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Basic principles in the theory of force and thermal force resistance of concrete

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Abstract. In the development of the ideas and approaches to the analysis of the force resistance of concrete of V.M. Bondarenko, the initial prerequisites for the model of the thermomechanical state of concrete under short-term sharp high-temperature exposure, characteristic of fire conditions, are formulated. The separation of force deformations into components is carried out on the basis of the connection with the accumulation of damage in the structure of the material, based on the principle of independence of the limiting structural stresses from temperature and the mode of force action, which makes it possible to establish basic thermomechanical relationships and determine the deformation parameters of concrete operating under conditions of unsteady heating in a loaded state. Based on the extension of the hypothesis of entropy damping of nonequilibrium processes to the area of action of an active destructive factor, the principle of normalization was formulated and a kinetic equation was proposed, from the solution of which exponential dependences having a single structure were obtained, which make it possible to describe the basic temperature parameters of concrete, the relationship of stresses with deformations, and other nonlinear characteristics. The application of the proposed principles creates a reliable theoretical basis for describing the mechanisms of thermal resistance of concrete and greatly simplifies the modeling of the effect of high temperature on the properties of concrete in the practical implementation of methods for the numerical calculation of reinforced concrete structures.

Keywords: calculation model prerequisites, invariants, structural stresses, heating under load, elasticity coefficient, kinetic equation

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Базовые принципы в теории силового и термосилового сопротивления бетона

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Аннотация. В развитие идей и подходов к анализу силового сопротивления бетона В.М. Бондаренко формулируются исходные предпосылки модели термомеханического состояния бетона при кратковременном резкорежимном высокотемпературном воздействии, характерном для условий пожара. Разделение силовых деформаций на компоненты осуществляется исходя из связи с накоплением повреждений в структуре материала, основываясь на принципе независимости предельных структурных напряжений от температуры и режима силового воздействия, что позволяет установить базовые термомеханические соотношения и определить параметры деформирования бетона, работающего в условиях нестационарного нагрева в нагруженном состоянии. На основе распространения гипотезы об энтропийном затухании неравновесных процессов на область действия активного разрушающего фактора сформулирован принцип нормализации и предложено кинетическое уравнение, из решения которого получены имеющие единую структуру экспоненциальные зависимости, позволяющие описывать базовые температурные параметры бетона, связь напряжений с деформациями и другие нелинейные характеристики. Применение предложенных принципов создает надежную теоретическую основу для описания механизмов термосилового сопротивления бетона и существенно упрощает моделирование влияния высокой температуры на свойства бетона в практической реализации методик численного расчета железобетонных конструкций.

Ключевые слова: предпосылки расчетной модели, инварианты, структурные напряжения, нагрев под нагрузкой, коэффициент упругости, кинетическое уравнение

Introduction

Regime heredity, disequilibrium, moment and lagged strains – this is an incomplete list of features, exhibited by concrete in conditions of resistance to external force action. There are many examples of calculations, where are used simplified models of material behavior. Nevertheless, since the very moment structural theory was born, researchers are concerned with the problem of the most adequate consideration of the real material behavior, which allows ones not to doubt obtained results. The processes having a place inside the material's structure mostly are hard to be directly observed, so there should be a reliable theory explaining known facts and predicting new ones as well.

From the beginning of the 20th century, especially with the appearance of such new materials as concrete and polymers, there were intensely taken efforts of creation theories that, on the one hand, were strict enough and, on the other, engineering-adapted and suitable for practical application. Research of long-lasting resistance to degradation and deformation features of the new materials led to the emergence of a new direction – *rheology* (this term was accepted at the 3rd symposium on plasticity in the USA). Temperature action leads to intensification of processes occurring in material over time. *Thermorheology* so is even more general, uniting impacts of

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strain, temperature and time factors. Though at the moment there are no universal thermorheological models of material behavior, consideration of thermal-force concrete resistance allows us to have quite another look at the stability of its long-lasting resistance.

Loaded concrete demonstrates several special properties even in conditions of harsh-regime high-temperature heating when long-lasting processes don't evaluate fully. The most important of such properties is a sufficient (nearly in twice) increase of strain capacity, in comparison with concrete heated to the same temperature and then loaded. This effect looks similar to short-term concrete creep in normal thermal conditions, but it's not a creep, because it doesn't depend on time and it is determined by a sequence of thermal and force impacts.

Through his works V.M. Bondarenko founded a method based on a clear *division of levels of the research object's model of development* [1; 2], this method helps to understand the reasons and consequences of thermal-force resistance of loaded concrete. Vitaly Mikhailovich used to cite Leonardo da Vinci: "there is as much science in the scientific work as there is mathematics in it..," but he always mentioned, that before writing math expressions (mathematical model), one should have necessarily imagined physics of the process (physical model) also he should have reasonably formulated starting hypotheses and basis (calculation model). Clarity of understanding the basis provides better understanding essence of physical and mathematical models, also it allows one to estimate the accuracy of taken decisions, because, as academic A.N. Krylov (1863–1945) wrote, "despite all the accuracy of the mathematical solution, it'll never be more accurate, than the basis on which it was made."

