Current Serbian Design Codes – Transfering from a Deterministic to a Semi-Probabilistic Approach

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The Serbian design code for reinforced concrete structures is somewhat out-of-date in its approach to structural design. In this paper, a study into the possible transition to a semi-probabilistic approach is presented. Firstly, the implicit reliability indices in the current Serbian reinforced concrete design code are determined for three design situations - bending, axial compression and shear and for various cases within each of them. The implicit reliability indices show that the Serbian design code is more conservative than Eurocode 2, but that design for shear without stirrups has a significantly low reliability index (2.27). Secondly, a calibration procedure was implemented in order to obtain partial safety factors for a target reliability index of 4.8 (calculated as the average of the implicit reliability indices). The obtained partial safety factors are ready-for-use with the current Serbian design code and, as expected, are higher than those in Eurocode 2.

Key words: *design codes, reinforced concrete, reliability index, code calibration*

1. INTRODUCTION

The current Serbian design code for reinforced concrete structures is somewhat out-of-date in its approach to structural design. It prescribes only partial safety factors for actions and its adopted values lead to uneven reliability indices in different design situations. This leads to economically, technically and societally unsustainable practice. For this reason most nations worldwide have adopted semi-probabilistic design codes calibrated to a target reliability index which is valid for all types of structures and materials (e.g. the Eurocodes). This is what is widely known as Load and Resistance Factor Design (LRFD).

The aim of this study is first to determine the different implicit reliability indices in Serbian design codes for concrete structures. Then a target reliability index for a new semi-probabilistic code can be adopted and a code calibration procedure carried out to determine the partial safety factors for both material resistances and loads. This would enable comparisons with other more modern codes such as the Eurocodes and foster more modern advanced design procedures in Serbian engineering practice.

2. IMPLICIT RELIABILITY INDICES IN CURRENT SERBIAN DESIGN CODE FOR REINFORCED CONCRETE STRUCTURES

2.1. An overview of the PBAB 1987 design code

Since 1987 the design code for reinforced concrete structures has been the “Rulebook of technical norms for concrete and reinforced concrete” (PBAB, 1987). At the time of its introduction it was a modern code that brought the use of limit state design to Serbian structural engineering but after 27 years of its use it can hardly be said that it still mirrors current trends in reinforced concrete design.

The (PBAB, 1987) follows the standard design check procedure of limit state design:

$$R_u \geq S_u \tag{1}$$

where R_u is the ultimate value of resistance and S_u the ultimate value of load effects.

Similar to other semi-probabilistic codes it defines characteristic values of material resistances and loads. In design, material resistances are used with their characteristic values and partial safety factors are only prescribed for load effects. The Application guidebook

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for PBAB 1987 (Aćić et al., 1989) states that the partial safety factors pertaining to material resistances are envisaged by the code, but are implicitly included in the load safety factors.

The load partial safety factors depend on the limit state being checked (defined by reinforcement strain) and on the load actions on the structure. For a reinforcement strain exceeding *3‰* in the ultimate limit state (which means a yielding of reinforcement and a ductile failure) the safety factor for permanent actions is set to $\gamma_g=1.6$ and for variable loads (such as live loads, wind, snow) to $\gamma_p=1.8$. For negative ultimate reinforcement strains (reinforcement in compression and brittle fracture) the safety factors are increased to $\gamma_g=1.9$ and $\gamma_p=2.1$. For reinforcement strains between *3‰* and *0‰* the safety factors are linearly interpolated. If an additional accidental load acts on the structure or if there is a favorable effect of permanent load, the factors change again. These situations however, will not be discussed here in detail.

According to (PBAB, 1987) the load effects, are to be determined using theory of elasticity. This implies that all sections reach their limit state simultaneously. Therefore, the limit state design according to (PBAB, 1987) deals only with resistances of member cross-sections while other limit states related to whole structure are not analyzed.

In this study a FORM analysis is carried out in order to determine the implicit reliability indices of the (PBAB, 1987) design code. Three different design situations are analyzed – failure of a cross-section in bending, axial compression and shear.

2.2. Bending of reinforced concrete members

In this study bending was analyzed on a $b/d=30/60$ *cm* rectangular cross-section.

