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ASSESSMENT OF THE PREDICTION METHODS OF RESTRAINED STRAINS AND SELF-STRESSES OF THE MEMBERS MADE OF EXPANSIVE CONCRETES

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Abstract

In various time periods, interest to RC structures made of expansive concretes was very different: from admiration after its successful utilization in the real practice of civil engineering works (for instance, jointless self-stressed and post-tensioned slab-on-ground with dimensions of 144 x 72 m^2) to great criticism and sarcasm when shrinkage cracking appeared after full self-stressing loosing or even self-damaging taking place in case, when «unbalanced» expansion and strength development were observed.

Nevertheless, the interest to the elements made of self-stressing concrete drastically increased in the last decades. By the way, it is necessary to mention that in its majority, there are elements with so-called composite reinforcement (bars). In such a bars, the reinforcing fibers are made of glass, aramid, carbon and etc. However, prediction methods of the restrained strains and self-stresses development are not always characterized by the accuracy in connection with the self-prestressing is a multifactorial process and, thus, these methods need for clarification and modification in future.

In this paper authors presented short historical review on expansive binders production for self-stressing and shrinkage-compensating concretes, discussed advantages and disadvantages of the known models for restrained strains and compressive stresses assessment as a result of self-prestrssing, as well as some own results of the behavior of self-stressed concrete structural elements, reinforced with both steel and glass fibers reinforced polymer (GFRP) bars on the both expansion (self-prestressing) stage and under the static loading.

Keywords: expansive concrete, self-stressed elements, restrained strains, conservation law of chemical energy, deformation approach.

ОЦЕНКА СУЩЕСТВУЮЩИХ МЕТОДИК РАСЧЁТА СОБСТВЕННЫХ ДЕФОРМАЦИЙ И САМОНАПРЯЖЕНИЙ ЭЛЕМЕНТОВ ИЗ НАПРЯГАЮЩИХ БЕТОНОВ

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Реферат

В различное время отношение к железобетонным конструкциям, выполненным из расширяющихся бетонов было различным: от признания перспективности материала после удачной реализации ряда объектов в реальной строительной практике (например, бесшовные самонапряжённые полы на упругом основании, выполненные с постнапряжением, размерами 144 × 72 м^2) до существенной критики и сарказма в отношении данного материала, когда в конструкциях наблюдали образование усадочных трещин после полной потери самонапряжения либо же констатировали саморазрушение элемента в случае несбалансированного развития динамики расширения бетона по отношению к динамике набора им прочности.

Тем не менее в последнее время возрос интерес к элементам из напрягающего бетона. В большей степени это элементы, выполненные с так называемым композитным армированием, где армирующим элементом является не арматурная сталь, волокна выполненные из стекла, арамида, карбона и т. д. Однако расчётные методики связанных деформаций и самонапряжений в таких элементах не всегда отличаются точностью в связи смногофакторностью процесса и требуют уточнения и доработки в дальнейшем.

В данной статье авторы представили короткий исторический обзор производства расширяющихся вяжущих для самонапрягающих бетонов и бетонов с компенсированной усадкой, обсудили преимущества и недостатки известных моделей для расчёта связанных деформаций и сжимающих напряжений в бетоне как результат самонапряжения, а также представили ряд собственных результатов исследования самонапряжённых бетонных элементов, армированными стальными и композитными стержнями как на стадии самонапряжения (расширения), так и на стадии статических испытаний.

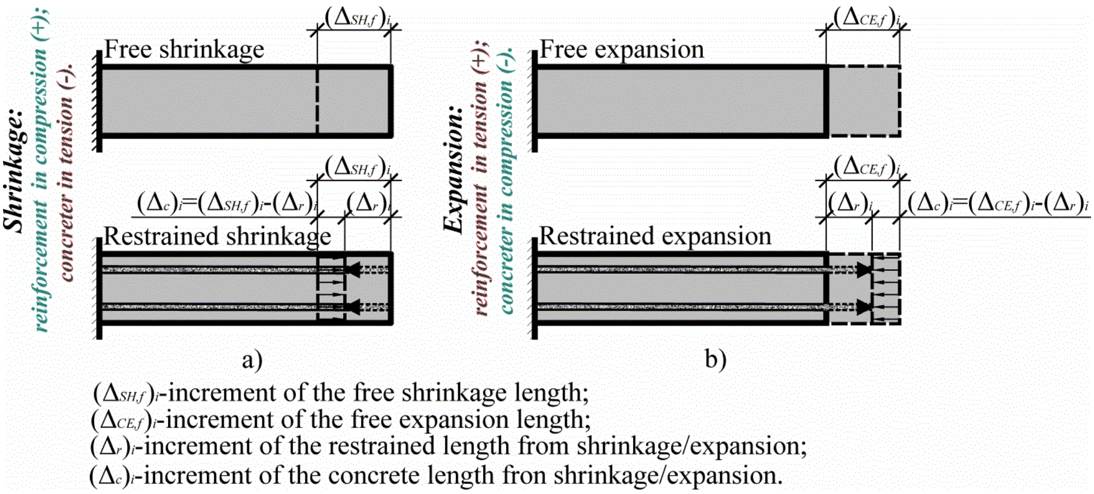
Ключевые слова: расширяющийся бетон, самонапряжённые элементы, связанные деформации, закон сохранения химической энергии расширения, деформационные подходы.

Introduction. Historical review on expansive concretes and binders development

In the recent years a new wave of interest in the research and practical use of self-stressing concrete has been observed, now, specifically~~,~~ in combination with various types of so-called non-traditional reinforcement. One of the widely used types of these reinforcements is FRP bars and long fibres (textile) (fibres made of glass, aramid carbon and etc.) [1]. The main problems of FRP reinforcement implementation as a structural reinforcement are related to the development of excessive deflections as well as crack opening under service loads. One of the most effective methods for concrete with FRP reinforcement performance enhancing is chemical prestressing with self-stressing (expansive) concretes.

Let's consider phenomena of concrete prestressing (chemical or mechanical) in general. Prestressing of concrete is a technique in which reinforcement is actively tensioned initially to later transfer, after reinforcement releasing, the stresses into hardened concrete (prestress). Such appeared compressive stresses in concrete counteracts the tensile stresses that occur at service stage and, hence, offset the risk of tensile cracking.

