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Procedia Engineering 117 (2015) 970 – 979

Procedia Engineering

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International Scientific Conference Urban Civil Engineering and Municipal Facilities, SPbUCEMF-2015

The Strength and Strain of High-Strength Concrete Elements with Confinement and Steel Fiber Reinforcement including the Conditions of the Effect of Elevated Temperatures

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Abstract

The article introduces the results of the experimental research of the effect of confinement and steel fiber reinforcement on the strength and strain of the compressed elements of high-modified as well as high-strength SFRC. Analytical expressions to describe the strength and strain of elements with confinement reinforcement under axial compression have been proposed. The results of the research of the effect of concrete age and temperatures elevated up to 150°C on the strength and strain of high-strength SFRC have been introduced.

Peer-review under responsibility of the organizing committee of SPbUCEMF-2015

Keywords: high-strength concrete, confinement reinforcement, steel fiber reinforced concrete (SFRC), short-term heating, elevated temperatures, axial compression, strain, strength.

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Peer-review under responsibility of the organizing committee of SPbUCEMF-2015
doi:10.1016/j.proeng.2015.08.192

1. Introduction

Application of modern high-strength concrete is the most effective way to reduce the weight of structures, the work content and the cost of their construction. An effective way to increase the bearing capacity of heavily loaded columns as well as parts of structures under local compression is application of confinement and fiber reinforcement [1-12]. The use of high-strength SFRC in the erection of structures designed to work in conditions of elevated technological temperatures seems perspective. The issue of the influence of the effectiveness of the above mentioned types of reinforcement as well as the influence of temperature effects on the strength and compressive deformation of structural elements has been studied insufficiently, nowadays, in reference to high-strength modified concrete.

2. Concrete mix proportion and test specimens

2.1. Specimens with confinement reinforcement

High-strength concrete with organic-mineral modifier 10-01 MB [13] in the amount of 20% of cement weight containing silica fume, fly ash, fluidifier [14] and concrete hardening controller has been used in the tests. Concrete mix components: 490 kg/m^3 of Portland cement 42.5 (M500); 549 kg/m^3 of silica sand with the fineness modulus of 1.9; 1100 kg/m^3 of crushed granite fractions 5...20 mm; 153 l/m^3 of water. The specimens maturing in formworks has lasted one day, subsequently they have been stored for 27 days in a room with the air temperature of $T = 20° \pm 2°C$ at a relative humidity of 90%.

Concrete mix characteristics: volume-weight = 2400 kg/m^3; slump = 21 cm; water-cement ratio W/C = 0.31; water-binding ratio W/(C + MB) = 0.26; sand-binding ratio S/(C + MB) = 0.93.

Test specimens are prisms with dimensions of 150×150×600 and 250×250×650 mm. Concrete specimens are reinforced with transverse mesh fabric reinforcement in combination with longitudinal rebars.

Fabric reinforcement for specimens with the section of 150×150 mm consist of reinforcement ∅5 B500 with the mesh of 40 mm, mesh spacing equal to 60 and 100 mm ($\rho_w = 2.2$ and 1.3% respectively). Longitudinal reinforcement used is 4∅10 A400C ($\rho_l = 1.4\%$). Prisms with the section of 250×250 mm have been reinforced with fabric reinforcement of rebars ∅10 A400C with the mesh of 70 mm, mesh spacing of – 60 and 100 mm ($\rho_w = 5.0$ and 3.0% respectively). Longitudinal reinforcement used is 4∅14 A400C ($\rho_l = 1.0\%$).

2.2. Specimens with steel fiber reinforcement

The proportion of high-strength concrete mix is: 545 kg/m^3 of Portland cement 42.5 (M500); 660 kg/m^3 of silica sand with the fineness modulus of 1.9; 870 kg/m^3 of crushed granite fractions 5-20 mm; 190 kg of the organic-modifier modifier; 153 l/m^3 of water.

Organic-mineral modifier has been obtained by using industrial secondary raw materials from the Donetsk region (Ukraine) and is available as a dry mix. Modifier proportion (in % of the weight) is: silica fume in a composition of SicaFume (20%), fine ash-and-slag composite from the Uglegorskaya Thermal Power Plant (30%), fluidifier [14] (2%) and fly ash from the Zuyevskaya Thermal Power Plant (48%).

