SELF-STRESSED CONCRETE MEMBERS REINFORCED WITH FRP-BARS

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Summary:
Problem of usage fiber reinforced polymer as reinforcing bars in self-stressed flexuring structural elements is considered. Comparison of reaching degree of self-stress in concrete, which was got on the stage of self-stressing and was got like result of calculation with usage theoretical equations is produced.

Keywords: reinforcement, FRP bars; self-stressing (expansive) concrete; value of the self-stress; reinforcement ratio.

Introduction

Concrete members with reinforcement made of fiber reinforced polymer (FRP) bars find a wide application in the world practice of manufacturing building structures, exploting under the influence of the aggressive environment and industrial buildings of special purpose. As a polymer matrix for manufacturing FRP bars use different thermoreactive, thermoplastic or hybrid polymers and for it's reinforcing use different types of fibers: glass, polymer, basalt, carbon and others.

High strength, low density, high corrosion resistance in comparison with steel, possibility of regulation in wide range of conductance and radioparency depending on the type of reinforcing fiber.

It should be noted, that FRP bars have high strength characteristics (the ultimate tensile strength comes up to 1 200 N/mm^2) and low modulus of elasticity (it's about 32 000 – 55 000 MPa). This feature of FRP bars creates difficulties with it's utilization. The main problems appear at service limit state (SLS) analysis (deflection and cracking control). According with sufficiently conservative guidelines of (*fib* 2005) the partial factor γ_{cf} shall be set equal to 1,5 for ULS, that substantially reduces it's design strength in comparison with value of characteristic strength; and for SLS stresses in FRP bars shall be limited under $0,3 \cdot f_{frpu}$.

The possibility of creation prestressed structures by means of usage FRP bars (in this case rise up the effectiveness of FRP reinforcement) exists due to perfectly elastic behavior of FRP bars. At the same time, as experience has shown, it's rather laborious and even somethimes impracticable to realize pretensioning of FRP bars with the usage of traditional technology (because of necessity of manufacturing special anchorage systems, tensioning devices and creating special conditions of concrete hardening by means of temperature limitation when heat treatment is applying etc.).

In this case could be effective the utilization of physicochemical method of prestressing, based on the usage of self-stressing (expansive) concrete. Experimentally-theoretical grounds of self-stressing structures are presented in work (Mihajlov, Litver 1974). In work (Tur 1998) is presented hypothesis of conditional reinforcement. In accordance with this hypothesis anyone restriction, that restrains free expansion strains development, can be presented as a quantity of equivalent steel reinforcement. The equivalent steel reinforcement are determined on the basis of equality of the restrictive bars stiffness. Such an approach gives in the first approximation a possibility of usage multiplicative model, that was developed by prof. Mihajlov V.V., Litver S.L., Budagianec L.I. (Mihajlov, Litver 1974) and that was updated by prof. Tur V.V.

It must be noted, that in the case of utilization steel reinforcement could be reached rather high stresses, that are coupled with low restrictive bars strains. This restrictive bars strains can be compensated as a result of concrete shrinkage in the air-dry curing.

FRP bars that have low modulus of elasticity and as a result less stiffness than steel reinforcement, reach more restrained expansion strain in comparison with steel reinforcement coupled with the same with it grade of self-stressing. As a result, greater level of the self-stressing would be saved in FRP bar after development concrete shrinkage in the air-dry curing.

Experimental researches

With the aim of checking offered theoretical approaches for evaluating self-stress value in the concrete members reinforced with FRP bars, special laboratory researches were done. For this researches were done specimens-prisms of series I and series II. The specimens-prisms had square cross section (100x100 mm) and were reinforced uniaxially with single bar. The degree of restraint (it depends on longitudinal axial stiffness of the restrictive bars); kind of the expansive concrete and it's self-stressing grade were variables parameters during this researches. One specimen-prism (reference specimen) reinforced with traditional steel bar (diameter 8 mm) that is equivalent to GFRP bar (diameter 14 mm) in terms of equality of the axial stiffness in the series I and series II was done. It was done with the aim of checking all degrees of self-stressing, that were reached during concrete expansion in the all specimens-prisms of series I and series II.

Program of the experimental researches is presented in the table 1, schemes of the experimental specimens-prisms are presented on the figure 1.