In the case of mechanical pre-tensioning of tendons, the following disadvantages exist: high needs for time and labor consumption, necessity of qualified personnel, usage of very expensive prestressing beds with loading jacks and anchorage systems. As well, traditional prestressing is feasible for one-directional prestress only. At the same time, the self-prestressing technology is becoming a viable alternative to traditional prestressing. Self-prestressing (see Figure 1) of concrete members is based solely on usage the restrained expansion capacity of self-stressing concrete during hardening to cause pretensioning of the reinforcement. Usage of self-stressing concrete for gradual prestressing allows to achieve the required levels of prestressing, as well as to save the major part of obtained prestress in time.



a) – plain concrete shrinkage; b) – self-stressing concrete expansion

Figure 1 – Schematic presentation of the shrinkage and expansion processes

Intensive development of the Portland cement concrete technology in the last decades allowed to obtain high-performance concrete (HPC) or even ultra-high performance concrete (UHPC) with a compressive strength above 120 MPa. Nevertheless, an inadequate ratio of such a concretes compressive to tensile strength allows to say, that concretes of the new generation still remain an artificial composite stone materials with a good performance under compression only in combination with inherent to concrete early-age and long-term effects (such as autogenous, plastic and drying shrinkage deformations, creep deformations, sensitivity to temperature differences). In its turn this situation unavoidable leads to decreasing of the serviceability parameters of concrete structures decreasing. For instance, restrained shrinkage and temperature deformations lead to the additional tensile stresses appearance in the concrete structure causing cracks (cracks of different sizes can be found almost in every reinforced concrete structures). Obviously, such cracking of concrete reduces structural durability in general. Based on the sustainable development strategy, presented at the *fib* Symposium 2020 in China defines concrete of the new generation as a high-durability concrete (HDC). To permit a more efficient utilization of structural concrete, the search for means of overcoming these weaknesses had led to mechanical prestressing of steel tendons. By keeping the concrete in compression, cracking is prevented. Considerable advantage can be derived from concrete expansion under the various types of restraint and in its turn to induce restrained strains and, respective to them, compressive pre-stressing of sufficient magnitude to compensate shrinkage effects (so-called shrinkage-compensating concrete) or to induce compressive stresses of a high enough magnitude, resulting in significant compression in the concrete after long-term and short-term processes have been realized in time and in such a way we obtain self-prestressing of concrete. The above-mentioned problem led to the idea of the physico-chemical (or sometimes called chemical) method of concrete structures volume prestressing. In 1953 **I. Giyon** wrote in his monograph: «*... In case we will reach a significant restrained expansion of the concrete, that could provide an adequate reinforcement pre-tensioning, without doubts, we will get a principally new method of the beams pre-stressing*».

The history of the development of expansive cement development and application (self-stressing and shrinkage-compensating concretes) counts about 90 years and it can be said, that it has originated from an investigation of ettringite in cement. Ettringite crystal (3CaO·Al2O3·3CaSO4·32H2O) represents by itself a phase, formed in different concrete phases during hydration of expansive cements, which are the source of the ability for expansion. It is comparable to the natural mineral of the same name. This high sulfate calcium sulfoaluminate is also formed by sulfate attack on mortar and concrete (so-called delayed ettringite) and was defined as «*cement bacillas*» in elder literature. **Candlot** reported in 1890s that this product resulted from reaction of ticalcium aluminate (C3A) with calcium sulfate (CaSO4). **Michaelis** in 1892 [2] suggested that ettringite was responsible for destructive expansion of Portland cement concretes in the presence of sulfates in ambient conditions.

One of the earliest investigators, recognized the potential of ettringite in the elimination of shrinkage and possibly of prestress inducing was **Lossier** [3]. His works continued more than 20 years, starting in the middle 1930s, and the cement he had developed was consisted of Portland cement, an expansive component (grinding gypsum, bauxites, chalk to slurry burning as the admixture to a clinker) and blast furnace slag.

Russian works, published by professor **Mikhajlov** in the field of expansive cements followed two different courses to obtain an expansive cements for the aims of repairing and/or waterproofing and self-prestressing. Expansive cement type-M either is a mixture of Portland cement, calcium aluminate cement and calcium sulfate or an interground product made with Portland cement clinker, calcium aluminate clinker and calcium sulfate. In monograph [4], we can find the first formulation of the solid-state or solid-phase expansion mechanism theory of the concrete matrix as a fundamental condition of concrete self-prestressing under restraint and the related to it requirements to the expansive cement compositions (for instance, the ratio Al_2O_3/SO_3 in both expansive additive and expansive concrete itself).

Studies performed by **Klein** [5] and his associates at the University of California are based on the formation of a stable anhydrous calcium sulfoaluminate compound by heat treating a mixture of bauxite, chalk, gypsum at about 2400F (≈1315 °C). While the ingredients were quite similar to those were used by the Lossier in his cements, the material selection and clinkering conditions, probably, contributed to the formation of an anhydrous calcium sulfoaluminate, calcium sulfate and lime. As a result produced cement could be handled much in the same manner as a regular cement and adjusted to offset shrinkage and produce large net of expansion.

In recent years some new types of the expansive cement and expansive additives to OPC are proposed on the market of building materials, but all these materials based on the reacton of the ettringite formation (CSA-type of additives). It should be mentioned that besides the use of expansive potential generated by ettringite formation, another type of expansive admixtures takes use of hydroxide formation. As well, periclase has been employed in dam construction as the expanding agent in China.

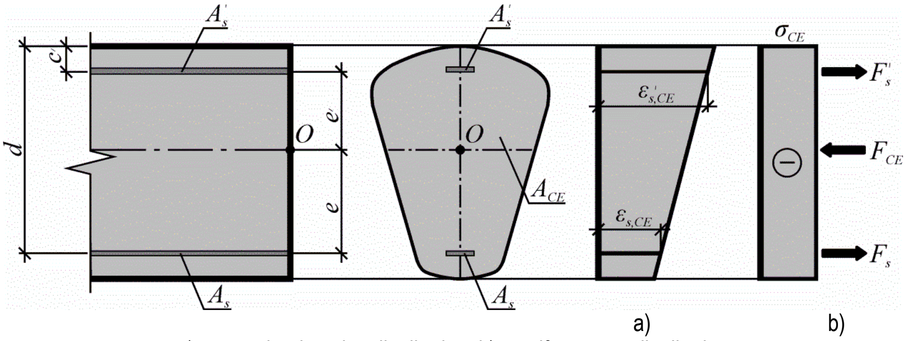
Models for assessment of the early-age stress-strain state in self-stressed concrete structures

In this paper, the attention is paid on the most controversial series of articles [6, 7] dedicated to textile-reinforced self-stressed concrete (TRSSC), mainly respectively the «*Theory of self-stressing distribution models*» and experimental results used for verification of these models. In recent decades, it was developed various types of reinforcing elements to replace conventional reinforcing steel (short fibres, various types of FRP bars). Firstly, let's pay attention on the replacement steel reinforcement by use of continuous fibres or grids that were made from continuous fibres began in the 1980s. Among experts, *this new, innovative composite building material* is known today as *textile-reinforced concrete* (TRC). In investigations *«self-stressing concrete (SSC) matrix was combined with textile to form a new composite material, namely, textile-reinforced self-stressing concrete (TRSSC). In this material, textile functions as expansion confinements to SSC to attain self-stress»*.

There is no doubts that to extend application of self-stressing concrete with any type of reinforcement in the practice of civil engineering the need for adequate models for assessment of stresses (strains) appeared in self-prestressed concrete structures is evident. In this paper the authors would like to show some most conservative and as well more complicated and prospective approaches used for stress-strain statement of self-prestressed structures prediction.