When designing a cross-section in bending according to (PBAB, 1987), the procedure is as follows:

1) Assume an effective depth of the beam $h \approx (9/10)x_d$

2) Calculate the coefficient k (equilibrium of bending moments) given by

$$k = \frac{h}{\sqrt{\frac{M_u}{b \cdot f_b}}} \qquad (2)$$

where M_u is the ultimate value of bending moment ($=1.6 \cdot M_g+1.8 \cdot M_p$), b the width of cross-section zone in compression and f_b the design value of concrete compressive strength

3) The coefficient k unambiguously determines concrete and reinforcement strains ε_c and ε_r; they in turn define the lever arm of internal forces through the coefficient ζ (all values are tabulated)

4) Using the coefficient ζ calculate the necessary reinforcement area (equilibrium of axial forces)

$$A_a = \frac{M_u}{\zeta \cdot h \cdot \sigma_v} \qquad (3)$$

where σ_v is the characteristic value of reinforcement yield stress.

In this study three design cases are evaluated. The Case B1 is a design situation where minimal reinforcement area is required (*0.2%* of cross-section area, under-reinforced section). The Case B2 is defined as “simultaneous failure” in (PBAB, 1987) where both concrete and reinforcement reach their respective ultimate strains at the same time (*3.5* and *10‰* respectively). The Case B3 stipulates the reinforcement ultimate strain of *3‰*, close to the so-called “balanced section” (over-reinforced section).

In (PBAB, 1987) the concrete compressive strength is defined by a concrete grade MB which is the 10-percentile of concrete compressive strength obtained by testing *20x20x20cm* cubes, given in MPa. This differs from the definition of concrete grade in (EN 1992, 2004) where a 5-percentile of concrete compressive strength obtained by testing *Ø15x30cm* cylinders is defined as the characteristic value.

The provisions in (PBAB, 1987) further decrease this value by multiplying it with approximately 0.7 in order to take into account the difference between the strength achieved in structural members versus test specimens (Aćić et al., 1989). The factor by which the characteristic value is multiplied decreases from 0.683 for MB 30 to 0.55 for MB 60.

Given that in bending, the contribution of concrete to bending resistance is significantly smaller compared to the reinforcement the reduction of concrete compressive strength wasn't varied with concrete grade in this study. In axial compression this variation in reduction of the characteristic value can be significant and is therefore investigated.

The reinforcement yield stress is defined as the 5-percentile of the reinforcement steel yield stress obtained by testing defined specimen as required in (EN 1992, 2004).

Basic variables for the design of the cross-section are given in Table 1. The ultimate bending moment is given by:

$$M_u = \gamma_g \cdot M_g + \gamma_p \cdot M_p \qquad (4)$$

For this ultimate moment, the ratio of permanent to variable load was varied using a coefficient $\alpha=[0,1]$ and equation (5):

$$\gamma_g \cdot M_g = \alpha \cdot M_u;\ \gamma_p \cdot M_p = (1-\alpha) \cdot M_u \qquad (5)$$

Table 1. Variables for design of cross-section in bending

Case	b (cm)	h (cm)	A_a (cm^2)	M_u (kNm)	MB (kN/cm^2)	σ_v (kN/cm^2)
B1	30.0	55.5	3.73	80.0	3.0	40.0
B2		53.5	17.29	330.0		
B3		51.5	34.53	552.0		

The limit state equation for FORM analysis is as follows (Vrouwenvelder and Siemes, 1987):

$$G = A_a \cdot \sigma_v \cdot \square \cdot \ 1 - 0.55\frac{A_a}{b \cdot h}\frac{\sigma_v}{0.683 \cdot MB} \qquad (6)$$
$$- M_g - M_p$$

In all three design cases the only differing parameters are the reinforcement area, effective depth, permanent and live load. In this study the geometric quantities are viewed as deterministic values. This, of course, doesn't mirror the reality, but is deemed sufficiently accurate for the given purpose. For the distribution of concrete compressive strength and reinforcement yield stress, a Log-normal distribution was selected, as recommended in literature (Vrouwenvelder and Siemes, 1987; JCSS, 2001).

For the distributions of the permanent and variable loads, the Normal and Gumbel distributions are selected, respectively. Their characteristic values are also selected according to (JCSS, 2001). This results in four random variables and for each variable two coefficients of variation (CoV) are selected according to (Vrouwenvelder and Siemse, 1987; JCSS, 2001).

This amounts to 16 combinations that are to be simulated using the limit state equation (6). For each combination a FORM analysis was carried out using the software VaP (developed by Markus Petschaher, PSP GmbH, Feldkirchen, Austria). In each combination of CoVs, four values of α factor are analyzed – *0, 0.3, 0.6* and *1.0*.