Tab. 1 Program of the experimental researches

Series of specimens	Specimens-prisms designation	Reinforcement		
		Reinforcement area $A_{f(s)}$, mm^2	Reinforcement ratio $\rho_{f(s)}$, %	Equivalent reinforcement ratio $\rho_{l,eff}$, %
Series I	PECC-1	12.56 / 1Ø4	0,126	0,035
	PECC-2	28.26 / 1Ø6	0,283	0,078
	PECC-3	50.24 / 1Ø8	0,505	0,139
	PECC-4	78.50 / 1Ø10	0,791	0,218

Series I	PECC-5	$\frac{153.86}{1\text{Ø}14}$	1,563	0,430
	PECS-6	$\frac{50.24}{1\text{Ø}8}$	0,505	0,505
Series II	PEFC-1	$\frac{12.56}{1\text{Ø}4}$	0,126	0,035
	PEFC-2	$\frac{28.26}{1\text{Ø}6}$	0,283	0,078
	PEFC-3	$\frac{50.24}{1\text{Ø}8}$	0,505	0,139
	PEFC-4	$\frac{78.50}{1\text{Ø}10}$	0,791	0,218
	PEFC-5	$\frac{153.86}{1\text{Ø}14}$	1,563	0,430
	PEFS-6	$\frac{50.24}{1\text{Ø}8}$	0,505	0,505

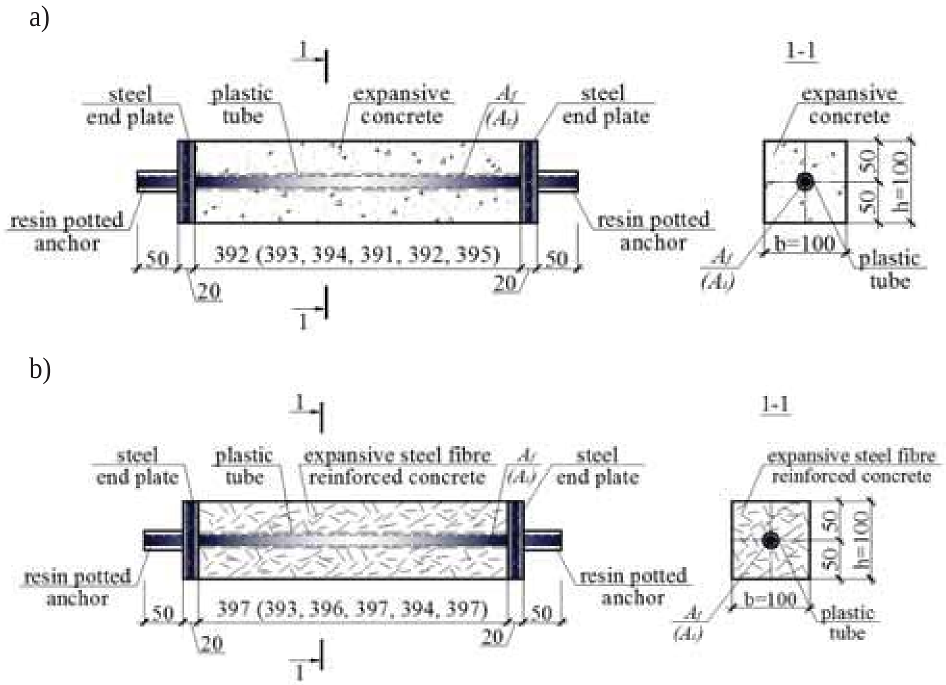


Fig. 1. Schemes of the experimental specimens-prisms
– specimens-prisms of the series I; b) – specimens-prisms of the series II)

For preparation of expansive concrete were used granite crushed stone (fraction 5-10 mm) and sand with fineness modulus equal to 3,4. Water-cement ratio was fixed equal to 0,4 (W/C=0,4). For preparation of 1 m^3 of expansive concrete were used: 500 kg of expansive cement; 960 kg of crushed stone and 750 kg of sand. Material consumption for preparation of 1 m^3 of the expansive concrete was the same as for preparation of 1 m^3 of steel fibre reinforced expansive concrete and was selected proceeding from the requirement of the efficient packing of the fractions (steel fibre consumption for preparation of 1 m^3 of the steel fibre reinforced expansive concrete was equal to 210 kg or 2,68% from the common volume content of the concrete mix or 8,4% from the common mass of the concrete mix). For the reaching required workability of the concrete mix hyper-plasticizer was used.

The main properties of the used expansive concrete were the next: the value of the self-stressing in the standard conductors was equal to 0,86 N/mm^2, the average concrete compressive strength in the free condition was equal to 36,3 N/mm^2. The main properties of the used steel fibre reinforced expansive concrete were the next: the value of the self-stressing in the standard conductors was equal to 0,63 N/mm^2, the average concrete compressive strength in the free condition was equal to 22,5 N/mm^2.