Three series of specimens with the weight content of fiber per 1 m^3 of concrete (in % of the volume) respectively: 1st – 0 kg ($\rho_{sfb} = 0\%$); 2nd – 50 kg ($\rho_{sfb} = 0.6\%$); 3rd – 200 kg ($\rho_{sfb} = 2.5\%$) have been tested.

Steel fiber with clinched ends produced by the Private Joint Stock Company “Production Association „STALKANAT-SILUR“ (Khartsyzsk, Donetsk region) [15] with the following characteristics: length $l = 60 \pm 6$ mm, diameter $d = 0.75 \pm 0.07$ mm; clinch length and clinch height respectively $l_1 = 5.0 \pm 1.0$ mm, $h_1 = 2.9 \pm 0.5$ mm; mean tensile strength value – $f_{yk} = 1160\text{-}1290$ MPa (ultimate tensile strength value $f_{yk,u} = 1325$ MPa) has been used.

Experimental specimens are cubes with the edge of 150 mm and prisms with the dimensions of 150×150×600 mm.

3. Test procedure

The programs of the experimental research include the study of the basic laws of test specimens deformation and fracture under axial compression. Measurable values during loading of test specimens are: longitudinal compressive force, longitudinal and transverse specimen deformations at each stage of loading.

The main variable parameter for specimens with confinement reinforcement is the percentage of transverse reinforcement: ρ_w = 1.3%, 2.2%, 3.0% and 5.0%. For specimens of this group, longitudinal and transverse rebars deformations have been measured additionally.

The main straining processes characteristics of steel fiber reinforcement samples under the study are stress-strain curves (“$\sigma_1 - \varepsilon_1$, ε_2, ε_3”), tangent modulus of elasticity E_{cm}, axial compression strength $f_{cm,sfb}$ and compressive strains ε_{c1}, ε_{c2}, ε_{c3} at peak stresses. Variable factors of influence are the percentage of three-dimensional reinforcement ρ_w = 0%, 0.6%, 2.5%; the age of the SFRC by the time of loading t_o = 28, 90 and 360 days; the test temperatures T = 20°, 90° and 150°C (short-term heating).

The heating of prism specimens for testing has been carried out using electric heating elements of special design consisting of three layers of heat-resistant fabric and an electric heating coil of nichrome wire. The heating temperature of the coil was controlled by changing the voltage in coil electrical circuit via the voltage regulator. The temperature rise rate in each heating step is 12-15°C per hour. The duration of short-term heating before press loadings at 90° and 150°C has been 15 and 13 hours respectively. This corresponds to the time during which usual heavy-weight concretes achieve the minimum strength under the same test heating temperature rates [16].

The compressive loading of test specimens has been performed according to the method [17] with a hydraulic press with the tonnage of 10 000 kN. The measurement of longitudinal and transverse strains of test specimens has been carried out using indicating gages. The accuracy of relative linear deformations measurement has been about $1 \cdot 10^{-5}$.

4. The results of the experimental research

4.1. The strain and compressive strength of concrete elements with confinement reinforcement

The effects of confinement reinforcement on the strength and strain of the prism specimens of high-strength concrete are similar to the laws identified during the research of usual heavy-weight concretes [9-11], namely: the higher the percentage of reinforcement is, the higher the increased strength and compressive strain compared with the plain concrete are. For prism specimens with the edge of 150 mm (f_{cm} = 69.2 MPa, ρ_w = 1.3% and 2.2%) the reduced strength $f_{cm,red}$ has risen on average by 1.31 and 1.43 times compared with the plain concrete (Fig. 1a), and the ultimate compressibility $\varepsilon_{c1,w}$ by 1.41 and 1.75 times respectively (Fig. 1b). For prism specimens with the edge of 250 mm (f_{cm} = 82.5 MPa, ρ_w = 3.0% and 5.0%) $f_{cm,red}$ has grown, on average, by 1.41 and 1.48 times compared with the plain concrete (Fig. 1a), and the ultimate compressibility $\varepsilon_{c1,w}$ by 1.7 and 2.4 times respectively (Fig. 1b).

The failure behavior of prism specimens with the increase of the percentage of confinement reinforcement has been smoother with a higher proportion of plastic deformation. The failure has been accompanied by the spalling of concrete outside the fabric reinforcement circuit and the yield of reinforcement. Specimens with fabric reinforcement ∅5 B500 have acquired gaps in the fabric of reinforcement bars.

