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METHODICAL BASES OF CALCULATION OF STRENGTH THE NORMAL CROSS SECTION OF RAINFORCED CONCRETE BEAMS WITH CONCRETE UPPER BELT AND EXTERNAL REINFORCEMENT

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Abstract. The proposed methods allows to make calculation of the strength the normal cross-section of reinforced concrete beams with concrete upper belt and external (outside imposed)reinforcement (RCBER) that depends on the stress-strain mode at the time of the destruction of its components (concrete and structural coerced steel profile).

Keywords: steel, concrete, beam, normal, cross-section, stress, strain, deformation, strength.

Introduction

Leading national scientists Y. G. Ametov, A. M. Bambura, O. V. Semko, Y. S. Slyusarenko, L. I. Storozhenko, V. G. Tarasyuk are author of existing regulatory documents (DBN 2010), recognize the necessarily of further editorial work on State construction norm “Steel reinforced concrete construction” (DBN 2010) in their researches (Ametov *et al.* 2008).

One of the directions of norms improvement (DBN 2010) is to develop specific practical methods of calculation and design of steel reinforced concrete construction with its main conditions and certain points of the (Evrokod 4. 2007), which used in the EU.

In this work (Muravlov 2012) author substantiated the effectiveness of concrete and composite structures with rendered reinforcement, make its general classification and analyzes the advantages and disadvantages of design decisions steel reinforced beams with external reinforcement, that were studied at research work (Ivanyuk 2012), (Krupchenko 2008) and (Kuch 2012).

Previously A. P. Vasilev and V. M. Golosov (Vasilev, Golosov 1981) made general analyze of present mode and prospects for effective implementation structures with external reinforcement. Results of analyze showed, for example, arrangement in buildings with steel or concrete frame of monolithic slabs with profiled steel decking allow comparison with monolithic reinforced concrete floor on steel beams, reduce labor costs by 1.5 times at the same cost of 1 m^2 of flooring and concrete consumption by 35 %.

Reducing of material capacity of bearing composite structures is achieved by using of high-efficient materials (concrete and structural steel), whose resistance is used in full; creation of new structural forms of cross-sections through the rational combination of rolled sections and concrete. Design of these structures is inhibited lack of optimal methods of calculation, the essence of which is to determine the minimum cross-section of reinforcement and structural steel, cross-section dimensions and reinforcement methods of complex structural elements.

Existing methods for calculating steel reinforced concrete (SRC) structures (elements) that bent, are based on the calculation of the threshold voltage using rectangular stress diagram for both materials (Evrokod 4. 2007; Rukovodstvo… 1979). The suggested in researches (Ametov *et al.* 2008; Bambura, Ametov 2004.) new concept of calculating, introducing the practice of the method of boundary deformations, that allowed to get closer to the actual stress-strain mode (SSM) of SRC structures (elements). At the same time suggested in researches (Ametov, *et al.* 2008; Bambura, Ametov 2004) and norms (DBN 2010) calculated provisions do not include: general deformation model of element, its cross-sectional design, the nature and strength of connections between the concrete and constructive reinforcement, the impact of efforts shift. For the last two years, methods of composite elements have undergone further development on the basis of the calculated deformation model that takes into account the real diagrams of concrete and rebar.

So in research (KushnIr *et al.* 2012; KushnIr *et al.* 2012; OvsIy *et al.* 2012) there are methods of calculation of steel concrete beams of solid rectangular cross section and double-T-section with a concrete top shelf for determining the strength of normal section based SSM components thereof at the time of destruction.

At the same time, in sphere of development level, methods of calculation of RCBER are lag behind methods of reinforced concrete elements, introducing in practice calculations using deformation models of concrete. For increasing the efficiency and proliferation RCBER is necessary to improve the theory and methods of calculation. The foregoing has identified the relevance of the research topic and great economic importance.

The overall aim of the research

Is develop methods of calculation of the strength the normal cross section of reinforced concrete beams with concrete upper belt and external (outside imposed) reinforcement (RCBER) that depends on the stress-strain mode at the time of the destruction of its components (concrete and structural coerced steel profile). Achieving the goal carried out by solving these tasks that consisted in:

– substantiation conditions of application deformation model and marginal criteria calculation;

– developing a method of calculating the optimum cross-section a constructive steel double-T profile (CSDP), which reinforced normal section RCBER;

– obtaining problem solving strength of normal cross-section RCBER, bent, depending on its SSM components: concrete and structural steel profiles at the time of the destruction.

Rationale of the obtained results

Methodical bases the calculation of the strength the normal cross section of reinforced concrete beams with concrete upper belt and external (outside imposed) reinforcement (RCBER) were developed according to estimated deformation model using basic scientific practical provisions of works the follow scientists: Y. M. Babych (Babich, E. *et al.* 2005; Babich, E. *et al.*1999), A. Y. Barashykov (Barashikov, ZadorozhnIkova 2005; Barashikov 2005), A. M. Bambura (Bambura, Ametov 2004; Bambura 2006), V.P. Mytrofanov (Mitrofanov *et al.* 2008; Mitrofanov 2004), D. V. Kochkarov, V. I. Babych (Kochkarov, Babich, V. 2012), James J. Mac Gregor, James K. White (Wight *et al.* 2011) and certain provisions of existing national and international standards (DBN 2010; DBN 2009; ACI Innovation…) and include the solution of two problems: the selection of structural steel double-T-section profile (CSDP) that depending on the height of the concrete belt RCBER, which is a direct task for optimization design, checking the strength of normal section RCBER.

1. Tasks of checking the strength and selection of section CSDP for normal section bendable RCBER, based on following points:

– task selection of the optimal section A_a CSDP, which is a supporting element the normal section RCBER, solved according to criterion:

$$A(\varepsilon_{cu};\varepsilon_{au}) = A_a = \min, \qquad (1)$$

where: $A_a = 2 \cdot A_f + A_w$ – sectional area of RCBER, which consists of the sum of squares of its shelves and ribs; ε_{cu} – limit relative deformation ratio in the extreme top fiber concrete compressed zone of normal concrete sectional shelves RCBER, are assumed to be equal $\varepsilon_{cu} = 0{,}0035$ ($f_{cd} = 8\ldots60$ MPa) or according to the table 1 (KushnIr *et al.* 2012; KushnIr *et al.* 2012; OvsIy *et al.* 2012, table.1.1) or table 3.1 (DBN 2009); ε_{au} – marginal relative tensile strain in the extreme lower fiber stretched zone KSDP, which is part of the normal section bearing RCBER, the values of which are taken under the provisions of 6.3.3 (DBN 2010).

