KAUNAS UNIVERSITY OF TECHNOLOGY

TADAS ZINGAILA

THE INVESTIGATION OF MECHANICAL PROPERTIES OF FLEXURAL REINFORCED CONCRETE COMPOSITE MEMBERS WITH ULTRA-HIGH PERFORMANCE CONCRETE LAYER

Summary of Doctoral Dissertation
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TADAS ZINGAILA

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INTRODUCTION

The relevance of scientific problem

Due to the extraordinary mechanical and durability properties, ultra-high performance concrete surpassed the ordinary concrete; however, the high price of this type of concrete especially limits the possibilities of structural elements manufacture, and its practical application is still preferred only in more developed countries. The idea to use combined ordinary and ultra-high performance concrete was suggested by the other scientists; however, the main attention was paid to the strengthening of existing reinforced concrete structures. In many countries, there exists a problem that the earlier designed reinforced concrete structures do not meet the essential structural requirements before the predicted design age of the structure has been reached (appearance of not allowed cracks, damage of concrete cover, corrosion of reinforcement, too large deflections); therefore, it inspired to think about possibilities how to use more advanced materials for strengthening the structures. However, contemporary scientists can suggest other solutions to extend the durability of newly constructed buildings as well. One of such solution is the creation of more advanced structures. In this case, it would be advisable to pay more attention to the analysis of new flexural reinforced concrete composite members, without which industrial production remains impossible, while there are no prepared reliable and alternative manufacturing technology and calculation methods. Such type of composite members exhibits better strength, stiffness, resistance to cracking and durability properties in comparison to the ordinary reinforced concrete structures; therefore, it could be used effectively in buildings with reinforced concrete frames, which have more strict serviceability limit state or durability requirements due to the influence of hazard environment or other negative actions. Currently, a larger contribution from scientists is necessary in this field, in the development of new and the improvement of old calculation methods and manufacturing technologies of composite structures. The mechanical properties and behaviour of ultra-high performance concrete and ordinary concrete are significantly different; therefore, the design regulations and standards of reinforced concrete structures are not directly suitable for the calculations of composite members of combined steel fibre and ordinary reinforcement reinforced concrete/ultra-high performance concrete. The existing calculation methods should be improved, or it is necessary to create new methods.

Research object and methodology

The flexural reinforced concrete composite members were analysed in this thesis. Based on the performed investigations, the methodology for the calculation of deformations of composite members uncracked and cracked sections were created taking into account the different behaviour of ordinary and combined steel fibre and ordinary reinforcement reinforced with ultra-high performance concrete. The influence of heat treatment and other factors on mechanical properties and interface strength of composite members were analysed as well. Experimental, analytical, iteration and numerical methods were applied in the thesis.

The aim of the thesis

The aim of the thesis is to create methodology for the calculation of deformations of flexural reinforced concrete composite members with combined steel fibre and ordinary reinforcement reinforced ultra-high performance concrete layer.

The objectives of the thesis

1. To perform the analysis of literature review on the topic of flexural reinforced concrete composite members investigations when ultra-high performance concrete is used and analyse the mechanical and physical properties of different strength concretes and factors influencing the bond strength between different composites.
2. To make experimental investigation of new flexural reinforced concrete composite members and determine the influence of heat treatment on composites mechanical properties and bond strength.
3. To create the analytical model for stress and strains calculations in uncracked and cracked sections of flexural reinforced concrete composite members.
4. To calculate the deflection of composite members according to the method given in Eurocode 2, applying the analytically determined average curvatures.
5. To calculate the deflection of flexural reinforced concrete composite members using finite element software “ABAQUS”.

Scientific novelty and its significance

Based on the results of performed experimental analysis on flexural combined steel fibre and ordinary reinforcement reinforced concrete composite members, the method to describe the variation of reduced residual tensile stresses in cracked section was created.

The practical value of research findings

The calculation method that has been created can be applied to the calculations of average curvature and deflection of flexural reinforced concrete composite members with combined steel fibre and ordinary reinforcement reinforced ultra-high performance concrete layer.

Statements presented for the defence

1. The suggested method describing the variation of reduced residual tensile stresses in a cracked section of flexural reinforced concrete composite members allows to evaluate the influence of steel fibre reinforced ultra-high performance concrete layer thickness and the amount of ordinary reinforcement of curvature calculations.
2. When applying the suggested calculation methodology, it is possible to determine the optimal thickness of flexural composite member layers considering its stiffness.

1. LITERATURE REVIEW

Ultra-high performance concrete is a relatively new type of concrete, which has been developed a few decades ago. It is a composite material with extraordinary mechanical properties and enhanced durability having compressive strength ≥150 MPa and tensile strength ≥7 MPa as well as exhibiting high residual tensile strength after the crack opening (AFGC, 2013) due to the big amount of steel fibre in the mix composition. The first more comprehensive analysis of structural behaviour of flexural reinforced concrete composite members, which is described in Habel's (2004) doctoral thesis, became the basis for the further research of composite members. Habel (2004), Habel, Denarié and Brühwiler (2006) have found that flexural reinforced concrete composite members exhibit higher stiffness and load bearing capacity in comparison to the ordinary reinforced concrete members. Furthermore, the crack width and spacing are reduced; the development of the macrocracks is delayed due to the high tensile strength and the influence of steel fibre, and the localization of the cracks begins at the higher level of loads. Due to the low water permeability and small crack widths, the strengthening layer perfectly provides protective function. During the performed experimental analysis, no significant debonding of layers has been observed. The further analysis was carried out by Wuest (2006, 2007), where the behaviour of tensile ultra-high performance fibre reinforced concrete and the influence of fibre orientation were investigated by applying the determined parameters to the calculations of composite members. Noshiravani and Brühwiler (2010, 2013a, 2013b) analysed the behaviour of composite beams under the combined action of bending moment and shear force. Bastien-Masse and Brühwiler (2013, 2014), Bastien-Masse et al. (2014) carried out an extensive analysis of strengthened composite slabs punching resistance. The investigations of strengthened existing flexural reinforced concrete structures, when ultra-high performance concrete was used, were performed as well by the other scientists (Brühwiler, 2012; Brühwiler, Denarié, 2008, 2013; Denarié, Habel and Brühwiler, 2003; Lampropoulos et al., 2016; Martinola et al., 2007, 2010; Tsioulou, Lampropoulos and Dritsos, 2012). High strength concrete was used for the strengthening purposes (Kheder, Al Kafaji and Dhiab, 2010; Lapko, Sadowska-Buraczewska and Tomaszewicz, 2005; Sadowska-Buraczewska, Lapko, 2007). More extensive investigations on newly cast composite members with ultra-high performance concrete layer were made by Hussein (2015), Hussein and Amleh (2015). These studies focused on the manufacture process of new composite members, and in the further phases of the experiments, on the investigations of shear capacity of composite members without shear reinforcement.