Separation of strain components. Invariance of ultimate structural stresses

One of the important premises used in the creation of equations of mechanical and thermomechanical states of concrete is *the premise about the separation of strain components*. Total relative strains are traditionally divided into non-force (thermal, humidity, etc.) and force (mechanical) particular strains, which are often considered as mutually independent and subsequently summed. Mechanical strains are divided by the principles of linearity and non-linearity, reversibility, and time. In particular, phenomenological theories of concrete creep for the region of non-linear strains are based on different variants of division of components of total mechanical strains:

– concept of N.H. Arutyunyan and V.B. Kolmanovskiy [3] suppose that all non-linear strains of concrete strain rely upon concrete creep;

– in the concept of V.M. Bondarenko [4; 5] total strains consist of moment (inelastic, non-linear) and lagged (non-linear) strains concrete creep;

– in the concept of A.A. Gvozdev and K.Z. Galustov [6; 7] total strains are divided into elastic and long-timed, consisting of two components – reversible (strains of elastic aftereffect, which depends on strains linearly and obey the principle of superposition), and irreversible, which non-linearly depend on stresses that caused them; a theory based on this concept is named by A.A. Gvozdev "two-component theory of concrete creep";

– in the V.M. Bondarenko and N.I. Karpenko concept of "non-equilibrium force deformation of concrete" [8] it's proposed to consider strains as the matter of structural changes in concrete. This concept continues those laid down in the works of O.Ya. Berg [9], E.N. Shcherbakov [10] the views on non-linear strains of concrete in a way of "quasi-plastic", caused by an accumulation of damages in the structure, which make close physical and phenomenological ways of creating a general theory of concrete resistance.

Summarizing the approaches mentioned above we propose to divide mechanical strains into two types based on their connection with the accumulation of damages in concrete structure [11]. In this case concrete structure should be considered as a set of bonds that are viscoelastic deformable under stress but degradable when some limiting level of stress is achieved. Similar representations based on the implementation of the detachable fracture mechanism are used in statistical theories (V. Veibull [12], S.D. Volkov [13], V.V. Bolotin [14], L.G. Sedrakyan [15], V.D. Harlab [16], M.M. Holmyanskiy [17], etc.). Presented bonds possess unequal stability following some statistical distribution. When a sample is put under stress, the stress distributes between bonds. Not strong enough bonds cleave and the stress comes to others. So cleavage of the weakest bonds is compensated by increased stress on remained ones. In case of sample is being loaded either for a short or for a long time there is an accumulation of defects demonstrated by bond cleavage. It could be presented as a decrease in the workable part of the sample section (Figure 1).

Stresses having a place in bonds are structural (true). They increase with the load increase but more intensively because of the redistribution of strains from broken bonds (inner loading). Assuming in a mental model that the distribution of stress is equal between bonds (though it's being distributed proportionally to the rigidness of bonds), structural stresses can be calculated by dividing the applied loading by the nominal area of the workable part of section (before the loading). In the same manner, dividing this loading by the total area of the sample section results in average stresses, which are usually considered “stresses” and are used in the determination of the strength characteristics of the material.

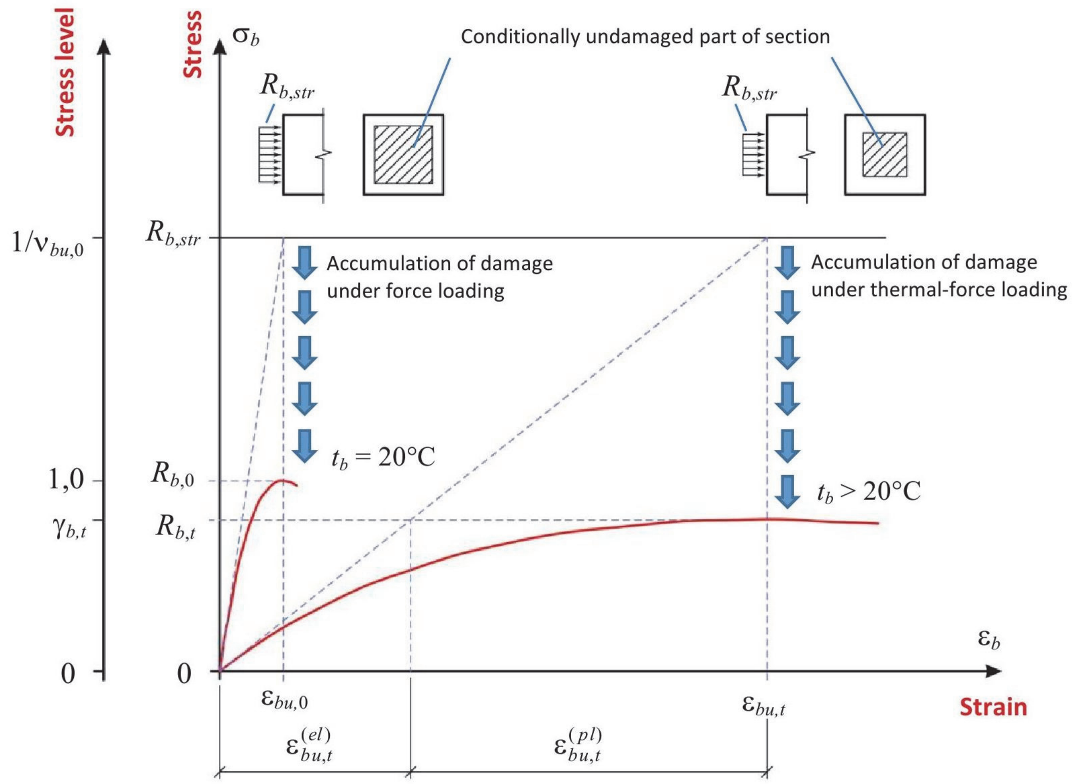


Figure 1. Graphical interpretation of the principle of independence of ultimate structural stresses