GFRP bars with diameters 4, 6, 8, 10, 14 mm (it's ultimate tensile strength was equal to 1 300 N/mm^2 and it's modulus of elasticity was equal to $E_{f(s)}$=55 000 N/mm^2) were used for restraint of the expansion strains of the self-stressing concrete and steel fiber reinforced self-stressing concrete. Steel reinforcing bars (S500) with diameter 8 mm (it's tensile strength was equal to 620 N/mm^2 and it's modulus of elasticity was equal to $E_{f(s)}$=200 000 N/mm^2) were used as equivalent to GFRP bars with diameter 14 mm in terms of equality of the uniaxial longitudinal stiffness.

Manufactured in accordance with accepted program of the experimental specimens-prisms researches were in the moist curing on the stage of self-stressing concrete expansion (before loading testing).

The following equation was used to compute the value of the self-stress ($\sigma_{CE,p}$) in the experimental specimens-prisms of series I and II expansive concrete:

$$\sigma_{CE,p} = \varepsilon_{CE,f(s)} \cdot E_{f(s)} \cdot \rho_{f(s)}, \tag{1}$$

where: $\varepsilon_{CE,f(s)}$ – GFRP (steel) reinforcing bars restrained strain at the moment of the experimental specimens-prisms of series I and II self-stressing concrete expansion stabilization; $E_{f(s)}$ – GFRP (steel) reinforcing bars modulus of elasticity; $\rho_{f(s)}$ – experimental specimens-prisms of series I and series II reinforcement ratio.

To compute the value of the theoretical self-stress ($\sigma_{CE,th}$) in the experimental specimens-prisms of series I and II expansive concrete were used equations from (TKP 45-5.03-158-2009 2010) and prof. Mihailov's formula (Mihajlov, Litver 1974).

In accordance with (TKP 45-5.03-158-2009, 2010) to compute the value of the theoretical self-stress ($\sigma_{CE,th}$) in the concrete of the experimental specimens-prisms the next equation was used:

$$\sigma_{CE,th} = \varepsilon_{CE,f(s)} \cdot E_s \cdot \rho_{l,eff}, \tag{2}$$

where: $\varepsilon_{CE,f(s)}$ – strain at the point of the centroid of reinforcing restrictive bars cross section (it is computed with usage of equation from (TKP 45-5.03-158-2009 2010)); E_s – steel reinforcement modulus of elasticity; $\rho_{l,eff}$ – equivalent reinforcement ratio, that was computed with usage of the next equation:

$$\rho_{l,eff} = \frac{A_{s,eff}}{A_c} = \frac{\alpha_f \cdot A_f}{A_c}, \tag{3}$$

where: $A_{s,eff}$ – cross sectional area of the steel reinforcement, equivalent to the cross sectional area of the FRP reinforcement; A_c – concrete section area; α_f – ratio, that takes in account steel reinforcement and FRP reinforcement stiffnesses ratio ($\alpha_f = \frac{E_f}{E_s}$); E_f – FRP reinforcement modulus of elasticity; A_f – cross sectional area of the FRP reinforcement.

In accordance with (Mihajlov, Litver 1974) to compute the value of the theoretical self-stress ($\sigma_{CE,th}$) in the concrete of the experimental specimens-prisms prof. V. V. Mihailov's formula was used:

$$\sigma_{CE,th} = 0.085 \cdot \left(f_{CE,d}\right)^{1,25} \cdot \left(\frac{1}{\varepsilon_{CE,f(s)}}\right)^{0,25}, \tag{4}$$

where: $f_{CE,d}$ – design value of the self-stressing; $\varepsilon_{CE,f(s)}$ – reinforcing bar restrained strain at the moment of the self-stressing concrete expansion stabilization.

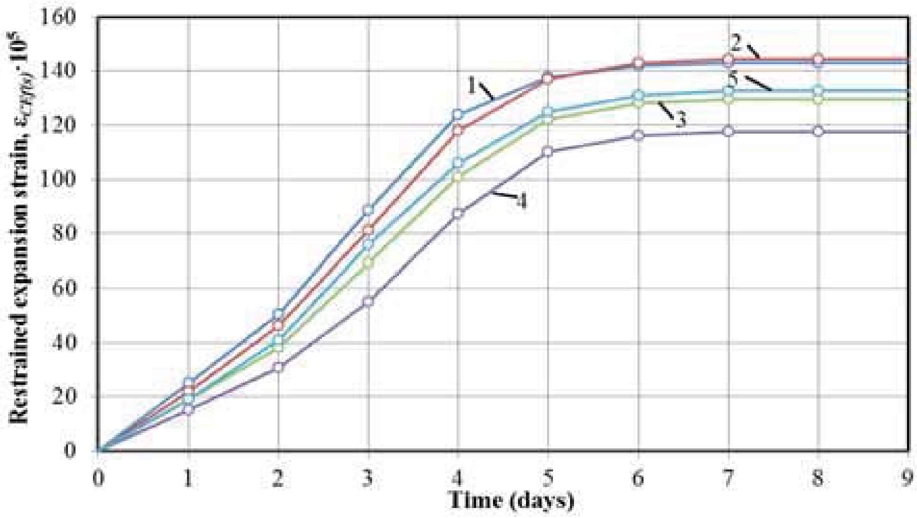
Specimens-prisms of series I and series II value of the average concrete tensile strength (f_{ctm}) was computed with usage of the following equation:

