2. Строительство. Предпроектная (предынвестиционная) документация. Состав, порядок разработки и утверждения: ТКП 45-1.02-298-2014 (02250). – Введ. 20.07.2014. – Минск : Минстройархитектуры, 2018. – 52 с.
3. Тарасевич, Е.И. Управление эксплуатацией недвижимости: учеб. / Е.И. Тарасевич. – Санкт-Петербург : Издательство «МКС», 2006. – 838 с.
4. Кочурко, А. Н. Экономическое обоснование вариантов проектов производства работ / А. Н. Кочурко, Л. Г. Срывкина // Вестник БрГТУ. – 2012. - № 1(73): Строительство и архитектура. – С. 142–146.
5. Экономика строительства. Курсовое и дипломное проектирование : учебное пособие / А. Н. Кочурко [и др.]. – Минск : Издательство Гревцова, 2012. – 396 с.

Материал поступил в редакцию 03.02.2019

KOCHURKO A. N., SRYVKINA L. G. The analysis of approaches to assessment of efficiency of alternatives of implementation of investment projects at the predesign stage

In this article we identify the problems of investment justification, which are not yet solved in current regulatory literature. We reason the consideration of alternative design solutions at the pre-project stage of the investment project implementation. We systemize the criteria for static and dynamic assessments of economic efficiency of investments.

UDC 624.012.46

Drahan A.V.

AN INNOVATIVE APPROACH TO CRACK WIDTH PREDICTION OF REINFORCED CONCRETE ELEMENTS

Introduction. Currently the CEN TC/250 is compliting the development of a new (second) generation of structural EuroCodes and among them pr EN1992. In accordance with the actual codes [3] serviceability limit States (SLS) for reinforced concrete structures are applied to ensure (for checking) their functionality and structural integrity under service loafing conditions. Cracking analysis (crack width estimation) constitutes a main step in the serviceability design of concrete structures, became cracks of excessive width contribute to corrosion of the reinforcement, surface degradation and consequently damage the structures durability. Moreover, cracks can to decrease axial and flexural stiffness of reinforced concrete member. It is recognized in prEN1992 [3] that crackling in reinforced concrete members may be of two forms (modes):

1. Cracking due to restraint provides by structure volume change (shrinkage, temperature, imposed strains);
2. Cracking due to applied loads. It should be pointed, that this paper only the cracking due to applied loads is discussed.

As it was shown in [1] to control the crack width at RC-members designer can use the guidelines prescribed in various design codes [2, 3, 2, 5], which are based on certain analytical solution or empirical equations to crack width assessment. Recently, the study on crack width control has been continued and numerous formulas of the crack width calculation are proposed. However, almost of them was developed based on regression analysis of experimental data. Detailed analysis shows that all approaches can be divided on the following groups:

1. Empirical (or full-empirical) approaches (ACI 224.2R-86 [5], Gergely P. and Lutz L. A. [6], Mulin N. M. [7], Gusha U. P. [8] etc.).
2. Fracture-Mechanics theory approaches (Piradov A. B., Gvelesiani L. O., Piradov K. A., Guzeev E. A. [9, 10], Oh B.H., Kang Y.-J. [11], Shah S. P., Swartz S. E. [12], etc.).
3. «Tension-Stiffening» theory approaches (CEB-fib Model Code 1990 [13], Pedziwiatr J. [14], SNB 5.03.01-02 [2], Murashev V. I. [15], Nemirovskij J. N. [16], etc.).
4. «Bond-slip» theory approach (Holmberg A [17], Farra B. [18], Noakowski P. [19], Alvares M. [20], an proposed method, etc.).

The crack width calculations are based on the basic case of a prismatic reinforced concrete bar subjected to tension, what is modeling the tensile zone of the RC-element. With regard to behavior under increasing tensile strain, four stages are distinguished in general case:

- the uncracked stage;
- the crack formation;
- the stabilized cracking stage;
- the steel yielding stage.

For carring out crack width calculations, it is necessary to know whether the crack formation stage or the stabilized cracking stage applies. It should be pointed, that the formulation given for the value of the crack width in general case provides an estimate of the surface crack width for members subjected to pure tension. For members, subjected to bending, the value represent the crack width at the level of reinforcement. In this case crack spacing and crack width will generally be larger of the extreme tensile fibre of the section. In order to estimate the value of crack width at the extreme tensile fibre, the crack width may be multiplied with factor $(h-x)/(d-x)$ in accordance with [17].

This study presents an analytical method to estimate the tensile and flexural crack width of reinforced concrete members based on the original conventional crack theory and bond-slip relation [17–20]. The validity, accuracy and efficiency of the proposed analytical method are established by comparing the results of the proposed model with experimental data as well as with results obtained from the analytical study.

The comparation between the proposed analytical solution and experimental data, the analytical solution of *fib* MC2010 [4] and project pr EN1992-1-1[3] was performed.

1. CODES PROVISION

1.1. *fib MC2010*

In accordance with *fib* MC2010 [4] requirements for all stages of cracking, the design crack width w_d may be calculated by:

$$w_d = 2l_{s,\max}\left(\varepsilon_{sm} - \varepsilon_{cm} - \varepsilon_{cs}\right), \qquad (1.1.1)$$

where:

$l_{s,\max}$ denoted the length over which slip between concrete and steel bars occurs. The steel and concrete strains, which occur within this length, contribute to the width of the crack. For the length $l_{s,\max}$ the following expression applies:

$$l_{s,\max} = k \cdot c + \frac{1}{4}\frac{f_{ctm}}{\tau_{bms}}\frac{\varnothing_s}{\rho_{s,ef}}, \qquad (1.1.2)$$

where:

k is an empirical parameter to take the influence of the concrete cover into consideration; as a simplifications, k = 1,0 can be assumed;

c is the concrete cover;

τ_{bms} is mean bond strength between steel and concrete (see Table 76-2 [4]).

