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DESIGN OF CONCRETE STRUCTURES ACCORDING TO EUROCODE 2 AND BAB 87: COMPARISON OF BASIC CALCULATIONS

Nenad PECIĆ[1]

ABSTRACT

Structural Eurocodes have superseded national design codes in a number of countries. The education of engineers is necessary for quick and successful adaptation to new standards. Qualified institutions should organize appropriate training programs and provide quality manuals with clear and precise instructions.

A possible approach for practicing the new regulations is to compare the procedures and results of the design according to the former regulations, which are well known by the designers, and the corresponding procedures in the new regulations.

This paper compares the basic design inputs in the Code for Concrete Structures of the Former Yugoslavia (BAB 87) and Eurocode 2 (EN 1992-1-1: 2004). In addition, a comparison of the basic ULS and SLS design procedures is presented, as well as an analysis of the obtained results. ULS design for bending and for shear are studied. Control of cracking and deflection control are covered in the SLS domain.

The comparisons are intended to indicate to the designers whether their previous experience in the design of structures is in accordance with the requirements of the new regulations. The derived conclusions can be useful in assessing the compliance of existing structures, designed according to BAB 87, with the requirements of Eurocode 2.

Keywords: concrete structures; design; BAB 87; Eurocode 2; comparison.

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1. INTRODUCTION

The implementation of Eurocodes as national standards regularly raises certain issues. From the aspect of experienced designers, who performed their professional work within a certain national standard, the basic question is whether it is necessary to change something in the conceptual design in order to meet the requirements of Eurocodes. A similar question arises in the assessment of structures designed according to national standards regarding the fulfilment of Eurocode requirements.

The Code for the design of concrete structures in the Former Yugoslavia, in practice named BAB 87 [1] according to the acronym and formal year of publication, has been applied in the republics for decades. At the time of publishing, it was a very advanced standard, which relied on the international experience regarding concrete structures contained in the CEB-FIP Model Code 1978. Eurocode 2 (EC2, EN 1992-1-1, [2]) originated from the next generation of these documents contained in the CEB-FIP Model Code 1990, in which many models from Model Code 1978 remained unchanged or with small modifications. It is the main reason for numerous similarities between BAB 87 and EC2 that facilitate the transition to new regulations. Besides, Eurocode standards

Eurocode standards contain a number of parameters whose value each country can adjust for its own use, the so-called "Nationally Determined Parameters" (NDP). Their values must be explicitly defined in the "National Annex" (NA), issued by the national standardization body. NA is mandatory with each Eurocode standard. EC2 has more than 100 clauses in which the setting of NDPs is allowed. NDPs can also be used as a useful tool to mitigate the transition from national standards to Structural Eurocodes.

Overview of basic design inputs and criteria, as well as comparisons of the results of the basic design procedures for concrete structures is presented in the following. Design for bending and for shear are analyzed in the ULS domain. Control of cracking and deflection control are covered in the SLS domain.

2. OVERVIEW OF BASIC INPUTS AND REQUIREMENTS

2.1 Factor of safety

BAB 87 applies partial safety factors only for loads, while EC2 has partial factors for both loads and material properties. Higher factors associated with BAB 87 include safety factors for materials and overall reliability is similar.

The partial factor of safety for permanent actions (loads) is 1.35 (EC2) and 1.60/1.90 (BAB 87, for dominant bending/compression), while for variable actions equals 1.50 (EC2) and 1.80/2.10 (BAB 87). Table 1 shows the values of the average safety factor for several ratios of permanent (g) and variable (q) loads.

Table 1. Average values of safety factors for loads.

$\gamma = (\gamma_g\, g + \gamma_q\, q)/(g + q)$	g = 1.0 q	g = 2.0 q	g = 3.0 q	g = 4.0 q	g = 5.0 q
γ_{EC2}	1.425	1.400	1.388	1.380	1.375
$\gamma_{PBAB\,87}$ (dominant bending)	1.700	1.667	1.650	1.640	1.633
$\gamma_{PBAB\,87}$ (dominant compression)	2.000	1.967	1.950	1.940	1.933
$\gamma_{PBAB\,87}$ / γ_{EC2} (dominant bending)	1.193	1.190	1.189	1.188	1.188
$\gamma_{PBAB\,87}$ / γ_{EC2} (dominant compression)	1.404	1.405	1.405	1.406	1.407

It may be seen from Table 1 that the average safety factor according to EC2 is lower than one that follows BAB 87. But, additional partial safety factors for the material properties are used when a design according to EC2 is performed: γ_s = 1.15, for steel, and γ_c = 1.50, for concrete. That gives a similar value to the safety factor as a whole. For further analysis, the value $\gamma_{PBAB\,87}/\gamma_{EC2}$ = 1.190 was adopted in case of dominant bending, and 1.405 for dominant compression.

2.2. Concrete compressive strength

The strength class designation in EC2 consists of the letter C followed by two numbers (C$f_{ck,cyl}$ /$f_{ck,cube}$). The first number is the characteristic strength measured on a standard cylinder with a diameter of 150 and a height of 300 mm. This number participates in all expressions. The second number is the strength measured on a 150 mm cube with statistical limit (fractile) of 5%, so that, for a usual dispersion, the conversion factor for 200 mm cube and 10 % limit (BAB 87) is near 1,0. This means that the second number in a strength class is close to the concrete grade according to BAB 87 (for example, C25/30 ≈ MB30).

2.3. Tensile strength of reinforcing steel

The characteristic yield stress of reinforcing steel has the same definition in both codes ($f_{yk} = \sigma_v$) since both apply 5% fractile. The basic reinforcement class is B500, i.e. $f_{yk} = \sigma_v = 500$ MPa.