$$f_{ctm} = \frac{\sigma_{flex}}{1,5}, \tag{5}$$

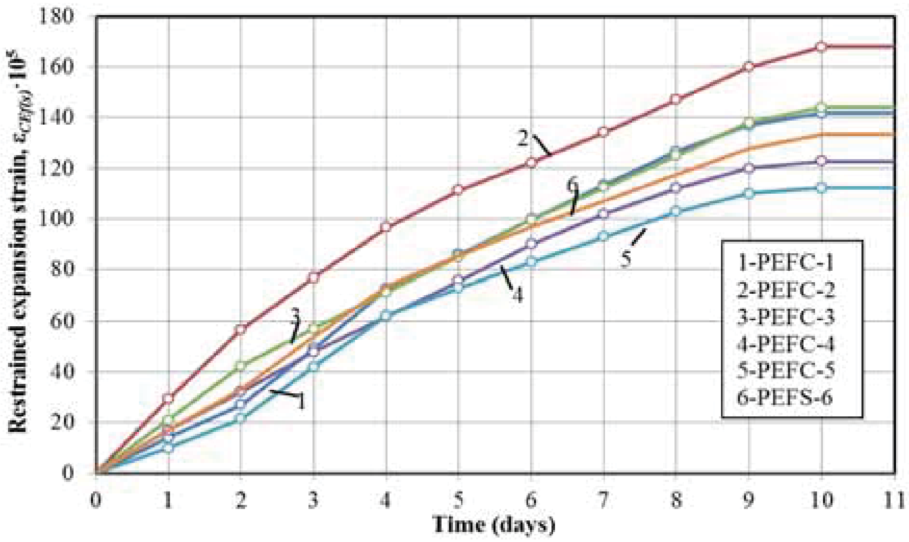
where: σ_{flex} – concrete flexural tensile strength that was got during specimens-prisms of series I and series II testing; 1,5 – conversion factor, applied for transition from concrete flexural tensile strength to concrete uniaxial tensile strength.

Results of the experimental studies

As was mentioned previously, specimens-prisms of series I and II were manufactured with various reinforcement ratio. Diagrams, shown restrained expansion strain development in uniaxially restrained expansive concrete of specimens-prisms of series I and series II (moist curing) are represented on the figure 2.



a)



b)

Fig. 2. Restrained expansion strain development
in uniaxially restrained expansive concrete (moist curing)
(a) – for specimens-prisms of series I; б) – for specimens-prisms of series II)

Tab. 2 Testing results on the phase of specimens-prisms of series I and series II self-stressing concrete expansion stabilization

Series of speci-mens	Specimens-pri-sms designation	Reinforcement ratio $\rho_{f(s)}$, %	Equivalent reinforcement ratio $\rho_{l,eff}$, %	Restrained expansion strain $\varepsilon_{CE,f(s)} \cdot 10^5$ at the moment of the self--stressing concrete expansion stabilization	Self-stress in the concrete $\sigma_{CE,p}$, MPa	Theoretical self-stress in the concrete $\sigma_{CE,th}$, MPa	
						equation (2)	equation (4)
1	2	3	4	5	6	7	8
Series I	PECC-1	0,126	0,035	143	0,1	0,26	0,36
	PECC-2	0,283	0,078	144	0,22	0,37	0,36
	PECC-3	0,505	0,139	129	0,36	0,48	0,37
	PECC-4	0,791	0,218	118	0,51	0,57	0,38
	PECC-5	1,563	0,430	133	1,14	0,71	0,37
Series II	PEFC-1	0,126	0,035	142	0,10	0,19	0,25
	PEFC-2	0,283	0,078	168	0,26	0,27	0,24
	PEFC-3	0,505	0,139	144	0,40	0,35	0,24
	PEFC-4	0,791	0,218	118	0,54	0,42	0,26
	PEFC-5	1,563	0,430	112	0,96	0,52	0,26
	PEFS-6	–	0,505	133	1,34	0,54	0,25

As follows from the diagrams, presented on the figure 2, specimens-prisms restrained strains had almost even development up to the moment of the self-stressing concrete expansion stabilization.