At the moment when the stability of remaining bonds isn't enough for load perception, the sample degrades and the stresses in the nominal workable part of section are ultimate structural stresses. Further increase of the sample loading is impossible and if there is no loading decrease there initiates the process of chain bond destruction with stress transfer to the remaining bonds. This results in avalanche destruction of all sample – such an effect is observed during concrete tests with a constant rate of stress increase.

If after reaching ultimate structural stresses it's possible to decrease loading like when it happens during concrete tests with a constant rate of strain increase, the process of remaining bonds destruction is being carried under control and there is a descending branch on the stress-strain curve.

The mentioned example of destruction is related to increasing loading, but if one tries to destruct the other sample of the same concrete in a slightly different manner by adding heat to increasing loading the general trend would be quite the same. Factors of destructive impact (loading and heat) would lead to the accumulation of structure damages, which results in the increase of structural stresses to the very same ultimate level. It would be observed as a decrease in sample stability in the context of the ultimate structural stresses (Figure 1).

In that case whatever would be the way of destruction ultimate structural stresses remain constant because the sample is made from the same material (concrete of certain contains, age, and curing conditions). Obtained experimental diagrams of concrete deformation in different temperatures [18; 19] illustrate this very well (Figure 2).

V.M. Bondarenko mentioned not once that invariants take a special place in theory building. Invariants are values that remain unchanged during transformations. In papers and books of V.M. Bondarenko there are often mentioned invariants obtained directly from experiments or consequent generalizations. Many times there are mentioned widely used invariants of affine similarity, energy invariants of stability theory (M. Reiners invariant of the constant potential energy of material degradation particularly), N.N. Davidenkovs invariant about hysteresis loop area independence (specific dissipative scattered energy) from stationary oscillation frequency, equivalent deformational invariants, etc.

Acceptance of a precondition about the invariance of ultimate structural stresses from a regime of destructive impacts leads to an important consequence, specifically, it allows to divide components of total concrete strain into two types depending on whether they do or do not connect with the decrease of concrete stability towards the ultimate structural stresses. Both of these components can be moment, lagged, and evolving over time. It allows us to use the proposed principle in case of short-time and also long-time force actions. A principle of ultimate structural stresses independence keeps correctness for diagram-isochron N.I. Karpenko (Figure 3) [20]. Despite the generality of the principle in this work, the emphasis is placed on short-time thermo-mechanical loading, which is specific to fire conditions when the intensity of thermal impact becomes more significant than the rate of evolving of transition nonequilibrium processes, which can't fully exhibit itself.

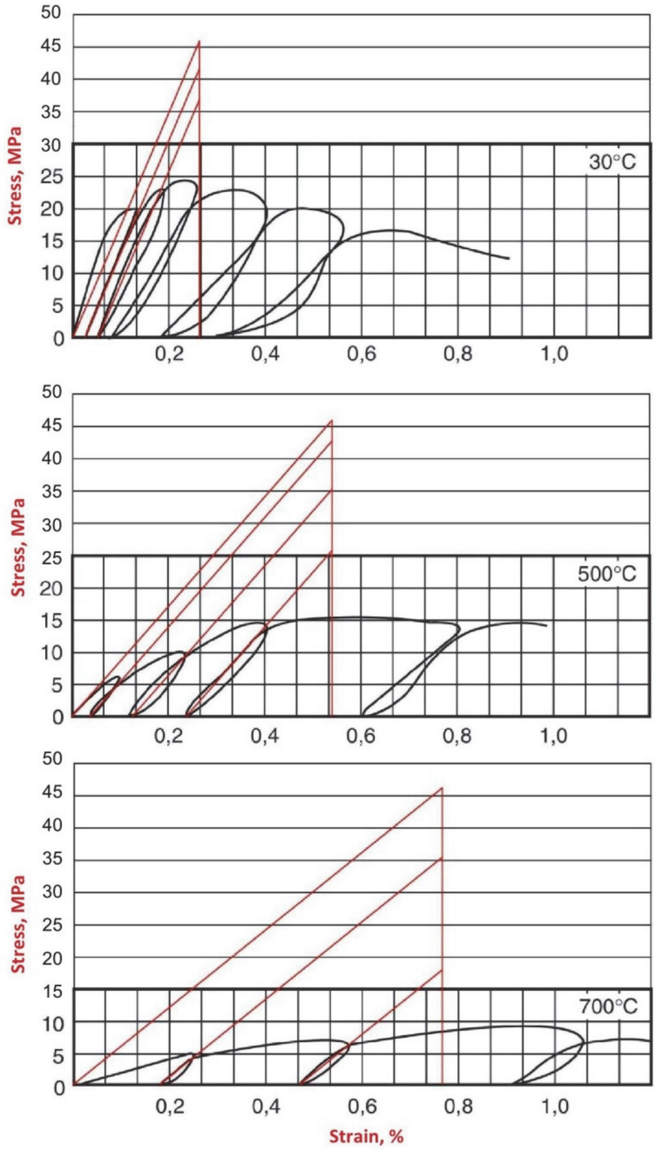
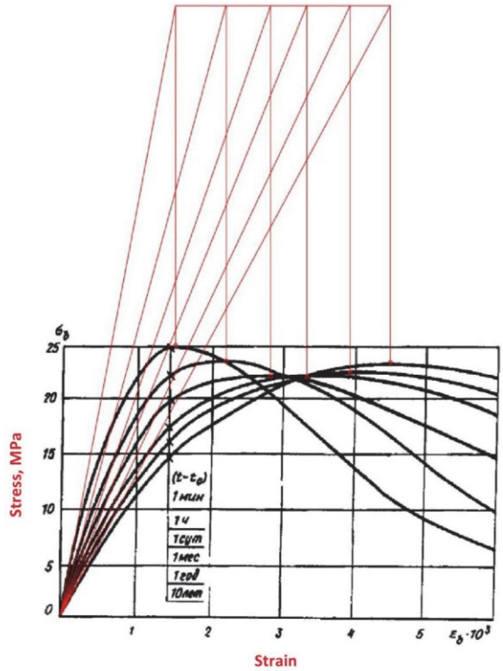