2.4. Concrete cover to the reinforcement

EC2 has a more detailed classification of environmental conditions compared to BAB 87. A total of 18 exposure classes (EC2) replace the simple division into mild, moderate or aggressive environmental conditions (BAB 87). The concrete cover by EC2 is designed in several steps, taking into account the design working life of the structure (basically 50 years), the exposure class, the concrete class and the type of member. Most of the values associated with concrete cover design are set as NDP.

BAB 87 simply provided a concrete cover in relation to the environment and the type of member. An overview of the required concrete covers according to EC2 and BAB 87 is shown in Table 2. A recommended tolerance of 10 mm was taken for EC2 values.

The values in Table 2 indicate that EC2 in most cases requires significantly larger concrete covers compared to BAB 87. The increase ranges from 5 mm for non-aggressive environments to as much as 20 mm for very aggressive conditions. This means that structures designed with a minimum concrete cover according to BAB 87 do not meet the recommended durability requirements according to EC2.

Table 2. Comparison of the required concrete cover (mm) according to EC2 and BAB 87.

EC2	Exposure class	X0, XC1	XC2, XC3, XC4	XD, XS
	Slabs	20	30 ÷ 35	40 ÷ 50
	Beams	20 ÷ 25	35 ÷ 40	45 ÷ 55
BAB 87	Slabs	15	20	30
	Beams	20	25	35
	Environment	mild	moderate	aggressive

An indirect consequence of a larger concrete cover is the reduced effective depth of the flexural elements, when the total height is fixed, and greater deflections of the elements are expected. On the whole, under the same design conditions, a slightly higher cross-section may be required.

2.5. Crack width limits

The maximum allowable crack widths in EC2 are significantly relaxed compared to the practice in previous decades. The overview is shown in Table 3. To make the comparison adequate, the maximum allowable crack widths shown in BAB 87 have been increased by 50%, in accordance with the provision that deals with the enlarged concrete cover, which is required by EC2.

Table 3. Comparison of the recommended maximum crack widths (mm) under long-term loads.

EC2	Exposure class	X0, XC1	XC2, XC3, XC4	XD, XS
	Maximum width	0.4	0.3	0.3
BAB 87	Maximum width (+50 %)	0.3	0.15	0.075
	Environment	mild	moderate	aggressive

In addition, the crack width calculated by the procedure presented in EC2 is smaller than that obtained by the procedure according to the CEB “Manual on cracking and deformations” [3], which will be commented below. Although the reinforcement B500 has higher service stresses compared to the previously used RA400/500, the smaller calculated crack width and the higher limit make it easier to satisfy the crack width.

2.6. Creep and shrinkage of concrete

BAB 87 provides tabulated values of creep coefficient and shrinkage strains. The values were derived from creep compliance and shrinkage of concrete according to Model Code 1978, with regard to conducted experiments and measured data in Yugoslavia. Creep coefficient provided in BAB 87 is a function of concrete age at loading, relative humidity and notional size of the element. In addition to these parameters, EC2 accounts for concrete strength and type of cement. Creep and shrinkage in EC2 are based on Model Code 1990 which defines creep coefficient as a product function and total shrinkage strain as a sum of two components – autogenous and drying shrinkage strain.

The creep coefficient in BAB 87 is related to modulus of elasticity $E_b(t_0)$ corresponding to the age of concrete at time of loading t_0. Unlike, the creep coefficient in EC2 is related to the tangent modulus of concrete E_c determined at an age of 28 days. For the purpose of comparison, values of the creep coefficient according to EC2 are adjusted with moduli ratio to match the approach used in BAB 87. Corresponding values of the creep coefficient from BAB 87 and EC2 for C25/30 ≈ MB30 and cement class N are presented in Table 4. EC2 values are calculated according to Annex B of [2].

Table 4. Comparison of final the creep coefficient for concrete C25/30 and cement class N.

Concrete age at loading t_0 (days/years)	Notional size of the element h_0 (mm)	Final values of creep coefficient $\varphi(\infty, t_0)$							
		Relative humidity - RH (%)							
		40		70		90		100	
		BAB 87	EC 2	BAB 87	EC 2	BAB 87	EC 2	BAB 87	EC 2
7	100	4.30	3.76	3.10	2.70	1.70	1.99	1.40	1.64
	200	4.10	3.32	2.90	2.48	1.60	1.92		
	400	3.80	2.98	2.70	2.31	1.60	1.86		
14	100	4.00	3.45	2.90	2.48	1.60	1.83	1.30	1.50
	200	3.80	3.05	2.70	2.28	1.50	1.76		
	400	3.60	2.73	2.50	2.12	1.50	1.71		
28	100	3.70	3.12	2.60	2.24	1.60	1.65	1.20	1.36
	200	3.60	2.76	2.60	2.06	1.50	1.59		
	400	3.40	2.47	2.50	1.91	1.40	1.55		
90	100	2.70	2.58	2.00	1.85	1.30	1.37	1.00	1.12
	200	2.80	2.28	2.10	1.70	1.30	1.32		
	400	2.90	2.04	2.10	1.58	1.30	1.28		
365	100	1.70	2.01	1.30	1.44	1.00	1.07	1.00	0.88
	200	1.80	1.78	1.40	1.33	1.10	1.03		
	400	2.00	1.59	1.50	1.23	1.10	1.00		
3 years	100	0.90	1.64	0.80	1.18	0.70	0.87	0.80	0.71
	200	1.10	1.45	0.90	1.08	0.80	0.84		
	400	1.20	1.30	1.00	1.00	0.80	0.81		

Presented values in Table 4 indicate that, in most cases, BAB 87 provides slightly higher values of the creep coefficient in comparison to EC2. The values differ more for lower humidity and early loading. Differences are smaller if the age at loading is higher. Significant differences in Table 4 are for relatively slender elements (h_0 = 100 mm) or for delayed loading (after 3 years), which are rare design situations. However, the differences are larger when the effect of concrete strength according to EC2 is considered.