Models based on the conservation law of chemical energy

Finding out the origins of the models based on the conservation law of chemical energy brought us to the early 1970s. The fundamental for these models assumption was formulated by the V. Mikhailov and S. Litver in [4]: «*… it was established within numerous investigations that regardless of the reinforcement areas in the both zones* (authors' note: in tensile (A_{st}) and in compressive (A_{sc}) zones under the loading) *of the section A_{sc} and A_{st}, i.e. when $A_{sc} \neq A_{st}$, self-stressing concrete of the structure within expansion process accumulate* ***uniformly distributed through the section depth compressive stresses (self-stresses)***». It can be explained by the fact that tensile forces in reinforcement in the both of these zones perform the equal work on the obtained within expansion strains, and, as a result, strains and forces in the reinforcement of these zones are different, the cross-section of the structure loses its linearity and the concrete is precompressed uniformly». On the above-mentioned fundametal statement was based a number of standards [8] in accordance with the which it was reported, that: «*self-stresses in the concrete are accepted to be uniformly distributed through the cross-section depth:* ***resultant of the compressive stresses (self-stresses) is arranged in the cross-section gravity center***» (see Figure 2).



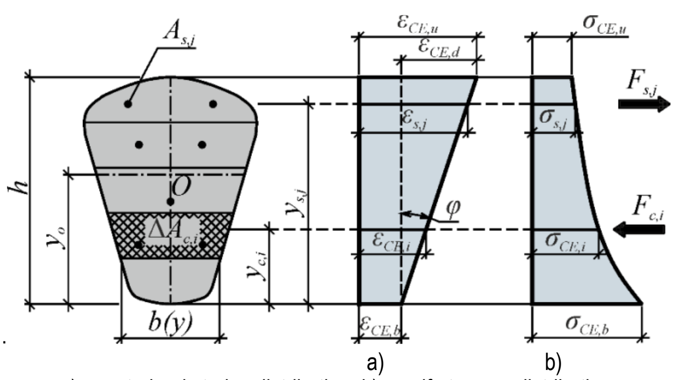
a) – restrained strains distribution; b) – self-stresses distribution

Figure 2 – Restrained strains and self-stresses distribution in the cross-section of the self-stressing concrete member in accordance with model [4, 8]

Evidently, that an accepted hypothesis essentially simplified assessment of the self-stresses value as far as for it became unnecessary to know the value of the concrete restrained strains. *It should be noted, that this circumstance led to the situation that for a long period of time resultant value of the free expansion strains to the expansion stabilization point was ignored as a necessary parameter for calculations.* As the main calculation parameters was accepted to consider $f_{CE,d}$ – reference self-stress of the concrete established in the standard restraint conditions to the concrete expansion stabilization, i.e. it is compressive stress in the self-stressing concrete prism under uniaxial symmetrical restraint with the stiffness equal to 1 % of the steel cross-sectional reinforcement ratio ($E_s = 200$ GPa).

Inspite of the evident simplicity of the models [4, 8], based on the *uniform self-stresses distribution through the section depth hypothesis*, it have to be mentioned that the hypothesis itself is not applicable at least for the case of the one-layer cross-sectional reinforcement arrangement: in such a case it is difficult to state that resultant force in the concrete in compression is in the concrete section gravity center (applied points mismatching of the resultant compressive force in concrete and resultant tensile force in the reinforcement causes appearance of the unbalanced moment in the cross-section to the concrete expansion stabilization). At the same time, an obtained value of the self-stress in concrete is a result of the elastic compressive strains amount accumulated in concrete to the expansion stabilization. Thus, in the self-stressed members in the same way like in the traditionally mechanically prestressed members, after the compression force transfer, cross-sectional equilibrium conditions have to be satisfied.

To the modified model based as well on the conservation law of chemical energy is referred model developed by Y. Tsuji [9] (see Figure 3).



a) – restrained strains distribution; b) – self-stresses distribution

Figure 3 – Restrained strains and self-stresses distribution in the cross-section of the self-stressing concrete member with multi-layer reinforcement arrangement in accordance with model [9]

This model is more complex in comparison with the above-prescribed model [4, 8], as far as apart of the fundamental hyphothesis of conservation law of chemical energy it takes into account cross-section equilibrium conditions. The model is based on the following approaches: a) to the expansion stabilization restrained concrete strains are distributed linear throughout the cross-section depth (plain cross-section hypothesis is valid); b) the amount of work U_{CE}, that expansive concrete performs against restraint per unit volume, is a constant value regardless of the degree of restraint; c) self-stressing concrete and restraint compatibility of strains are respected throughout the expansion stage.

The same like in [9] $f_{CE,d}$ value is an «energetic» parameter that represents self-stress in concrete reached in standard restraint conditions, in [9] self-stressing concrete work amount U_{CE} (that is established in the same standard restraint conditions: centrally uniaxially arranged steel ($E_s = 200$ GPa) restraint in the concrete cross-section with reinforcement ratio $\rho_{l,s} = 1\%$) as well is an «energetic» parameter that is established in accordance with the following equation:

$$U_{CE} = \frac{\sigma_{CE} \cdot \varepsilon_{CE}}{2}, \tag{1}$$

where σ_{CE} and ε_{CE} – self-stress and respective to it concrete restrained strains respectively established in standard restraint conditions ($E_s = 200$ GPa, $\rho_{l,s} = 1\%$) to the expansion stabilization.

In the article [10] disadvantages of the model [4, 8, 9] were found out on the basis of the numerical studies and described in details.

Summarizing all of the above-stated models based on the conservation law of chemical energy, it is possible to make following conclusions: any models based on predescribed hypothesis have a pretty tight diapason when they are applicable, in the other cases calculation results contradict the experimental values or, even, don't demonstrate a physical sense in general. Moreover, it is necessary to underline, that these models are valid only when as a free expansion strains restriction steel reinforcement is acting. It can be explained by the fact that reference energetic parameters in these models (such as $f_{CE,d}$ in [4, 8] and U_{CE} in [9]) are established in conditions when steel reinforcement is acting as a restraint. The models [4, 8, 9] are not applicable when, as a restraint, reinforcement with the different from steel elasticity modulus is utilized. It is connected with the fact, that these models don't take into account reinforcement properties itself such as stresses relaxation, development of which one is different for materials with not the same elasticity modulus.

Models based on the deformation approach

In the recent years a numerous of international publications [11] are devoted to the description of the self-stressing concrete expansion process with the deformation models. These models are universal as far as with it application it is possible to describe a physical side of the expansion process and to take into account a wide range of factors influenced on the kinetics of the self-stressing concrete expansion (the factors such as continuously changing at the self-stressing stage ratio between axial stiffness of the early-age concrete and of the restraint, as well as creep of early-age concrete), i.e. it allows to take into account continuous redistribution of the internal forces in the member cross-section as a result of non-elastic properties of the self-stressing concrete at early-age. Moreover, deformation models are able not only to describe expansion processes, but they can be easily extended on the following after expansion long-term processes (shrinkage, long-term creep) and its consideration allows to extract a number of self-stressing parameters (self-stresses, concrete restrained strains, accumulated in concrete elastic strains) to the any time-point of the expansion process (not only to the expansion stabilization).

In general case deformation models are based on the consideration of the following basic equation:

$$d\varepsilon_{CE,f} = d\varepsilon_r + d\varepsilon_{c,el} + d\varepsilon_{c,pl}, \tag{2}$$

where $d\varepsilon_{CE,f}$ – increment of the self-stressing concrete free expansion strains;

$d\varepsilon_r$ – increment of the self-stressing concrete restrained strains on the depth of the restraint;

$d\varepsilon_{c,el}$ and $d\varepsilon_{c,pl}$ – increment of the self-stressing concrete elastic and plastic strains respectively.