Table 1. Average value of ε_{cu} for compressed zone of concrete rectangular shape

f_{cd} , MPa	12	16	20	25	30	35	40	45	50
$\varepsilon_{cu} \times 10^{-3}$	3,8	3,7	3,6	3,5	3,4	3,3	3,2	3,1	3,0

– task of checking the strength of a normal rectangular steel concrete beams based on the criteria:

$$M(\varepsilon_{cu};\varepsilon_a > \varepsilon_{au}) = \max;\ M(\varepsilon_{cu};\varepsilon_{au}) = \max;$$
$$M(\varepsilon_{cu};\varepsilon_a < \varepsilon_{au}) = \max, \qquad (2)$$

where: M – the maximum value of the bending moment, which can absorb normal cross-section steel concrete beams with concrete top belt and the outside (rendered) reinforcement; ε_a – relative deformation in the lowest fiber stretched zones CSDP.

2. To solve tasks have been marked the following prerequisites calculation:

– on the marginal distribution of the relative phase of deformation strain on the height of the normal section RCBER carried out by the linear dependence (3)and (4) that proved the hypothesis of flat sections:

provided $\varepsilon_a = \varepsilon_{au}$ $\quad (\varepsilon_{cu} + \varepsilon_{au})/h = \varepsilon_{cu}/Y_B, \qquad (3)$

provided $\varepsilon_a > \varepsilon_{au}\,;\varepsilon_a < \varepsilon_{au}$ $\quad (\varepsilon_{cu} + \varepsilon_a)/\mathrm{h} = \varepsilon_{cu}/Y_B, \qquad (4)$

where; h, Y_B – respectively height of RCBER and distance between top fiber concrete shelf to the horizontal zero line of it normal section;

There are ties in the form of rigid vertical or inclined rods between the concrete top shelf CSDP. It's provide compatible (simultaneously) its work in the normal section RCBER, resulting in the apparent maximum composite properties of their components. It means that relative deformation of concrete and CSDP height cross section normal to change the law of plane sections;

– normal concentrated force (F_C) compressed in the area of concrete section top shelf RCBER. The mode RCBER in moment of destruction can be described by diagram "stress – relative deformative" ("$\sigma_C - \varepsilon_C$") (img. 1), determined by dependencies (5), which was suggested by scientists (Wight *et al.* 2011; ACI Innovation…; Kaar *et al.* 1978), and (6):

$$F_C = 0{,}85 \cdot f_{Cd} \cdot \beta_1 \cdot Y_B \cdot b, \quad \text{at} \quad \beta_1 \cdot Y_B \le T_f, \qquad (5)$$

$$F_C = 0{,}85 \cdot f_{Cd} \cdot T_f \cdot b, \quad \text{at} \quad \beta_1 \cdot Y_B > T_f, \qquad (6)$$

where: f_{Cd} – design value of the compressive strength of concrete; b – top shelf section width of RCBER; β_1 – given rate of compressed zone of concrete in its height, which was suggested as result of experimental research of James J. Mac Gregor and James K. White in 1997 (Wight *et al.* 2011) and defined by dependences, that showed in table 2. formed according to data of scientific work (Kaar *et al.*1978) and norms (ACI Innovation…):

– distance between the most compressed concrete fiber top shelf in the normal section RCBER and its axis to focus the efforts of compression (F_C,)determined by dependencies (7). This model was suggested by O. F. Ilyuin in his research (Ilyin 1980, (3)), and (8):

$$Y_B - z_1 = \left[0{,}5 - \frac{2381 \cdot f_{cd}^{0{,}69}}{E_a} \right] \cdot \beta_1 \cdot Y_B, \quad \text{at} \quad \beta_1 \cdot Y_B \le T_f \ ,(7)$$

$$\frac{T_f}{2}, \quad \text{at} \quad \beta_1 \cdot Y_B > T_f \,. \qquad (8)$$

– to calculate the strength of normal section RCBER was used simplified diagram "stress – relative deformative" ("$\sigma_a - \varepsilon_a$") of structural steel (img. 2) according to recommendations point 6.3.2 (DBN 2010). The value of marginal deformations of structural steel is determined in accordance with the recommendations point 6.3.3 (DBN 2010) based on dependences:

$$\varepsilon_{au} = \frac{16{,}5 \cdot f_y}{E_a}, \qquad (9)$$

where: f_y – characteristic impedance of reinforcing steel in liquid limit; E_a – modulus of elasticity of structural steel.

– calculation of the strength and optimal space CSDP of normal section RCBER based on the estimated deformation model using criteria occurrence limit state, that showed in position points 5.6.1.1, 5.6.1.6 and 5.6.2.3 of state construction norms (DBN 2010), and diagram of condition material (fig. 1 & fig. 2). The main criterion for the appearance of the boundary condition in the normal section RCBER is an extreme test achievement deformations of compressed concrete limit values ε_{cu}, during which carrying capacity is the maximum (M_{max}) (fig. 3);

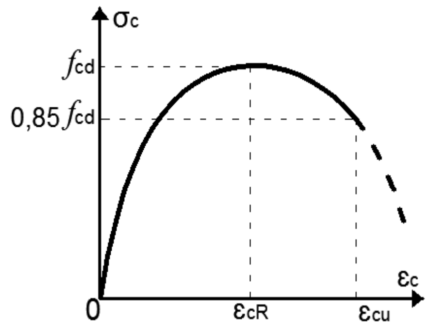

Fig. 1. Diagram of compression concrete $\sigma_C - \varepsilon_C$

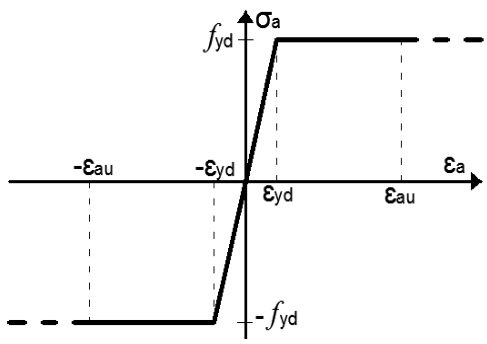

Fig. 2. Diagram compression-tensile structural steel $\sigma_a - \varepsilon_a$

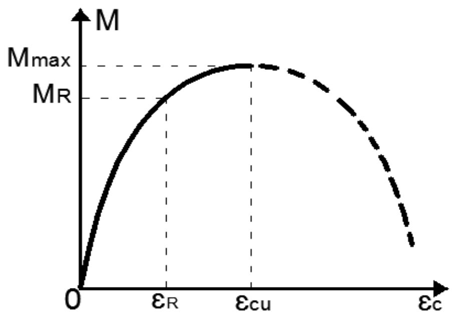

Fig. 3. Diagram of mode RCBER that transformed from diagram $\sigma_C - \varepsilon_C$