In equation (1.1.1):

ε_{sm} is the average steel strain over the length $l_{s,\max}$;

ε_{cm} is the average concrete strain over the length $l_{s,\max}$;

ε_{cs} is the strain of the concrete due to free shrinkage.

***Драган Алексей Вячеславович,** к. т. н., зав. кафедрой архитектуры Брестского государственного технического университета. Беларусь, 224017, БрГТУ, г. Брест, ул. Московская, 267.*

The relative mean strain in eq (1.1.1) follows from:

$$\varepsilon_{sm}-\varepsilon_{cm}-\varepsilon_{cs}=\frac{\sigma_s-\beta\cdot\sigma_{sr}}{E_s}-\eta_r\cdot\varepsilon_{sh}\,, \qquad (1.1.3)$$

where:

σ_s is the steel stress in a crack;

σ_{sr} is the maximum steel stress in a crack in the crack formation stage, which, for pure tension, is:

$$\sigma_{sr}=\frac{f_{ctm}}{\rho_{s,ef}}\left(1+\alpha_e\rho_{s,ef}\right), \qquad (1.1.4)$$

where: $\rho_{s,ef}=\dfrac{A_s}{A_{c,eff}}$,

with $A_{c,eff}$ effective area of concrete in tension (Fig 7.6 – 4 [4])

1.2. prEN1992-1-1

In according to 9.2.8. prEN1992-1-1 the calculated surface crack ($w_{k,cal}$) may be determined from following expression:

$$w_{k,cal}=\left(\varepsilon_{sm}-\varepsilon_{cm}+\eta_r\cdot\varepsilon_{cs}\right)\cdot s_{r,\max,cal}\,, \qquad (1.2.1)$$

where:

$s_{r,\max,cal}$ is the calculated maximum crack spacing is stabilized or alternatively the maximum length along which there is slip between concrete and steel in phase of crack formation.

For elements subjected to direct loads (stabilized cracking) or subjected to imposed strains (crack formation phase), ($\varepsilon_{sm}-\varepsilon_{cm}$) may be calculated from expression:

$$\left(\varepsilon_{sm}-\varepsilon_{cm}\right)=\frac{\sigma_s-k_t\dfrac{f_{ct,\mathrm{ef}}}{\rho_{ef}}\left(1+\alpha_e\rho_{ef}\right)}{E_s}\geq 0{,}6\frac{\sigma_s}{E_s}\,, \qquad (1.2.2)$$

σ_s is the stress in the tension reinforcement closest to tensioned concrete surface assuming a crack section.

k_t is a coefficient dependent on the duration and nature of the load:

k_t = 0,6 for short term instaneous loading in crack formation stage;

k_t = 0,4 for long term, repeated loading and stabilized cracking.

2. Theoretical background of the proposed analytical model

As it was shown in [21] in general case the crack width calculation are based on the basic case of a prismatic (cylindrical) reinforced concrete bar (element), subjected to axial tension. The proposed analytical model for crack width estimation was developed based on theoretical and experimental studies of the axially loaded tensioned reinforced concrete elements.

The proposed analytical model is based on the following assumptions:

1. The «bond stress-slip» relation $\tau = f(\delta_s)$ between steel bars and surrounding concrete was adopted in accordance with *fib*MC1990;
2. The «stess-strain» relation $\sigma = \sigma(\varepsilon)$ for material, was adopted according to [3]. For concrete in tension this relation assumed as a linear approximation with ascending only accordance with [3].
3. Crack formation in any section observed when the tensile strain ε_{ct} exceeds the ultimate tensile strain ε_{ctu}.

In accordance with proposed model crack formation and development process can be presented as a follows. When ultimate tensile strains for concrete is reached, a crack will form and adjacent tensile done will no longer be acted on by direct tension force. The formation of this crack («conventional crack» in Figure 2.1) lead to a local redistribution of stresses within section. At the crack (or «conventional crack» at the ends of analyzed element, see Figure 2.1), all tensile force will be redistributed to the reinforcement and the stress in concrete immediately adjacent to the crack must clearly be zero. But by applying of the additional tensile force, causes further direct tension stresses to develop of the distance from the crack (see Figure 2.1, zone), tensile force is transferred by bond from reinforcement to concrete until, at the some distance from crack, the strain and stress distribution within section remains unchanged from that it was before crack formed (see Figure 2.1). This in turn causes further cracks to form and process continues until the distance, does not permit sufficient tensile stresses to develop and cause further cracking.

As further load is applied, the second crack will form at the next weakest section, though it will not form within of the first crack since the stresses within the region will have been reduced by formation of the first crack. Tensile stresses in the concrete surrounding reinforcement bars are caused by bond increase as the strain in the reinforcement decreases (see figure 2.1,b). These stresses increase with distance from the primary cracks and may eventually cause further cracks to form approximately mid-way between the primary («conventional») cracks.

Further increases in loading will lead to the formation of further cracks until, eventually, there is no remains area of member surface which is not within so of previously formed crack. After all the cracks have formed, further loading will result in widening of the existing cracks but no new cracks formation (so called, "the stabilized cracking stage"). Stresses in the concrete will be relived by limited "bond-slip" near the crack faces and by the formation of internal cracks. This process leads to further reduction of stiffness, but clearly, the stiffness cannot reduce to bellow that of the reinforcement bar.