As previously said the experimental data of the influence of confinement reinforcement on the strength and ultimate strain of reinforced concrete elements are shown in Fig. 1. The possibility of application of the methodology SNiP 2.03.01-84* [12] to calculate the strength of elements with confinement reinforcement of high-strength concrete classes C50/60…C65/80 (B60…B80) has been confirmed (Fig. 1):

$$f_{cm,red} = f_{cm} + \varphi \cdot \rho_w \cdot f_{ywd}. \quad (1)$$

Here:

f_{cm} is the value of unreinforced concrete prism compressive strength (given in MPa);
f_{ywd} is the design strength of the fabric reinforcement (given in MPa);

ρ_w is the reinforcement ratio numerically equal to the ratio of the steel volume in fabric reinforcement to the volume of its surrounding concrete;

φ is the effectiveness factor of the confinement ratio given by [21]:

$$\varphi = \frac{1}{0.23 + \psi}, \tag{2}$$

where $\psi = \dfrac{\rho_w f_{ywd}}{f_{cm} + 10}$. (3)

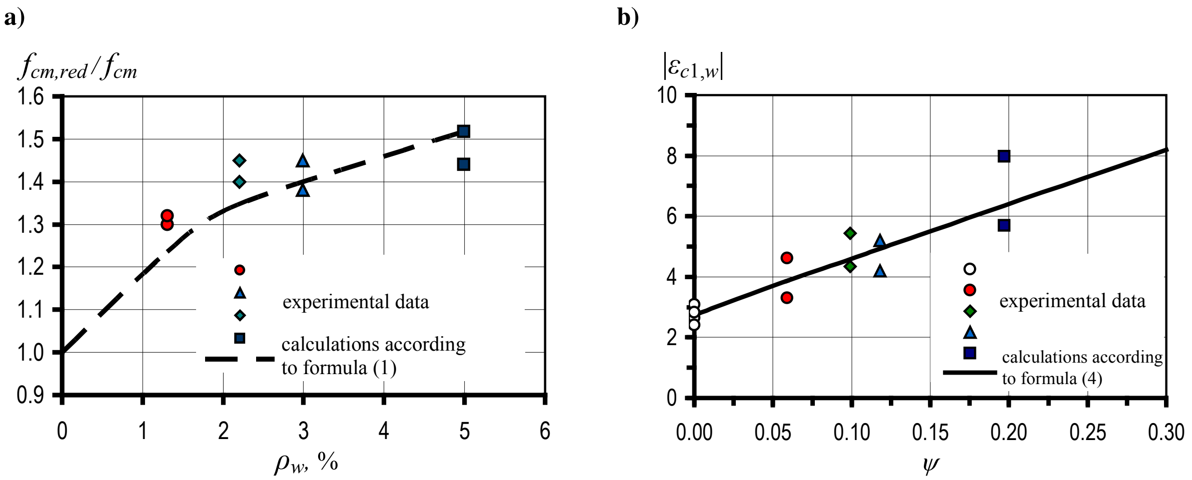


Fig. 1. The influence of confinement reinforcement on the reduced strength (a) and ultimate compressibility (b) of concrete.

For the analytical description of the deformations of high-strength concrete elements with confinement reinforcement, the following expressions have been offered:

- a modified analytical expression to determine the ultimate strains $\varepsilon_{c1,w}$ of structural elements depending on the concrete ultimate compressibility ε_{c1} and intensity of confinement reinforcement on the basis of the formula [20]:

$$|\varepsilon_{c1,w}| = |\varepsilon_{c1}| + 18 \cdot \psi \cdot 10^{-3}; \tag{4}$$

- a modified expression as a polynomial in the 3rd degree on the basis of proposals [22] to describe the dimensionless constitutive law of elements with confinement reinforcement:

$$\eta_\sigma = a_1 \cdot \eta_\varepsilon + a_2 \cdot \eta_\varepsilon^2 + a_3 \cdot \eta_\varepsilon^3, \tag{5}$$

where $\eta_\sigma = \dfrac{|\sigma_{c,red}|}{f_{cm,red}}$; $\eta_\varepsilon = \dfrac{\varepsilon_{c,w}}{\varepsilon_{c1,w}} \le 1$; $a_1 = 3$; $a_2 = -3$; $a_3 = 1$.