Table 2. The coefficient of reduction the compressed zone of concrete β_1 to the height Y_B according to calculated value of compressive strength of concrete f_{Cd}

Sources	a		б		в	
	Limits of variance f_{Cd}	β_1	Limits of variance f_{Cd}	β_1	Limits of variance f_{Cd}	β_1
[*]	f_{Cd} ≤28 MPa	0.85	28 MPa< f_{Cd} < 56 MPa	0.85 – 0.05 × (f_{Cd} –28 MPa)/ 7 MPa	f_{Cd} ≤56 MPa	0.65
[**]	f_{Cd} ≤56 MPa	0.85	56 MPa< f_{Cd} < 126 MPa	0.97 – 0.015× f_{Cd} /7 MPa	f_{Cd} ≤126 MPa	0.7

– during the comparison the sections of steel concrete beams with concrete and steel top shelf bearing element having a section in the form of beams (a), square(б) round (в) tube, located at different distances from the shelf and were obtained characteristic lines (Fig. 5). All of this show the dependence of their individual values. Typological analysis of different options reinforcing proper section RCBER and of strength values shown that all cross-sections can lead to reduced one (Fig. 4).

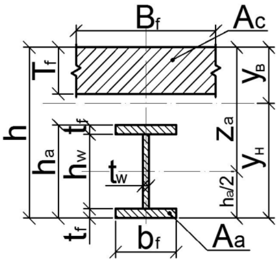

Fig. 4. Total given cross section is RCBER

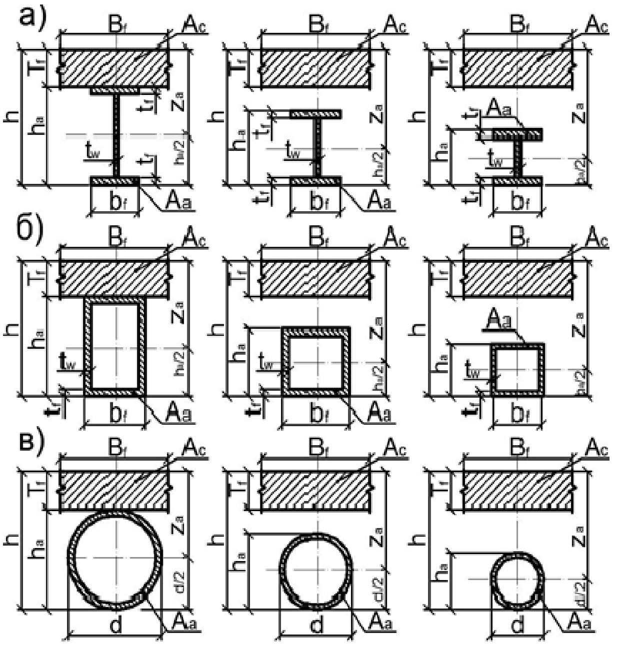

Fig. 5. Characteristic ranks of sections of steel concrete beams with top shelf and brought out concrete reinforcement (RCBER)

Than calculated depending between analytical problem solving test of strength and recruitment section CSDP (hard reinforcement) of normal section RCBER, which were obtained by numerical solutions.

3. Solution of task: *selection of required section CSDP* (A_a), which reinforced normal section RCBER. The main aims of task is definition of optimal section CSDP of RCBER, during the strain in the extreme of upper(concrete) and bottom (a steel) fiber of the normal section while reaching under the limit values ε_{cu} and ε_{au}.

The optimal cross-sectional area CSDP (A_a) of RCBER proposed to define the following dependencies:

$$A_a = A_b \cdot \mu_{onm};$$

$$\mu_{onm} = \frac{2 \cdot \Delta_z \cdot \Delta_\varepsilon (1 + \Delta_h) - \Delta_\varepsilon - 1}{\alpha_a \cdot [2 \cdot \Delta_Z \cdot (1 + \Delta_h) - \Delta_h \cdot (1 + \Delta_\varepsilon)]}, \quad (10)$$

where: $\alpha_a = E_a / E_b$ – ratio of elastic modulus structural steel and concrete; $\Delta_\varepsilon = \varepsilon_{cu} / \varepsilon_{au}$; – ratio values relative marginal deformations of concrete (ε_{cu}) and CSDP(ε_{au}); $\Delta_h = h_a / T_f$; – ratio of height values CSDP (h_a) to the height of the shelf RCBER (T_f); $\Delta_Z = \dfrac{Z_a}{T_f + \dfrac{h_a}{2}}$ – ratio values reduced the normal elevation section of RCBER; $\mu_{onm} = A_a / A_b$ – optimal coefficient constructive reinforcement steel I-profile (CSDP) of normal section RCBER; $A_b = T_f \cdot B_f$ – sectional area of the upper concrete shelf RCBER.

As a result of the transformation from dependence (10) and received the dependence on relative values Δ_ε and Δ_h:

$$\Delta_\varepsilon = \frac{1 + \alpha_a \cdot \mu [2\Delta_Z (1 + \Delta_h) - \Delta_h]}{\alpha_a \mu \Delta_h + 2\Delta_Z (1 + \Delta_h) - 1}, \quad (11)$$

$$\Delta_h = \frac{\alpha_a \mu \cdot (2\Delta_Z - \Delta_\varepsilon - 1) - 2\Delta_\varepsilon \Delta_Z}{2\Delta_\varepsilon \Delta_Z - 2\alpha_a \mu \Delta_Z - \Delta_\varepsilon - 1}. \quad (12)$$