Shrinkage strain provided in BAB 87 is a function relative humidity and notional size of the element. In addition to these parameters, EC 2 accounts for concrete strength and type of cement.

Table 5. Comparison of the total shrinkage strain for concrete C25/30 with cement class N.

Notional size of the element h_0 (mm)	Final values of total shrinkage strain (‰)							
	Relative humidity - *RH* (%)							
	40		70		90		100	
	BAB 87	EC 2	BAB 87	EC 2	BAB 87	EC 2	BAB 87	EC 2
100	0.56	0.59	0.40	0.42	0.15	0.20	0.00	0.04
200	0.48	0.50	0.34	0.36	0.12	0.17		
400	0.42	0.43	0.30	0.32	0.10	0.15		

Corresponding values of the final shrinkage strain from BAB 87 and EC2 are presented in Table 5. EC 2 values are calculated for concrete class C25/30 and cement class *N* according to Annex B of EC2. The values from both codes are similar. Impact of the concrete strength is relatively small. However, one should have in mind that the use of cement classes *S* and *R* leads to larger differences than those presented in Table 5.

Tabulated values of the creep coefficient and the total shrinkage strain, calculated according to Annex B of EN 1992-1-1:2004 for all combinations of relevant parameters, may be found in [4].

2.7. Stress-strain relations for the design of cross-sections

The shape of the stress-strain diagrams both for concrete and reinforcing steel is identical in both codes, Figure 1. Transition strain of 2.0 ‰ and maximum strain of 3.5‰ are applied in EC2 for concrete class up to C50/60. Maximum strain for reinforcing steel is not limited, as in BAB 87 (10 ‰), in the case of diagram with a horizontal top branch.

EC2 applies partial safety factors for material properties and design values of ultimate stresses are obtained as $f_d = f_k/\gamma$. γ_c is 1.50 for concrete and γ_s is 1.15 for steel. An additional factor α_{cc} is applied to account for unfavourable effects on the concrete compressive strength. α_{cc} is NDP and recommended value is 1.0 (no reduction). However, the recommended value in ENV 1992-1-1:1991 was 0.85, and the new prEN1992-1-1:2021 also recommends 0.85. Serbian National Annex sets its value at 0.85.

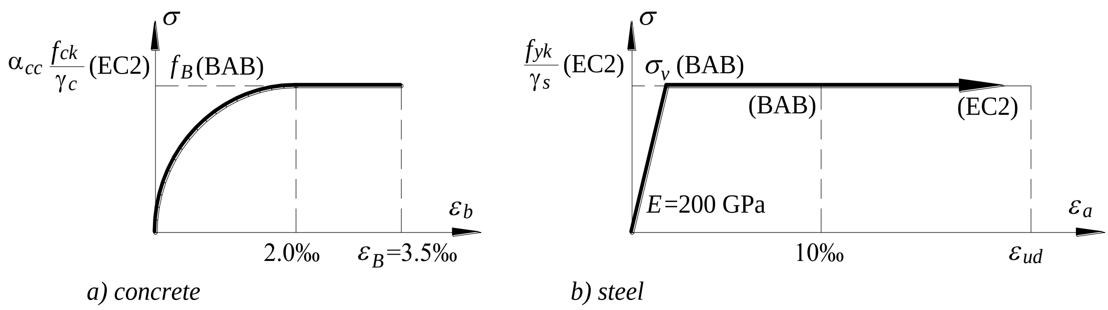

Fig. 1. Stress-strain diagrams for concrete (a) and reinforcing steel (*b*)

It is shown below that the partial safety factors for materials from EC2 together with those for loads provide similar effects as the higher safety factors for loads from BAB 87. The reduction of α_{cc} to 0.85 has small effect to the flexural capacity of the beams and moderate effect to the axial resistance of the columns.

Table 6: Ratio of the maximum compressive stresses for α_{cc} = 1,0

	C25/30 ≈ MB30	C35/45 ≈ MB45	C50/60 ≈ MB60
$\alpha_{cc}f_{ck}/\gamma_c$ (EC2) MPa	16.67	23.33	33.33
f_B (PBAB 87) MPa	20.50	27.75	33.00
$f_B/(\alpha_{cc}f_{ck}/\gamma_c)$	1.23	1.19	0.99

3. DESIGN FOR BENDING WITH OR WITHOUT AXIAL FORCE

The usual design assumptions apply in both codes: sections remain plane, the tensile strength of concrete is ignored, the strain of bonded reinforcement is equal to the strain of the surrounding concrete. The strain distribution is linear and the possible range is shown in Figure 2.

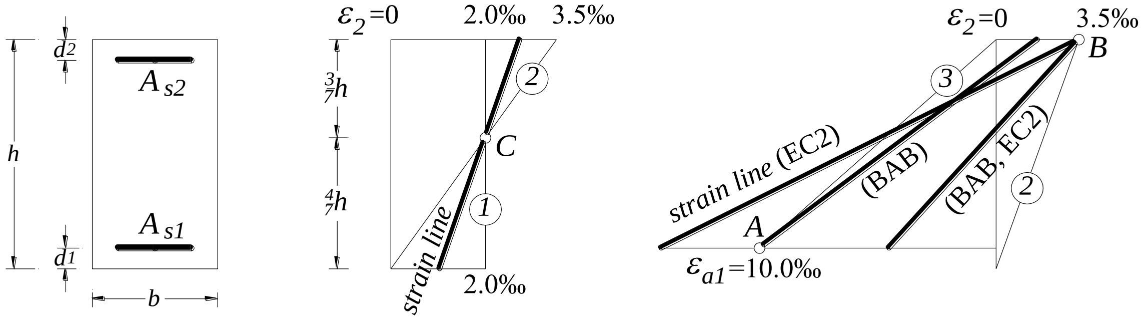

Fig 2. Possible strain distributions in the ULS: the whole section is in compression (middle); the part of section is in tension (right)