In general case, for any stage of process, the crack width can be calculated from following expression based on "bond-slip" theory:

$$w_m=\int_{l_t-\frac{L_m}{2}}^{l_t}\left(\varepsilon_s(x)-\varepsilon_{ct}(x)\right)dx\,, \qquad (2.1)$$

where:

$\varepsilon_s(x)$, $\varepsilon_{ct}(x)$ are the steel and concrete strain distributions over the transfer length l_t, respectively (see Figure 3.1);

l_t – transfer zone length;

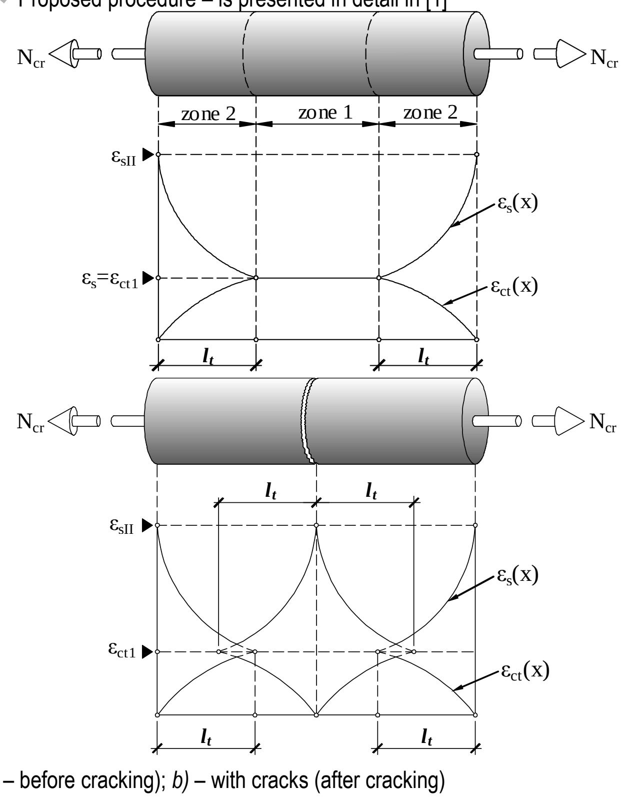
L_m – average length of the block associated with spacing between two adjacent cracks.

For obtaining of the basic model parameters or variables (l_t, $\varepsilon_s(x)$, $\varepsilon_{ct}(x)$), which are used for crack width calculation according to (2.1), an iterative procedure was proposed based on expressions (2.2) and (2.3), strain compatibility diagram (see Figure 3.1) and «bond stress-slip» relation according to [3].

$$\boldsymbol{\Delta}_{1\,i}=\sigma_{s\,i}-\sigma_{s\,i-1}-\boldsymbol{\Delta x}\cdot\left(\frac{\tau_{b\,i}+\tau_{b\,i-1}}{2}\right)\cdot\frac{4}{\varnothing_s}=0\,; \qquad (2.2)$$

$$\boldsymbol{\Delta}_{2\,i}=\sigma_{ct\,i-1}-\sigma_{ct\,i}-\boldsymbol{\Delta x}\cdot\left(\frac{\tau_{b\,i}+\tau_{b\,i-1}}{2}\right)\cdot\frac{4\cdot A_s}{\varnothing_s}\cdot\frac{1}{A_{ct,netto}}=0\,\cdot \qquad (2.3)$$

Proposed procedure – is presented in detail in [1]



a) – before cracking); *b)* – with cracks (after cracking)

Figure 2.1 – Schemes of strain distribution on length of an element with allocation of characteristic zones

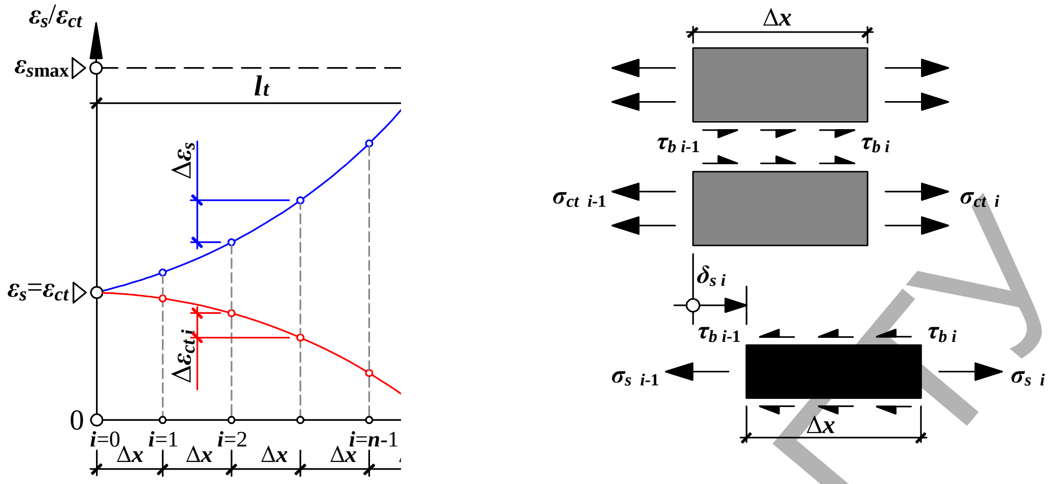
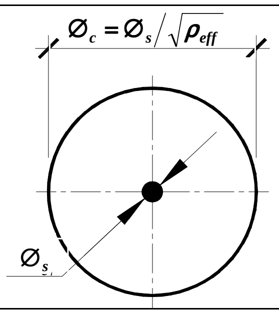