Coordinates neutral line height section (h) RCBER get the possibility to definite according to dependences with $h = Y_B \cdot Y_H$:

$$Y_B = h \cdot \left[\frac{\frac{1}{2} + \alpha_a \mu \Delta_Z \cdot \left(1 + \frac{\Delta_h}{2}\right)}{(1 + \alpha_a \mu) \cdot \left(\Delta_Z + \frac{\Delta_Z \Delta_h}{2} + \frac{\Delta_h}{2}\right)} \right], \quad (13)$$

$$Y_H = h \cdot \left[\frac{\Delta_Z \cdot (\Delta_h + 2) + (1 + \alpha_a \mu) \Delta_h - 1}{\left(2(1 + \alpha_a \mu) \cdot \left(\Delta_Z + \frac{\Delta_Z \Delta_h}{2} + \frac{\Delta_h}{2}\right)\right)} \right]. \quad (14)$$

As a result, the calculations were obtained numerical relationship between the dimensionless coefficients of correlations Δ_ε, Δ_h, Δ_Z and composition $\alpha_a \mu_{onm}$. So value of product $\alpha_a \mu_{onm}$ according to value coefficient correlations between Δ_h and Δ_ε for normal section RCBER with coefficient $\Delta_Z = 1$ shown in table 3, and with coefficients $\Delta_Z = 5$ and $\Delta_Z = 10$ – showen in table 4 and table 5.

Table 3. Value of product $\alpha_a \mu_{opt}$, that depends on the coefficient correlations between Δ_h & Δ_ε with a coefficient $\Delta_z = 1$

$\Delta_h \Delta_\varepsilon$	1	2	3	4	5	6	7	8	9	10
0.05	-	-	-	-	-	-	-	-	-	0.0043
0.06	-	-	-	-	-	-	-	0.0021	0.0134	0.0228
0.07	-	-	-	-	-	-	0.0059	0.0201	0.0318	0.0416
0.08	-	-	-	-	-	0.0053	0.0237	0.0385	0.0506	0.0607
0.09	-	-	-	-	-	0.0228	0.0418	0.0571	0.0697	0.0802
0.10	-	-	-	-	0.0154	0.0405	0.0602	0.0761	0.0891	0.100
0.11	-	-	-	-	0.0326	0.0586	0.0790	0.0954	0.1089	0.1202
0.12	-	-	-	0.0145	0.0500	0.0769	0.0980	0.1150	0.1290	0.1407
0.13	-	-	-	0.0310	0.0677	0.0956	0.1174	0.1350	0.1495	0.1617
0.14	-	-	-	0.0478	0.0857	0.1145	0.1372	0.1554	0.1704	0.1830
0.15	-	-	0.0110	0.0648	0.1040	0.1338	0.1572	0.1761	0.1917	0.2048
0.16	-	-	0.0265	0.0821	0.1226	0.1534	0.1777	0.1972	0.2134	0.2269
0.17	-	-	0.0423	0.0996	0.1415	0.1734	0.1985	0.2188	0.2355	0.2495
0.18	-	-	0.0583	0.1174	0.1607	0.1936	0.2196	0.2407	0.2580	0.2725
0.19	-	-	0.0745	0.1355	0.1802	0.2143	0.2412	0.2630	0.2809	0.2960
0.20	-	0.0001	0.0909	0.1538	0.2000	0.2353	0.2632	0.2857	0.3043	0.3200
0.21	-	0.0140	0.1076	0.1725	0.2202	0.2567	0.2855	0.3089	0.3282	0.3444
0.22	-	0.0281	0.1244	0.1914	0.2407	0.2784	0.3083	0.3325	0.3525	0.3694
0.23	-	0.0424	0.1415	0.2106	0.2615	0.3006	0.3315	0.3566	0.3774	0.3948
0.24	-	0.0568	0.1589	0.2302	0.2828	0.3232	0.3552	0.3812	0.4027	0.4208
0.25	-	0.0714	0.1765	0.2500	0.3043	0.3462	0.3793	0.4063	0.4286	0.4474
0.26	-	0.0862	0.1943	0.2702	0.3263	0.3696	0.4039	0.4318	0.4550	0.4745
0.27	-	0.1012	0.2124	0.2907	0.3487	0.3934	0.4290	0.4579	0.4819	0.5022
0.28	-	0.1163	0.2308	0.3115	0.3714	0.4177	0.4545	0.4845	0.5094	0.5304
0.29	-	0.1316	0.2494	0.3326	0.3946	0.4425	0.4806	0.5117	0.5375	0.5593
0.30	-	0.1471	0.2683	0.3542	0.4182	0.4677	0.5072	0.5395	0.5663	0.5889
0.31	-	0.1627	0.2875	0.3761	0.4422	0.4935	0.5344	0.5678	0.5956	0.6191

Table 4. Value of product $\alpha_a \mu_{opt}$, that depends on the coefficient correlations between Δ_h & Δ_ε with a coefficient $\Delta_z = 5$