Expressions (4) ÷ (5) allow to describe accurately the stress-strain curves of high-strength concrete elements with the intensity of confinement reinforcement in the range up to $\rho_w = 5.0\%$ (Fig. 2).

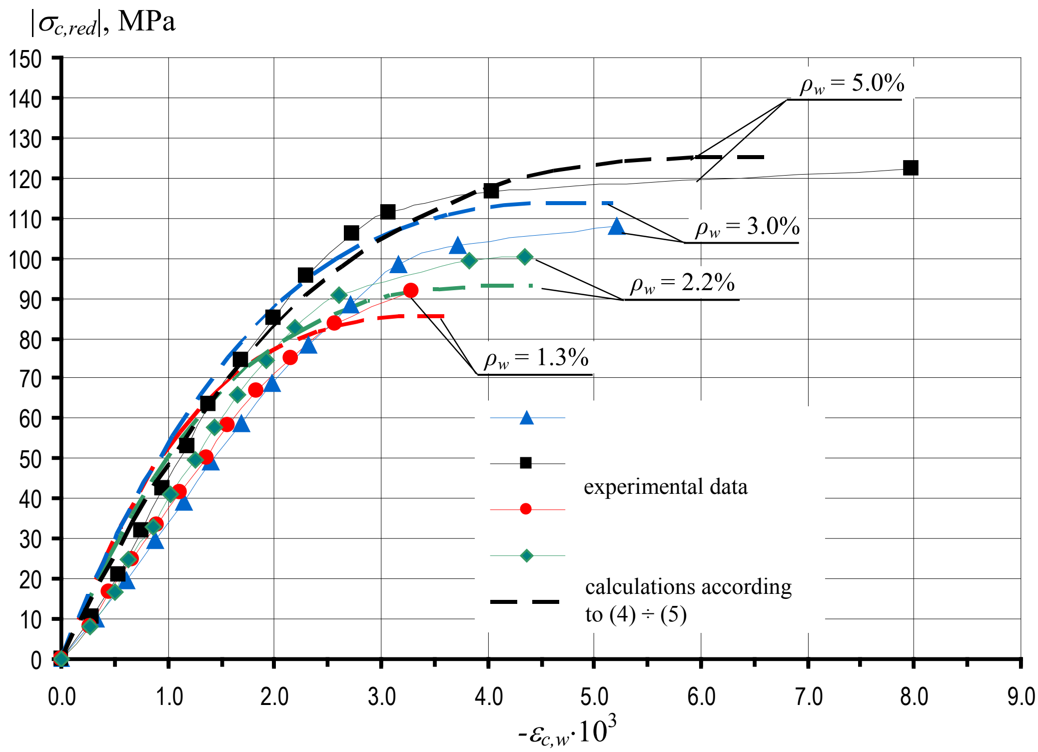


Fig. 2. Constitutive laws of reinforced prism specimens made of high-strength modified concrete with confinement reinforcement.

4.2. The compressive strain and strength of high-strength SFRC at normal temperature

Mean values of prism strength $f_{cm,sfb}$ of 150×150×600 mm standard prism specimens under axial compression at normal temperature have shown the following values: 83.2 MPa for $\rho_{sfb} = 0\%$, 89.8 MPa for $\rho_{sfb} = 0.6\%$ and 102.5 MPa for $\rho_{sfb} = 2.5\%$. The strength increase for specimens with steel fiber reinforcing in the range of $\rho_{sfb} = 0.6$-2.5% has been 10-23% compared to the one of high-strength concrete unreinforced specimens ($\rho_{sfb} = 0\%$). The diagrams of the dependence of prism strength and ultimate compressibility of specimens of high-strength SFRC on reinforcement ratio are shown in Fig. 3.

At the age of 28 days the mean value of prism strength $f_{cm,sfb}$ of standard prism specimens with $\rho_{sfb} = 0.6\%$ under axial compression has been 89.8 MPa. At the age of 90 days the SFRC strength compared to the corresponding one at the age of 28 days, has been 3-5% higher and, at the age of 360 days – 7-11% higher. The mean value of the ratio of prism strength $f_{cm,sfb}$ to the cube strength $f_{sfb,cube}$ of high-strength SFRC does not substantially depend on the age of concrete and is equal to $K_{ps} = f_{cm,sfb} / f_{sfb,cube} = 0.88$. The values of tangent modulus of elasticity E_{cm} under axial compression amount on average to $53.6 \cdot 10^3$ MPa and do not significantly depend on the age of concrete at the time of the test.