All of the known modifications of the deformation models are consisted in the consideration of the basic equation (2) on the elementary time-steps Δt_i, where the increment of the self-stressing concrete free expansion strains takes place on the background of the non-stopped development of the materials physico-mechanical properties at early-age, including creep. From the equation (2) it is possible to find out $d\varepsilon_r$ on the elementary time-step Δt_i. Detailed description of the deformation models are presented in [11].

It should be mentioned, that in contrast to the models based on the conservation law of chemical energy, where reference concrete characteristics ($f_{CE,d}$, U_{CE}) are established in the uniaxial standard restraint conditions, in deformation models self-stressing concrete free expansion (shrinkage) strains temporal progress (in such a models it is essential not only concrete free expansion (shrinkage) strains value to the expansion stabilization, but kinetics of its development) are considered as a basic concrete characteristic. To the input data for deformation models an appropriate early-age concrete creep function and function of the early-age concrete elasticity modulus development are referred as well.

Calculation results of the concrete restrained strained in accordance with the general concept of the deformation models demonstrate a good fit with experimental data [11]. It should be stated that that verification of the deformation models was performed, mostly, for concretes with free expansion strains to stabilization not higher than 0,05% and expansion stabilization was observed to the end of the first week of concrete age. Thus, such an expansive concrete can be referred to the shrinkage-compensating. Nevertheless, verification of the basic deformation model for the case of structural members made of self-stressing concrete (with high expansion energy capacity: concrete free expansion strains to the stabilization are higher than 0,2%, reference self-stress value to the expansion stabilization is about 3 MPa and expansion stabilization is observed on the 14 days or more of concrete age) doesn't demonstrate so good fit of the calculated and predicted by the model restrained strains value as for members made of shrinkage-compensating concretes. Increasing of the experimental and predicted values mismatching (predicted values become sufficiently higher those, established experimentally) increases with the concrete age increasing and with the increasing of the cross-section reinforcement ratio. In [12] one essential circumstance was noted: with the cross-section reinforcement ratio increasing, the higher part of the free expansion energy of concrete is spent on the elastic and early-age creep deforming of the composite itself. Besides, as it was suggested by the authors of this article [12], utilizing of the high expansion energy capacity concrete allows to deform restraint and the forces increment in the restraint reinforcement is acting as an additional restriction for the further reinforcement stretching and as well for further restrained concrete strains development. Thus, modification of the basic deformation model for the assessment of the accumulated restrained strains in the high expansion energy capacity self-stressing concrete members is necessary. Suggested by the authors modified strains development model (MSDM) was developed and verified for the uniaxially reinforced self-stressing concrete members with different reinforcement types (steel, FRP bars) and it is described in details in [12].

As well, it is necessary to point attention, that in the last years artificial neural network (ANN) and fuzzy inference system model (FIS) for predicting free expansion have been developed [13].

Discussion

All of the listed above models proposed for assessment early age restrained strains and/or self-stresses in the self-prestressed concrete elements are based on the following fundamental assumptions: (1) self-stressing is the specific type of the pre-stressing, in which the tensile force in the tendons and equal resultant compression force in concrete are induced gradually in time as a result of the work that self-stressing concrete performs against restraint; (2) expansive strains are linearly distributed in the direction of cross-sectional height (plain section hypothesis is valid). In the first approximation, the cases considered are those when misalignments are not produced at the respective boundaries between expansive concrete and reinforcing bars. Based on the assumption that self-stresses in concrete are considered as the product of the elastic strains and modulus of elasticity $E_{cm}(t)$, at the state of stabilization of the expansion stresses are distributed linearly too.

On the other hand, the distribution of the stresses in the cross-section can be calculated as well as for the mechanically pre-stressed structure. Here, the resultant force in restraint is considered as a pre-stressing force.

Anyway, in some articles [6, 7] we can read following statement: *"Self-stress is distributed along the fiber bundle in the textile and exhibits similar effect to that of mechanical pre-stress."* This statement complies with assumptions adopted in the models listed above and considered TRSSC element as the pre-stressed element, but the following model assumptions about stresses distributions look strange and speculative and require comments.

According to [6, 7] *analysis of cracking load is based on the following three assumptions:*

a. The beam is in elastic stage and conforms to the assumption of small deformation before cracking.

b. The relative displacement between matrix and the woven fabric is ignored.

c. ***The self-stress value was distributed identically within range of 5mm above and below the textile*** *(see Figure 4).*

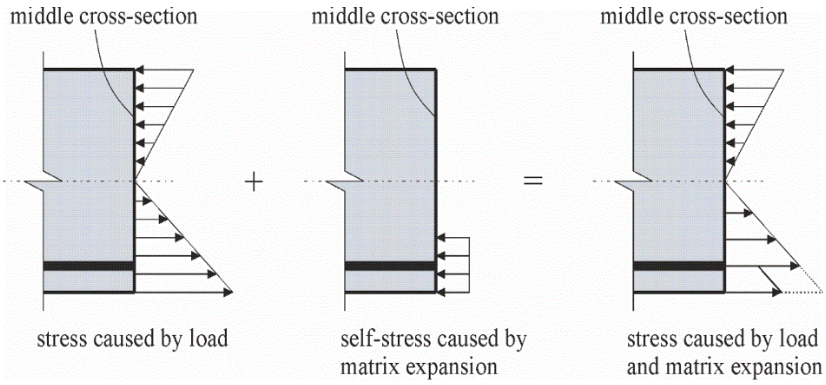


Figure 4 – Stress distribution in TRSSC beam section according to [6, 7]

Analysis of presented assumptions (mainly assumption c) and Figure 4 initiates following questions:

(1) Why self-stresses are distributed uniformly at the local area limited within the range of 5 mm above and below the textile? (Why is it not 5,5 mm; 7 mm; 5,6 mm...? What is the influence of bundle area? What is the scientific background of this range?);

(2) Does for a such new composites plain section law at the stage of expansion is not valid and does it mean, that authors observed in test deplanation of the section?

(3) If self-stresses concentrate within a limited range above and below textile reinforcement, could we say, that the rest part of section is free from self-stressing?

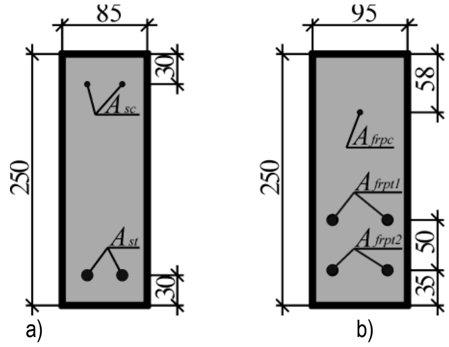
Experimental studies of self-stressed element reinforced with FRP

Let's consider the results of the research work [14], in which we used self-stressing concrete for beam elements reinforced with FRP bars. In [14], the possibility of applying self-stressing concrete to increase FRP reinforcement's effectiveness, mainly increasing crack resistance and flexural stiffness (satisfying the serviceability limit state requirements) was studied. We adopted the following work hypothesis: expanding of self-stressing concrete in restraint conditions developed by FRP bars induces tensile force in restraint and compression forces (self-stresses) in concrete. The relatively low axial stiffness of FRP bars allows sufficient restrained strains that cannot fully be compensated for shrinkage development.