$\Delta_h \Delta_\varepsilon$	1	2	3	4	5	6	7	8	9	10
0.05	-	0.0161	0.0258	0.0317	0.0356	0.0385	0.0406	0.0423	0.0436	0.0447
0.06	0.0074	0.0265	0.0364	0.0424	0.0464	0.0493	0.0515	0.0532	0.0546	0.0557
0.07	0.0174	0.0370	0.0470	0.0531	0.0573	0.0602	0.0625	0.0642	0.0656	0.0668
0.08	0.0275	0.0474	0.0577	0.0639	0.0681	0.0712	0.0734	0.0752	0.0767	0.0778
0.09	0.0375	0.0579	0.0683	0.0747	0.0790	0.0821	0.0844	0.0862	0.0877	0.0889
0.10	0.0476	0.0683	0.0790	0.0855	0.0899	0.0931	0.0954	0.0973	0.0988	0.1000
0.11	0.0577	0.0788	0.0897	0.0964	0.1008	0.1040	0.1065	0.1084	0.1099	0.1111
0.12	0.0678	0.0893	0.1004	0.1072	0.1118	0.1150	0.1175	0.1194	0.1210	0.1223
0.13	0.0779	0.0999	0.1112	0.1181	0.1227	0.1261	0.1286	0.1306	0.1321	0.1334
0.14	0.0880	0.1104	0.1219	0.1290	0.1337	0.1371	0.1397	0.1417	0.1433	0.1446
0.15	0.0981	0.1209	0.1327	0.1399	0.1447	0.1482	0.1508	0.1528	0.1545	0.1558
0.16	0.1083	0.1315	0.1435	0.1508	0.1557	0.1593	0.1619	0.1640	0.1657	0.1671
0.17	0.1184	0.1421	0.1543	0.1617	0.1668	0.1704	0.1731	0.1752	0.1769	0.1783
0.18	0.1286	0.1527	0.1651	0.1727	0.1778	0.1815	0.1843	0.1864	0.1882	0.1896
0.19	0.1388	0.1633	0.1760	0.1837	0.1889	0.1927	0.1955	0.1977	0.1995	0.2009
0.20	0.1489	0.1739	0.1868	0.1947	0.2000	0.2038	0.2067	0.2090	0.2108	0.2122
0.21	0.1591	0.1846	0.1977	0.2057	0.2111	0.2150	0.2180	0.2202	0.2221	0.2236
0.22	0.1693	0.1952	0.2086	0.2168	0.2223	0.2262	0.2292	0.2316	0.2334	0.2350
0.23	0.1795	0.2059	0.2195	0.2278	0.2334	0.2375	0.2405	0.2429	0.2448	0.2464
0.24	0.1898	0.2166	0.2304	0.2389	0.2446	0.2487	0.2518	0.2542	0.2562	0.2578
0.25	0.2000	0.2273	0.2414	0.2500	0.2558	0.2600	0.2632	0.2656	0.2676	0.2692
0.26	0.2102	0.2380	0.2523	0.2611	0.2670	0.2713	0.2745	0.2770	0.2790	0.2807
0.27	0.2205	0.2487	0.2633	0.2723	0.2783	0.2826	0.2859	0.2885	0.2905	0.2922
0.28	0.2308	0.2595	0.2743	0.2834	0.2896	0.2940	0.2973	0.2999	0.3020	0.3037
0.29	0.2410	0.2702	0.2854	0.2946	0.3008	0.3053	0.3087	0.3114	0.3135	0.3152
0.30	0.2513	0.2810	0.2964	0.3058	0.3121	0.3167	0.3202	0.3229	0.3250	0.3268
0.31	0.2616	0.2918	0.3075	0.3170	0.3235	0.3281	0.3316	0.3344	0.3366	0.3384

Table 5. Value Value of product $\alpha_a \mu_{opt}$, that depends on the coefficient correlations between Δ_h & Δ_ε with a coefficient $\Delta_z = 10$

$\Delta_h \Delta_\varepsilon$	1	2	3	4	5	6	7	8	9	10
0.05	0.0244	0.0337	0.0384	0.0412	0.0431	0.0445	0.0445	0.0463	0.0470	0.0475
0.06	0.0344	0.0439	0.0487	0.0516	0.0535	0.0549	0.0560	0.0568	0.0574	0.0580
0.07	0.0444	0.0541	0.0590	0.0620	0.0639	0.0654	0.0664	0.0673	0.0679	0.0685
0.08	0.0545	0.0643	0.0693	0.0723	0.0743	0.0758	0.0769	0.0777	0.0784	0.0790
0.09	0.0645	0.0745	0.0796	0.0827	0.0848	0.0862	0.0874	0.0882	0.0889	0.0895
0.10	0.0746	0.0848	0.0900	0.0931	0.0952	0.0967	0.0978	0.0987	0.0994	0.1000
0.11	0.0846	0.0950	0.1003	0.1035	0.1056	0.1072	0.1083	0.1092	0.1099	0.1105
0.12	0.0947	0.1053	0.1106	0.1139	0.1161	0.1176	0.1188	0.1197	0.1205	0.1211
0.13	0.1047	0.1155	0.1210	0.1243	0.1265	0.1281	0.1293	0.1303	0.1310	0.1316
0.14	0.1148	0.1258	0.1314	0.1347	0.1370	0.1386	0.1399	0.1408	0.1416	0.1422
0.15	0.1248	0.1360	0.1417	0.1452	0.1475	0.1491	0.1504	0.1513	0.1521	0.1528
0.16	0.1349	0.1463	0.1521	0.1556	0.1580	0.1597	0.1609	0.1619	0.1627	0.1633
0.17	0.1450	0.1566	0.1625	0.1661	0.1685	0.1702	0.1715	0.1725	0.1733	0.1739
0.18	0.1551	0.1669	0.1729	0.1765	0.1790	0.1807	0.1820	0.1830	0.1839	0.1845
0.19	0.1652	0.1772	0.1833	0.1870	0.1895	0.1913	0.1926	0.1936	0.1945	0.1951
0.20	0.1753	0.1875	0.1937	0.1975	0.2000	0.2018	0.2032	0.2042	0.2051	0.2058
0.21	0.1854	0.1978	0.2041	0.2080	0.2105	0.2124	0.2138	0.2148	0.2157	0.2164
0.22	0.1955	0.2081	0.2146	0.2185	0.2211	0.2229	0.2243	0.2254	0.2263	0.2270
0.23	0.2056	0.2185	0.2250	0.2290	0.2316	0.2335	0.2350	0.2361	0.2370	0.2377
0.24	0.2157	0.2288	0.2354	0.2395	0.2422	0.2441	0.2456	0.2467	0.2476	0.2484
0.25	0.2258	0.2391	0.2459	0.2500	0.2527	0.2547	0.2562	0.2574	0.2583	0.2590
0.26	0.2339	0.2495	0.2564	0.2605	0.2633	0.2653	0.2668	0.2680	0.2689	0.2697
0.27	0.2461	0.2598	0.2668	0.2711	0.2739	0.2759	0.2775	0.2787	0.2796	0.2804
0.28	0.2562	0.2702	0.2773	0.2816	0.2845	0.2866	0.2881	0.2893	0.2903	0.2911
0.29	0.2663	0.2806	0.2878	0.2922	0.2951	0.2972	0.2988	0.3000	0.3010	0.3018
0.30	0.2765	0.2909	0.2964	0.3027	0.3057	0.3079	0.3095	0.3107	0.3117	0.3126
0.31	0.2866	0.3013	0.3088	0.3133	0.3164	0.3185	0.3202	0.3214	0.3225	0.3233

The optimal cross-sectional area CSDP (A_a) of RCBER defined by dependence (10), with initial data: of concrete upper chord beams B_f and T_f; strength characteristics of concrete and steel: E_c, E_a, ε_{cu} and ε_{au}; ratio of height cross sections CSDP and beams: $\Delta_h = \frac{h_a}{T_f}$; ratio values of heights Z_a: $\Delta_Z = \frac{Z_a}{T_f + \frac{h_a}{2}}$.