Mean values of ultimate deformations of shortening for specimens tested at the age of 28, 90 and 360 days amount to $\varepsilon_{c1,sfb} = 2.53 \cdot 10^{-3}$, $2.64 \cdot 10^{-3}$ and $2.82 \cdot 10^{-3}$ respectively. At loading levels up to $\eta_\sigma = \sigma_c / f_{cm,sfb} = (0.8\text{-}0.9)$ the deformation behavior of the modified high-strength SFRC is close to the elastic one. Deformations of the relative change of SFRC volume have developed almost linearly up to the levels of loading $\eta_\sigma = 0.88$-0.92 of the prism strength $f_{cm,sfb}$.

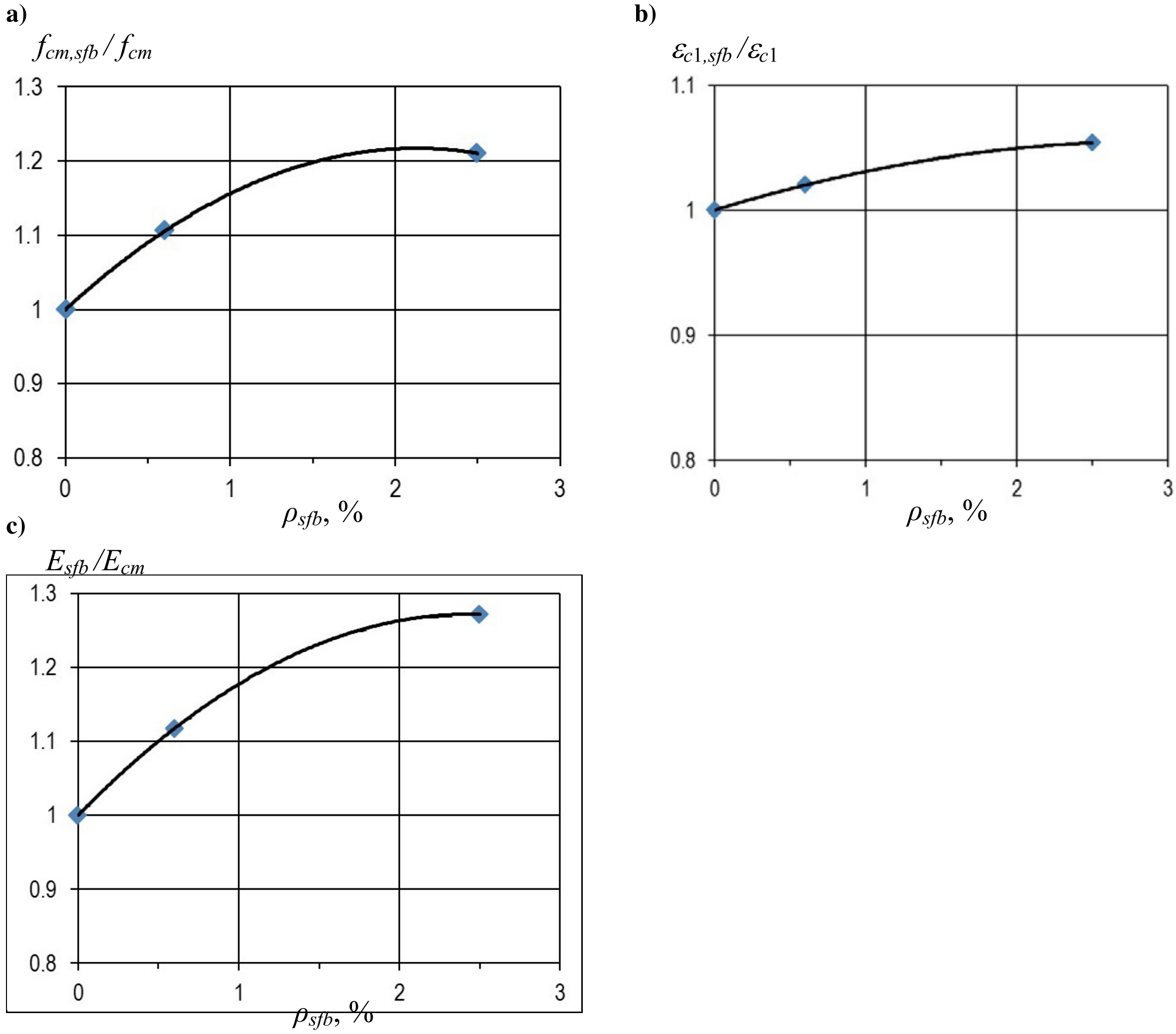
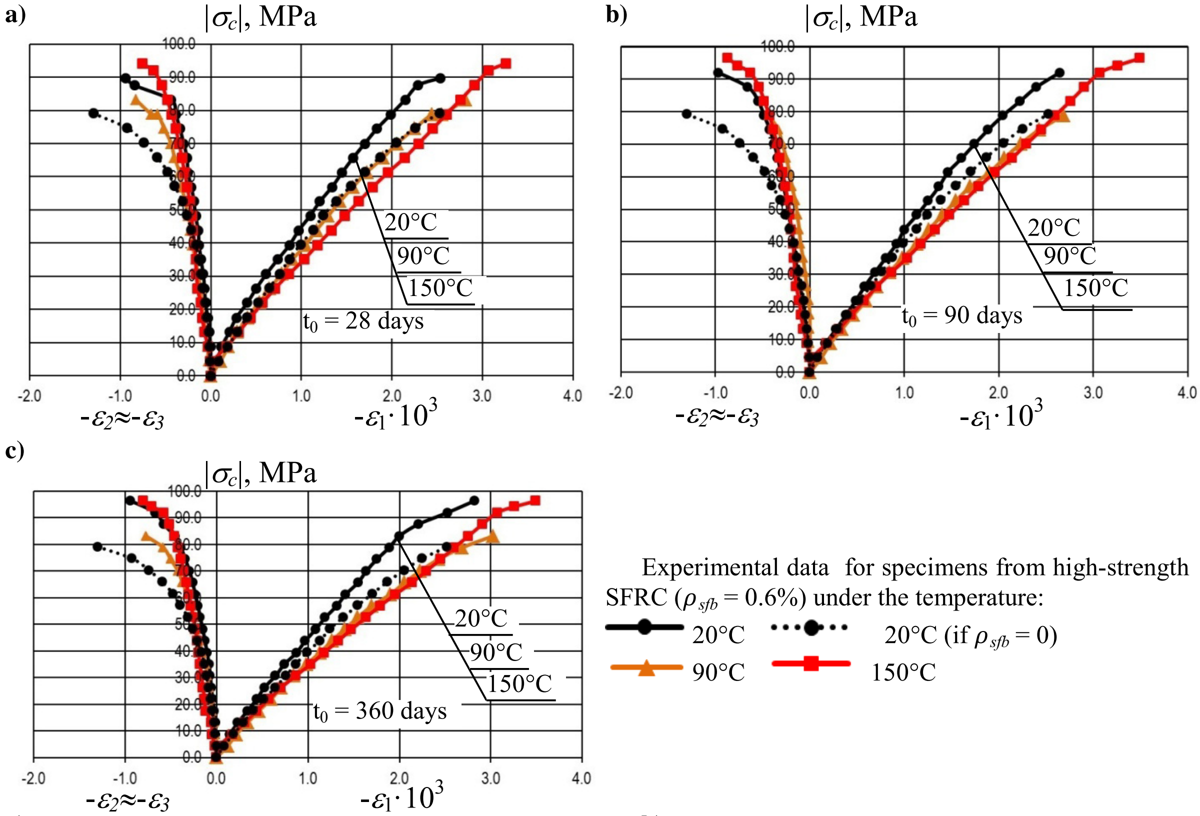


Fig. 3. The effect of fiber reinforcement ratio ρsfb on the prism strength (a), ultimate compressibility (b) and tangent initial modulus of elasticity (c) of high-strength SFRC.

4.3. The effect of elevated temperatures on the strength and strain of high-strength SFRC

The changes of the stress-strain diagrams of high-strength SFRC under axial compression at normal and elevated temperatures during short-term heating at the age of 28, 90 and 360 days are shown in Fig. 4, 5. Loading levels practically corresponding to the elastic straining of concrete are nearly the same for all the temperatures tested and are in the range of 0.8-0.9 of the prism strength values $f_{cm,sfb}$. The process of the elastic reduction of specimens' volume of high-strength modified SFRC has been replaced by its increase (manifestation of the dilatation effect) at loading levels of about $0.9{\cdot}f_{cp,sfb}$ (Fig. 4, 5).