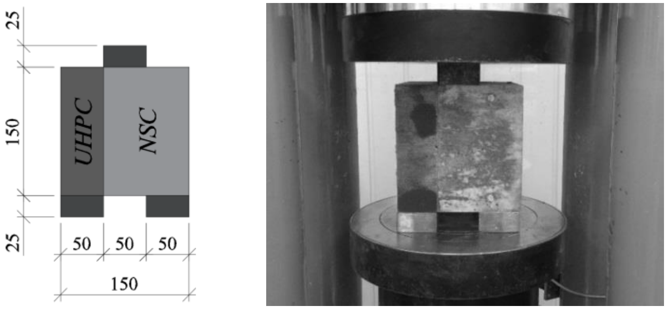
The behaviour of steel fibre reinforced ultra-high performance concrete was analysed by Xu and Wille (2015), Naaman (2008), Wille, El-Tawil and Naaman (2014), López et al. (2015), Fehling et al. (2013), Leutbecher (2008), Leutbecher and Fehling (2012). The comprehensive analysis of mechanical properties and durability of ultra-high performance concrete was carried out by the other scientists as well (Graybeal, 2007, 2014a, 2014b; Graybeal, Tanesi, 2007; Graybeal, Davis, 2008; Máca, Sovják and Vavřiník, 2013; Voit, Kirnbauer, 2014; Wille, Naaman, 2010), which made a significant contribution to the development of this kind of concrete.

2. EXPERIMENTAL RESEARCH

2.1. Research to determine the influence of heat treatment on mechanical properties of concretes

In order to determine the influence of heat treatment on the shear-bond strength of different concretes, the experiments were performed according to the method proposed by Momayez et al. (2002). Momayez et al. (2002, 2004, 2005) and Mousa (2015) determined that the type of the test and size of the specimens have influence on the test results; however, for relative comparison, when only the influence of heat treatment is analysed, it was decided to use Momayez et al. (2002) proposed simple bi-surface shear test method. In this case, standard 150×150×150 mm formworks can be used, where 2/3 of it is filled with ordinary concrete, and ultra-high performance concrete is cast in the left space. It is necessary to mention that by using this method, the average shear-bond strength is measured. Additional inaccuracies can appear as well due to the influence of two shear planes. The specimens are loaded through three 150×50×25 mm steel plates. According to the other scientists' (Santos, Santos and Dias-da-Costa, 2012) experience, the specimens were loaded with constant load of 2 kN/s. The arrangement of bi-surface shear test is given in Fig. 1.

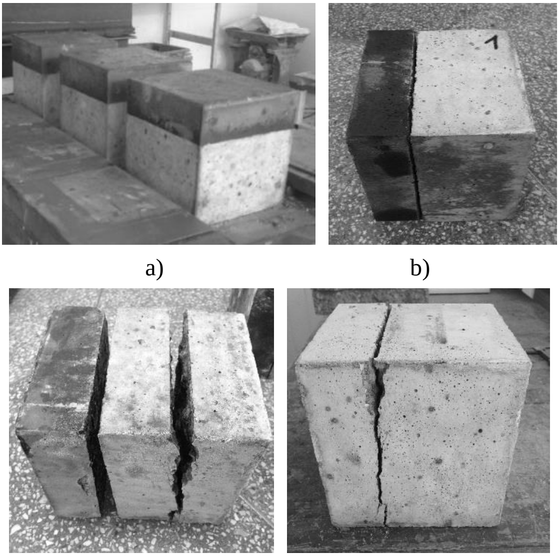
The insufficient bond of two different concrete layers in newly cast flexural reinforced concrete composite member can have influence on the rapid strength reduction of such elements or even failure. Without the influence of heat treatment, there are many other factors, which have an influence on the interface strength; however, it is difficult to distinguish it from the others.



a) b)

Fig. 1 Bi-surface shear tests: a) scheme of bi-surface shear test, b) testing of composite specimen

The composite specimens before and after the failure are given in Fig. 2 a), and Fig. 2 b), c). For the comparison of results, continuous specimens from ordinary concrete were made as well, and the sample after the failure is given in Fig. 2 d).



a) b)

c) d)

Fig. 2 Composite and continuous concrete specimens: a) composite specimens before failure, b) and c) composite specimens after failure, d) continuous specimen after failure

The experimental shear-bond strength results of composite specimens are given in Table 1. However, it is quite complicated to draw conclusions. In scientific publications (Momayez et al., 2002, 2004, 2005; Mousa, 2015; Nagaonkar, Bhusari, 2014; Santos, Júlio, 2010; Santos et al., 2012; Tayeh et al., 2012, 2013), it has been observed that the roughness of surface, curing conditions, different shrinkage deformations and modulus of elasticity, time interval between casting, volume of silica fume in concrete mix composition, reinforcement crossing the interface etc. have influence on the shear-bond strength of different concretes. Under the circumstances of performed experiments, the best results were obtained for the control specimens *B-NSC/UHPC-WHT*, which were cured in natural conditions (in +20 °C water for 28 days). Heat treated specimens (at 65±2 and 90±2 °C temperature) had lower average shear-bond strength 62.87% and 60.48%, respectively. However, it should be emphasized that the variation of results is very wide and depending on the case, it reaches up to 38.30%, 43.40% and 27.62%. Despite this fact, according to the results given in Table 1, the influence of heat treatment can be seen clearly.

Table 1. The influence of heat treatment on average shear strength of *NSC/UHPC* composites

Type of heat treatment	Average shear strength τ, MPa	Standard deviation, MPa	*COV*, %	Failure mode
B-NSC/UHPC-WHT	5.01 (100%)	1.92	38.30	Interface In 2 planes Interface
B-NSC/UHPC-65*HT*	1.86 (37.13%)	0.81	43.40	Interface Interface Interface
B-NSC/UHPC-90*HT*	1.98 (39.52%)	0.55	27.62	Interface Interface Interface

After the heat treatment at high temperature, the shrinkage deformations of ultra-high performance concrete come close to zero (AFGC, 2013); therefore, despite the influence of other factors, the rapid shrinkage of ultra-high performance concrete during the heat treatment process could determine the reduction of the strength as well.

2.2. Research on strength and stiffness of flexural composite members

New flexural reinforced concrete composite beams with a tensile layer of combined steel fibre and ordinary reinforcement reinforced with ultra-high performance concrete were cast during the doctoral studies, and then, the experimental analysis of strength, stiffness and cracking was performed. The purpose of these investigations was not only to evaluate the behaviour of composite members but to avoid influence of possible technological effects on mechanical properties of such structures as well. Twelve intermediate size beams with geometry of 1300(l)×160(b)×200(h) mm were cast and tested during the experimental research. Eight units of composite beams were made from high strength and ultra-high performance concrete, and 4 units of high strength concrete were made additionally as control specimens for the comparison of results and prediction of composite beams effectiveness. The types of the beams are presented in Fig. 3.