4. Solution of task: *test the strength of normal section RCBER.* The aim of the task is to determine the threshold parameter bending moment (M_u) given normal section RCBER and compare it to the current point in it (M) from external loads:

$$M_u = \geq M. \tag{15}$$

As a result of consolidation has been allocated three separate cases of stress-strain mode(SSM) normal section RCBER at the stage of collapse or at the boundary condition depending on the position of the neutral axis with respect to the steel profile and the top shelf (fig. 6):

– variant "a": when the extreme upper fiber compressed concrete sectional area relative strain of concrete reaches a compressive strain limit $\varepsilon_b = \varepsilon_{cu}$, and in the lowest fiber stretch, relative deformation CSDP change within $\varepsilon_a > \varepsilon_{au}$, there is plastic deformation zone;

– variant "б": when the relative strain of concrete reaches $\varepsilon_b = \varepsilon_{cu}$, and the relative deformation CSDP –values $\varepsilon_a = \varepsilon_{au}$;

– variant "в": when the relative strain of concrete reaches $\varepsilon_b = \varepsilon_{cu}$, and the relative deformation CSDP change within $\varepsilon_a < \varepsilon_{au}$.

General equilibrium equation for each of the cases SSM normal section RCBER are:

– in variant 1a, 2a:

$$M_u = F_c \cdot z_1 + F_a \cdot z_2 + F_a^{p^1} \cdot z_3; \tag{16}$$

– in variant 1b, 1c, 2b, 2c:

$$M_u = F_c \cdot z_1 + F_a \cdot z_2; \tag{17}$$

– in variant 3a:

$$M_u = F_c \cdot z_1 + F_a \cdot z_2 + F_a^{p^1} \cdot z_3 + F_a^{/} \cdot z_4; \tag{18}$$

– in variant 3б 3в:

$$M_u = F_c \cdot z_1 + F_a \cdot z_2 + F_a^{/} \cdot z_4, \tag{19}$$

where: F_c; F_a'; F_a; $F_a^{p^1}$ – total normal force in section beams under its section compressed concrete and structural steel profile and spread its area of construction steel profile, working in the elastic and plastic stages; z_1; z_2; z_3; z_4 – vertical distance from the neutral line of efforts to section (fig. 6).

In the first step of calculating the strength of normal section CSDP with specified parameters (ε_{cu}; ε_{au}; E_c; E_a; f_{cd}; f_y; $A_c = B_f \cdot T_f$; $A_a = 2 \cdot h_f b_f + h_w \cdot t_w$; check the condition:

$$\alpha_a \mu \geq \alpha_a \mu_{onm} \ . \qquad (20)$$

If the condition is satisfied SSM with normal section RCBER responsible SSM for variant “c”, if none –so SSM with variant “a”.

With conditions $\alpha_a \mu = \alpha_a \mu_{onm}$ – SSM section RCBER directly responsible SSM with variant “b”.

In the second stage calculation determines the position of the neutral axis with respect to the normal section CSDP in terms of (21) and (22):

$$h - Y_B \leq h_a, \qquad (21)$$

$$h - T_f \leq h - Y_B, \qquad (22)$$

where: value Y_B which find by dependence (13), and the value of $h_a = 2 \cdot h_f + h_w$.

And if conditions (21) and (22) not satisfied, then the neutral axis passes between the lower edge of the upper shelf and the upper edge section RCBER (case 2), if the condition (21) satisfied – then the neutral axis passes through the section CSDP (case 3), if the condition (21) satisfied – then the neutral axis passes through the upper section of the concrete cross section normal shelf RCBER (case 1).

In the third stage of the calculation we make the equation of equilibrium of bending moments relative to the neutral line of the normal section RCBER under certain cases SSM and check for dependency (15) compliance with the terms of its strength.

Conclusions

The basic position of the normal methods of calculating the strength of the reduced section RCBER according to SSM concrete and CSDP. Suggested allow to distinguish between cases of calculating bearing capacity of RCBER, that give the possibility and the process of simplifying the calculation of the deformation model. The aim of further research is:

– To develop a method for calculating the optimum area of structural steel double-T profile (CSDP), which is one of the main components of the normal reduced section steel concrete beams with concrete top shelf and outer (rendered) reinforcement (RCBER);

– To research of common algorithm and analytical dependences (equilibrium equations of bending moments) calculation of normal strength sections RCBER, bent, depending on the options for SSM.

