk} z_{sxk}^2 E_{sTk} \nu_{sk} + \sum_j A_{sj} z_{sxj}^2 E_{sTj} \nu_{smj};\\
D_{12} &= \sum_i A_{bi} z_{bxi} z_{byi} E_{bT} \nu_{bi} + \sum_k A_{sk} z_{sxk} z_{zyk} E_{sTk} \nu_{sk} + \sum_j A_{sj} z_{sxj} z_{syj} E_{sTj} \nu_{smj};\\
D_{13} &= \sum_i A_{bi} z_{bxi} E_{bT} \nu_{bi} + \sum_k A_{sk} z_{sxk} E_{sTk} \nu_{sk} + \sum_j A_{sj} z_{sxj} E_{sTj} \nu_{smj};\\
D_{22} &= \sum_i A_{bi} z_{byi}^2 E_{bT} \nu_{bi} + \sum_k A_{sk} z_{syk}^2 E_{sTk} \nu_{skj} + \sum_j A_{sj} z_{syj}^2 E_{sTj} \nu_{smj};\\
D_{23} &= \sum_i A_{bi} z_{byi} E_{bT} \nu_{bi} + \sum_k A_{sk} z_{syk} E_{sTk} \nu_{sk} + \sum_j A_{sj} z_{syj} E_{sTj} \nu_{smj};\\
D_{33} &= \sum_i A_{bi} E_{bT} \nu_{bi} + \sum_k A_{sk} E_{sTk} \nu_{sk} + \sum_j A_{sj} E_{sTj} \nu_{smj},
\end{aligned}\right\} \quad (26)$$

ν_{bi}, ν_{sk} ν_{smj} – coefficients of secant modules which are calculated on the basis of analytical dependences (10), (11), (13), (18), (19) for diagrams deformation of concrete and reinforcement.

The established physical relations can be used in various constructions of finite elements.

Due to physical nonlinearity, step-iterative methods are usually used to calculate structures (determination of forces and displacements).

Physical relations (25) for complex constructive systems are rationally record in finite increments, which leads to economical weakly iterative and non-iterative methods for solving physically nonlinear tasks.

It should be noted that the results of the studies carried out in this work on the construction of diagrams of deformation of heavy concrete corresponds:

– with presented in the works of S. N. Leonovich [7], and Yu. V. Zaitsev [8] physical models of the phase transition of water into ice in the pores and capillaries cement stone of concrete;

– with the theories and hypotheses of frost destruction of concrete according the characteristics of its structure, in particular its differential porosity, which has a significant impact on the development of hydraulic pressure in the gel pores and pores-capillaries of the cement stone concrete; the latter, provided that the original moisture content of concrete W exceeds the limit of W ($W>W_{cr}$, where W_{cr} corresponds to the degree of saturation of cement stone of the concrete is the critical value for more than 90%) [1, 2]. This can lead first of all to the development of irreversible microcracks in the walls of the above pores and capillaries, and later – to the formation and development of the so-called "magistral (main) crack" [8] and then – to the progressive destruction of concrete;

– with the nature of the diagrams "axial compression stresses in frozen concrete under different temperatures – longitudinal deformations of concrete" given in [7, 8], as well as in the works of foreign researchers [3, 9–12].

6. Conclusion

The general physical relations connecting moments M_x, M_y and normal force N with generalized deformations (curvatures $1/r_x$, $1/r_y$ and ε_0 – relative deformations at the level of the chosen z axis) taking into account influence of low negative temperatures are established. Physical relations allow to calculate reinforced concrete structures taking into account the influence of temperature factors by modern computational methods, for example, the finite element method. The initial basis of the physical relations are diagrams linking the stresses and strains of concrete and reinforcement under the action of low temperatures. Based on the generalization of the available experimental data, the correction of concrete diagrams was performed. This takes into account the effect of low temperatures on increasing the strength of concrete, its initial modulus of elasticity and relative deformation at the tops of the diagrams to a temperature of approximately minus 70 °C. It is shown that the increase in strength, the initial modulus of elasticity and the limiting relative deformations at the tops of the diagrams largely depends on the moisture of the concrete at the time of its freezing. At temperatures lower than minus 70 °C, most experiments indicate that the increase in the strength and deformation characteristics of concrete is practically stopped, which can be justified by the results of studies step-by-step process of water-to-ice phase transition, namely, the physical end of this process, when the temperature reaches minus 70 °C. However, this issue still requires further targeted research. The influence of these factors also affects the deformation diagrams of reinforcement, but to a much lesser extent than in the diagrams of concrete deformation.

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