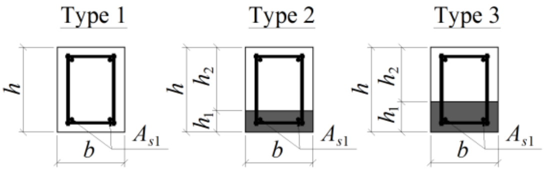


Fig. 3 Types of specimens

The beams can be grouped according to the thickness of ultra-high performance concrete layer as three different types, and two types can be distinguished according to the ratio of reinforcement. The thickness of ultra-high performance concrete layer was selected taking into account the issue of protecting longitudinal reinforcement in the tension side of the beam. Type 1 beams were made from high strength concrete and reinforced only with ordinary reinforcement (h=200 mm), type 2 and type 3 beams were strengthened with h_1=50 and h_1=70 mm layer of combined steel fibre and ordinary reinforcement reinforced ultra-high performance concrete. The percentage of longitudinal reinforcement was ρ_l=0.577% and ρ_l=1.132%, respectively. The data on the geometry and reinforcement of the beams are presented in Table 2.

The example of specimen notation: S3-50/150-2d10, where S3 is beam number; 50/150 is ultra-high performance concrete/high strength concrete layer thickness, mm (for beams which are not composite, the full height of the beam is given in mm), 2d10 is the number and diameter of reinforcement in the tension side of the beam, mm.

Table 2. Geometry and reinforcement of beams

No.	Specimen	h, mm	h_1/h_2, mm	b, mm	l, mm	A_{s1}, mm^2	ρ_l, %
1	S1-199-2d10	199	–	159	1300	157	0.577
2	S2-198-2d10	198	–	161			
3	S3-49/152-2d10	201	49/152	160			
4	S4-50/150-2d10	200	50/150	160			
5	S5-65/137-2d10	202	65/137	160			
6	S6-55/147-2d10	202	55/147	162			
7	S7-198-2d14	198	–	159		308	1.132
8	S8-199-2d14	199	–	160			
9	S9-47/152-2d14	199	47/152	159			
10	S10-48/150-2d14	198	48/150	160			
11	S11-70/129-2d14	199	70/129	159			
12	S12-68/131-2d14	199	68/131	161			

The test setup and reinforcement arrangement details are given in Fig. 4 and Fig. 5. Similar cases were applied to the ordinary reinforced concrete beams as well.

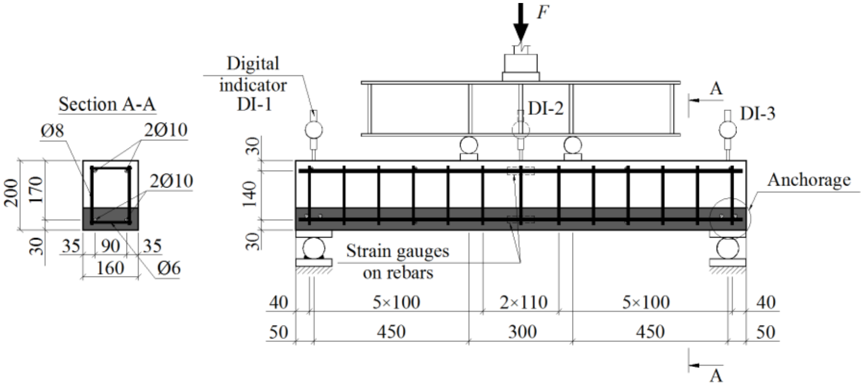


Fig. 4 Test setup and details of S3-S6 concrete-UHPFRC/RC composite beams

All the beams were tested by using hydraulic force equipment with the capacity of 200 kN. The loading of the beams was performed according to the load control by manually increasing the load by steps of 2,0 – 4,0 kN. Three digital indicators "Mitutoyo" (accuracy – 1μm) were used to measure the vertical beam deflection at the supports and in the middle of the beam.

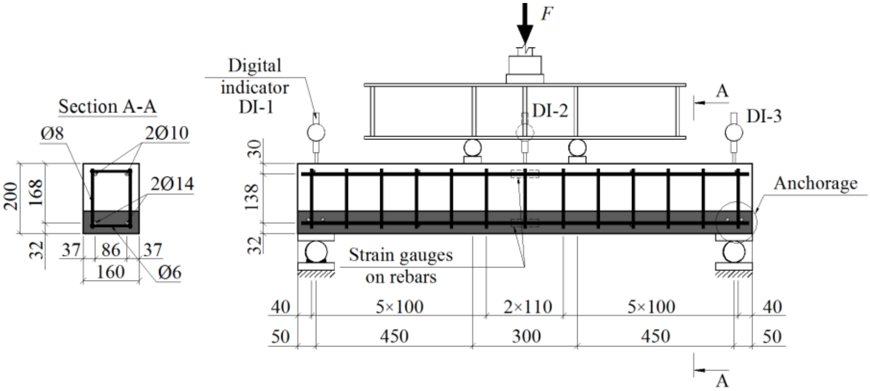


Fig. 5 Test setup and details of S7-S12 concrete-UHPFRC/RC composite beams

Analysing the influence of ultra-high performance concrete layer thickness, a higher efficiency was observed for the beams with lower reinforcement percentage (S3 – S6 beams). In the case of higher reinforcement percentage (S9 – S12 beams), the influence of ultra-high performance concrete layer thickness was minimal. In this case, it is necessary to emphasize that the differences between the thickness of ultra-high performance concrete layers were relatively minimal, and at the greater difference, this effect would be more significant. However, the purpose of these studies was not only to increase the strength or stiffness of the elements but to protect the problematic tensile zone from cracking as well. As shown in Fig. 6, in order to increase the strength or stiffness of the element, it would be more efficient to add more reinforcement.

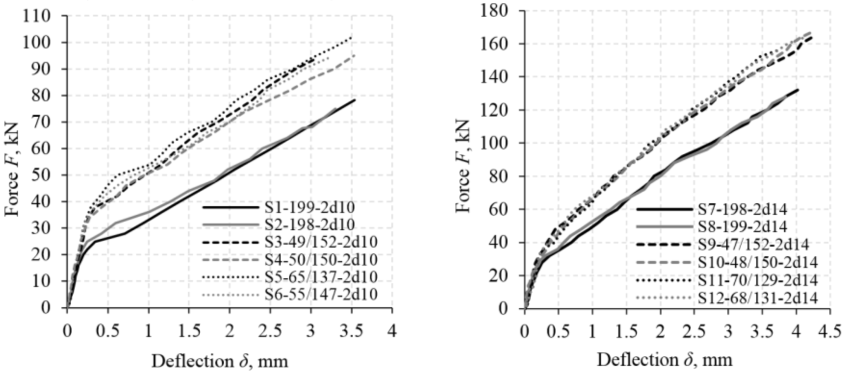


Fig. 6 Force – deflection relationship approximately till the yielding of reinforcement: a) beams S1 – S6 (reinforcement percentage 0.577%, 2d10 rebars), b) beams S7 – S12 (reinforcement percentage 1.132%, 2d14 rebars)

In the experimental analysis of stress in reinforcement and the curvature of cracked section of flexural reinforced concrete composite members, the positive effect of ultra-high performance concrete reinforced with steel fibre was observed.

The main values of bending moments are given in Fig. 7.