Figure 2.2 – Distribution of the strains (a) and equilibrium conditions (b) in proposed model

Table 3.1 – Values of the basic variables (in put data for numerical simulations)

Cross-section	Calculating parameters	
	$\varnothing_s$, mm	10; 12; 14; 16; 18; 20; 22; 25; 28; 32; 36; 40
	ρ_{eff}, %	0,25; 1,0; 2,0; 3,0 ;4,0
	f_{cm}, MPa	1,6; 1,9; 2,6; 3,2; 3,5; 4,1
	f_{sm}, MPa	240; 400; 500

3. Numerical investigations (studies) of the basic parameters of the proposed design model

Based on proposed an iterative procedure the numerical studies was performed for obtaining of the transfer zone length $lt = f(N)$, steel $\varepsilon_s(x)$ and concrete $\varepsilon_c(x)$ distributions for the different values of the basical variables (bar diameter ø, type of bar surface (plane, ribbed), reinforcement ratio ρ_{eff}, concrete (f_{cm}) and steel strength (f_{sm}) – see Table 3.1).

Based on the obtained results of numerical studies, which was presented in detail in [1], the following expression for transfer length l_t calculation was developed with usage of regression analysis:

$$l_t = k_p \frac{N_{ult}}{u\cdot(1+\rho_{eff}\alpha_E)}\sqrt{\frac{N}{N_{ult}}}, \tag{3.1}$$

where:

k_p is semi-empirical coefficient, characterizing steel-concrete bond conditions (mm²/N);

u perimeter of the steel bar;

N_{ult} ultimate tensile force for steel bar, kN;

ρ_{eff} the effective reinforcement ratio ($\rho_{eff} = A_s/A_{c,eff}$);

$A_{c,eff}$ the effective area of concrete in tension, which is calculated as follows:

$$A_{ct,eff} = \frac{E_s\cdot A_s\left(\varepsilon_{sII}-\varepsilon_{ct1}\right)}{f_{ctm}}, \tag{3.2}$$

where:

ε_{sII} strain of reinforcement in section with a crack (see Figure 2.2);

ε_{ct1} limiting extensibility of concrete (see Figure 2.2).

Steel reinforcement and concrete strain distribution over the transfer length l_t can be expressed as follows:

$$\varepsilon_s(x)=\varepsilon_{sII}\cdot\left[a\cdot\left(\frac{x}{l_t}\right)^{\frac{1+\alpha}{1-\alpha}}+b\right]; \tag{3.3}$$

$$\varepsilon_{ct}(x)=\varepsilon_{sII}\cdot\left[1-\left[a\cdot\left(\frac{x}{l_t}\right)^{\frac{1+\alpha}{1-\alpha}}+b\right]\right]\cdot\rho_{eff}\cdot\alpha_E. \tag{3.4}$$

Coefficient a and b in equations (3.3) and (3.4) are dimensionless coefficient characters characterizing a relation between stiffness characteristic of reinforcement and concrete cross section and calculated by following formulas:

$$a=\frac{1}{1+\rho_{eff}\cdot\alpha_E},\quad b=\frac{1}{1+\frac{1}{\rho_{eff}\cdot\alpha_E}}. \tag{3.5}$$

Taking into account Eq. (3.3) and (3.4), expression (3.1) for average crack width calculation can be rewritten as:

$$w_m = 0,6\cdot\varepsilon_{sII}\cdot l_t\cdot\left[1-\left(1-\frac{L_m}{2\cdot l_t}\right)^{\frac{2}{1-\alpha}}\right], \tag{3.6}$$

where:

ε_{sII} steel strain at the cracked section;

$l_t = f(N)$ transfer length, calculated by Eq.(3.1);

L_m average spacing between cracks;

α empirical coefficient, is equal 0,4.

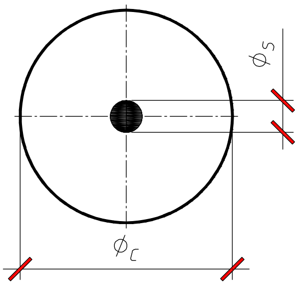
Based on the results of a numerical investigation the following formula for average block length L_m calculation (for the crack formation stabilized stage) was proposed:

$$L_m = l_{t1} = k_p\frac{N_{ult}}{\pi\cdot\varnothing_s\cdot(1+\rho_{eff}\cdot\alpha_E)}\cdot\sqrt{\frac{N_{crc1}}{N_{ult}}}, \tag{3.7}$$

Taking into account Eq.(3.6) and Eq.(3.7), expression for w_m finally can be rewritten is close form as:

$$w_m = 0{,}15\cdot k_p\cdot\varepsilon_{sII}\cdot\frac{\varnothing_s}{1+\rho_{eff}\cdot\alpha_E}\cdot\sqrt{\frac{\sigma_{sII}}{f_{yk}}}\cdot\left[1-\left(1-\frac{1}{2}\cdot\sqrt{\frac{f_{ctm}}{\sigma_{sII}\cdot\rho_{eff}}}\right)\right]. \tag{3.8}$$