Figure 2. Invariance of ultimate structural stresses under thermal force loading (experimental data by Furamura [18; 19])



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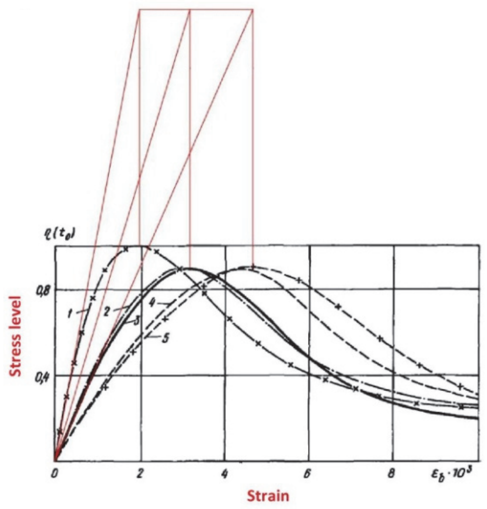


b

Figure 3. Invariance of ultimate structural stresses under long-term loading:
a – isochrone diagrams by N.I. Karpenko under axial compression for various periods of time; *b* – reference diagram for short-term loading (1), isochrones of soft (2, 3) and hard (4, 5) loading modes according to N.I. Karpenko

The first strain component not connected with the accumulation of defects and not leading to stability decrease is linear towards the strains and is expressed by a straight line in the deformation diagram. This line connects the origin and a point above the top of the diagram on the level of ultimate structural stresses (Figure 1). Physically it's a viscoelastic bonds strain without their cleavage; an elastic component is a strain of crystal joint and a viscous component is a strain of calcium hydrosilicate gel, lagging from elastic one.

Experiments of unloading samples witness only partial reversibility of linear components of strains (Figure 4). A moment linear-elastic component with initial deformation module E_0 = tgα_0 appears right after loading then appears viscous component that is non-linear towards the strains but it is reversible. The viscous component is followed by the redistribution of strains from gel to joint. Everything goes the same way during unloading: firstly the unloading line goes at an angle of inclination α_0 (elastic component impact), then it deviates going back practically to the origin (viscous component impact). Because of losing energy in inner friction (hysteresis losses) deformation is restored partially. Despite some of the described demonstrations of non-linearity, the total strain (viscous and elastic) during loading is linear towards the stress and it's characterized by initial module of concrete deformations $E_{b,0}$ = tgα. The line goes from the origin exactly at an angle to the point of ultimate structural stresses.