When the entire cross-section is in compression ("small eccentricity"), the strain distribution in EC2 is the same as in BAB 87. The fixed strain at 3/7 of the section height is 2.0 ‰ (for classes C12 - C50), measured from the more pressed edge (Figure 2 – middle). When the part of the cross section is in tension ("large eccentricity"), the edge strain of compressed concrete is fixed to 3.5 ‰ (limit strain of concrete), but the strain of the tensile reinforcement in EC2 is not limited as in BAB 87 (Fig. 2 – right).

3.1. Bending without axial force

The results of the analysis of the moment of resistance M_u of a singly-reinforced rectangular section are shown in Fig.3, [5]. Dimensionless values of the design resistance are plotted against the mechanical ratio of the tensile reinforcement for selected concrete grades. The notation (strength parameters and cross-sectional geometry) are according to BAB 87. None of the compression reinforcement is included.

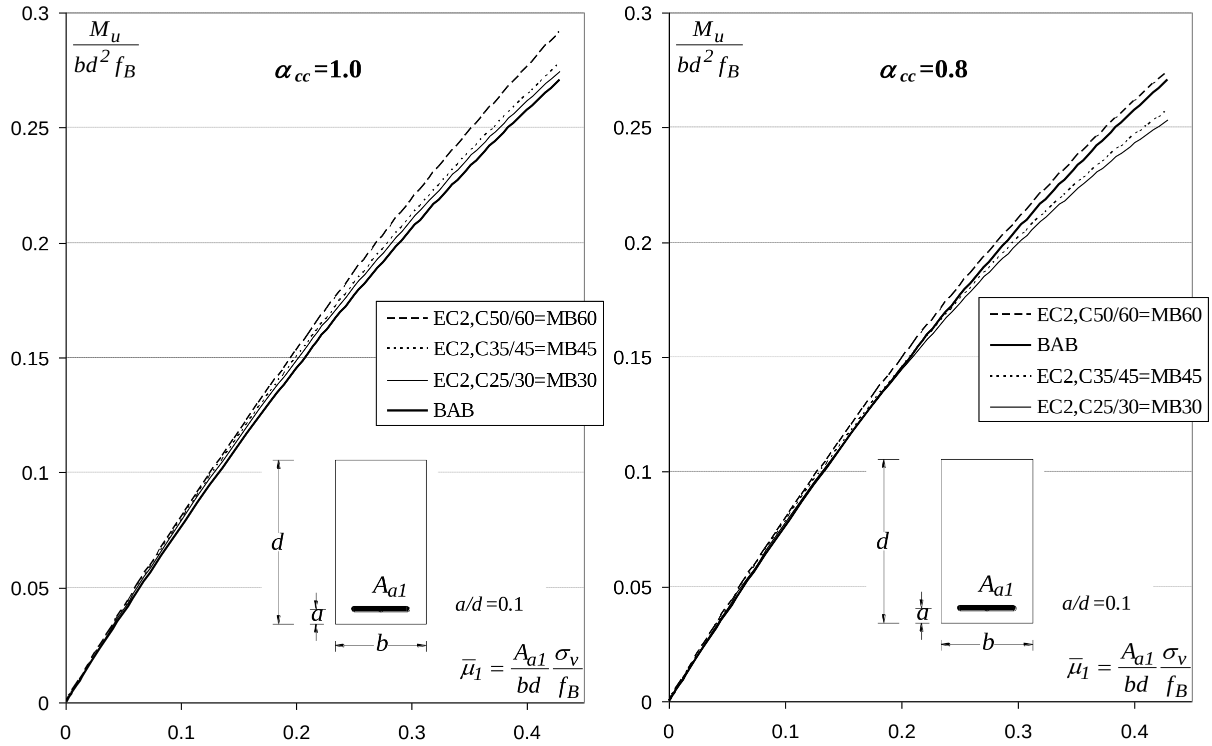

Fig. 3. The moment of resistance of singly reinforced section (α_{cc} = 1.0 and α_{cc} = 0.8)

With the selected chart axis labels in Fig. 3, one line covers all concrete grades for BAB, while EC2 requires a separate line for each concrete class, since the ratio $f_B/(\alpha_{cc}f_{ck}/\gamma_c)$ is not constant (Figure 1 and Table 6). (A particular concrete class (EC2) corresponds to a certain concrete grade (BAB 87); The mechanical ratio μ determines the reinforcement area and the resistance according to EC2 can be calculated).

It is obvious from Fig. 3 that the values for BAB 87 and EC2 are close for small and moderate amounts of tensile reinforcement for both α_{cc} = 1.0 and α_{cc} = 0.8. This is the result of a similar value of the load-to-design-steel-stress ratio. If we compare the ratio of average safety factors for loads (1.19, Table 1) with the ratio of steel stresses at ULS ($\sigma_v/(f_{yk}/\gamma_s) = \gamma_s$ = 1.15), we conclude that EC2 requires 3-4% less reinforcement for the same tensile force. But, if we take concrete C35/45 (MB45), the ratio of maximum compressive stresses is 1.19 (Table 6) and BAB 87 requires a smaller area of compressed concrete. It gives a slightly larger lever arm of the internal forces and compensates for the steel stress. However, the large reinforcement area requires a larger area of compressed concrete, so the reduction factor α_{cc} has a greater influence through the size of the internal lever arm on the calculated resistance.