Experimental studies [14] were carried out on two series of self-stressed concrete beams with different type of reinforcing bars. Experimental beams cross-section geometry with reinforcement areas and arrangement are shown in Figure 5.

Expansive cement composition was consisted of 3 components in the following proportions (by weight): Ordinary Portland cement (CEMI-42,5N) – 71 %; metakaolin – 14 %; gypsum ($CaSO_4 \cdot 2H_2O$) – 15 %. The main mechanical characteristics of the hardened expansive cement are presented in Table 1.

Self-stressed beams of the both series were made of self-stressing concrete with characteristics presented in Table 2.



a) – self-stressed beams of the series I (I-BECS-(1…4): A_{sc} = 25,1 mm^2 (2Ø4); A_{st} = 157,0 mm^2 (2Ø10)); b) – self-stressed beams of the series II (II-BECF-(1,2): A_{frpc} = 13,7 mm^2 (1Ø4); A_{frpt1} = 143,5 mm^2 (2Ø10); A_{frpt2} = 143,5 mm^2 (2Ø10); II-BECF-(3): A_{frpc} = 13,7 mm^2 (1Ø4); A_{frpt1} = 143,5 mm^2 (2Ø10); A_{frpt2} = 330,5 mm^2 (2Ø14))

Figure 5 – Experimental beams cross-section geometry with reinforcement areas and arrangement [14]

Table 1 – Expansive cement characteristics

Expansion		Strength	
free expansion strain ε_f, %	reference self-stress $f_{CE,d}$, MPa	flexural f_{flex}, MPa	compressive f_{cm}, MPa
1,21	5,9	5,5	50,8

Notes: *1. Expansion and strength characteristics were established at the 28 days age of the mortar bars hardened in the unrestrained conditions.*
2. Reference self-stress, $f_{CE,d}$, was established in standard restraint conditions:
ρ_l = 1 % and E_s = 200 GPa.

Table 2 – Average values of the self-stressing concrete characteristics

Series	Expansion characteristics at the concrete expansion stabilization		Mechanical characteristics	
	free expansion strain $\varepsilon_{CE,f}$, %	reference self-stress $f_{CE,d}$, MPa	compressive strength $f_{cm,28}$, MPa	modulus of elasticity $E_{cm,28}$, GPa
I	0,47	2,4	33,2	25,3
II	0,55	2,8	37,8	25,7

Notes: *1. Free expansion strain, $\varepsilon_{CE,f}$, was established on the unrestrained specimens;*
2. Reference self-stress, $f_{CE,d}$, was established in the standard restraint conditions:
ρ_l = 1 % and E_s = 200 GPa;
3. Modulus of elasticity was established on the cylindrical specimens (Ø = 150 mm, h = 300 mm).

Steel and FRP reinforcing bars characteristics are listed in Table 3 and Table 4.

Table 3 – Average values of the mechanical characteristics of steel reinforcing bars (experimental values)

Nominal diameter, mm	Yield stress f_{ym}, MPa	Modulus of elasticity E_{sm}, GPa
4	573,2	200,0
10	625,7	

Table 4 – Average values of the mechanical characteristics of FRP reinforcing bars (experimental values)

Nominal diameter, mm	Type of fibers	Modulus of elasticity E_{frpm}, GPa	Tensile strength f_{frpm}, MPa	Ultimate tensile strain ε_{frpm}, %
5	Basalt	51,5	1262	2,45
10	Glass	45,2	1027	2,27
14	Glass			

Experimental values of the restrained strains and self-stresses in concrete on the depth of the cross-section gravity center immediately before static loading are listed in the Table 5.

Table 5 – Experimental values of restrained strains and self-stresses immediately before static loading

Unit code	Restrained strains, [%]			Self-stress σ_{CE}, [MPa]
	$\Sigma(\Delta\varepsilon_{CE,t})_i$	$\Sigma(\Delta\varepsilon_{CE,m})_i$	$\Sigma(\Delta\varepsilon_{CE,b})_i$	
I-BECS-(1)	0,342	–	0,128	2,69
I-BECS-(2)	0,372	–	0,144	2,95
I-BECS-(3)	0,443	–	0,144	3,00
I-BECS-(4)	0,499	–	0,154	3,46
II-BECF-(1)	0,481	0,330	0,269	1,78
II-BECF-(2)	0,556	0,365	0,276	1,92
II-BECF-(3)	0,429	0,267	0,197	2,10

As shown in Table 5, in all the tested beams, the initial value of self-stresses was got in the range from 1,8 to 3,5 MPa depending on the reinforcing bars' type, area, and arrangement. Reached pre-tensioning in reinforcing bars were at average 46 % from yield strain and 14 % from ultimate tensile strain for steel and FRP reinforcing bars respectively. It should be pointed that for the members pre-stressed with FRP reinforcing bars in accordance with [15], initial values of the pre-stress should be limited by the 24 % from the ultimate tensile strength.

Beams initial restrained expansion curvature values obtained on the basis of measured restrained strains and measures deflections varied in the diapason (1,16–1,82)·10^{-5} mm^{-1} and (3,7–4,1) mm respectively. These values of the initial restrained expansion curvature of the beams got at the self-stressing stage should be considered because two developed in time superposed basic processes: (1) on the one hand–self-stressing concrete expansion in asymmetrical restraint conditions and (2) on the other hand–concrete elastic compressive strains accumulating under monotonically increasing in time restraint reaction. It should be pointed that plane section hypothesis was valid for all tested beams. The so-called beam initial «elastic» curvature (that is determined from the accumulated concrete elastic compressive strains distribution) only have an influence on the self-stressed member behaviour under the applied static load in terms of traditional decompression. In contrast with traditional pre-stressed members, in the self-stressed members the values of the beam initial «elastic» curvature is not possible to establish based on the direct strains measurement, but it can be obtained under the proposed MSDM concept [12].

After the self-stressing concrete expansion stabilization was reached, self-stressed beams were tested with monotonically increasing load by means of two concentrated forces applied at the 1/3 and 2/3 points of the 1200 mm span. The main aim of the static loading consisted in the investigation of the influence of the achieved initial stress-strain state obtained to the self-stressing concrete expansion stabilization on the behavior of the tested beams under the load.

The moment-curvature and moment-deflection curves for specimens of series I and series II are shown in Figure 6.

Test results obtained within loading of the self-stressed beams are listed in Table 6 and Table 7.