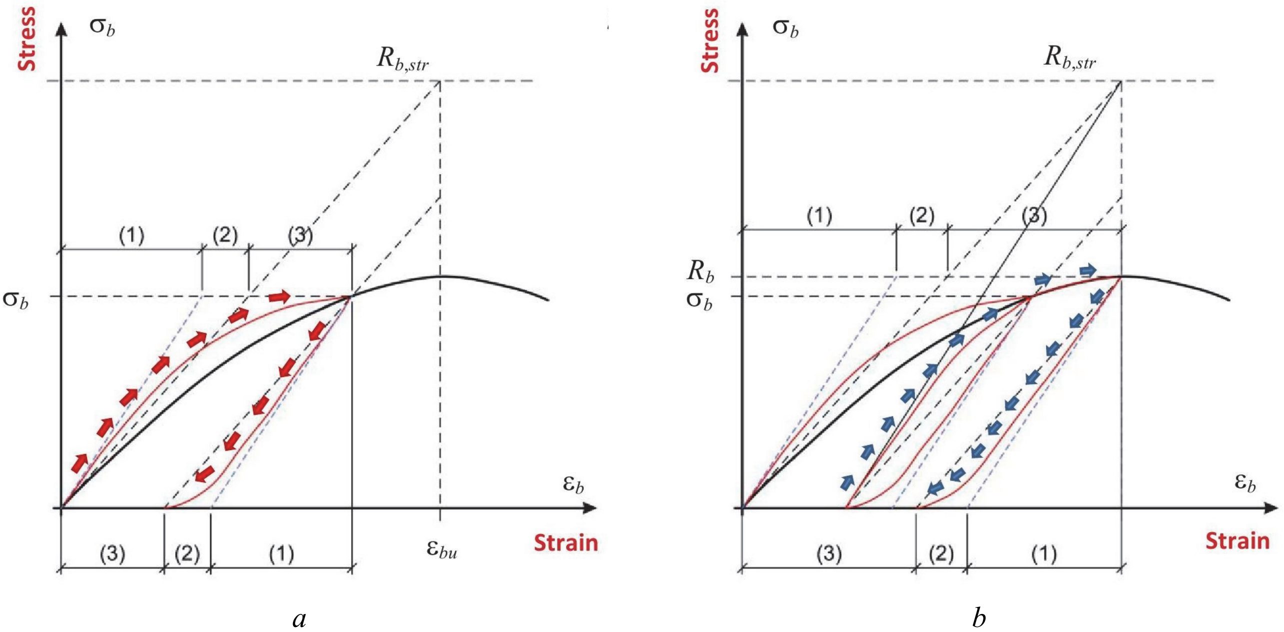


Figure 4. Strains of concrete under modal loading-unloading:
a – first cycle; *b* – second cycle; *(1)* – instantaneous linear strains; *(2)* – delayed linear strains; *(3)* – instantaneous nonlinear strains

The second deformation component connected with the accumulation of defects and leading to stability decrease is non-linear towards acting stress (physical non-linearity). It's known that the accumulation of defects in the form of bonds cleavage in concrete structure is a thermofluctuation process and increases with the growth of strains and heating temperature. Overtime accumulation of damages in a sample during isothermal endurance under constant load is characterized by classical creep curves with three distinctive regions. Time before destruction (longevity of concrete) is described by the Boltzmann – Zhurkov kinetic equation [21].

Diagram non-linearity is characterized by an elasticity coefficient of concrete in V.I. Murashev terminology [22] or by a coefficient of secant modulus in N.I. Karpenko terminology [20]. The mentioned coefficient is calculated by dividing elastic deformations by total deformations and shows a ratio between acting average and structural stress, particularly for ultimate strain in normal temperature

$$\nu_{bu,0} = \frac{\varepsilon_{bu,0}^{(el)}}{\varepsilon_{bu,0}} = \frac{R_{b,0}}{E_{b,0}\varepsilon_{bu,0}} = \frac{R_{b,0}}{R_{b,\mathrm{str}}}, \tag{1}$$

where $R_{b,0}$, $E_{b,0}$ и $\varepsilon_{bu,0}$ – compressive strength, initial modulus, and limit (peak) strains of concrete before heating respectively; $R_{b,\mathrm{str}}$ – ultimate structural stresses (structure stability) of concrete.

Heating temperature impact on mechanic concrete properties in domestic and foreign regulatory and scientific literature is usually characterized by two main thermal parameters – stability decrease coefficient $\gamma_{b,t}$ and elasticity module decrease coefficient $\beta_{b,t}$ during heating, which are calculated as the ratio of the characteristics in the heated state ($\sigma_{bu,t}$, $E_{b,t}$) to the corresponding values before heating ($R_{b,0}$, $E_{b,0}$):

$$\gamma_{b,t} = \frac{\sigma_{bu,t}}{R_{b,0}}; \tag{2}$$

$$\beta_{b,t} = \frac{E_{b,t}}{E_{b,0}}. \tag{3}$$

Temperature dependences of these parameters are set as analytical expressions, tables, and graphs.

As the third temperature parameter, it's usually used either limiting deformations of concrete $\varepsilon_{bu,t}$ (peak values, corresponding to the top of the diagram), or limiting elasticity coefficient $\nu_{bu,t}$, but when limiting structural strains independence principle is applied, there is no need for normalization of temperature dependences for these characteristics because they are expressed through general parameters as basic thermomechanical ratios:

– for ultimate strains through the initial module decrease coefficient

$$\varepsilon_{bu,t} = \frac{\varepsilon_{bu,0}}{\beta_{b,t}}; \tag{4}$$

– for ultimate the elasticity coefficient through the stability decrease coefficient

$$\nu_{bu,t} = \nu_{bu,0}\gamma_{b,t}. \tag{5}$$

Proposed ratios are easy to prove by geometrical conditions. The first one of these ratios represents the fact that limiting (peak) concrete strains value when heating are defined only by a initial module decrease:

$$\frac{\varepsilon_{bu,0}}{\varepsilon_{bu,t}} = \frac{R_{b,\mathrm{str}}}{E_{b,0}} \cdot \frac{E_{b,t}}{R_{b,\mathrm{str}}} = \frac{E_{b,t}}{E_{b,0}} = \beta_{b,t}. \tag{6}$$

The second ratio witnesses about the increase of deformation curve non-linearity increase as the concrete stability increase:

$$\frac{\nu_{bu,t}}{\nu_{bu,0}} = \frac{R_{b,0}\gamma_{b,t}}{R_{b,\mathrm{str}}} \cdot \frac{R_{b,\mathrm{str}}}{R_{b,0}} = \gamma_{b,t}. \tag{7}$$

Limiting structural strains' independence from temperature allows not only to simplify the normalization of temperature parameters but also to explain peculiarities of its behavior during heating under loading.