It must be noted that almost all specimens-prisms, regardless of it's actual GFRP reinforcement ratio (r_f varies from 0,126 % to 1,563 %) had rather close restrained strains values at the moment of the self-stressing concrete expansion stabilization ($\varepsilon_{CE,f}$ 0.14%). This is explained by the fact that in terms steel reinforcement effective area (on the basis of uniaxial stiffness equality), equivalent steel reinforcement ratio $r_{l,eff}$ varied over the range from 0,035 % to 0,43 %. For traditional self-stressed structures presented equivalent steel reinforcement ratio are related to the restraint conditions similar to free expansion.

Expansive concrete cross-section of specimens-prisms of series I and II uniaxial stiffness development towards GFRP bars axial stiffness are presented on the figure 3.

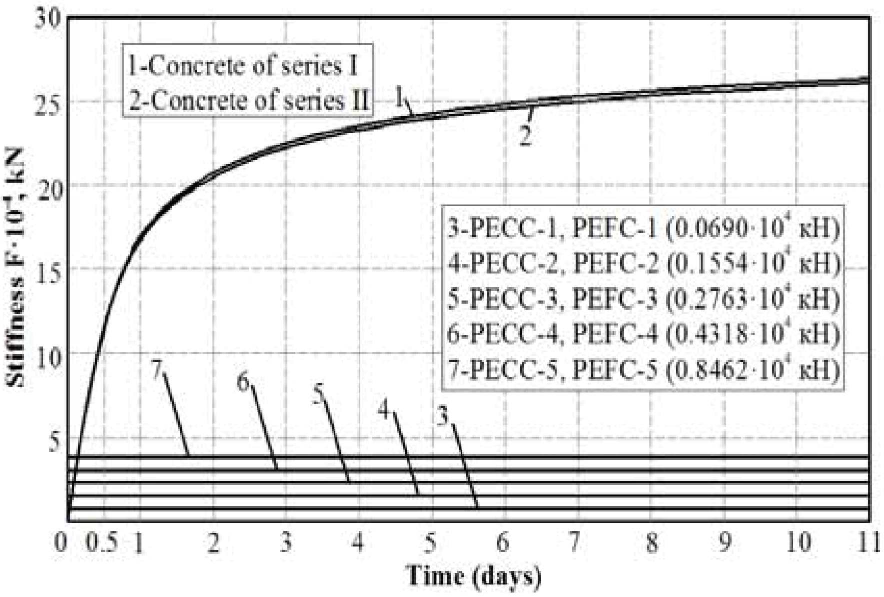
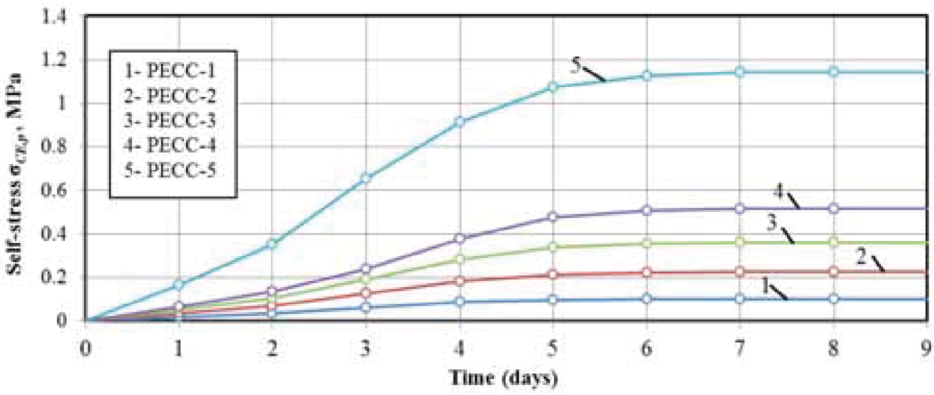
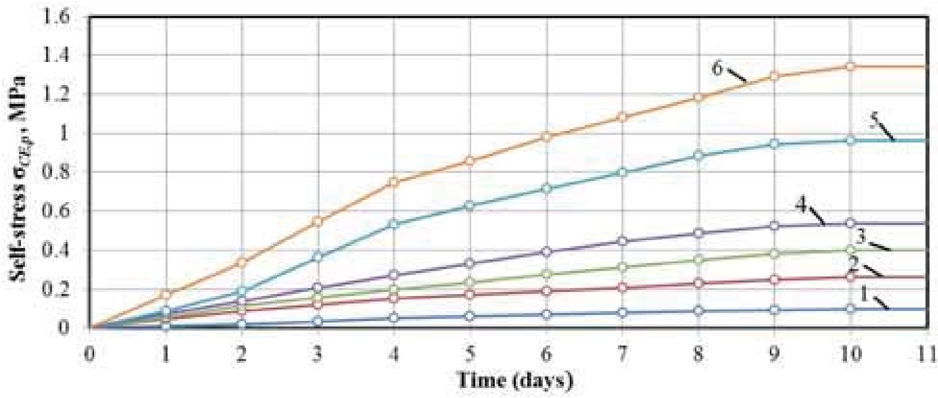