3.2. Bending with axial force

Elements that are entirely under compression, such as columns, are analyzed. NDP α_{cc} has a significant effect on the compressive resistance of the member. Since it is multiplier, its effect is generally similar at any stress level and can be displayed on the selected concrete class (grade).

In Figure 4, [5], the notation according to BAB 87 is applied. The calculated values of the axial resistance $N_u/(bdf_B)$ are plotted against mechanical ratio of the total reinforcement $\mu = A_a\sigma_v/(bdf_b)$, for the eccentricity equal to $d/30$ ($M_u = N_u$×(d/30), or, $M_{Ed} = N_{Ed}$×(h/30) in EC2 notation). Again, one line covers all concrete grades for BAB 87, but EC2 requires a separate line for each concrete class or α_{cc} value. Results for MB 30 ≈ C25/30 and α_{cc} = 0.8, 0.9 and 1.0 are presented in Figure 4. The values for EC2 are multiplied by the average value of the safety factor ratio $\gamma_{PBAB\ 87}/\gamma_{EC2}$ = 1.405 (Table 1) to allow comparison in terms of load capacity.

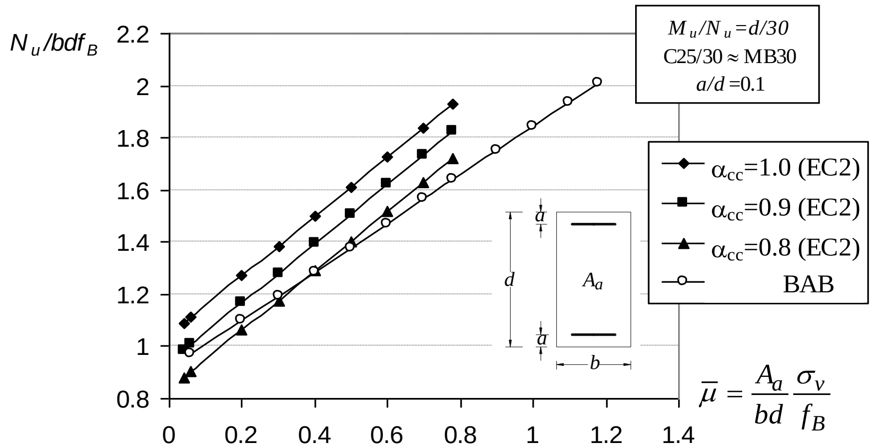

Fig. 4. Compressive resistance of symmetrically reinforced section (MB30, $e = d$/30)

Figure 4 shows that EC2 generally gives higher resistance for the same material, except for α_{cc} = 0.8 with light reinforcement. The increase in the case of maximum reinforcement (EC2: 0.04 of concrete area, also NDP) and α_{cc} = 1.0 is about 18%. Also, the ratio of overall safety factor for reinforcement (load-to-steel-stress at ULS) is BAB 87 : EC2 = 1.405/γ_s = 1.405/1.15 = 1.22, i.e. the slope of the EC2 lines in Figure 4 is 22% higher than the BAB 87 line. In other words - EC2 requires less reinforcement for the same resistance.

The ratio of the design compressive strength of concrete (f_B; $\alpha_{cc}f_{ck}/\gamma_c$) and the required specimen strength for the concrete grade (class) also significantly affects the resistance of the member. BAB 87 gradually reduces this ratio for higher grades, while for EC2 it remains constant ($1/\gamma_c$ = 1/1.5). The BAB 87/EC2 ratio is shown in Table 6.

As a result of the conservative the design strength value for high grades, EC2 shows a significant increase in the compressive resistance compared to BAB 87. The values for C25/30, C35/45 and C50/60, for α_{cc} = 1.0, are shown in Figure 5. It is apparent that the EC2 C50/60 line is high above BAB 87 (MB60) line. With a maximum (4%) reinforcement area, the EC2/BAB 87 ratio is about 1.35.

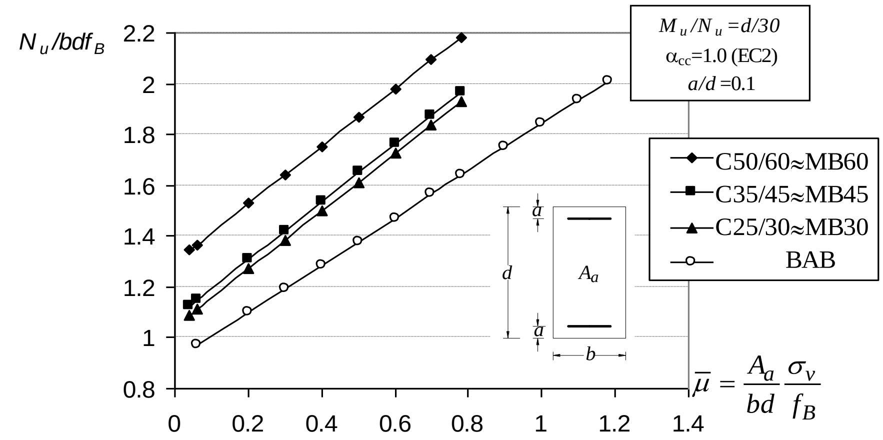

Figure 5: Compressive resistance of symmetrically reinforced section (α_{cc} = 1.0; e = d/30)

Similar results are obtained in the case of higher eccentricity *d/15* (the entire cross-section is still in compression; greater eccentricity involves tension). Favourable values of the design compressive strength of concrete and the ratio of overall safety factors again provide higher resistance according to EC2, Figure 6.