Resistance of loaded concrete during heating. Energy criterion for strain coupling

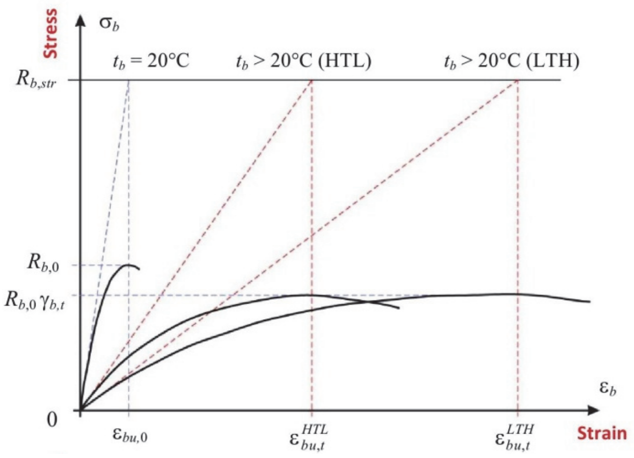
Non-stationary heating of concrete samples under the action of initial compressive stresses as shown [19] leads to its stability increase (growth of $\gamma_{b,t}$ coefficient) in comparison with concrete which was heated firstly and then loaded to destruction. It's explained by the restraining effect of the load on the development of microcracks (cracks caused by thermal incompatibility between aggregate and cement stone matrix first of all), which growth is unhindered in unloaded samples [23]. It has been proven by research [24] at least for residual crack density calculations.

But heating in a loaded state has a much more significant impact on concrete strains. During non-stationary heating of loaded samples mechanical strains (obtained by subtracting the temperature component from the total strains) were almost double the strain of samples which were loaded after heating.

Assume that sequence of applying heating and force actions has no impact on concrete strength and conditionally assume that it remains exactly the same at a given heating temperature, regardless of the loading conditions. In this case deformability increase while maintaining the constancy of the ultimate structural stresses, it can be ensured only by increasing the linear component. Thus, the connection between stress and strains during loaded concrete heating is expressed as a diagram with the same value of elasticity coefficient as the one when loading has a place after heating (due to the fact that the stability to ultimate structural stresses does not decrease) but with less initial module (Figure 5, *a*). In this case, the difference in the description of diagrams will be fewer values of $\beta_{b,t}$, coefficient and it leads to an increase of peak strains from $\varepsilon_{bu,t}^{\mathrm{HTL}}$ up $\varepsilon_{bu,t}^{\mathrm{LTH}}$ to because of the thermomechanical equation (6).

The energy interpretation of the considered effect of increasing deformability is that when heated under load, the external force performs additional work on displacements caused by a decrease in the elastic properties of the material. Assuming that the value of work that external force has to make for the achievement of given deformation is the same, we obtain all the required work is done because of external force increase when loading occurs before heating. When the last ones occur at the same time part of this work is done because of initial module of material deformation decrease caused by heating, so the same deformation can be achieved with less value of force or the same force will lead to bigger deformation. Graphically, this is expressed in equality of shaded areas in Figure 5, *b*. It results in the following ratio between strains when concrete heated under load $\varepsilon_{bu,t}^{\mathrm{LTH}}$ (load-then-heat regime, LTH) and loading after heating $\varepsilon_{bu,t}^{\mathrm{HTL}}$ (heat-then-load regime, HTL), showing their nearly two times increase corresponding quite well with experimental data [25]:

$$\varepsilon_{bu,t}^{\mathrm{LTH}} = \varepsilon_{bu,t}^{\mathrm{HTL}} + \sqrt{\varepsilon_{bu,t}^{\mathrm{HTL}}(\varepsilon_{bu,t}^{\mathrm{HTL}} - \varepsilon_{bu,0})}. \qquad (8)$$



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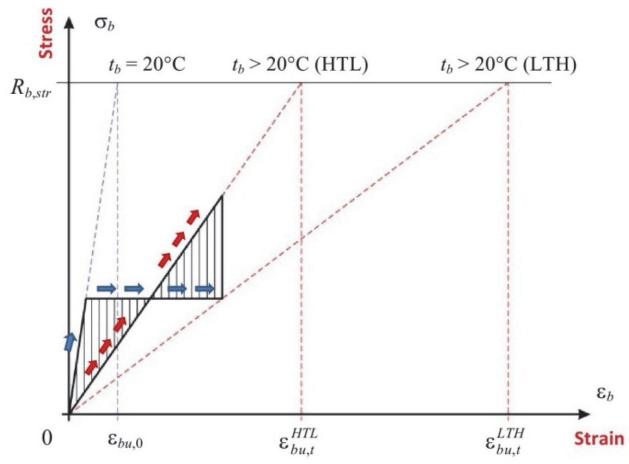

b

Figure 5. Isothermal stress-strain diagrams of concrete for loading conditions after heating and heating under load (*a*); energy criterion for strain coupling (*b*)

As noted many times by V.M. Bondarenko, besides evidence-based calculation explanation any fact should be given a clear physical interpretation. In the case of concrete, it should be done from the building materials science point of view, which is intended not only to explain the structure and composition of concrete, but also the mechanism of its deformation.