Table 4.1 **–** Basic parameters of, the specimens

Speciment seria	Section	Steel bars		Concrete member size, $\varnothing_c$, mm	Material strength, MPa		
		$\varnothing_s$, mm	ρ_{eff}		f_{cm}	f_{ctm}	f_{ym}
1.		20	0,01	200±5	40,5	2,47	400
2.		25	0,015		40,5	2,47	400
3.		36	0,03		40,5	2,47	400

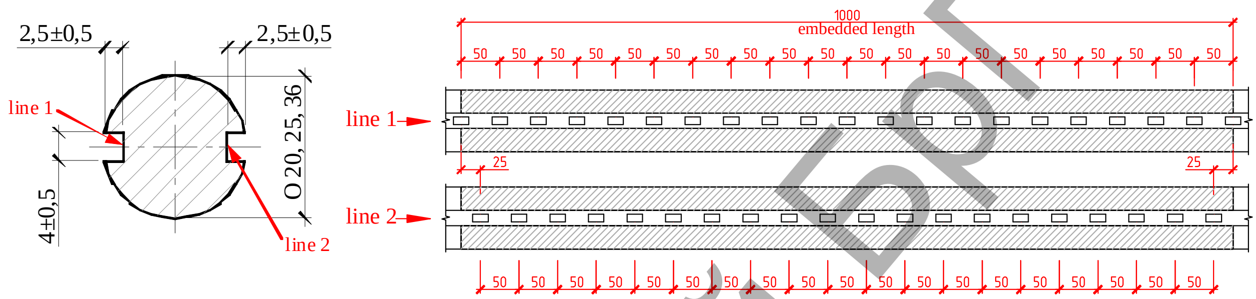


Figure 4.1 **–** Layout of strain-gages on the reinforcement bar

4. Verification of the proposed model. Comparison between proposed analytical solutions, codes propositions and experimental date

Experimental program

For verification of the proposed analytical model and comparison with new approaches in accordance with *fib*MC2010 [3] and pr EN1992-1-1 [4] special experimental studies was performed. Some series of the axially loaded reinforced concrete members was casted and tested. Short program of the experimental studies is listed in Table 4.1.

For measurement of the steel strain distribution over embedding length strain-gage method was applied. As a main measuring devices strain gages (with the basic length 5mm) have been used. Strain-gage were situated in the grooves on the lateral surface of reinforcement bars along the longitudinal edges of a profile, as it shown in Figure 4.1.

During the experimental the following parameters was registered:

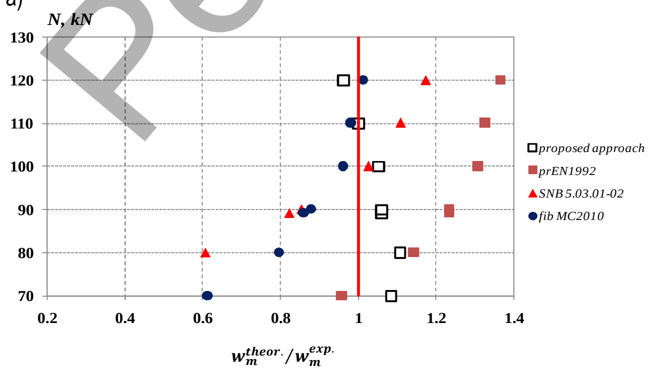
- steel strain distribution over the bar length with usage computer complex «TISSA-B-485/65»;
- crack width (opening) by microscope MP3 with the division price 0,02 mm.

5. Results and discussion

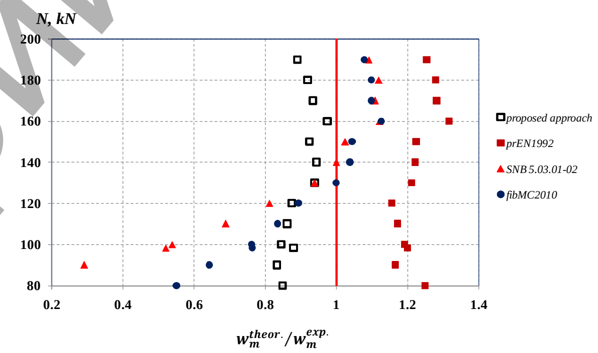
In Figures 5.1 and 5.2 shown experimental and theoretical (calculated) steel strain ($\varepsilon_s(x)$) distributions over the bar length obtained for stages before and after crack formation. It should be noted, that the strain distributions obtained by calculation in accordance with proposed model demonstrates good agreement with experimental distributions, for different values of input parameters.

Comparation of the average value of the crack width obtained by proposed model, *fib*MC2010 model [4] SNB 5.03.01-02[2] and prEN1992 [3] shown in Figure 5.3.

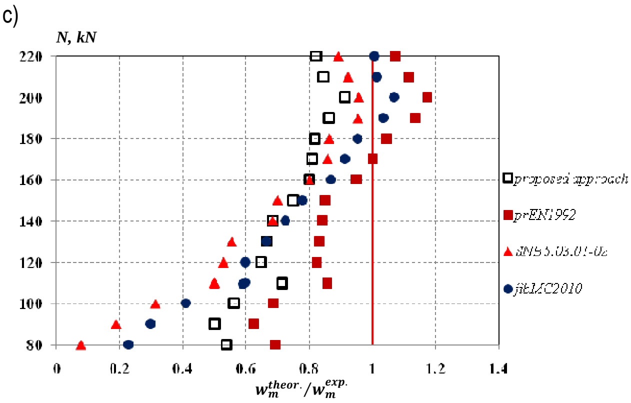
a)



b)



c)