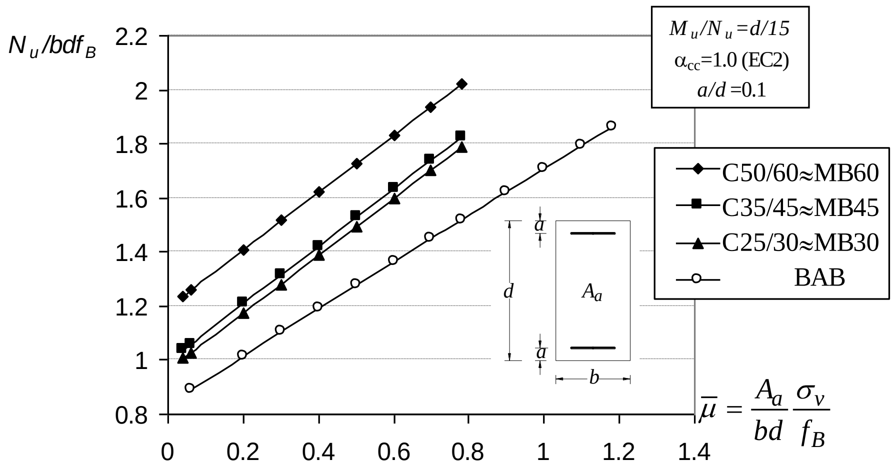

Figure 6: Compressive resistance of symmetrically reinforced section (α_{cc} = 1.0; e = d/15)

However, if α_{cc} = 0.80, the load resistance of the members in compression in BAB 87 and EC2 are quite similar, Figure 4. A similar conclusion was drawn in Chapter 3.1 for flexural elements. Therefore, no problems are expected with the assessment of existing structures regarding bending with or without axial force if α_{cc} is set lower than 1.0, as in Serbian NA to EC2.

4. DESIGN FOR SHEAR

It is assumed that reinforced concrete elements subjected to transverse load without axial compression are cracked. The design criteria for shear are expressed in terms of shear stress (BAB 87) or in terms of shear force (EC2).

BAB 87: Nominal shear stress at ULS τ_u is obtained dividing shear force by internal lever arm z and section width b_w. Shear design is based on three limits of shear stress: $\tau_u \leq \tau_r$, (design shear reinforced not required), $\tau_r < \tau_u \leq 3\tau_r$ (design shear reinforcement required; part of the shear, decreasing with level of the shear stress, is resisted by concrete), and $3\tau_r < \tau_u \leq 5\tau_r$ (total shear is resisted by reinforcement).

Limit $5\tau_r$ denotes capacity of diagonal compression which is not allowed to overcome. Shear limits τ_r are provided in relation to concrete grade MB. The procedure relies mainly on the Model Code 1978, with some modifications.

The truss model with variable inclination angle of concrete compressive struts is adopted in the current EC2. The designer is allowed to select the inclination angle within the range from 22° to 45°. This model does not apply to elements not requiring shear reinforcement and empirical equations are provided for the shear resistance of concrete $V_{Rd,c}$ in such case. Only minimum shear reinforcement should be provided where shear force at ultimate $V_{Ed} \leq V_{Rd,c}$. However, it may be omitted for slabs. In case that $V_{Ed} > V_{Rd,c}$ the concrete resistance to shear does not account any more, and the total shear should be resisted by shear reinforcement, $V_{Rd,s} \geq V_{Ed}$. V_{Ed} should not exceed the maximum shear resistance $V_{Rd,max}$ which is derived from crushing of the compressive struts.

BAB 87 and EC2 concepts are similar in case of the concrete resistance ($\tau_u \leq \tau_r$ or $V_{Ed} \leq V_{Rd,c}$) and for the maximum resistance ($5\tau_r$ or $V_{Rd,max}$), but effective limits may significantly differ in value, as it will be commented below. Within these boundaries (elements requiring design shear reinforcement), concepts are different.

4.1. Elements not requiring design shear reinforcement

The design value for the shear resistance in EC2 is given by Eq. 1:

$$V_{Rd,c} = \left[C_{Rd,c}\, k\,(100\rho_l\, f_{ck})^{1/3} + k_1\sigma_{cp}\right]b_w d \tag{1}$$

but not less than:

$$V_{Rd,c} = \left[v_{\min} + k_1\sigma_{cp}\right]b_w d \tag{2}$$

where f_{ck} is the concrete strength in MPa; $k = 1 + \sqrt{200/d} \leq 2.0$, with the structural depth d in mm; $\rho_l = A_{sl}/(b_w d) \leq 0.02$ is the reinforcement ratio for the longitudinal reinforcement; b_w is the smallest width of the cross-section in the tensile area (mm); σ_{cp} is the section stress due to axial force. The recommended value for is $C_{Rd,c} = 0.12$ and

$$v_{\min} = 0.035\, k^{3/2} f_{ck}^{1/2}. \tag{3}$$

Shear resistance depends on concrete class, structural depth and longitudinal reinforcement ratio. The range of the shear resistance by Eqs. (1,2) is evaluated for concrete classes C25/30, C35/45, C50/60, structural depth d from 200 to 600 mm, and reinforcement ratio ρ_l from 0.001 to 0.02, without axial force ($\sigma_{cp} = 0$), [6]. Obtained values of $V_{Rd,c}/b_w d$ are presented in the Table 7 (EC2min: d = 600 mm, ρ_l = 0.001; EC2max: d = 200 mm, ρ_l = 0.02). The column (4) shows maximum values of the shear stress of an element not requiring design shear reinforcement by BAB 87 (for a corresponding concrete grade, designated in column (7)). Due to comparisons, the limit stress τ_r is weighted by the ratio of ULS shear forces (1.19) and the ratio internal lever-arm-to-structural-depth (z/d = 0.9).