The strength $f_{cp,sfb}$ of specimens of high-strength SFRC at the age of 28, 90 and 360 days at the first short-term heating up to temperatures 90° and 150°C have been 94% and 104%, 96% and 105%, 95% and 100% respectively of the strength values of concrete which has not been exposed to heating (Fig. 6a). At the temperature of the first short-term heating of 90°C the maximum reduction of the strength of high-strength SFRC has been approximately 6% which is slightly less than the reduction of the strength of plane high-strength concrete [23-26] and significantly lower than the strength reduction at the same heating temperature of usual heavy-weight concretes [16, 23-24]. The increase of the strength of high-strength SFRC compared with the corresponding one of concrete without heating is characteristic for the temperature of short-term heating equal to 150°C (Fig. 6a).



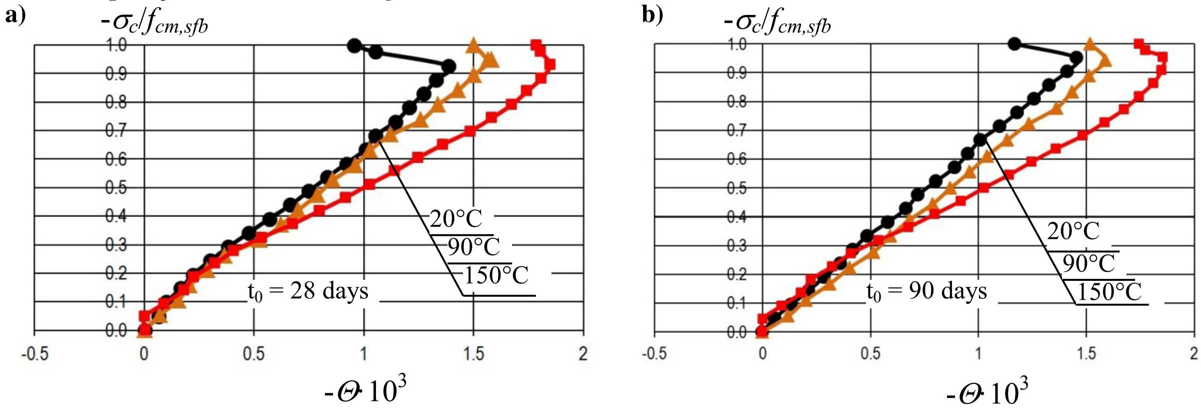


Fig. 4. The effect of short-term heating at temperatures of 90° and 150°C on deformation curves under axial compression at the age of t_o = 28, 90, and 360 days (a, b, c).

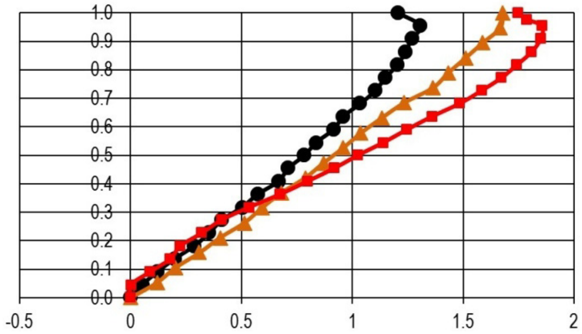


Fig. 5. The effect of short-term heating at temperatures of 90° and 150°C on volume strain of high-strength SFRC (□sfb = 0.6%) under axial compression at the age of to = 28, 90, and 360 days (a, b, c). The legend is the same as on Figure 4.