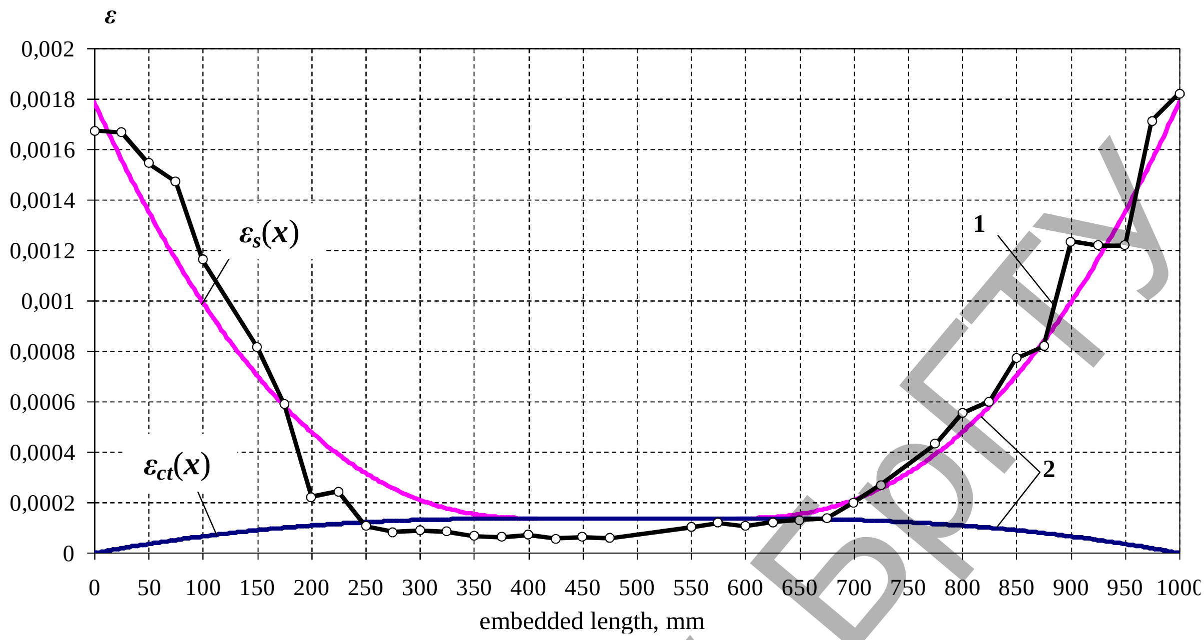
a) –1Ø20S400; *b)* – 1Ø25S400; *c)* – 1Ø36S400

Figure 5.3 **–** Comparison of average values of crack width

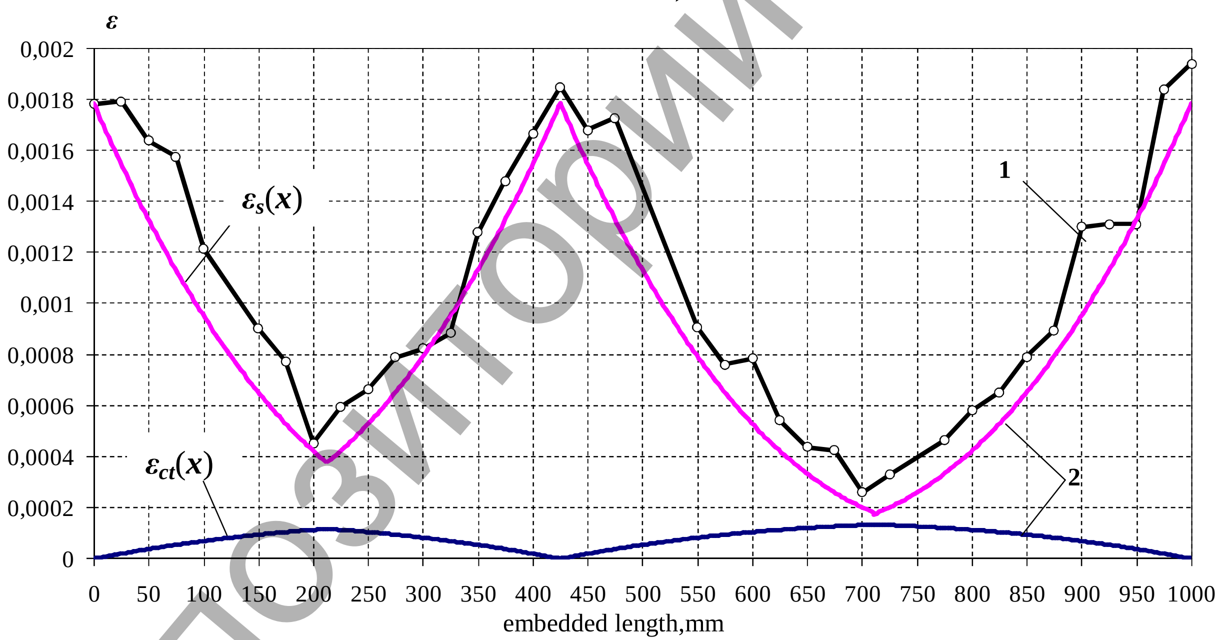
Conclusions

An innovative design method for crack width control is proposed. This method is based on modified «bond-slip» approach and allows to assess transfer length l_t in accordance with proposed Eq.(3.1). The verification of the proposed model shows good agreement with experimental data, obtained in the own studies.

a)