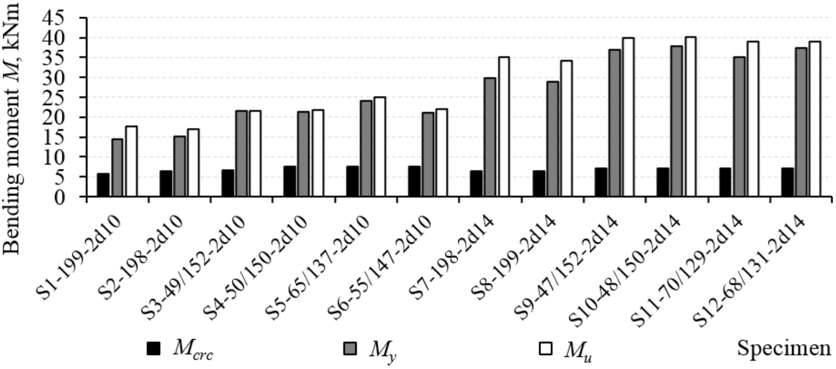


Fig. 7 Bending moments M_{crc}, M_y and M_u of S1-S12 beams

3. THEORETICAL RESEARCH

3.1. Method to determine the optimal ultra-high performance concrete layer thickness of flexural reinforced concrete composite member

In the analysis of flexural reinforced concrete composite members (Fig. 8), the question arises about the determination of optimal thickness of stiffer layer; therefore, the calculation method was proposed. Increasing the thickness of the stiffer layer of rectangular cross-section as well results in the increase of the effective moment of inertia. However, this increment of moment of inertia is not continuous with the growth of thickness of the layer. Initially, when the thickness of the stiffer layer forms a small part of the cross-section, the increase of this layer results in a significant increase in the effective moment of inertia.

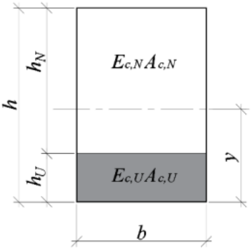


Fig. 8 Composite section with different stiffness of layers

However, this increase in thickness begins to play a smaller role when the thickness becomes close to the distance from the edge of the tensile zone to the centre of gravity of the element. In order to determine the effective thickness of the layer, it is necessary to express the inertia moment dependence on the thickness of the layers. Analysing the curve "1" (Fig. 9), it can be observed that the effective moment of inertia grows to a certain value of stiffer layer thickness (point "*A*"), from which the subsequent increase of the layer becomes insignificant until the layer overpass the side of the compression zone.

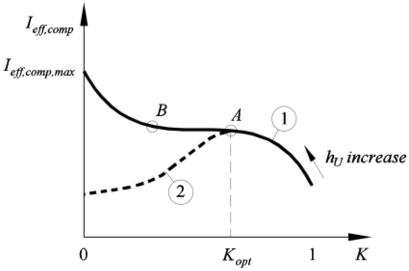


Fig. 9 Dependence of the effective moment of inertia on the thickness of the stiffer layer

When the stiffer layer overpass compression zone, the growth of the effective moment of inertia, even up to the point “*B*”, is not intense, but by increasing the thickness of stiffer layer, this growth begins to increase significantly. Since the work focuses on the tensile zone of the composite member, the main goal of this study is to determine the point “A” on the curve “1”, which as well describes the optimum thickness of the stiffer layer:

$$h_U = (1-K)h = (1-K_{opt})h; \tag{1}$$

where K_{opt} is the optimal ratio of thickness of the ordinary concrete layer to the total cross-section height.

Formula (2), which satisfies the condition $K = \{0 \ldots 1\}$, describes the optimum thickness of the layer.

$$\begin{aligned} &K^4\left(0{,}5-2\alpha_C+3\alpha_C^2-2\alpha_C^3+0{,}5\alpha_C^4\right)-K^3\left(0{,}25-2{,}75\alpha_C+6{,}75\alpha_C^2-6{,}25\alpha_C^3+2\alpha_C^4\right)-\\ &-K^2\left(\alpha_C-5\alpha_C^2+7\alpha_C^3-3\alpha_C^4\right)-K\left(1{,}25\alpha_C{}^2-3{,}25\alpha_C{}^3+2\alpha_C^4\right)+0{,}5\left(\alpha_C{}^4-\alpha_C{}^3\right)=0. \end{aligned} \tag{2}$$

3.2. Application of the proposed model to the analytical and iterative layer methods for the deformational analysis of the flexural composite members

The effectiveness of steel fibre in flexural steel fibre reinforced concrete structures immediately after the crack opening is greater than for the flexural combined steel fibre and ordinary reinforcement reinforced concrete structures, because the ordinary reinforcement together with steel fibre transfers the tensile stress after the crack opening and delays the full activation and effectiveness of steel fibre. In that situation, when reinforced concrete structures are additionally reinforced with high amount of steel fibre (≥ 2%), it is a usual case to obtain strain hardening behaviour. The assumption is made in the model that the plastic behaviour of steel fibre begins after the crack opening, which does not exceed the limit value of tensile strength of concrete matrix, and only the partial influence of steel fibre is taken into account, which increases together with the increment of load. Therefore, while the crack width is relatively small, the bigger part of tensile load is transferred through stiffer reinforcement, and with the increment of crack width, the effectiveness of steel fibre increases further. The limit value, when the maximum effectiveness of steel fibre is achieved, can vary depending on the properties of steel fibre and reinforcement; however, in the suggested model, it is assumed that the maximum effectiveness of steel fibre is achieved with the beginning of reinforcement yielding.

In order to evaluate the curvature of cracked section more precisely, based on the performed experimental results, the empirical formula (3) was suggested in this work for the calculation of residual tensile stress reduction coefficient, and it shows what part of residual tensile stress is applied to the calculations of flexural combined steel fibre and ordinary reinforcement reinforced concrete members immediately after the crack opening. It has been observed during the investigation that it is not possible to propose the constant value of the α_{red} coefficient for all the cases; therefore, based on the boundary conditions of performed experiments, the influence of ultra-high performance concrete layer thickness and reinforcement ratio were taken into account:

$$\alpha_{red} = \left(\frac{1}{\rho_{sl}^{0,3}}\right)\left(\frac{a_2 \rho_{sl}}{1,35}\right)^{0,54 a_1} \times \left(1 - 1,35 \rho \rho_{sl}^{-0,9} a_1^{0,01 a_2}\right) \leq 1; \tag{3}$$

where $\rho = A_s/(bd)$, % is the reinforcement percentage of the whole section; $\rho_{sl} = A_s/(bh_{sl})$, % is the reinforcement percentage of ultra-high performance concrete layer; $a_1 = \rho/\rho_{sl}$; $a_2 = \rho_{sl}/\rho$.

The variation of residual tensile stress from the value of cracking moment M_{crc} to the beginning of reinforcement yielding M_y is described according to the formula (4) assuming the reduced plastic behaviour of fibres through the whole thickness of ultra-high performance concrete layer. The principal scheme of the model is given in Fig. 10.

$$\alpha_{fb} = \left[\frac{M_{Ek} - M_{crc}}{(\beta_y - 1) M_{crc}}\right](1 - \alpha_{red}) + \alpha_{red}; \tag{4}$$

where α_{red} is the minimum value of residual tensile stress reduction coefficient; $\beta_y = M_y/M_{crc}$; M_y is the value of bending moment, when yielding strains of reinforcement is reached; M_{crc} is the cracking moment.