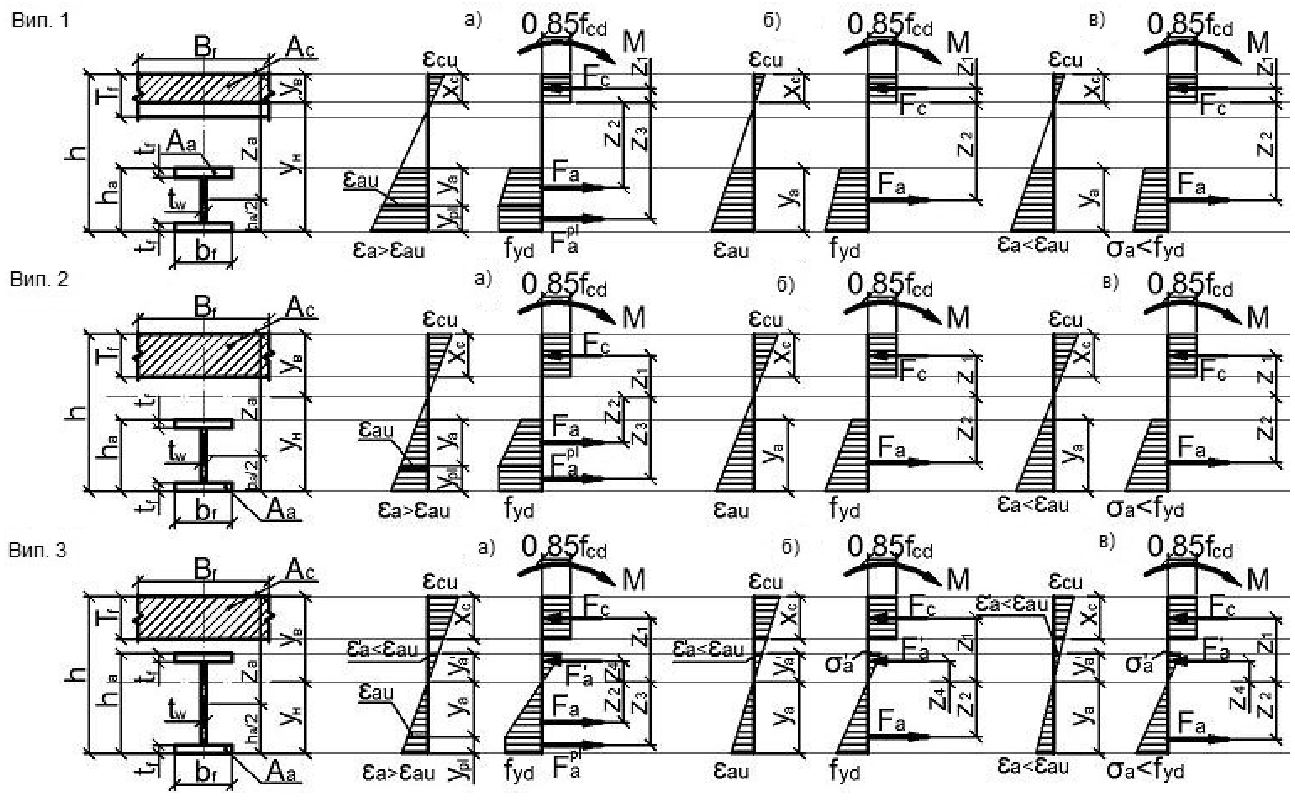

Fig. 6. Instances of the stress-strain state of the normal section RCBER depending on the position of the neutral axis

References

ACI Innovation Task Group 4: Structural Design and Detailing for High-Strength Concrete in Moderate to High Seismic Applications (ACI ITG 4.3) // American Concrete Institute. Farmington Hills: MI p. 212. Available from Internet: http://www.constructionbook.com/aci-itg-42r-06-materials-quality-considerations-for-high-strength-concrete-in-moderate-to-high-seismic-applications-itg4206/aci-standards/ (in English).

Ametov, Yu. G.; Bambura, A. N.; Semko, O. V.; Slyusarenko, Yu. S.; Storozhenko, L. I.; Tarasyuk, V. G. 2008. Problemi rozrobki natsIonalnogo normativnogo dokumenta «StalezalIzobetonnI konstruktsIYi» BudIvelnI konstruktsIYi: Zb. nauk. prats. – KiYiv – Vip.70.: 10–14. (in Ukrainian).

Babich, E. M.; Babich, V. E.; Savitskiy V. V. 2005. Rozrahunok nerozrIznih zalIzobetonnih balok Iz vikoristannyam deformatsIynoYi modelI: RekomendatsIYi.– RIvne: Vid-tvo NUVGP – 37 s. (in Ukrainian).

Babich, E. M.; Krus, Yu. O.; Babich V. E. 1999. Napruzheno-deformovaniy stan normalnih pererIzIv zalIzobetonnih balok z urahuvannyam ne lInIynostI deformuvannya betonu/ VIsnik RDTU: ZbIrnik naukovih prats. – RIvne: Vid-vo RDTU. Vip. 2. – Chastina 3.: 13–20. (in Ukrainian).

Bambura, A.M. 2006. EksperimentalnI osnovi prikladno YideformatsIyno YiteorIYizalI zobetonu: avtoref. dis. na zdobuttya nauk. Stupenya dokt. tehn. nauk: spets. 05.23.01 "BudIvelnI konstruktsIYi, budIvlI ta sporudi"/ Bambura AndrIy Mikolayovich; HDTUBA. – HarkIv – 49 s. Available from Internet: http://www.disslib.org/eksperymentalni-osnovy-prykladnoyi-deformatsiynoyi-teoriyi-zalizobetonu.html (in Ukrainian).

Bambura, A.N.; Ametov, Yu.G. 2004. Do otsInki zdatnostI stalebetonnih elementIv, scho zginayutsya, na osnovI deformatsIynogo metodu I realnih dIagram deformatsIYi materIalIv / StalezalIzobetonnI konstruktsIYi, Kriviy RIg: KTU, – Vip.6.: 71–76. (in Ukrainian).

Barashikov, A. Ya. 2005. Metodika rozrahunku zalIzobetonnih konstruktsIy za deformatsIynoyu modellyu zgIdno z proektom novih norm UkraYini – Suchasne promislove ta tsivIlne virobnitstvo. Tom 1, № 1.: 13–18.Available from Internet: http://prev.donnasa.edu.ua/win/publish/issues/spgs/nombe_en/st2.pdf (in Ukrainian).

Barashikov, A. Ya.; ZadorozhnIkova, I. V. 2005. SproschenI rozrahunki nesuchoYi zdatnostI normalnih pererIzIv zginalnih zalIzobetonnih elementIv za deformatsIynoyu modellyu / ResursoekonomnI materIali, konstruktsIYi, budIvlI I sporudi: Zb. nauk. statey. Vip.12. – RIvne.: 109-115. (in Ukrainian).

DBN V.2.6-160:2010. StalezalIzobetonnI konstruktsIYi. OsnovnI polozhennya: Zatv. MInregIonbudom UkraYini vId 15.11.2010 r №447 ta vId 30.12.2010 r. №571, chinnI z 01.09.2011 r. K.: DP "UkrarhbudInform" – 81 s. Available from Internet: http://budmart.kiev.ua/wp-8.pdf.

DBN V.2.6-98:2009. BetonnI ta zalIzobetonnI konstruktsIYi. OsnovnI polozhennya: Zatv. MInregIonbudom UkraYini vId 24.12.2009 № 680, chinnI z 01.07.2011 r. K.: DP "UkrarhbudInform", 2011 – 75 s. Available from Internet: http://dbn.at.ua/load/normativy/dbn/1-1-0-792 (in Ukrainian).

Evrokod 4. 2007. Proektuvannya kombInovanih stale zalIzobetonnih konstruktsIy – Chastina 1-1: ZagalnI normi I pravila dlya budIvel / UkraYinskiy pereklad anglomovnoYi versIYi: NDIBK – KiYiv – 118 s. (in Ukrainian).

Ilyin, O. F. 1980. The generalized method of calculation of durability of normal sections taking into account features of properties of various concrete / Behaviour of concrete and elements of ferroconcrete designs at influence of various duration. M.: PAM VNIIS of the State Committee for Construction of the USSR, Page 47–54. Available from Internet: Available from Internet: http://eprints.kname.edu.ua/25229/. (in Russian).