Table 2. Variables and their distribution parameters for FORM analysis of bending

Property	MB	σ_v	M_g	M_p
Distribution	Log-normal	Log-normal	Normal	Gumbel
Prob. of exceeding charact. value	0.95	0.90	0.50	0.02
CoV 1	0.15	0.05	0.05	0.30
CoV 2	0.25	0.10	0.10	0.50

The distribution parameters of the variables for cases B1-3 are given in Table 2.

2.3. Axial compression of reinforced concrete members

In this study axial compression was analyzed on a square cross-section with *b=50 cm*. Second-order effects are not considered. When designing a cross-section in axial compression according to (PBAB, 1987), the axial forces equilibrium condition is given by:

$$N_u = A_b \cdot f_b + A_a \cdot \sigma_v \qquad (7)$$

where N_u is the ultimate value of axial force ($=1.9 \cdot N_g + 2.1 \cdot N_p$), A_b the concrete cross-section area and A_a the reinforcement area ($=\mu \cdot A_b$, where μ is the reinforcement ratio)

In this study four design cases (C1-C4) are evaluated. The reduction factor to obtain the characteristic value of the concrete compressive strength from concrete grade depends on the grade and this dependency cannot be neglected for axial compression. Hence, in this study two concrete grades are analyzed – MB 30 and MB 60 and for each grade two reinforcement ratios μ are chosen – the minimum ratio of *0.6%* and a sufficiently large reinforcement ratio of *3%* (the maximum allowed being *6%*). The basic variables for the design of the cross-section are given in Table 3. The ultimate axial force is analogous to Eq. (4) and the ratio of permanent to variable load α is defined in the same way as in bending.

Table 3. Variables for design of cross-section in axial compression

Case	b (cm)	μ (%)	N_u (kN)	MB (kN/cm^2)	σ_v (kN/cm^2)
C1	50.0	0.6	5725	3.0	40.0
C2		3.0	8125	3.0	
C3		0.6	8850	6.0	
C4		3.0	11250	6.0	

The limit state equation for FORM analysis is as follows:

$$G = A_b \cdot C \cdot MB + A_a \cdot \sigma_v - N_g - N_p \qquad (8)$$

where C is the reduction coefficient for concrete compressive strength (0.683 for MB 30 and 0.55 for MB 60)

Table 4. Variables and their distribution parameters for FORM analysis of axial compression

Property	f_b	σ_v	N_g	N_p
Distribution	Log-normal	Log-normal	Normal	Gumbel
Prob. of exceeding charact. value	0.95	0.90	0.50	0.02
CoV 1	0.15	0.05	0.05	0.30
CoV 2	0.25	0.10	0.10	0.50

The distribution parameters of the variables for cases C1-4 are given in Table 4. There are 16 combinations of CoVs and three values of α factor are analyzed – *0, 0.5* and *1.0*.

2.4. Shear in reinforced concrete members

In this study shear was analyzed on a *60 cm* high T-section with a web width of *30 cm*.

When designing a cross-section in shear according to (PBAB, 1987), there are several specific issues. Firstly the basic design check format for members in shear is:

$$\tau_n = \frac{T_u}{b \cdot z} \leqslant \tau_r \tag{9}$$

where τ_n stands for the nominal shear stress, T_u for the ultimate shear force ($=1.6 \cdot T_g + 1.8 \cdot T_p$), b for the section width, z for the internal lever arm ($=0.9 \cdot h$) and τ_r for the shear strength design value

The design value of the shear strength is derived by dividing the mean value of the uniaxial tensile strength f_{bzm} by approximately 2.2 (PBAB, 1987). This means that, unlike bending and compression, in shear, a basic random variable isn't the concrete compressive strength (with a 10-percentile), but the concrete uniaxial tensile strength f_{bzm} with its mean value. Also associated with the concrete tensile strength is the significantly larger coefficient of variation.

Unlike other modern design codes (PBAB, 1987) prescribes three different possibilities when checking Eq. (9). In the case S1 i.e. $\tau_n < \tau_r$, no transverse reinforcement is required.

If $\tau_r < \tau_n < 3 \cdot \tau_r$ (case S2) transverse reinforcement is required but a part of the shear force is assumed to be transferred by aggregate interlock. The reinforcement is calculated for the remaining nominal shear stress.