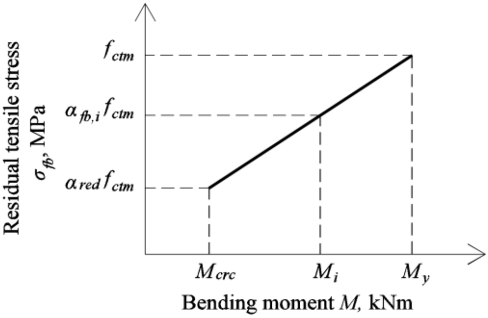


Fig. 10 Variation of residual tensile stress in cracked section of ulta-high performance concrete layer of flexural reinforced concrete composite member according to the proposed method

The assumptions and limitations of the proposed method:

- the method is created on the basis of performed experimental results of composite members, where the ratio of ultra-high performance concrete layer and the effective height of the beam was $h_U/d \leq 0.45$;

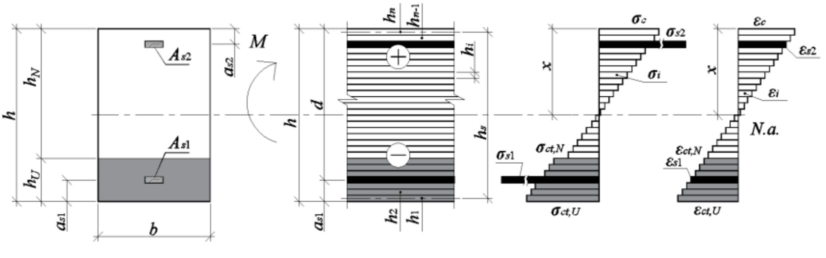
- the volume of steel fibre ≥ 2% (157 kg/m^3), type – straight steel fibre, l_f = 13 mm, d_f = 0.2 mm, ratio l_f/d_f = 65, $f_u \approx$ 2750 MPa;
- the methodology is related to the tensile strength of concrete matrix, which can be determined indirectly from flexural tests (5 formula) or calculated according to the compressive strength of the concrete (6 formula);

$$f_{ct,el} = f_{ct,fl} \frac{\alpha \cdot a^{0,7}}{1+\alpha \cdot a^{0,7}}, \tag{5}$$

$$f_{ctm} = 0.30 \times f_{ck}^{(2/3)}. \tag{6}$$

- the maximum effectiveness of steel fibre, which is equal to the tensile strength of concrete matrix, is achieved at the beginning of reinforcement yielding.

The adjusted layer iterative method can be used for the calculations of stress, strains and curvature in uncracked and cracked sections of flexural reinforced concrete composite members. The original version of this method is given in Augonis and Zadlauskas (2013) publication, where it is applied to the calculations of ordinary reinforced concrete members. The sections of various geometrical configurations can be calculated by using this method and taking into account nonlinear properties of materials as well as the influence of steel fibre after the crack opening. The principal schemes of iterative layer method with different loading stages are given in Fig. 11 and Fig. 12.



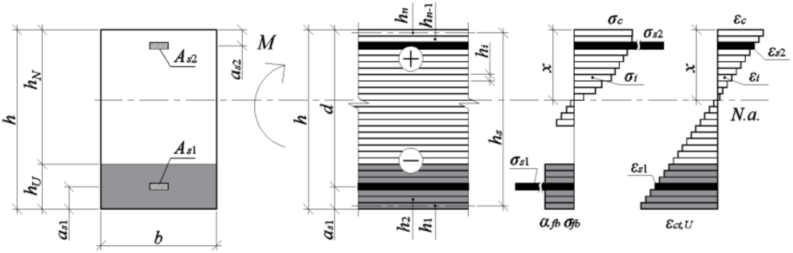
a) b) c) d)

Fig. 11 Calculation scheme in uncracked reinforced concrete composite member stage using the iterative layer method: a) cross-section of reinforced concrete composite member, b) longitudinal member section, c) stress in uncracked section, d) strains in uncracked section

The main assumptions using the iterative layer method for the calculations of flexural reinforced concrete members:

- plane section hypothesis is valid, the variety of strains through the height of the section is linear;
- perfect bond between reinforcement and concrete;
- full bond between each layer;

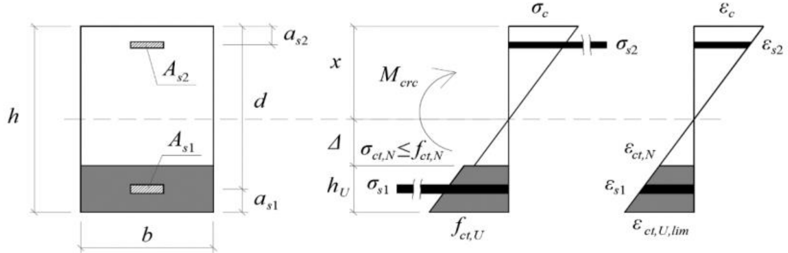
- full bond between different concrete composite layers;
- elastic behaviour of concrete is assumed in primary iterations, and when the nonlinear behaviour begins, deformation modulus of material is recalculated;
- the shrinkage of concrete is not taken into account due to the simplification of the model making an assumption that its influence will not be critical in the calculations of composite members.



a) b) c) d)

Fig. 12 Calculation scheme in cracked reinforced concrete composite member stage using the iterative layer method: a) cross-section of reinforced concrete composite member, b) longitudinal member section, c) stress in cracked section, d) strains in cracked section

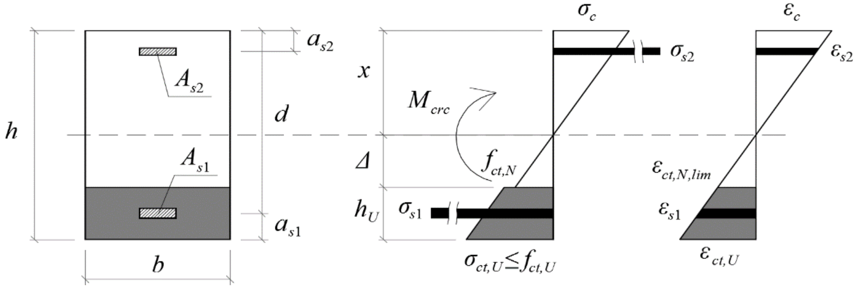
Stress, strains and curvature in uncracked and cracked sections of composite flexural member can be calculated as well by applying the simplified analytical calculation methods, assuming the elastic material behaviour before cracking or achieving maximum stress. The first case of composite flexural member cracking moment calculation is described in Fig. 13 when the layer of ultra-high performance concrete is only on the tension side of the section (i.e., $\Delta>0$), and the cracking begins from the most tensile fibre of ultra-high performance concrete layer.



a) b) c)

Fig. 13 Reinforced concrete composite member cracking moment calculation scheme, when the first cracks open in the layer of ultra-high performance concrete: a) cross-section of reinforced concrete composite member, b) stress in uncracked section, c) strains in uncracked section