Ivanyuk, A. V. 2012. Napruzheno-deformovaniy stan ta nesucha zdatnIst stale zalIzobetonnih balkovih konstruktsIy z armuvannyam vertikalnimi listami [Tekst]: avtoref. dis.... kand. tehn. nauk: 05.23.01 "BudIvelnI konstruktsIYi, budIvlI ta sporudi" / Ivanyuk AndrIy Volodimirovich; Poltav. nats. tehn. un-t ImenI YurIya Kondratyuka. – Poltava. – 21 s. (in Ukrainian).

Kaar, P. H.; Hanson, N. W.; Capell, H. T. 1978. Stress–Strain Characteristics of High Strength Concrete // Douglas McHenry International Symposium on Concrete Structures, ACI Publication SP-55, American Concrete Institute. Detroit: MI, pp. 161–185. (in English).

Kochkarov, D. V.; Babich, V. I. 2012. Praktichniy rozrahunok zalIzobetonnih elementIv na mItsnIst za dIYi zginalnogo momentu na bazI DBN V.2.6-98:2009 / Komunalne gospodarstvo mIst: nauk.-tehn. zbIrnik. – № 103.: 46–57 Available from Internet: http://eprints.kname.edu.ua/25445/ (in Ukrainian).

Krupchenko, O. A. 2008. Napruzheno-deformovaniy stan ta mItsnIst stale zalIzobetonnih dvotavrovih balok Iz zalIzobetonnim verhnIm poyasom [Tekst]: avtoref. dis... kand. tehn. nauk: 05.23.01 – "BudIvelnI konstruktsIYi, budIvlI ta sporudi" / Krupchenko Oleksandr AnatolIyovich; Poltavskiy natsIonalniy tehnIchniy un-t Im. YurIya Kondratyuka. – Poltava. – 20 s. (in Ukrainian).

Kuch, T. P. 2012. Napruzheno-deformovaniy stan ta nesucha zdatnIst stale zalIzobetonnih balkovih konstruktsIy z vinesenim armuvannyam trubami [Tekst]: avtoref. dis. ... kand. tehn. nauk: 05.23.01 –"BudIvelnIkonstruktsIYi, budIvlI ta sporudi" / Kuch Tetyana PetrIvna; Poltav. nats. tehn. un-t ImenI YurIya Kondratyuka. – Poltava. – 21 s. (in Ukrainian).

KushnIr, Yu. O.; Pents, V. F.; OvsIy, M. O. 2012. MetodichnI osnovi rozrahunku nesuchoYi zdatnostI normalnogo pryamokutnogo privedenogo pererIzu poperedno-napruzhenih stalebetonnih balok na osnovI rozrahunkovoYi deformatsIynoYi modelI /

Suchasne promislove ta tsivIlne budIvnitstvo. – DonNABA, – Tom 8, № 3.: 107–122. Available from Internet: http://donnasa.edu.ua/publish_house/journals/spgs/2012-3/01_kuschnir_penc_ovsij.pdf (in Ukrainian).

KushnIr, Yu. O.; Pents, V. F.; OvsIy, M. O. 2012. MetodichnI osnovi rozrahunku nesuchoYi zdatnostI normalnogo pryamokutnogo privedenogo pererIzu stalebetonnih balok na osnovI rozrahunkovoYi deformatsIynoYi modelI. ResursoekonomnI materIali, konstruktsIYi, budIvlI ta sporudi: zb. nauk. prats. – RIvne: NUVGP. 2012.– Vip. 24.: 167–179. (in Ukrainian).

Mitrofanov, V. P. 2004. Prakticheskoe primenenie deformatsionnoy modeli s ekstremalnyim kriteriem prochnosti zhelezobetonnyih elementov / Kommunalnoe hazyaystvo gorodov:nauchn.-tehn. sbornik. – Vip. 60.: 29–48. Available from Internet: http://khg.kname.edu.ua/index.php/khg/article/view/2390 (in Russian).

Mitrofanov, V. P.; Shkurupiy, A. A.; Lazarev, D. N. 2008. O predelnoy szhimaemosti betona v deformatsionnyih modelyah zhelezobetonnyih elementov / ResursoekonomnI materIali, konstruktsIYi, budIvlI ta sporudi: zbIrnik naukovih prats. – RIvne: NUVGP, – Vip. 16. Ch.2.: 264–271. (in Russian).

Muravlov, V. V. 2012. Problemi ta perspektivi rozvitku zalIzobetonnih konstruktsIy z vinesenim armuvannyam / Zb. nauk. prats (galuzeve mashinobuduvannya, budIvnitstvo). – Poltava: Polt. NTU. – Vip. 3 (33).: 141–147. Available from Internet: http://www.irbis-nbuv.gov.ua/cgi-bin/irbis_nbuv (in Ukrainian).

OvsIy, M. O.; Pents, V. F.; GalInska, T. A. 2012. MetodichnI osnovi rozrahunku nesuchoYi zdatnostI normalnogo pererIzu stalebetonnih dvotavrovih balok Iz betonnim verhnIm poyasom na osnovI rozrahunkovoYi deformatsIynoYi modelI / Zb. nauk. prats (galuzeve mashinobuduvannya, budIvnitstvo). – Poltava: Polt.NTU. – Vip. 3 (33).: 152–161. Available from Internet: http://www.irbis-nbuv.gov.ua/cgi-bin/irbis_nbuv/cgiirbis_64.exe?Z21ID=&I21DBN=UJRN&P21DBN=UJRN&S21STN=1&S21REF=10&S21FMT=juu_all&C21COM=S&S21CNR=20&S21P01=0&S21P02=0&S21P03=PREF=&S21COLORTERMS=0&S21STR=Znpgmb(in Ukrainian).

Rukovodstvo po proektirovaniyu zhelezobetonnyih konstruktsiy s zhYostkoy armaturoy: NIIZhB, TsNIIPromzdaniy. – M.: Stroyizdat, 1978 – 55 s. (in Russian).

Vasilev, A. P.; Golosov, V. N. 1981. Sostoyanie i perspektivyi razvitiya konstruktsiy s vneshnim armirovaniem. Beton i zhelezobeton. №3.: 23–24. (in Russian).

Wight, J. K.; Richart, Jr., F. E.; Macgregor, J. G. 2011. Reinforced Concrete: Mechanics and Design. – 6th ed. pp. 1177. Available from Internet: http://ua.bookfi.org/g/James%20G.%20MacGregor (in English).