Fig. 3 Expansive concrete cross-section uniaxial stiffness development towards GFRP bars axial stiffness

As follows from the diagram on the figure 3 approximately at the age of 12 hours, concrete element stiffness becomes more than axial stiffness of GFRP bars and in the provided bond conditions (in the presence of steel end plates and resin potted anchors on the GFRP bars end zone) solid-phase expansion realizes. Whereby GFRP bars are exposed to tension that leads to concrete compression (self-stressing). It must be noted that extension of the expansion strains took place in the conditions of rather low compression stresses values. Values of the fixed in the course of experiments self-stresses at the moment of the self-stressing concrete expansion stabilization are presented in the table 2 and self-stress development is presented on the figure 4. As follows from the presented in the table 2 and on the diagram (see figure 4) data, regardless of the GFRP bars reinforcement ratio development of the restrained strains was almost the same for all tested specimens-prisms and approached to the free expansion (specimens with value of the r_f under 0,5 %) strains.



a)

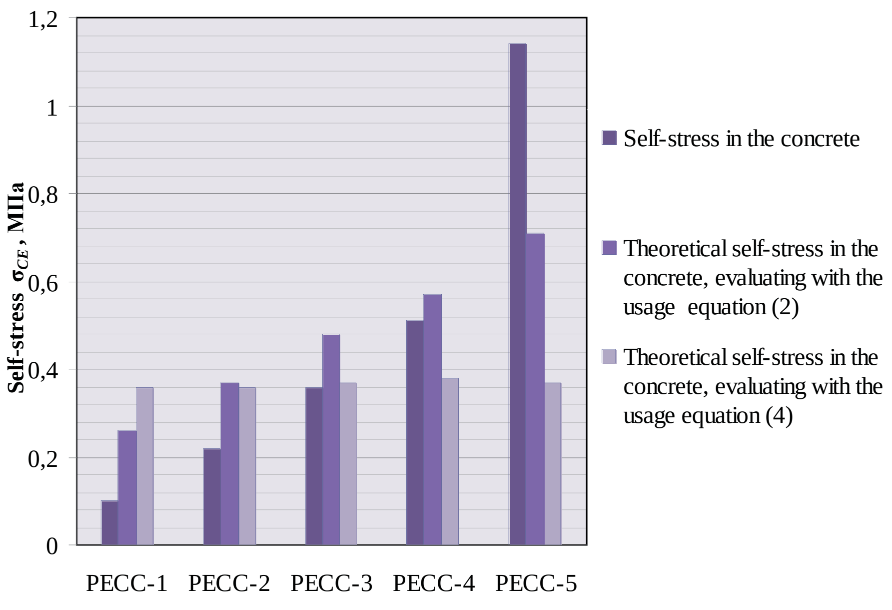


b)

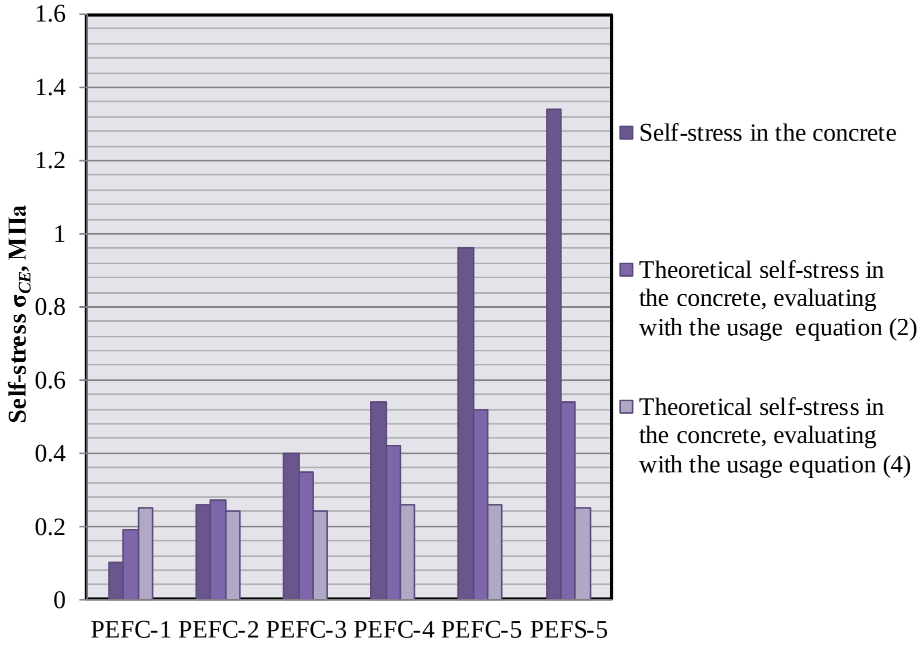
Fig. 4 Self-stress development in uniaxially restrained expansive concrete
(a) – for specimens-prisms of series I; b) – for specimens-prisms of series II)