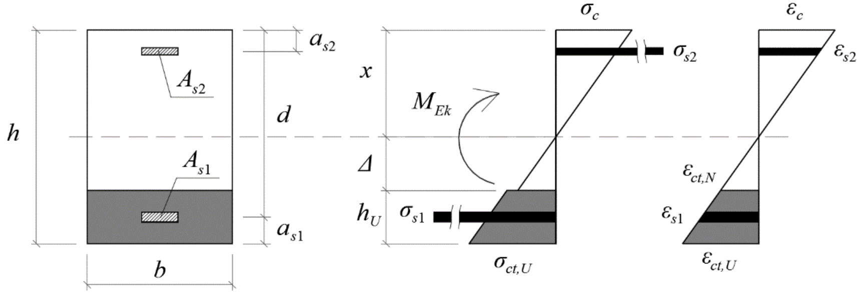
Another possible case of the cracking moment calculation exists when the cracking of the member begins from the most tensile fibre of the ordinary concrete (Fig. 14).



a) b) c)

Fig. 14 Reinforced concrete composite member cracking moment calculation scheme when the first cracks open in the layer of ordinary concrete: a) cross-section of reinforced concrete composite member, b) stress in uncracked section, c) strains in uncracked section

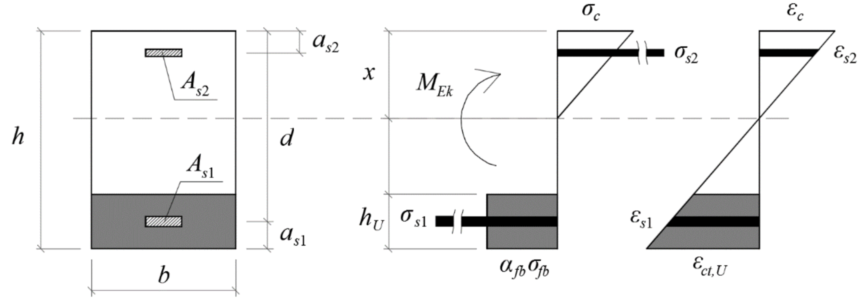
Due to the action of external bending moment, the stress, strains and curvature in the uncracked section of composite member can be calculated according to the principal scheme given in Fig. 15.



a) b) c)

Fig. 15 Reinforced concrete composite member uncracked section calculation scheme: a) cross-section of reinforced concrete composite member, b) stress in uncracked section, c) strains in uncracked section

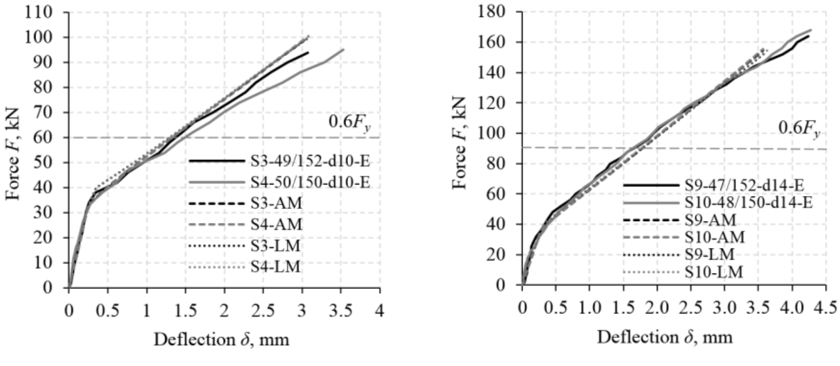
The calculation of stress and strains in cracked section is performed on the basis of the principal scheme given in Fig. 16. In this case, the influence of steel fibre has to be taken into account after the crack opening.



a) b) c)

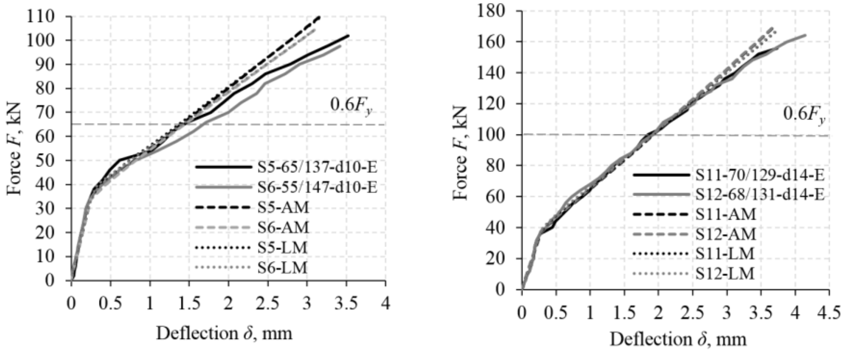
Fig. 16 Reinforced concrete composite member cracked section calculation scheme: a) cross-section of reinforced concrete composite member, b) stress in cracked section, c) strains in cracked section

Theoretical analysis of stress and strains of flexural reinforced concrete composite members was made by analytical and iterative layer methods, and the results were compared with experimentally obtained values. Stress and strains were calculated in uncracked and cracked sections of composite beams with both methods; then, the average curvatures and midspan deflections were determined according to the method given in Eurocode 2 (Fig. 17 and Fig. 18).



a) b)

Fig. 17 Theoretically calculated (AM – analytical method, LM – layer method) and experimentally obtained (E) $F - \delta$ relationships of composite beams: a) beams S3 and S4 ($\rho = 0.577\%$, bottom rebars – 2d10, amount of steel fibre in *UHPC* – 157 kg/m^3), b) beams S9 and S10 ($\rho = 1.132\%$, bottom rebars – 2d14, amount of steel fibre in *UHPC* – 157 kg/m^3)



a) b)

Fig. 18 Theoretically calculated (AM – analytical method, LM – layer method) and experimentally obtained (E) $F - \delta$ relationships of composite beams: a) beams S5 and S6 (ρ = 0.577%, bottom rebars – 2d10, amount of steel fibre in *UHPC* – 157 kg/m^3), b) beams S11 and S12 (ρ = 1.132%, bottom rebars – 2d14, amount of steel fibre in *UHPC* – 157 kg/m^3)

The range of results analysed in the work is assumed to be significant until the approximate value of $0.6F_y$, which is important in the serviceability limit state calculations.

4. NUMERICAL RESEARCH

The strains, curvatures and deflections of the middle section of flexural reinforced concrete composite members were analysed with finite element software Abaqus when the behaviour of tensile ordinary concrete and steel fibre reinforced ultra-high performance concrete was defined through the parameters of fracture energy. The behaviour of ordinary tensile concrete was described according to the method given in CEB/FIP Model Code 2010; however, additional assumption was made that before the opening of the crack, the concrete deforms elastically. Ultra-high performance fibre reinforced concrete was defined using linear material model given in the finite element software Abaqus when the tensile strength of the concrete matrix and the value of fracture energy after the crack opening are known as inputs. However, the strain-hardening behaviour of tensile ultra-high performance fibre reinforced concrete cannot be taken into account by using this model and has to be neglected. The elastic behaviour of tensile UHPFRC until the cracking was assumed. The influence of variation of ultra-high performance concrete tensile strength was analysed in this work as well.