Table 6 – Failure modes and experimental value of cracking and ultimate loads obtained within self-stressed beams testing

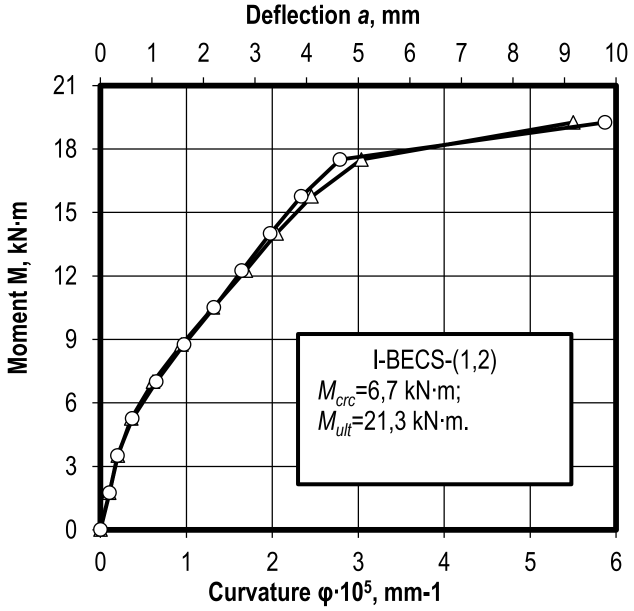
Unit code	Cracking load (force) P_{crc}, kN (M_{crc}, kN·m)	Ultimate load (force) P_{ult}, kN (M_{ult}, kN·m)	Failure mode
I-BECS-(1)	34 (6,8)	108 (21,6)	«B»
I-BECS-(2)	37,3 (6,5)	120 (21,0)	
I-BECS-(3)	39,5 (6,9)	120 (21,0)	
I-BECS-(4)	46,6 (8,2)	125,4 (22,0)	
II-BECF-(1)	40,5 (8,1)	150 (30,0)	«Sh»
II-BECF-(2)	43,5 (8,7)	130 (26,0)	
II-BECF-(3)	39,0 (7,8)	150 (30,0)	

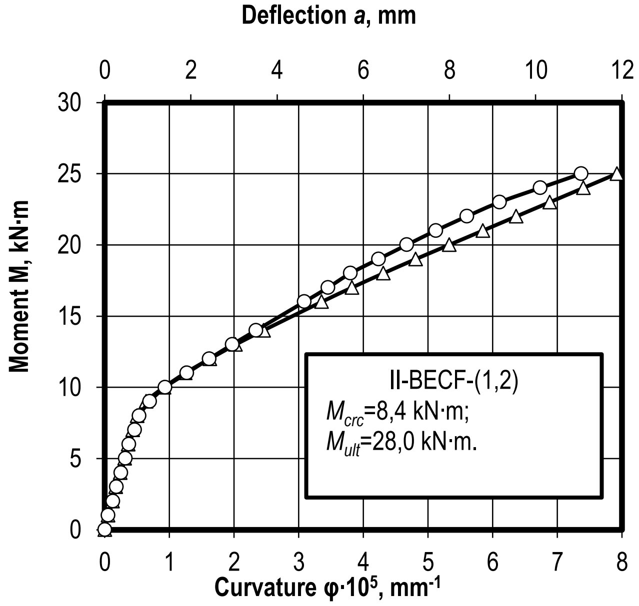
Note *– «B» – flexural failure mode; «Sh» – shear failure mode.*

Table 7 – Experimental values of the deflection and crack width obtained within self-stressed beams testing

Unit code	Deflection *a*, mm	Crack width (w_{max}/w_m), mm
I-BECS-(1)	2,3	0,1/0,1
I-BECS-(2)	2,7	0,15/0,07
I-BECS-(3)	2,9	0,1/0,09
I-BECS-(4)	3,2	0,1/0,1
II-BECF-(1)	4,9	0,7/0,59
II-BECF-(2)	4,6	0,6/0,38
II-BECF-(3)	4,6	0,6/0,47

Note *– In the table values of deflections, maximum and average crack width correspond to the loading rate of ≈0,6·P_{ult}, where P_{ult} – ultimate load.*





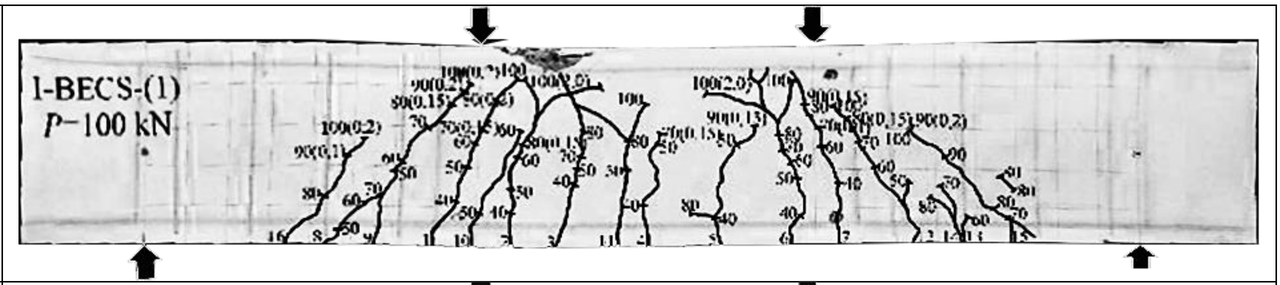
a) – self-stressed beams of the series I; b) – self-stressed beams of the series II

Figure 6 – Relations «*M-φ*» and «*M-a*» obtained on the static loading stage

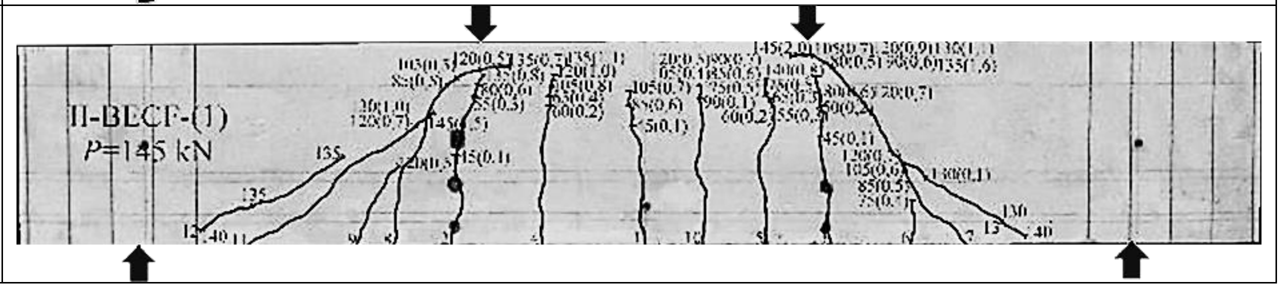
For beams of series I and series II, the first cracks occurred in the pure bending region at the load of 44 kN (7,1 kN·m) and 41 kN (8,2 kN·m) on average, respectively. After that in case of FRP reinforced beams, the slope of moment-curvature (moment-deflection) curves showed considerable drop and it was kept almost constant up to failure, as it is shown in Figure 3. In case of steel reinforced beams, three characteristic branch sections with different slopes was observed: the first branch section – up to cracking; the second branch section – from cracking and up to reinforcing steel yielding; the third branch section – from reinforcing steel yielding and up to the failure (see Figure 3). With increasing of the bending moment up to 24 kN·m, in the FRP reinforced beams, multiple inclined flexural shear cracks occurred outside the pure bending region and extended to a distance approximately 20 mm from the top surface of the beam. When applied load reached 143,3 kN (28,7 kN·m) at average, diagonal tension flexural shear failure mode was reached, but to this time FRP reinforcing bars didn't reach its ultimate tensile strains (in accordance with test results: $\varepsilon_{rt,frp}$ = 0,933 %). Taking into account that FRP reinforced self-stressed beams reinforcement ratio was equal to 1,6 % and 2,1 % for II-BECF-(1,2) and II-BECF-(3) respectively, that is considerably higher of the both balanced reinforcement ratio (ρ_{bal} = 0,3 %) and recommended reinforcement ratio 1,4· ρ_{bal} = 0,42 %. For the real reinforcement ratio of the tested beams, expected failure mode is due to crushing of the concrete in compression, but an observed failure mode had changed on the flexural shear without crushing of the concrete in compression. Moreover, registered within testing value of the ultimate moment was at average in 2 times higher than predicted value of the ultimate moment in accordance with [16] and based on the mean and established in tests values of the materials characteristics. In opposite to the FRP reinforced beams, failure mode and value of the ultimate load for steel reinforced self-stressed beams of series I was the same as it was predicted in accordance with [16] (ratio between predicted and established within loading ultimate bending moments was equal to 0,90).