The closest demonstration of considered deformability increase when loaded concrete is being heated is drying creep also known as *Pickett effect* [26]. In normal thermal conditions creep deformations of drying concrete are bigger than ones of sealed (hermetic) samples. It's explained by diffusion of free (not chemically bound) moisture from the micropores of the cement gel into neighboring larger capillary macropores, accompanied by sliding of the layers caused by long-time action of loading.

During high-temperature heating, a similar process happens. It's caused by the migration of free moisture firstly and after 250 °C, when it fully evaporates, it's followed by migration of chemically bounded moisture, released during the dehydratation of concrete stone minerals [27]. The observed facts prove that deformability increases during the heating of loaded concrete so as heated unloaded concrete elasticity module decrease happens during only the first heating and is irreversible [25].

Analytical dependencies description. Principle of normalization

The examples shown above of using the affinity properties of the stress-strain- curves of concrete at different temperatures both under loading after heating and under heating under load are a consequence of a more general *principle of normalization*. According to this principle the processes occurring in the material over time or under the action of the active factor, despite their different physical nature, can be described by analytical dependencies that have a single structure due to the general laws that the observed processes obey.

In the works of V.M. Bondarenko [5; 28, etc.], when analytically describing the ongoing processes of force or environmental resistance of concrete, whether it is the non-equilibrium development of deformations or the advancement of the front of corrosion damage, the solution of the kinetic equation was used. Such a solution shows that the intensity of change of observed process parameter in conditions of environment and energetic constancy is proportional to the deficit of this parameter towards limiting values:

$$\frac{d(y-a)}{dt} = -k(y-a)^m, \tag{9}$$

where y and a – correspondingly present and ultimate values of observed process parameter; t – time; k and m – some empirical characteristics.

Represented equation reflects the hypotheses about entropy damping of non-equilibrium processes. This hypothesis is a consequence of the physicochemical law of mass action, which was formulated by C.M. Guldberg and P. Waage in 1867.

Concerning the conditions of the active action of the destructive factor (temperature or load), which are characteristic of the thermal strength resistance of concrete, the intensity of the change in the observed process parameter is proposed to be taken not only proportional to the deficit of this parameter towards the limiting values, but also to the value of the active factor itself. So, the kinetic equation is written as

$$\frac{dy}{dt} = -k(y-a)ut^{u-1}, \tag{10}$$

where t – a value of active factor; k – a process damping parameter; u – an intensity index.

The solution of this equation makes it possible to use in the analytical description of ongoing processes of various nature (loading, heating, damage accumulation, development of strains or thermal degradation of the mechanical properties of concrete, as well as the development of the temperature field over the cross-section [29]) uniform analytical dependences in the form of exponential functions, which is depending on the given boundary conditions acquire one form or another (Figure 6):

– at $a = 0$ and initial condition $y = A$ at $t = 0$

$$y = A\exp[-kt^u]; \tag{11}$$

– at $a \neq 0$ and initial condition $y = 0$ at $t = 0$

$$y = a(1-\exp[-kt^u]). \tag{12}$$

The first solution at $A = 1$ is suitable for describing the coefficient of reduction concrete stability decrease during heating (for $u > 1$) and the coefficient of thermal degradation of the initial modulus of concrete deformation (for $0 < u \leq 1$):

$$\gamma_{b,t} = \exp\left[-\gamma\left(\frac{t_b - 20}{1000}\right)^m\right]; \tag{13}$$

$$\beta_{b,t} = \exp\left[-\beta\left(\frac{t_b - 20}{1000}\right)^n\right], \tag{14}$$

where t_b – temperature of concrete heating, °C (active action factor); γ, m, β, n – experimental dimensionless parameters for a given concrete composition.

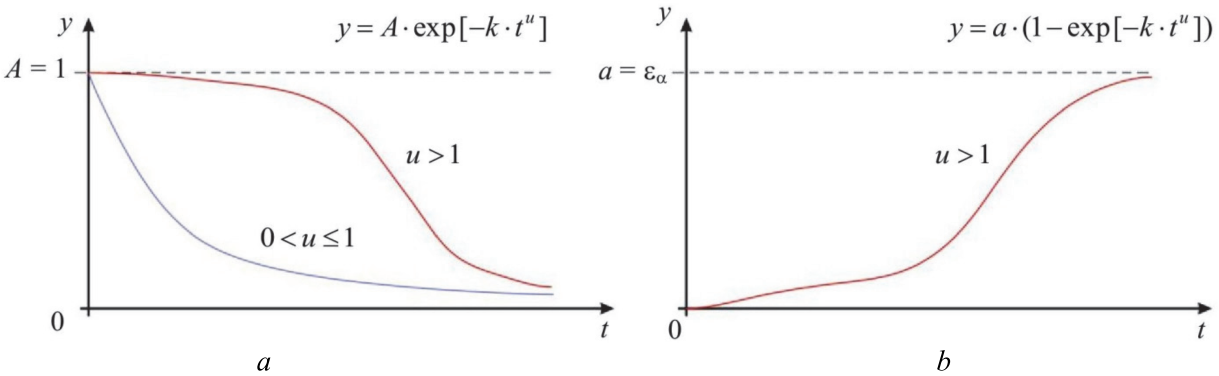

Figure 6. Characteristic dependences of changes in the main thermomechanical parameters of concrete:
a – reduction in strength and initial modulus of deformation during heating; *b* – free temperature strains

Intensity index u is considered here as a parameter that characterizes material resistance to destructive factor t_b: due to its high values (u = 3–5) concrete stability practically doesn't decrease in temperatures below 400 °C. And with u increase stability is constant till higher temperature, but it falls sharply afterward. At the same time values $0 < u \le 1$ make it possible to reflect the sharp falling of deformation modulus at the start of heating and a more gradual subsequent decrease at higher temperatures.