The calculations of force – deflection relationships of flexural reinforced concrete composite members using finite element analysis show that by applying the above mentioned fracture energy model for ultra-high performance fibre reinforced concrete, the essential influence on the results has tensile strength of the concrete matrix. When the tensile strength of the concrete matrix is reached, the behaviour of steel fibre becomes close to plastic because of the high value of fracture energy. Moreover, the results were substantially affected by the amount of tensile reinforcement. As it has been observed during the analysis, the difference between numerical and experimental results reduces when the ratio reinforcement increases. When using this model in finite element analysis, the cracking moment of composite members and the stiffness of these members after imaginary cracks opening (when the tensile concrete reaches plastic deformations) are overestimated. It has been observed that the differences between numerical and experimental deflections depend on the value of tensile strength of concrete and the level of force when the deflection is measured.

Calculated deflections of type 50/150-d10 composite beams were smaller than experimentally measured, approximately: F = 50 kN, $\Delta\delta \approx 38 - 56\%$; F = 70 kN, $\Delta\delta \approx 16 - 52\%$. The results of type 50/150-d14 composite beams show smaller errors: F = 60 kN, $\Delta\delta \approx 22 - 40\%$; F = 100 kN, the error $\Delta\delta$ varies approximately from –23% to +2%. In the latter case, when the force F reaches more than 100 – 140 kN, the theoretical deflections of composite beams become bigger than experimental.

The actual thickness of ultra-high performance concrete layer of 70/130-d10 type composite beams was slightly smaller than theoretical; therefore, only one beam was compared. The calculated deflections were smaller than experimental: F = 60 kN, $\Delta\delta \approx 32 - 56\%$; F = 80 kN, $\Delta\delta \approx 14 - 50\%$. The errors of deflections of 70/130-d14 type beams were bigger than for 50/150-d14 type; however, it is necessary to mention that the experimental deflections of both types of the beams were similar in both cases independently from the thickness of ultra-high performance concrete layer. In this case, the calculated deflections of 70/130-d14 type beams were smaller than experimental: F = 60 kN, $\Delta\delta \approx 31 - 45\%$; F = 100 kN, $\Delta\delta \approx 12 - 34\%$.

CONCLUSIONS

1. After the analysis of scientific papers on the topic of flexural reinforced concrete composite members investigations when ordinary concrete and combined steel fibre and ordinary reinforcement reinforced ultra-high performance concrete are used, it has been determined that the main part of the research is oriented towards the strengthening of existing structures, paying insufficient attention to the manufacture and preparation of calculation methodologies of new composite members. The mechanical and physical properties of such type of concretes as well as their behaviour are significantly different. It has been found in scientific literature that the calculation methods of shear strength between different composites, which are given in design standards and recommendations of reinforced concrete structures, do not take into account such essential parameters as different

shrinkage deformations of concretes, curing conditions and different modulus of elasticities.

2. On the basis of experimental analysis performed during the doctoral studies, it has been determined that:

   a) the effectiveness of new reinforced concrete composite members using steel fibre reinforced ultra-high performance concrete is observable in both ultimate and serviceability limit states: enhanced member load bearing capacity, stiffness and resistance to cracking, reduced crack widths and crack spacing as well as stress in tension reinforcement;

   b) the analysis of composite elements without using heat treatment showed that the selection of correct casting succession does not require additional surface preparation;

   c) the heat treatment in 90±2 °C temperature can be less effective for composite members made from ordinary and ultra-high performance concrete than for the elements which are made only from ultra-high performance concrete; however, there are a lot of factors that can influence results.
3. On the basis of experimental results of flexural reinforced concrete composite members, the calculation method, which can evaluate the variation of reduced residual tensile stress in ultra-high performance fibre reinforced concrete layer after the crack opening, was created. The composed calculation method can be applied to determine the stress and strains as well as to calculate the curvature in cracked section by using different methods (layer (iterative), analytical, etc.). According to the composed optimal layer calculation method and considering the stiffness of layers of composite member, it has been determined that the optimal thickness of ultra-high performance concrete layer is about 30% of the whole section height.
4. When applying the created model and based on the calculation method of deformations given in Eurocode 2, the average curvatures and midspan deflections of the beams were calculated. The comparison of experimentally and theoretically obtained results showed that the reliability of model is enough in the calculations of serviceability limit state, i.e., approximately till the limit value of $0.6M_y$, when the maximum errors of deflections were +39% and –21%. After performing more experiments or collecting and analysing more results from the other researchers' investigations, the model may be revised and expanded in the future.
5. In the finite element modelling of composite beams, the essential influence on stress in the reinforcement, curvature and deflection has the variation of reduced residual tensile stress in ultra-high performance fibre reinforced concrete layer after the crack opening. The calculations are significantly simplified when the behaviour of steel fibre is described through the fracture energy; however, in the analysed case, when there was a small amount of reinforcement, up to ≈ 62% of errors of deflection have been obtained.

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REZIUMĖ

Disertacijos darbe nagrinėjami nauji kombinuotai plieno plaušu ir armatūra armuoti lenkiamieji kompozitiniai gelžbetoniniai elementai, pagaminti iš įprastinio ir ypač stipraus betono. Eksperimentinių tyrimų metu nustatytos skirtingo stiprumo betonų mechaninės savybės ir kompozitinių elementų sandūros atsparumas šlyčiai esant skirtingoms bandinių kietinimo sąlygoms. Ištirtos vidutinio dydžio lenkiamosios kompozitinės gelžbetoninės sijos ir išmatuoti jų įlinkiai, įtempiai armatūroje bei apskaičiuoti kreiviai. Remiantis atliktų eksperimentinių tyrimų rezultatais, sukurta metodika, kuria aprašomas plieno plaušo liekamųjų tempimo įtempių kitimas supleišėjusiame ypač stipraus betono sluoksnyje. Taikant pasiūlytą metodiką skaičiavimai gali būti atliekami analitiniu, sluoksnių (iteraciniu) ir kt. metodais, įvertinant tampriai plastinę tempiamojo betono elgseną prieš atsiveriant plyšiams arba jos nevertinant. Žinant nesupleišėjusio ir supleišėjusio pjūvių kreivius, gali būti apskaičiuojami vidutiniai kompozitinių elementų kreiviai ir įlinkiai. Darbe taip pat pasiūlyta optimalaus ypač stipraus betono sluoksnio storio apskaičiavimo metodika, kuria atsižvelgiama į kompozitinio elemento sluoksnių standumus. Modeliuojant lenkiamuosius kompozitinius gelžbetoninius elementus baigtinių elementų metodu, nagrinėti atvejai, kai plieno plaušu armuoto ypač stipraus betono įtaka atsivėrus plyšiams įvertinama per irimo energiją.