Table 7: Shear resistance of an element not requiring design shear reinforcement, [6].

Concrete class EC2 (1)	EC2min (MPa) $V_{Rd,c}/(b_{wd})$ (2)	EC2max (MPa) $V_{Rd,c}/(b_{wd})$ (3)	τ_r×0.9/1.19 (MPa) (4)	(5)=(4)/(2) (5)	(6)=(4)/(3) (6)	Concrete grade BAB 87 (7)
C25/30	0.35	0.88	0.83	2.40	0.94	MB30
C35/45	0.41	0.99	1.06	2.58	1.07	MB45
C50/60	0.49	1.11	1.21	2.47	1.09	MB60

Table 7 shows that, in case of a low reinforcement ratio, EC2 requires shear reinforcement at a significantly lower stress level compared to BAB 87 (column (5)). Shear resistance of the concrete of lightly reinforced elements can be 2.5 times smaller than the one that is allowed according to BAB 87, i.e. problems in verifying the load-bearing capacity of previously designed structures are likely to occur. In the case of a high ratio of longitudinal reinforcement, the shear resistance of concrete is similar, column (6).

4.2. Maximum shear resistance

The design value of maximum shear force that can be sustained by an element is limited by crushing of the compression struts. For elements with vertical shear reinforcement and the inclination of compression struts of 45°, expression (6.9) of EC2 gives:

$$V_{Rd,max} = 0.5\alpha_{cc}\, b_w\, z\, \nu f_{ck} / \gamma_c \tag{4}$$

where:

$$\nu = 0.6\left[1 - \frac{f_{ck}}{250}\right], \; f_{ck} \text{ in MPa} \tag{5}$$

Comparison of $V_{Rd,max} / z b_w$ (EC2) with $5\tau_r$ (BAB 87, weighted by the ratio of ULS shear forces (1.19)) is presented in Table 8. α_{cc} is taken 0.85.

Table 8: Maximum shear resistance (MPa), [6]

Concrete class EC2 (1)	$V_{Rd,max}/(b_w z)$ (2)	$5\tau_r/1.19$ (3)	(4) = (3)/(2) (4)	Concrete grade BAB (5)
C25/30	3.83	4.62	1.21	MB30
C35/45	5.12	5.88	1.15	MB45
C50/60	6.80	6.72	0.99	MB60

Table 8 shows that EC2's maximum shear resistance is lower than one allowed by BAB 87, except for MB60 grade (which has reduced strength parameters in BAB 87). This is due to the reduced value of $\alpha_{cc} = 0.85$. For $\alpha_{cc} = 1.0$ the values are quite similar.

4.3. Minimum area of shear reinforcement

Both EC2 and BAB 87 set the minimum area of shear reinforcement, whenever the shear capacity of concrete is exceeded. BAB 87 states that the ratio of shear reinforcement should not be less than 0.2 %. The recommended minimum shear reinforcement ratio in EC2 is given by:

$$\rho_{w,\min} = \frac{0.08\sqrt{f_{ck}}}{f_{yk}} \tag{6}$$

Calculated values of $\rho_{w,min}$ for reinforcing steels B500 and former RA 400/500 (f_{yk} = 400 MPa) and GA 240/360 (f_{yk} = 240 MPa) are presented in the Table 9.

Table 9. Minimum shear reinforcement ratio (EC2), [6].

Concrete class (1)	B500 (2)	RA 400/500 (3)	GA 240/360 (4)	Concrete grade BAB (5)
C25/30	0.080 %	0.100 %	0.167 %	MB30
C35/45	0.095 %	0.118 %	0.197 %	MB45
C50/60	0.113 %	0.141 %	0.236 %	MB60

The minimum shear reinforcement according to EC2 is in most cases significantly lower than in BAB 87, so no problems are expected here when assessing existing structures.

The amount of shear force (stress) that can be resisted by the minimum shear reinforcement is

$$\frac{V_{Rd,s,\min}}{b_w z} = \rho_{w,\min}\, f_{yd} = \rho_{w,\min} \frac{f_{yk}}{\gamma_s} = 0.0696\sqrt{f_{ck}} \quad (f_{ck} \text{ in MPa}). \tag{7}$$

This value should be multiplied by the ratio $z/d \approx 0.9$ for comparison with the values of $V_{Rd,c}/(b_w d)$. The results are shown in Table 10, [6].

It is apparent from Table 10 that minimum shear reinforcement in no case cover the shear capacity of concrete. As a result, discontinuity appears in the transition region. The required shear reinforcement in vicinity of $V_{Rd,c}$ ($V_{Ed} = V_{Rd,c}^{+}$) can be twice as large as the minimum.

Table 10: Shear resistance $V_{Rd,s,min}$ of min. shear reinforcement vs. resistance of concrete $V_{Rd,c}$ (EC2).

(MPa)	C25/30	C35/45	C50/60
$V_{Rd,s,min} / b_w d$	0.31	0.37	0.44
$V_{Rd,c}/(b_w d)$ (min-max) d = 200 mm	0.49-0.88	0.59-0.99	0.70-1.11
$V_{Rd,c}/(b_w d)$ (min-max) d = 400 mm	0.39-0.75	0.46-0.84	0.55-0.95
$V_{Rd,c}/(b_w d)$ (min-max) d = 600 mm	0.35-0.70	0.41-0.78	0.49-0.88

4.4. Elements requiring design shear reinforcement

In case that the shear stress (ULS according to BAB 87) exceeded value $3\tau_r$, the required area of vertical links is slightly bigger than one by EC2. Partial safety factor for steel γ_s = 1.15 combined with the ultimate load ratio 1.19 gives the total ratio (EC2 : BAB 87) = 1.15/1.19 = 0.966. But, with the shear ranging from τ_r to $3\tau_r$, BAB 87 takes into account the shear resistance of concrete, while EC2 accounts for the resistance of reinforcement only. Due to the reduced shear stress which the reinforcement should resist, BAB 87 requires less shear reinforcement than EC2 in the range $\tau_r \div 3\tau_r$.