Characteristic modes of failure and crack patterns for beams of the both series I and series II are shown in the Figure 7.

a)



b)



a) – self-stressed beams of the series I; b) – self-stressed beams of the series II

Figure 7 – General view of the beam crack patterns after test

Based on the analysis of the obtained experimental results, it can be stated, that initial early age stress-strain state obtained on the expansion stage influenced on the beams behavior during loading. It was observed that for the both series I and series II self-stressed beams cracking load was near 30 % from the ultimate load (see Table 6). Flexural cracks development through the concrete cross-section depth was following: arised flexural cracks extended on the average depth about 180 mm and 195 mm (≈75 % from cross-section depth) for series I and series II beams respectively and saved this position almost up to the failure on the background of the gradually increasing cracks number and its opening. This effect is explained that in the self-stressed structures initial compressive stresses are saved in concrete under the crack. An observed cracks patterns in the member tensile zone (see Figure 7) with an average distance between cracks 60±15 mm indicated about practically uniform distribution of the stresses longwise reinforcing bars in tension, that is inherent for pre-stressed structures.

To analyze results obtained within static loading of the self-stressed beams with non-symmetric both FRP and steel reinforcement arrangement the «M-$\varepsilon_{rt,x}$» diagram was proposed (where M is a bending moment; $\varepsilon_{rt,x}$ is a longitudinal tensile strain from the loading on depth of gravity center of the reinforcement in tension). The general view of the diagram is presented in the Figure 8.

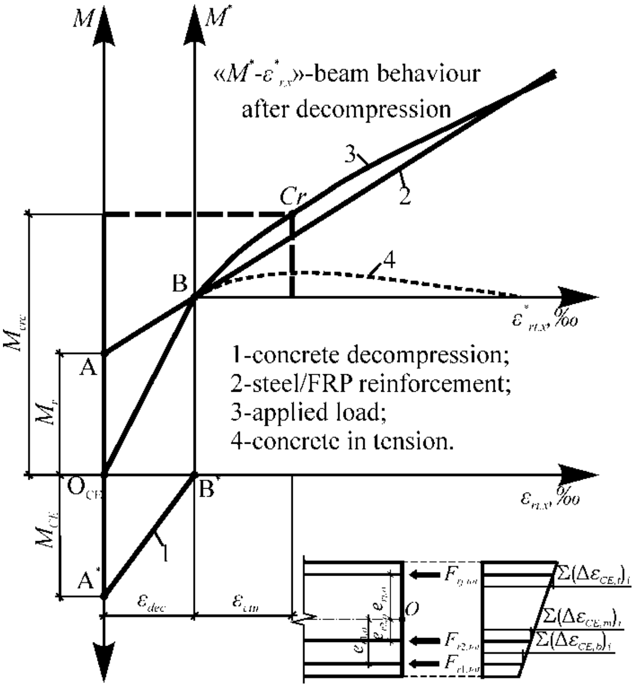


Figure 8 – Diagram for analysis of the initial stress-strain state influence on the behaviour under the loading of the non-symmetrically reinforced beams

Before load applying, in the beam cross-section balanced internal forces, obtained within self-stressing concrete expansion, are acting (see Figure 8: $O_{CE}A$=$O_{CE}A^*$, $M_{CE} = M_r$). Moments from the internal forces, accumulated to the end of the self-stressing stage, respectively concrete cross-section gravity center can be determined with respect to the value of the fixed restrained strains in reinforcement:

$$M_{CE} = M_r = \sum_{j=1}^{n} F_{rj,tot} \cdot e_{rj,o}, \tag{3}$$

where M_{CE} and M_r are the balanced moments from self-stressing;

$e_{rj,o}$ – eccentricity of the force in the j-th restraint reinforcement line respectively concrete cross-section gravity center;

$F_{rj,tot}$ – force in the j-th restraint reinforcement line, accumulated on the self-stressing stage to the concrete expansion stabilization, that is determined as follows:

$$F_{rj,tot} = \varepsilon_{rj}\left(t_{tot}\right) \cdot E_r \cdot A_r, \tag{4}$$

where $\varepsilon_{rj}\left(t_{tot}\right)$ – strain in the j-th restraint reinforcement line, accumulated on the self-stressing stage to the concrete expansion stabilization, calculated in accordance with MSDM model [12];

E_r, A_r – modulus of elasticity and area of the restraint reinforcement respectively.

After applying and further monotonically increasing of the load, reducing of the initial concrete cross-section pre-compression, obtained on the self-stressing stage, was observed. Besides, up to decompression point B* (see diagram in the Figure 8), cross-sectional tensile force is sustained by the reinforcement only (like it is in the traditional pre-stressed structures, line AB). Increment of the strains in reinforcement and increment of the bending moment, sustained by the reinforcement, before concrete decompression point B* is characterized by the AB line on the diagram in the Figure 6. At the same time, reducing of the concrete initial compressive stresses corresponds to the internal moment changing along the A*B* line. At the point B* (see Figure 8) concrete initial elastic compressive strains on the depth of gravity center of the reinforcement in tension reduces to 0 (so-called decompression stage). At the point B, line AB has the common point with the line O_{CE}B, characterized changing of the bending moment from the externally applied load. Within further loading after decompression point B*, behavior of the self-stressed member is the same like behavior of the conventional RC-beam without any initial pre-stressing (part of the diagram in the «M^*– $\varepsilon^*_{rt,x}$ » axises). At this loading stage, a tensile force in concrete cross-section is sustained together by the concrete in tension and reinforcement right up to the flexural cracks appearing. Flexural cracks appear when tensile strains in concrete exceeds its ultimate values ε_{ctu} (see diagram in «M^*– $\varepsilon^*_{rt,x}$ » axises in the Figure 8).

Thus, to the flexural cracks formation, the total strains respect to cracking $\varepsilon_{rt,crc}$ on the depth of reinforcement gravity center, is considered as a sum of decompression starins ε_{dec} and ultimate concrete tensile strains ε_{ctu}.