The second solution is supposed to be used for the description of free thermal strain at $a = \varepsilon_\alpha$:

$$\varepsilon_{b,th} = \alpha_1(t_b - 20) + \varepsilon_\alpha\left(1 - \exp\left[-\alpha_2\left(\frac{t_b - 20}{1000}\right)^p\right]\right), \tag{13}$$

where ε_α – maximal temperature strain of concrete; α_1, α_2, p – experimental dimensionless parameters for given concrete composition.

While construing it's taken into consideration that free thermal expansion can possess a linear component, characterized by α_1 coefficient, and a non-linear one, described by exponential dependence with α_2, p parameters, and ε_α tending to its maximum value. Because of fire shrinkage, the given equation doesn't consider deformation decrease in temperatures over 600 °C.

The solution of the equation mentioned above can also be used for the description of the connection between stress and strains of concrete. Achieved strain level $\eta_\varepsilon = \varepsilon_b / \varepsilon_{bu}$ is considered an active acting factor because strains are linearly related to structural stress and their value forms an accumulation of concrete structural damages as a result of force action [30].

Assume as starting a ν_b dependence of concrete in form of non-linear Hooke's law while considering coefficient as an integral function of the density distribution of structural bonds strength. Write an expression for σ–ε elasticity coefficient (secant modulus) as an exponential function of deformations level η_ε with k, u parameters like in (11).

$$\sigma_b = \varepsilon_b E_b \nu_b. \tag{14}$$

$$\nu_b = \exp[-k(\eta_\varepsilon)^u]. \tag{15}$$

Unlike in previously mentioned cases *k*, *u* parameters aren't independent and are defined by limiting conditions to which should correspond a stress-strain curve.

Parameter *k* defines conditions that at the top of the diagram ($\eta_\varepsilon = 1$) elasticity coefficient is equal to its limiting value ν_{bu}:

$$\nu_{bu} = \exp[-k]. \tag{16}$$

$$k = -\ln \nu_{bu}. \tag{17}$$

The parameter *u* is found from the condition that the angle of arrival of the tangent at the top of the diagram is zero. Calculate the first derivative:

$$\frac{d\sigma_b}{d\varepsilon_b} = E_b\nu_b + \varepsilon_b E_b \nu_b \frac{d\nu_b}{d\varepsilon_b} = E_b\nu_b + \varepsilon_b E_b \nu_b \left(-ku\left(\eta_\varepsilon\right)^{u-1}\right)\frac{1}{\varepsilon_{bu}}. \tag{18}$$

At $\eta_\varepsilon = 1$ elasticity coefficient $\nu_b = \nu_{bu}$, strain $\varepsilon_b = \varepsilon_{bu}$, substituting into (18) and equating to zero, obtain

$$\frac{d\sigma_b}{d\varepsilon_b} = E_b\nu_{bu} + E_b\nu_{bu}\left(-ku\right) = E_b\nu_{bu}(1-ku) = 0, \tag{19}$$

where

$$u = \frac{1}{k}. \tag{20}$$

Finally, the expression for the elasticity coefficient is:

$$\nu_b = \exp\left[-k(\eta_\varepsilon)^{1/k}\right]. \tag{21}$$

Proposed dependence corresponds to limiting conditions, which should be complied with by the curve of connection between stress and strains. This dependence also meets the requirements of continuity and differentiability over the entire range of concrete strain values. These properties do not create difficulties for its practical application in numerical analysis.

Conclusion

Evolving the ideas of V.M. Bondarenko and standing on the basic premises of the calculation model of concrete force resistance proposed by him, we propose basic principles of analysis of loaded concrete behavior in sharp-regime temperature action conditions.

Division of total strain components supposes one to consider un-force (temperature, moisture, etc.) and force (moment and lagged) partial strains which are taken as independent.

We suppose to divide mechanical strains into linear and non-linear depending on whether they cause damages accumulation in the concrete structure and reduce its stability in relation to ultimate structural stresses. The principle of superposition is right for linear deformations. Non-linear deformations are estimated using the elasticity coefficient which reflects the material structure degradation index.

Regardless of the level, regime, and duration of loading and temperature actions mechanical strains are described by the affinity principle.

Remise about the variability of characteristics under conditions of environmental and energy constancy follows non-linear entropy laws, used by V.M. Bondarenko for the analytical description of anticorrosion, age, and other resistances was extended to the scope of the active destructive factor (temperature, load, etc.). This has made possible its application in the description of thermomechanical equations of concrete state.

Considering the concrete resistance to degradation and deformation from the structural changes and statistics point of view is mostly a staged direction. Nevertheless, it allows one to make a based and consistent explanation of many observed patterns of concrete behavior. And in this sense, V.M. Bondarenkos favorite citation of the famous mathematician and science popularizer G. Pólya (1887–1985) should be recited: "Nobody has reached Polar Star yet, but many had found the right way looking at it."

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