In this case the design procedure is as follows:

- Reduction of the shear stress:

$$\tau_{Ru} = 1.5 \left(\frac{T_u}{b \cdot z} - \tau_r \right) \tag{10}$$

- Calculation of necessary stirrups to support the stress τ_{Ru}:

$$\tau_{Ru} = \frac{A_u}{b \cdot e_u} \sigma_v \tag{11}$$

where A_u is the area of reinforcement in one section (= number of legs x area of one stirrup bar) and e_u stands for stirrup spacing

In case S3 $3 \cdot \tau_r < \tau_n < 5 \cdot \tau_r$ shear reinforcement is required but no reduction of the shear force is allowed. The case $\tau_n > 5 \cdot \tau_r$ is not permitted.

In this case the design procedure consists only of calculating the necessary stirrups to support the nominal shear stress τ_n, using Eq.(11) replacing τ_{Ru} with τ_n .

For a selected concrete grade MB 30 the mean value of the uniaxial tensile strength f_{bzm} is 0.24 *kN/cm²*, the shear strength design value is 0.11 *kN/cm²*, and the reduction factor 2.18 (PBAB, 1987). The ultimate shear force is analogous to Eq. (4) and the ratio of permanent to variable load α is defined in the same way as in bending.

In the case of $\tau_n < \tau_r$ FORM analysis is carried out by using the following limit state equation:

$$G = 0.9 \cdot f_{bzm} \cdot b \cdot h / 2.18 - T_g - T_p \tag{12}$$

In the case of $\tau_n < 3 \cdot \tau_r$ the limit state equation is:

$$G = \frac{0.9 \cdot A_u \cdot h \cdot \sigma_v}{e_u} + \frac{1.35 \cdot f_{bzm} \cdot b \cdot h}{2.18} - 1.5 \cdot (T_g + T_p) \tag{13}$$

and in the case of $3 \cdot \tau_r < \tau_n < 5 \cdot \tau_r$ the limit state equation is:

$$G = \frac{0.9 A_u \cdot h \cdot \sigma_v}{e_u} - T_g - T_p \tag{14}$$

Table 5. Variables for design of cross-section in bending

Case	b (cm)	h (cm)	A_u (cm²)	e_u (cm)	T_u (kNm)	f_{bzm} (kN/cm²)	σ_v (kN/cm²)
S1	30.0	55.0	–	–	164.0	0.24	–
S2			1.57	12.6	327.0	0.24	40.0
S3			3.14	9.5	654.0	–	40.0

Basic variables for the design of the cross-section are given in Table 5. The distribution parameters of the variables for cases C1-4 are given in Tables 6. In cases S1 and S2 there are 8 combinations of CoVs while in case S3 there are 16 (two material resistances instead of one). Three values of α factor are analyzed – *0, 0.5* and *1.0*.

Table 6. Variables and their distribution parameters for FORM analysis of shear

Property	f_{bzm}	σ_v	T_g	T_p
Distribution	Log-normal	Log-normal	Normal	Gumbel
Prob. of exceeding charact. value	0.50	0.90	0.50	0.02
CoV 1	0.20	0.05	0.05	0.30
CoV 2	0.40	0.10	0.10	0.50

3. CALIBRATING A PROBABILISTIC CODE TO A TARGET RELIABILITY INDEX

3.1. Overview of the implicit reliability indices in the PBAB 1987

Ten design cases were analyzed and the average reliability indices for each one are presented in Table

7. Also a “most-likely” reliability index is shown for each case, chosen on the basis of the most-likely coefficient of variation for each variable and most-likely α factor (as described in Table 7). Within the cases for each type of analysis (bending, axial compression, shear) the reliability index increases as the role of reinforcement increases (due to the lower CoV of reinforcement properties).

Table 7. Average reliability indices for reinforced concrete structures

			average	**4.83**
	Type	**Case**	**β_{ML}[a]**	**β**
1	Bending	B1	4.35[b]	5.08
2	Bending	B2	4.34[b]	5.03
3	Bending	B3	4.32[b]	4.66
4	Axial compression	C1	5.07[c]	4.78
5	Axila compression	C2	5.45[c]	5.56
6	Axial compression	C3	5.23[c]	4.73
7	Axial compression	C4	5.39[c]	5.22
8	Shear	S1	3.22[d]	2.27
9	Shear	S2	5.01[e]	4.83
10	Shear	S3	5.40[f]	6.17

[a] Most-likely β
[b] α=0.3, CoV: f_b 0.15, σ_v 0.10, M_g 0.1, M_p 0.5
[c] α=0.5, CoV: f_b 0.15, σ_v 0.10, M_g 0.1, M_p 0.5
[d] α=0.3, CoV: f_{bzm} 0.2, M_g 0.1, M_p 0.5
[e] α=0.5, CoV: f_{bzm} 0.2, σ_v 0.10, M_g 0.1, M_p 0.5
[e] α=0.5, CoV: σ_v 0.10, M_g 0.1, M_p 0.5