Resultant value of the cumulative concrete elastic strains $\varepsilon_{CE,el}\left(t_{sl}\right)$, that corresponds to the decompression strains ε_{dec} at the static loading should be calculated as follows:

$$\varepsilon_{dec} = \varepsilon_{CE,el}\left(t_{sl}\right) = \frac{\varepsilon_{CE,eltot}\left(t_i\right) \cdot E_{c,aw}\left(t_i\right)}{E_{cm,sl}}, \tag{5}$$

where $\varepsilon_{CE,eltot}\left(t_i\right)$ – concrete elastic strains accumulated to the end of the expansion stage and saved in structural memeber immediately before loading. It have to be calculated in accordance with proposed MSDM model [12];

$E_{c,aw}\left(t_i\right)$ – «average-weighted» expansive concrete modulus of elasticity, calculation procedure of it is presented in detail in [12];

$E_{cm,sl}$ – concrete modulus of elasticity to the static loading time;

t_i – age of concrete immediately before static loading.

Considering that decompression strains are a parameter that allows assessing the effectiveness of the initial self-stressing and to predict its further influence on the crack behaviour of the beams, this parameter (ε_{dec}) was got from experimental results analysis with diagram «M-$\varepsilon_{rt,x}$ » using and compared with the total tensile strains immediately before cracking measured on the depth of the reinforcement gravity centre

$\varepsilon_{rt,crc}$. This analysis of the self-stressing effectiveness was based on the assessment of the ratio between decompression strains ($\varepsilon_{dec,exp}$) and total tensile strains ($\varepsilon_{rt,crc}$), that is presented in Table 8.

Table 8 – Experimental values of the concrete tensile strains on the depth of the reinforcement gravity center

Unit code	$\varepsilon_{dec,exp}$, ‰	$\varepsilon_{rt,crc}$, ‰	(2)/(3)
(1)	(2)	(3)	(4)
I-BECS-(1)	0,189	0,528	0,36
I-BECS-(2)	0,241	0,542	0,44
I-BECS-(3)	0,229	0,533	0,43
I-BECS-(4)	0,312	0,658	0,47
II-BECF-(1)	0,091	0,494	0,18
II-BECF-(2)	0,095	0,480	0,20
II-BECF-(3)	0,101	0,490	0,21

As it is shown in Table 8, from experimental research [14] this ratio was at average 0,43 and 0,20 for self-stressed beams of the series I and series II respectively.

For effectiveness of the FRP reinforcing bars application in the pre-stressed (self-stressed) structures, «*M*–$\varepsilon_{rt,x}$» diagram was utilized (see Figure 8). It was assessed from the experimental results, that before loading in the beams of Series I and Series II almost equal values of the moments created by the pre-compression forces was obtained (was at average 3 kN·m). Therefore decompression strains in case of FRP bars using were less approximately in two times in comparison with decompression strains registered in self-stressed beams with steel reinforcement (see Table 8). It was stated, that up to decompression point, resultant force in tensile zone of the cross-section is sustained by the reinforcing bars only (at this stage concrete is under the initial compressive stresses). Taking into account that steel and FRP bars are characterized by the different values of modulus of elasticity (FRP bars modulus of elasticity E_{frpm} = 45,2 GPa, that was close to the concrete modulus of elasticity E_{cm} = 25,7 GPa), a different values of the moment increment was observed for the same levels of the longitudinal tensile strains in reinforcement (in case of FRP reinforcement, such increments were sufficiently less). To obtain equal values of the moment increments in case of FRP and steel bars utilizing, required area of FRP reinforcement have to be increased considerably and can be found based on the optimization procedure (it consists in the assessment of the FRP reinforcement axial stiffness, that is necessary to provide desired values of the moment increments within decompression stage as well as initial self-stresses at the expansion stage).

Nevertheless, it should be pointed that obtained self-stressing parameters in the members reinforced with FRP bars not only lead to the cracking moment increasing, but change series II self-stressed beams post-cracking behavior. A number of cracks, comparable with cracks number in series I self-stressed beams with steel reinforcing bars was observed (N = 9 and N = 12 at average respectively), and maximum flexural crack width was not exceed 0,6 mm under the loading rate near 0,6·P_{ult}.

Conclusions

1. A self-stressed structure is a prestressed structure, in which we create the tension of the reinforcement by the work that self-stressing concrete performed against restraint at the expansion stage [17]. Resultant pre-stressing force transfers from tendons to expanding concrete by the bond or anchorage and depends on the degree of restraint. The cases considered are those when misalignments are not produced at the respective contact surface between expansive concrete and reinforcing bars [18].

2. Independently from the type of restraint (steel bars or FRP bars) transferring of the chemical pre-stressing force to self-stressing concrete is realized like for traditional pre-stressed structure [19]. At all stages of the self-stressing expansive strains are linearly distributed in the direction of the cross-sectional height [20]. Considering that self-stresses distribution is related to the restrained strain distribution It is difficult to imagine why such local stress distribution was adopted by some authors as a basic assumption in the «theory of self-stressing distribution model» [7] and repeated in a more controversial form as an assumption to «calculation model of cracking load and deflection of textile reinforced self-stressing concrete» [6].

3. Self-stressing is related to the elastic part of deformations only. All rules applied to the design of the pre-stressed structures (for checking of the serviceability limit states) are valid for self-stressed structures reinforced with FRP [21]. In such a case, why do we have to apply the finite difference method for the calculation of cracking load and deflection of TRSSC beams? According to the modern crack resistance theory cracking load depends mainly on the ultimate tensile strain of concrete (no tensile strength). Based on the obtained test results by the authors, it is possible to make a conclusion, that *«the comparison of calculated and test values indicates an error of less than 30%, which is consistent with each other, thus verifying the applicability of calculation method»*. It is a very optimistic statement!

The following conclusion is optimistic too: *«self-stress can significantly improve the cracking resistance of TRSSC beams. Although the tensile strength of the matrix of TRSSC is 26% lower than that TCR, the cracking loads of the TRSSC beam are increased by 33% and 30%»*. In first, in the experiment self-stressing cement grade 4,0 (self-stress in standard condition is equal to $f_{ct,m}$ = 4,0 MPa) was used. Matrix specimens were cured before tensile testing in non-restrained conditions. In such conditions unbalanced expansion of the active self-stressing cement matrix, leads to self-damaging of the own material structure and decreasing of the tensile (and compressive) strength. Testing these specimens after curing in the restrained conditions (like it was in tested prisms) will get higher values of the tensile strength. Now it is difficult to assess what is the value of tensile strength we have to account for when we want to verify the proposed crack resistance model. Moreover, experimental results presented in [6, 7] are very unclear and non-representative (for instance, the same mix proportions for matrix type NC and SSC; dimensions of the reinforced TRSSC beams (prisms 100 × 100 × 400 mm for testing so sensitive parameter as crack resistance); measurement (with unknown error) of the longitudinal deformations with the usage of the laser rangefinder only at the level of the layer of textile; curing under standard conditions, etc.)

Influence of the initial self-stressing on the concrete member behavior under the monotonically increasing loading was studied with the proposed diagram method. Obtained within self-stressing concrete expansion stress-strain state in the both steel and FRP reinforced self-stressed beams positively influenced on these member behavior under the applied load. Nevertheless, considerable difference in the behavior of the self-stressed beams with steel and FRP reinforcement was observed, especially up to decompression point. For FRP reinforcement effective utilizing, optimization procedure based on the joint consideration of the proposed both MSDM [12] and diagram method have to be applied for certain design case.

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