It means that in connection with GFRP reinforcement restraint low axial stiffness, concrete specimens-prisms expanded almost free. The main influence of the restrictive bond was observed at the early age concrete hardening and expansion (under 1 day). At the moment of the self-stressing concrete expansion stabilization the value of the concrete restrained strain for all specimens-prisms was equal to $\varepsilon_{CE,f} = 140\times10^{-5}$; at the same time the value of the restrained strain for specimens-prisms, hardened in the conditions of the standard restraint (power conductor with stiffness, equivalent to the steel reinforcement restraint with stiffness equal to $r_l = 1\%$) was up to $\varepsilon_{CE,d} = 43\times10^{-5}$ and to $\varepsilon_{CE,d} = 31,5\times10^{-5}$ for specimens-prisms of series I and II respectively. It must be noted, that for admitted restraint stiffness characteristics (GFRP reinforcement) were reached rather low self-stress values (under 0,4 MPa) in the concrete for both experimental specimens-prisms of series I and II (for specimens-prisms with r_f under 0,5 %); at the same time in specimens-prisms with $r_f = 1,563\%$ were reached rather high self-stress values – 1,14 MPa and 0,96 MPa for experimental specimens-prisms of series I (PTCC-5) and series II (PTCF-5) respectively.

Comparison self-stress values at the moment of self-stressing concrete expansion stabilization obtained in the result of the computation are represented in the table 2 and on the diagrams on the figure 5. Difference between self-stress values comes up to 3,6 times for specimens-prisms of series I and to 2,5 times for specimens-prisms of series II. It would be related with the next important effects: all equations, concerning estimation of the self-stress value are empirical and originally were received for steel reinforcement, coming out as free expansion strains restraint. During receiving this equations, low reinforcement ratio area (under 0,3%) was not considered. In this restraint area in accordance with (Mihajlov, Litver 1974) equation for evaluating self-stress value was received due to the test data from the area of high reinforcement ratio approximation and with taking into account fact, that function must pass through the origin of the axes $\frac{\sigma_{CE}}{f_{CE,d}} - r_e$.



a)



b)

Fig. 5 Comparison self-stress values obtained in the result of computations at the moment of self--stressing concrete expansion stabilization
(a) – for specimens-prisms of series I; b) – for specimens-prisms of series II)

In connection with this fact, conversion of the self-stress values for GFRP reinforcement with usage equivalent steel reinforcement ratio (it defines from the axial stiffness equality) is not quite reasonably in the low reinforcement ratio area. Approaches, based on the grounds from (Mihajlov, Litver 1974) in mind of restrained strains development, plastic strains progress in the result of creep of concrete at the young age and stress relaxation should be applied for adequate analytical model for evaluation of the self-stress values obtaining. Self-stressing process definition based on consideration values of the external restrictive bond fixed strains only don't allow take into account internal stresses, appeared in the concrete structure on the self-stressing phase. The following specimens-prisms loading tests results, presented in the table 3, sustain described above fact.

Tab. 3 Specimens-prisms loading tests results

Series of specimens	Designation of specimens-prisms	Restraint conditions $\rho_{f(s)}$, %	Cracking features		Axial tensile concrete strength σ_{ct}, MPa	$\Delta\sigma_{ct}$, MPa
			Force P, kN	Flexural tensile concrete strength s_{flex}, MPa		
	PEC-1	Free prism	8,98	4,04	2,69	
Series I	PECC-1	0,126 %	11,28	5,08	3,39	0,7
	PECC-2	0,283%	11,16	5,02	3,35	0,66
	PECC 3	0,505%	11,92	5,36	3,57	0,88
	PECC 4	0,791%	13,56	6,10	4,07	1,38
	PECC-5	1,563%	11,86	5,34	3,56	0,87*
	PECS-6	0,505%	7,5	3,38	2,25	–*
	PEF-1	Free prism	14,86	6,69	4,46	
Series II	PEFC-1	0,126 %	18,38	8,27	5,51	1,05
	PEFC-2	0,283%	14,04	6,32	4,21	–*
	PEFC-3	0,505%	23,24	10,46	6,97	2,51
	PEFC-4	0,791%	19,33	8,70	5,80	1,34*
	PEFC-5	1,563%	28,39	12,77	8,51	4.05
	PEFS-6	0,505%	21,05	9,47	6,31	1,85*

*Note. Within static tests in the experimental specimen-prism slipping of bar has happened.