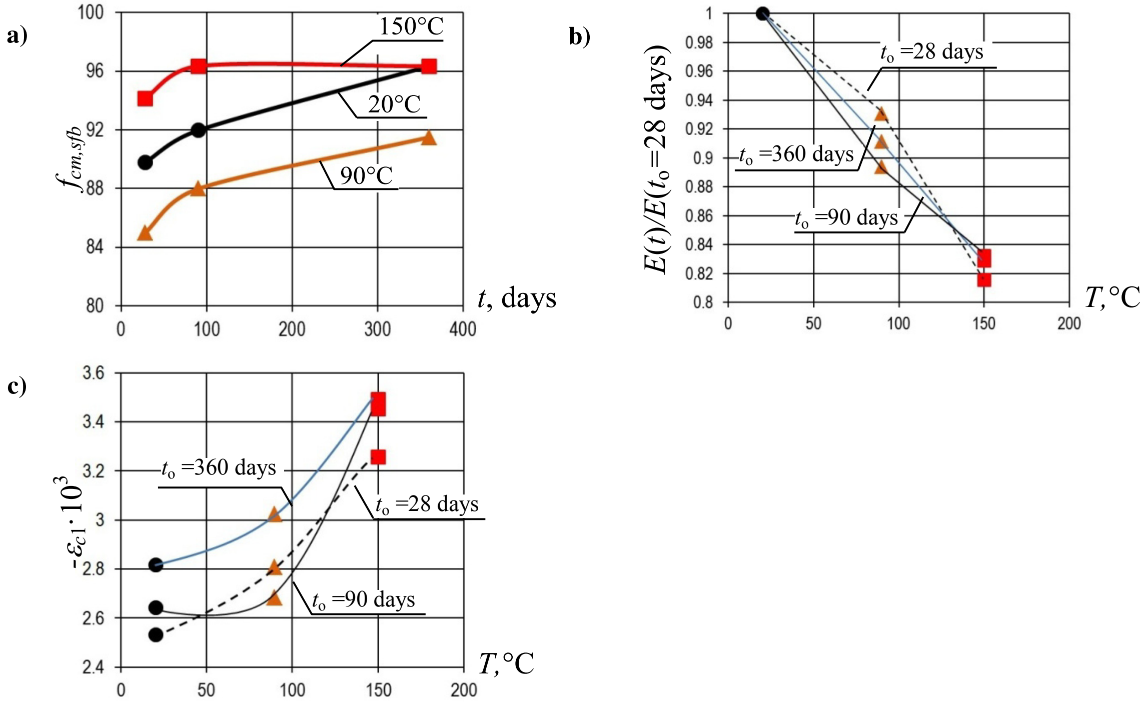


Fig. 6. The effect of the age to and the temperature T of short-term heating on the strength (a), tangent modulus of elasticity (b) and ultimate compressibility (c) of high-strength SFRC (□sfb = 0.6%) under axial compression at the age of to = 28, 90, and 360 days. The legend is the same as on Figure 4.

The values of the tangent modulus of elasticity $E_{ck}(t_o, T)$ of specimens of high-strength SFRC at the age of 28, 90 and 360 days during the first short-term heating up to 90° and 150°C decrease relatively to the values of concrete unexposed to heating by 8% and 18.5% respectively (Fig. 6b).

The ultimate compressibility of specimens of high-strength SFRC during the first short-term heating up to 90° and 150°C has increased on average by 12% and 28% respectively compared the values at normal temperature (Fig. 6c). In this case, the relative values of strain increment do not significantly dependent on the age of concrete by the beginning of the heating.

5. Conclusions

1. Confinement reinforcement of elements of high-strength modified concrete with the intensity of ρ_w = 1.3-5.0% increases the reduced prism strength of concrete by 30-75% and the ultimate strains of shortening ε_{c1} at maximum stresses by 1.4-2.4 times, in comparison with plane concrete.

2. The reduced prism strength of modified concrete classes up to C65/80 (B80) with confinement reinforcement can be determined with the sufficient accuracy according to formula (1).

3. The proposed modified analytical expressions (4) ÷ (5) allow to determine accurately the strains of elements of high-strength modified concrete with confinement reinforcement under axial compression loading.

4. Application of fiber reinforcement in the amount ρ_{sfb} = 0.6-2.5% leads to the increase of strength under axial compression of high-strength SFRC $f_{cm,sfb}$ on average by 10-23%, tangent modulus of elasticity E_{sfb} – by 12-28%, ultimate strain of shortening $\varepsilon_{c1,sfb}$ – by 2-6%.

5. As for the conditions of short-term heating up to 150°C application of fiber reinforcement increases the compressive strength value of high-strength SFRC on average by 4% and the tangent modulus of elasticity by 34% compared with the corresponding values for high-strength plane concretes at the same temperatures.

6. The values of the relative change of strength, tangent modulus and ultimate compressibility of high-strength SFRC are more dependent on the heating temperature rather than on the age of concrete at the time of loading.

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