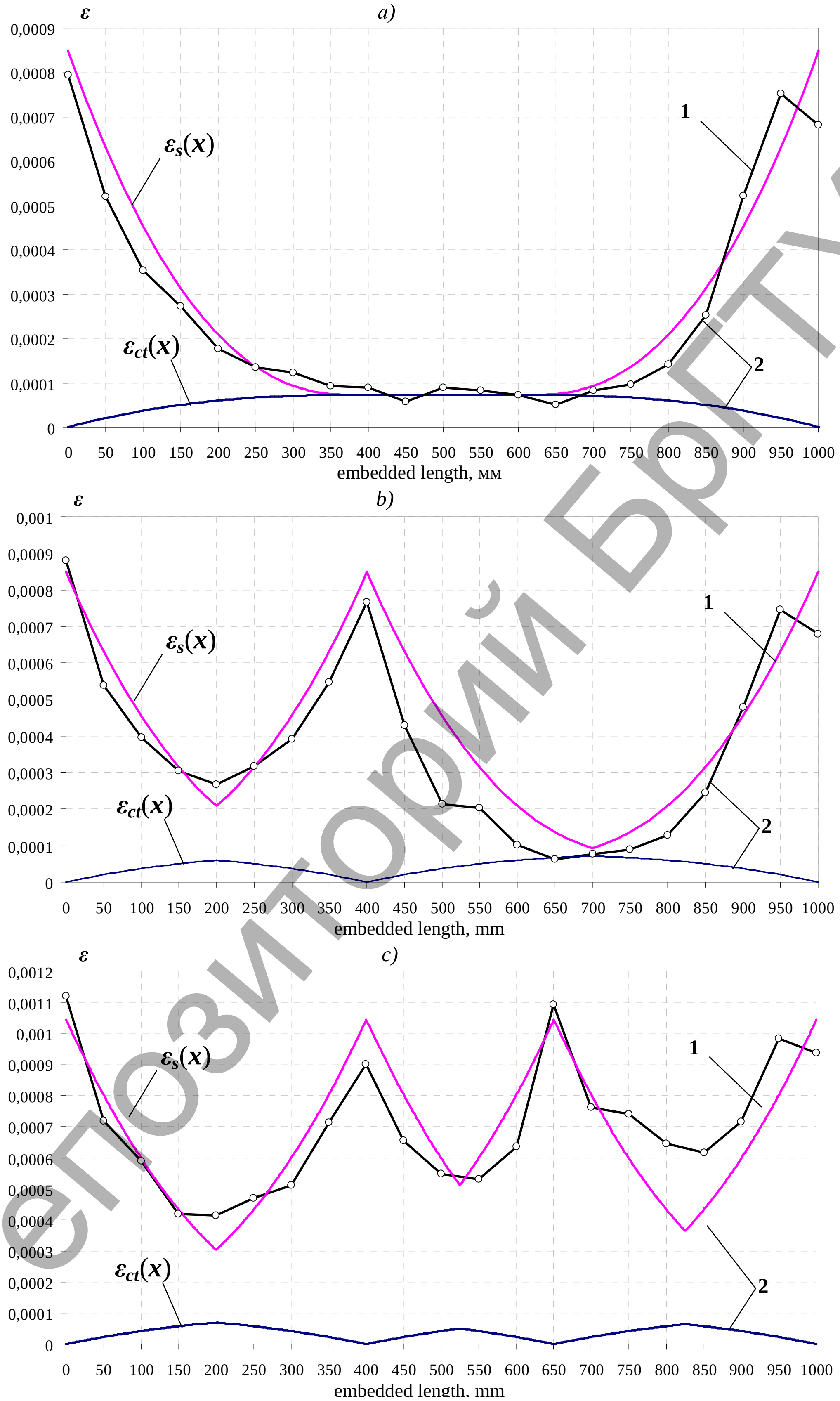


b)



a) – before cracking (*N*=100 kN); *b)* – after crack formation of (*N*=105 kN)

Figure 5.1 – Comparison of distributions of deformations, received experimentally (1) and analytically (2), for the experimental specimen with following characteristics – 1Ø20S400; ρ_{eff} = 0,01; f_{ctm}=2,7 N/mm^2



a) – before formation of crack (N=80kN); *b)* – after the first crack formation (N=80kN); *c)* – after the second crack formation (98,4kN)

Figure 5.2 – Comparison of distributions of deformations, received experimentally (1) and analytically (2), for the experimental specimen with following characteristics – 1Ø25S400; ρ_{eff} = 0,015; f_{ctm}=2,47 N/mm²