The overall average reliability index is *4.83* while the value range is *2.27 - 6.17*. It can be concluded that when designing reinforced concrete structures according to (PBAB, 1987) a higher reliability index is to be expected compared to the target reliability index of *3.8* used to calibrate (EN 1992, 2004). This is not valid for the design case of shear with $\tau_n<\tau_r$ (S1). This case requires correction of the design process since the obtained reliability index can be dangerously low. Also, it should be noted that in this study only member cross-sections are analyzed.

3.2. CodeCal software

Calibration of the partial safety factors is performed using the software CodeCal (Faber et al., 2003). CodeCal is specifically designed for the calibration of design codes according to (JCSS, 2001).

In this study calibration of partial safety factors to a target reliability index is performed for a design case consisting of permanent and one variable load as well as two material resistance variables.

The design equation is:

$$z = \frac{\gamma_m}{R_k}\ \alpha \cdot \gamma_G \cdot G_k + \ 1 - \alpha\ \cdot \gamma_Q Q_k \qquad (15)$$

where z is the design variable, γ_m the partial safety factor for material resistance (i.e. γ_c– concrete; γ_r – reinforcement), R_k the characteristic value of resistance variable, α the coefficient representing permanent to variable load ratio, γ_G the partial safety factor for permanent load, G_k the characteristic value of permanent load, γ_Q the partial safety factor for variable load and Q_k the characteristic value of variable load.

3.3. Calibration results and discussion

Code calibration is performed for a target reliability index of $\beta_{target}=4.8$. Calibration is performed for two scenarios. In scenario 1 all of the partial safety factors are calibrated and in scenario 2 only factors γ_m and γ_Q (material resistance and variable load safety factors) while the permanent load safety factor is kept constant at $\gamma_G=1.35$.

Calibration is performed for 32 combinations of material and load CoVs. The combinations are derived from two cases – bending and compression.

When analyzing bending, reinforcement is given a weight coefficient of *0.9* while concrete is given a weight coefficient of *0.1*. In compression the situation is reversed and concrete is given a weight coefficient of *0.9* and reinforcing steel *0.1*.

Shear isn’t analyzed because currently (PBAB, 1987) prescribes the use of a different material resistance variable for concrete shear strength (defined through a reduction of the uniaxial tensile strength) and not the concrete compressive strength.

In order to be able to apply the obtained partial safety factors to shear, it is necessary to formulate the shear strength as a function of the concrete compressive strength. This can be done via clause 51 of (PBAB, 1987) that defines the mean uniaxial tensile strength as:

$$f_{bzm} = 0.25 \cdot MB^{2\ 3} \qquad (16)$$

This value should be viewed as being too conservative since a mean value of a material property is empirically defined by a characteristic value of another material property. The shear strength design value can then be formulated as:

$$\tau_r = f_{bzm}\ 2.2 = 0.114 \cdot MB^{2\ 3} \qquad (17)$$

In this way, the obtained results are also applicable to shear.

The results of the code calibration are presented in Table 8.

Table 8. Calibration of partial safety factors to $\beta_{target}=4.8$

			γ_c	γ_r	γ_G	γ_Q
1	Calibration of all partial safety factors	a	1.62	1.33	1.29	1.67
		b	2.31	1.33	1.29	1.77
2	Keeping $\gamma_G=1.35$	a	1.54	1.26	1.35	1.77
		b	2.20	1.26	1.35	1.77
3	EN 1992		1.50	1.15	1.35	1.50
a – without compressive strength reduction b – with compressive strength reduction						

In comparison with (EN 1992, 2004) the calibrated safety factors are, as expected higher than in (EN 1992, 2004). As commented in (Jacobs, 2008) the calibration of Eurocode 2 yielded $\gamma_c=1.30$. This safety factor was multiplied by *1.15* in order to account for the uncertainty arising from the fact that concrete is tested on concrete from test specimens and not directly on the structure. The code (PBAB, 1987) already prescribes a reduction of the compressive strength by *0.7*. If this factor were applied directly on the characteristic value (by increasing the partial safety factor), *0.7* could be eliminated from the design equations. In this way the partial safety factor for concrete would be $\gamma_c^{1'}$=1.62/0.7=2.31 and $\gamma_c^{2'}$=1.54/0.7=2.20 (for calibration scenarios 1 and 2, respectively).