As follows from the specimens-prisms loading tests results (see table 3), reinforced experimental specimens-prisms had higher value of the flexural tensile strength in comparison with free specimens, hardened in the same with it conditions.

In addition it must be noted that hardening in restrained conditions (involving compression stresses effect) provided creation of the cleared conditions for structure formation, that in turn leads to average concrete axial tensile strength rising.

More important results were reached due to parthnering GFRP reinforcement and steel fibre reinforcement (specimens-prisms of series II).

Diagrams, shown restrained strains development of specimens-prisms of series II (with additional steel fibre reinforcement), presented on the figure 2 b) and values of the fixed self-stress at the moment of self-stressing concrete expansion stabilization are presented in the table 2.

It must be noted that experimental specimens-prisms of series II (made of steel fibre reinforced expansive concrete) strains were approximately the same with experimental specimens-prisms of series I (made of expansive concrete) strains (see table 2 and figure 2). Diagrams of self-stress development of specimens-prisms of series II, constructed in terms of fixed strains, are presented on the figure 4.

Conclusions

Due to completed stage researches following conclusions can be done:

1. Owing to restrictive bond made of FRP reinforcement bars low axial stiffness (it's modulus of elasticity almost commensurable with concrete modulus of elasticity) possibility of jacking with usage physicochemical method appears. Physicochemical method of the prestressing (by means of expansive concrete utilization) applies to the as it is called «soft» conditions of prestressing. Due to this method of prestressing the main shortcoming of the mechanical method of prestressing GFRP bars consisted in the limitation of the permissible stresses in the FRP tendons at jacking and transfer at the level of (30 – 50%) from it's ultimate tensile strength, otherwise creep strains (this strains inevitably led to tendons rupture) will indefinitely grow up and accumulate as a result of the FRP tendons low modulus of elasticity can be removed. In addition due to physicochemical method of prestressing it succeeded in minimizing of presrtess losses due to shrinkage of concrete.
2. In the course of self-stressing process research on the specimens-prisms of series I and series II was established that specimens with GFRP bars reinforcement ratio from 0 to 0,8% during self-stressing concrete expansion got strains almost the same with free specimens strains. As a result for such specimens comparable and rather low values of the self-stress were got. However though this fact specimens with GFRP reinforcement ratio from 0 to 0,8% tensile strength value (and as a result crack resistance) was higher in 1,4 times and in 1,6 times for specimens-prisms of series I and series II properly than free specimens from this series tensile strength value. At the same time we can say that specimens-prisms of series I and series II with GFRP reinforcement ratio in 1,563 % had rather massive restrictive bond, judging by the strains values it had at the moment of self-stressing concrete expansion stabilization. In turn it led to the obtaining by this experimental specimens rather sizeable values of it's practically self-stressing stresses: 1,14 MPa and 0,96 MPa for specimen from series I and series II properly. Mentioned specimens uniaxial tensile strength appeared in 1,9 times higher than proper free specimens uniaxial tensile strength.
3. As a result of additional restriction in the capacity of three-dimensional steel fibre reinforcement reduction of the longitudinal strains at the restrictive reinforcement depth were observed. Therewith completed loading tests showed that specimens-prisms of series I and series II made of steel fibre reinforced expansive concrete had the highest value of the axial tensile strength (specimens-prisms of series II cracking resistance moment was higher than specimens-prisms of series I made of expansive concrete cracking resistance moment in 2 times; was higher than specimen-prism with equivalent in terms of axial stiffness equality steel reinforcement ratio cracking resistance moment in 1,3 times; was higher than free specimen-prism made of steel fibre reinforced expansive concrete cracking resistance moment in 2 times).

4. Essential distinctions between theoretical self-stress value and obtained during experimental studies at the moment of self-stressing concrete expansion stabilization self-stress value must be noted (distinction between this values up to 3,6 times for specimens-prisms of series I and to 2,5 times for specimens-prisms of series II). It would be related with the next important effects: all equations concerning equation of self-stress value are impirical. In addition this equations were obtained for steel reinforcement as a restraint. Upon receipt this calculation equations was not considered the area of the low reinforcement ratio (under 0,3 %). Adequate analytical model can be developed taking into account the following values (some of this values are time-varying): self-stressing grade of concrete, restrictive bond stiffness, reinforcement ratio and surrounding it concrete stiffnesses, plastic strains in the result of creep of concrete and stress relaxation must also be taken into account.

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