REFERENCES

1. TUR, V. V. To calculation of crack width in reinforced-concrete elements at axial tension forces / V. V. TUR, A. V. DRAHAN // Building science and technique. – № 1 (22):40-54, 2009.
2. Concrete and reinforced-concrete stuctures : SNB 5.03.01–02. – Minsk : Minstrojarchitectura, 2002.
3. prEN 1992-1-1:2018 Eurocode 2: Design of concrete structures – Part 1-1: General rules, rules for buildings, bridges and civil engineering structures.
4. fib ModelCode for Concrete Structures 2010, International Federation for Structural Concrete (fib), Lausanne, Switzerland, 2010;
5. American Concrete Institute Committee 224. Cracking of concrete members in direct tension. ACI 224.2R-86. ACI Journal., 84(1): 3–13, 1986.
6. GERGELY P., LUTZ L. A. Maximum crack width in reinforced flexural members. Causes, Mechanism and Control of Cracking in Concrete. American Concrete Institute SP-20: 87–117, 1968.
7. MULIN N. M. Reinforcement of reinforced-concrete members. Stroyizdat, 1974.
8. GUSCHA J. P. Crack width in elements of reinforced-concrete constructers / Limiting conditions of elements of reinforced-concrete constructers: 30–44, 1976.
9. PIRADOV A. B., GVELESIANI L. O. Width and length of crack opening in elements of concrete designs at long loading. Izvestije vuzov. 12:88-90, 1991
10. GUZEEV E. A. Mechanics of destruction of concrete: theory and practice questions . Publishing house BPI. 1999 – 215.
11. OH B. H., KANG Y-J. New formulas for maximum crack width and crack spacing in reinforced concrete flexural members . ACI Structural Journal, 85(2):103–112, 1987.
12. SHAH S. P. Fracture Mechanics of Concrete: Applications of Fracture Mechanics to Concrete, Rock and Other Quasi-Brittle Materials. John Wiley & Sons, 1995.
13. CEB-FIP Model Code 1978: Design Code. Comite–Euro–International du Beton (CEB Bulletin d'Information No.124/125.) ., London, 1978.
14. PEDZIWIATR J. Tension stiffening effect a przyczepnosc na przykladzie osiowo rozciaganych elementow zelbetowych. 54 konferencja naukowa: Problemy naukowo-badawcze budownictwa, pages 241-248: Krynica, 2008 г.
15. MURASHEV V. I. Reinforced-concrete structures. Publishing house of literature on building, architecture and building materials, 1962.
16. NEMIROVSKY J. N. Revision of some positions of the theory of crack opening Concrete and reinforced-concrete, 3: 13-16, 1970.
17. HOLMBERG A. Unbonded Portions Between Concrete Cracks. Danish Society for Structural Science and Engineering, 55(4): 113-117, 1984
18. FARRA B., JACCOUD J.-P. Bond behaviour, tension stiffening and crack prediction of high strength concrete. Proceedings of International Symposium «Bond in Concrete», Riga, 1992.
19. NOAKOWSKI P. Determination of crack width: Simple and close to reality method considerating tensile strength of concrete and steel bonding: Technology of Concrete. Supplement. – Gdansk, 2004.
20. ALVAREZ M. Einfluss des Verbundverhaltens auf das Verformungsvermögen von Stahlbeton. Institut für Baustatik und Konstruktion, Eidgenössische Technische Hochschule Zürich, 1998.

Материал поступил в редакцию 20.05.2019

DRAHAN A.V. An innovative approach to crack width prediction of reinforced concrete elements

In article is presented an innovative design method for crack width control. This method is based on modified «bond-slip» approach and allows to assess transfer length l_t and crack width w_m. The verification of the proposed model shows good agreement with experimental data, obtained in the own studies.

УДК 624.012

Тарасевич А. Н., Дедок В. Н.

ПРОЕКТИРОВАНИЕ ФУНДАМЕНТОВ НА ЕСТЕСТВЕННОМ ОСНОВАНИИ (РАСПРЕДЕЛЯЮЩИХ) СОГЛАСНО ЕВРОКОДУ 7

Введение. В 1975 году в Европейском сообществе (ЕС) было принято решение о разработке новой единой европейской системы норм и правил строительного проектирования – Еврокодов, которые на первом этапе были альтернативными нормами, а затем заменили национальные нормы. При этом вводились соответствующие Национальные приложения, учитывающие особенности проектирования в каждой стране. Национальные приложения содержат, главным образом, только те положения, которые отнесены к т. н. национальноустанавливаемым параметрам (NDP). Целью этой программы было устранение технических препятствий в международном сотрудничестве, создание единого нормативного поля ЕС для работы проектных и строительных фирм.

Национальная адаптация Еврокодов предусматривала публикацию полного текста с титульным листом Госстандарта РБ, с национальным введением и приложением, в котором перечислены параметры, изменяемые Национальным приложением (NDP) применительно к геотехническому проектированию. В РБ были изданы нормативные документы: СТБ ISO 14688-1-2009, СТБ ISO 14688-2-2009, ТКП EN 1997-1-2009, ТКП EN 1997-2-2009. Этими изданиями и закончилась гармонизация национальных норм с Еврокодом 7 при этом, в ТКП 45-5.01-254-2012 нет ссылок на вышеуказанные документы. В статье рассматриваются структура, содержание и основные подходы, принятые в Еврокоде 7 по проектированию фундаментов на естественном основании, и приведены результаты расчета оснований по разным подходам, рекомендованным ТКП EN 1997-1.

Геотехническое проектирование предусматривает определение физико-механических характеристик и расчетного сопротивления грунтов основания как материала. Величины характеристик грунтов являются определяющими при расчете оснований и фундаментов. Еврокод 7 состоит из двух частей: EN 1997 – 1. Общие положения и EN 1997 – 2. Исследования и испытания грунтов. Первая часть состоит из 12 разделов и 9 приложений, вторая – из 6 разделов и 24 приложений.

Положения Еврокода 7, как и всех других конструкционных Еврокодов, подразделяются на принципы (P) и правила. Принципы – это безальтернативные требования, которые должны быть выполнены в проекте (напр. осадка меньше допустимой), правила – это набор

Тарасевич Алексей Николаевич, *к. т. н., декан инженерно-экономического факультета заочного образования Брестского государственного технического университета.*
Дедок Владимир Николаевич, *ст. преподаватель кафедры геотехники и транспортных коммуникаций Брестского государственного технического университета.*
Беларусь, 224017, БрГТУ, г. Брест, ул. Московская, 267.