Darbo uždaviniai

1. Atlikti lenkiamųjų kompozitinių gelžbetoninių elementų, kuriuose panaudotas ypač stiprus betonas, tyrimų analizę. Apžvelgti įvairių stiprių betonų mechanines bei fizines savybes ir skirtingų kompozitų sukibimo stiprumą lemiančius veiksnius.
2. Eksperimentiškai ištirti naujų lenkiamųjų kompozitinių elementų elgseną ir terminio kietinimo įtaką kompozitų mechaninėms savybėms ir sukibimo stiprumui.
3. Sukurti analitinį modelį, kurį taikant būtų galima apskaičiuoti lenkiamųjų kompozitinių gelžbetoninių elementų įtempius ir deformacijas nesupleišėjusiame ir supleišėjusiame pjūviuose.
4. Taikant analitiniu būdu gautus vidutinius skerspjūvių kreivius, pagal EC2 metodiką apskaičiuoti kompozitinių sijų įlinkius.
5. Apskaičiuoti lenkiamųjų kompozitinių gelžbetoninių elementų įlinkius naudojant baigtinių elementų metodo programą „Abaqus“.

Darbo mokslinis naujumas ir reikšmė

Remiantis atliktų kombinuotai plieno plaušu ir armatūra armuotų lenkiamųjų kompozitinių gelžbetoninių elementų eksperimentinių tyrimų rezultatais, sukurtas metodas, kuriuo aprašomas redukuotųjų liekamųjų tempimo įtempių kitimas supleišėjusiame skerspjūvyje.

Tyrimų objektas ir metodai

Darbe nagrinėjami lenkiamieji kompozitiniai gelžbetoniniai elementai. Remiantis atliktais tyrimais kuriama metodika, skirta kompozitinių elementų deformacijoms nesupleišėjusiame ir supleišėjusiame pjūviuose apskaičiuoti įvertinant skirtingą įprastinio ir plieno plaušu bei armatūra armuoto ypač stipraus betono elgseną. Taip pat analizuojama terminio kietinimo ir kitų veiksnių įtaka kompozitinių elementų mechaninėms savybėms ir sandūros stiprumui. Darbe yra taikomi eksperimentiniai, analitiniai, iteraciniai ir skaitiniai tyrimų metodai.

Ginamieji teiginiai

1. Pasiūlytas metodas, kuriuo aprašomas lenkiamųjų kompozitinių gelžbetoninių elementų redukuotųjų liekamųjų tempimo įtempių kitimas supleišėjusiame skerspjūvyje, leidžia įvertinti ypač stipraus betono sluoksnio storio ir armatūros kiekio įtaką kreiviui.
2. Taikant pasiūlytą skaičiavimo metodiką galima apskaičiuoti optimalius lenkiamojo kompozitinio elemento sluoksnių storius, atsižvelgiant į jų standumus.

IŠVADOS

1. Išanalizavus mokslinėse publikacijoje aprašomus lenkiamųjų kompozitinių gelžbetoninių elementų tyrimus, kuriems buvo naudojamas įprastinis ir kombinuotai plieno plaušu ir armatūra armuotas ypač stiprus betonas, nustatyta, kad didžioji dalis atliktų tyrimų orientuoti į esamų gelžbetoninių konstrukcijų sustiprinimą ir nepakankamai dėmesio skiriama naujų kompozitinių konstrukcijų gamybai ir skaičiavimo metodikų parengimui, o šių betonų mechaninės bei fizinės savybės ir elgsena gerokai skiriasi. Mokslinėje literatūroje teigiama, kad gelžbetoninių konstrukcijų projektavimo normose ir rekomendacijose pateikiamoje šlyties tarp skirtingų kompozitų skaičiavimo metodikoje neatsižvelgiama į tokius esminius parametrus, kaip skirtingos betonų susitraukimo deformacijos, kietinimo sąlygos ir skirtingi tamprumo moduliai.
2. Atlikus eksperimentinius tyrimus nustatyta, kad:
   a) naujų kompozitinių gelžbetoninių elementų efektyvumas panaudojant plieno plaušu armuotą ypač stiprų betoną pasireiškia vertinant tiek saugos, tiek tinkamumo ribinį būvį: padidinama elemento laikomoji galia, standumas ir pleišėjimo momentas, sumažinami plyšių pločiai ir atstumas tarp jų, taip pat įtempiai tempiamojoje armatūroje;
   b) kompozitinių elementų tyrimai netaikant terminio kietinimo parodė, kad, parinkus tinkamą betonavimo procedūrų eiliškumą, kompozitų sandūros paviršiaus papildomai paruošti nereikia;
   c) terminis kietinimas 90±2 °C temperatūroje kompozitiniams elementams iš įprastinio ir ypač stipraus betono gali būti ne toks efektyvus kaip

elementams, pagamintiems tik iš ypač stipraus betono. Tačiau yra daugybė rezultatus lemiančių veiksnių.

3. Remiantis atliktais lenkiamųjų kompozitinių gelžbetoninių elementų eksperimentiniais tyrimais, sukurta skaičiavimo metodika, kurią taikant įvertinamas plieno plaušo redukuotųjų liekamųjų tempimo įtempių kitimas ypač stipraus betono sluoksnyje atsivėrus plyšiui. Sudaryta skaičiavimo metodika gali būti taikoma kompozitinių elementų supleišėjusio skerspjūvio įtempiams, deformacijoms ir kreiviui apskaičiuoti pasitelkiant skirtingus metodus (sluoksnių (iteracinį), analitinį ir t. t.). Taikant sudarytą optimalaus sluoksnio storio apskaičiavimo metodiką, nustatyta, kad, atsižvelgiant į kompozitinio elemento sluoksnių standumus, optimalus ypač stipraus betono sluoksnis sudaro apie 30 % viso skerspjūvio aukščio.
4. Taikant sukurtą modelį ir remiantis „Eurokode 2“ pateikiama deformacijų skaičiavimo metodika, apskaičiuoti kompozitinių elementų skerspjūvių vidutiniai kreiviai ir sijų vidurio įlinkiai. Palyginus eksperimentinius ir teoriškai gautus rezultatus, nustatyta, kad modelio patikimumas pakankamas skaičiuojant pagal tinkamumo ribinį būvį, t. y. apytiksliai iki $0{,}6M_y$ ribos, kai didžiausia įlinkio paklaida siekė +39 % ir –21 %. Atlikus daugiau eksperimentinių tyrimų arba surinkus ir apdorojus daugiau kitų mokslininkų tyrimų rezultatų, ateityje būtų galima tikslinti modelį ir plėsti jo taikymo galimybes.
5. Modeliuojant kompozitines sijas baigtinių elementų metodu, esminę įtaką armatūros įtempiams, kreiviui ir įlinkiui turi plieno plaušu armuoto ypač stipraus betono liekamųjų tempimo įtempių kitimas atsivėrus plyšiui. Aprašant plieno plaušo elgseną per irimo energiją, gerokai supaprastinami skaičiavimai, tačiau nagrinėtu atveju, esant mažam armatūros kiekiui, gaunamos reikšmingos, apytiksliai iki 62 % siekiančios įlinkio paklaidos.

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Padėka

Disertacijos autorius reiškia padėką moksliniams vadovams – KTU Statybos ir architektūros fakulteto docentui dr. Mindaugui Augoniui ir KTU Architektūros ir statybos instituto direktoriui dr. Raimondui Bliūdžiui – už pagalbą ir rekomendacijas rengiant šį mokslinį darbą.

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