Further harmonization is possible with (EN 1992, 2004) so that a better comparison can be made. The current code (PBAB, 1987) defines the characteristic value of the concrete compressive strength as a 10-percentile obtained on *20x20x20cm* cubic samples whereas Eurocode 2 defines this characteristic value as a 5-percentile obtained on *Ø15x30cm* cylindrical samples. If the concrete grade is to be converted to a 5-percentile value and a *Ø15x30cm* cylindrical sample it should be multiplied by *0.84-0.93* for the percentile difference (for a Log-normal distribution and CoV*0.15* and *0.25*) and by *1/1.2* for the sample size difference (PBAB, 1987) which means a new MB^{EN}=0.9·MB/1.2=0.75·*MB*.

Table 9. Partial safety factors for MB defined as in (EN 1992, 2004)

			γ_c	γ_r	γ_G	γ_Q
1	Calibration of all partial safety factors	a	1.22	1.33	1.29	1.67
		b	1.74	1.33	1.29	1.67
2	Keeping $\gamma_G=1.35$	a	1.16	1.26	1.35	1.77
		b	1.66	1.26	1.35	1.77
3	EN 1992		1.50	1.15	1.35	1.50
a – without compressive strength reduction b – with compressive strength reduction						

In this way the partial safety factors for concrete can be computed as $\gamma_c^{1,EN}$= 0.75·1.62 = 1.22 and $\gamma_c^{2,EN}$ = 0.75·1.54 = 1.16. The aforementioned factor of *0.7* can now be applied to calculate the final value of partial safety factors that are compatible with concrete grade as defined in (EN 1992, 2004), Table 9.

4. CONCLUSIONS

In this paper, a study of limited scope into the implicit reliability of current Serbian design codes is presented. From the analyses and calculations carried out the following conclusions can be drawn:

- The current Serbian reinforced concrete structures design code is, in most cases, too conservative. This is evident in the achieved reliability indices. The average reliability index for bending, compression and shear is higher than the target reliability index of *3.80* for (EN 1992, 2004).
- The exception to these findings is a design case which has a dangerously low reliability index: shear with the nominal shear stress lower than the shear strength (no transverse reinforcement required). The average achieved reliability index is *2.27*.
- In the current Serbian design code for reinforced concrete structures (PBAB, 1987) the use of safety factors for loads is unnecessarily complicated. Their values are tied to reinforcement strain and dependent on the types of load acting on the structure. This is a complicated approach prone to errors in engineering practice.
- It is possible to calibrate a semi-probabilistic code to target reliability indices implicit in the current code. Partial safety factors are calibrated either for direct use with (PBAB, 1987) or for use with material properties harmonized with (EN 1992, 2004).
- The calibrated partial safety factors are higher than those in the (EN 1992, 2004), as is expected due to the higher target reliability index.
- The use of the partial safety factors obtained in this study can facilitate Serbia's transition to the Eurocodes for practicing engineers and designers.

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REZIME

TRENUTNI SRPSKI PROPISI ZA PRORAČUN - PRELAZAK SA DETERMINISTIČKOG NA POLU-PROBABILISTIČKI PRISTUP

Srpski propis za proračun armiranobetonskih konstrukcija je relativno zastareo u svom pristupu. U ovom radu sprovedena je analiza mogućnosti prelaska na polu-probabilistički pristup definišući i parcijalne faktore sigurnosti i na strani nosivosti. Prvo su određeni indeksi sigurnosti, implicitno sadržani u važećem propisu, za tri proračunske situacije – savijanje, centrični pritisak i smicanje i za nekoliko slučajeva u okviru svake situacije. Izračunati indeksi sigurnosti pokazuju da je važeći srpski propis konzervativniji od Evrokoda 2 osim proračunski slučaj smicanja bez armature koji ima izuzetno nizak indeks sigurnosti (2.27). U drugom delu je primenjen postupak kalibracije radi dobijanja parcijalnih koeficijenata sigurnosti za ciljani indeks sigurnosti od 4.8 (dobijen kao srednja vrednost implicitnih indeksa sigurnosti). Rezultujući parcijalni koeficijenti sigurnosti su, očekivano, veći nego oni u Evrokodu 2, a moguća je njihova direktna upotreba uz važeći srpski propis.

Ključne reči: *propisi za proračun, armirani beton, indeks sigurnosti, kalibracija propisa*