Shear reinforcement ratio is given as $\rho_w = A_{sw}/(b_w s)$, where A_{sw} is the cross-sectional area of shear reinforcement at the spacing s. The required reinforcement ratio ρ_w for concrete C35/45 (MB45) and for three steel grades is presented on Figure 7a over the shear stress $V_{Ed}/(b_w z)$, [6]. Figure 7b shows the ratio of required shear reinforcement by EC2 and BAB 87. This ratio is independent of steel grade. Figure 7b refers to the concrete class C35/45.The ratio is similar for other classes.

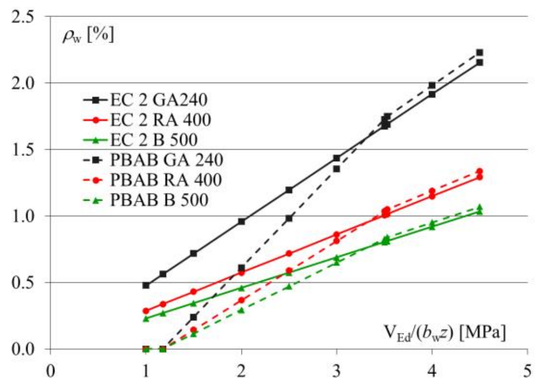

Figure 7a. Required reinforcement ratio ρ_w for C35/45 (MB45) according to EC2 and PBAB as a function of the shear stress

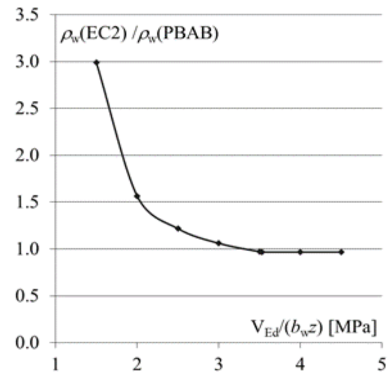

Figure 7b. Ratio ρ_w(EC2)/ρ_w(PBAB) for C35/45 (MB45) as a function of the shear stress

Figure 7b shows that elements designed for shear according to BAB 87, for lower and medium levels of shear stress, can show a large deficiency of links when assessed according to EC2.

5. CRACK CONTROL

The crack width ($a_{p,k}$) calculation procedure according to the CEB manual [3] was commonly used in combination with the BAB 87 code. Detailed instructions for the calculation are given in the Manual [7]. The crack width calculation procedure presented in EC2 follows the same approach.

The crack width is obtained as a product of the maximum crack spacing ($l_{p,k}$ or $s_{r,\max}$) and the relative mean dilatation of the tensile reinforcement $\varepsilon_{as,R}$ (the difference between the mean dilatations of the reinforcement ε_{sm} and the surrounding tensioned concrete ε_{cm}):

$$a_{p,k}\,(\text{BAB87}) = l_{p,k} \times \varepsilon_{as,R} \quad \text{or} \quad w_k\,(\text{EC2}) = s_{r,\max}(\varepsilon_{sm} - \varepsilon_{cm}) \tag{8}$$

However, the instructions for calculating the terms in equations (8) differ somewhat:

$$l_{p,k}\,(\text{BAB87}) = 1.7 \times [\,2(a_0 + 0.1 e_{\varnothing}) \;+\; k_1 k_2 \varnothing / \mu_{z,ef}\,] \tag{9}$$

$$s_{r,max}(\text{EC2}) \;=\; k_3 c \,+\, k_1 k_2 k_4 \varnothing / \rho_{p,eff} \tag{10}$$

where:

- $a_0 = c$ is the cover to the longitudinal reinforcement,
- $e_{\varnothing}$ is the spacing of the reinforcement,
- $\varnothing$ is the bar diameter,
- $\mu_{z,ef}$, $\rho_{p,eff}$ is the effective reinforcement ratio – area of the tensile reinforcement divided by the effective area of concrete in tension surrounding the reinforcement. The depth of tensioned concrete is up to 7.5$\varnothing$ from the highest row of bars (BAB 87) or 2.5 times the distance ***a*** from edge to the centroid of tensile reinforcement (EC2)
- k_1, k_2, k_3, k_4 are coefficients.

For en element subjected to bending with one layer of high bond bars of diameter $\varnothing$, and with the horizontal spacing $e_{\varnothing}$, the expression (9) becomes, [8],

$$l_{p,k}\,(\text{BAB87}) \;=\; 3.4\,a_0 \;+\, 0.11(a + 10.5\varnothing)\frac{e_{\varnothing}}{\varnothing} \tag{11}$$

Similarly, in same notation, the expression (10) becomes:

$$s_{r,\max}(\text{EC2}) \;=\; 3.4\,a_0 \;+\, 0.54\,a\,\frac{e_{\varnothing}}{\varnothing} \tag{12}$$

Expressions (11) and (12) have many similarities and, in most cases, give close values.
The relative mean dilatation is calculated in both BAB 87 and EC2 by reducing the dilatation of the tensile reinforcement $\varepsilon_s = \sigma_s/E_s$ calculated for the cracked section. However, the approach to determining the reduction factor (multiplier less than 1.0) differs somewhat.
In Manual [7], the reduction factor, denoted by ω in this text, is

$$\omega(\text{BAB 87}) \;= 1 - 0.5\left(\frac{\sigma_{s,cr}}{\sigma_s}\right)^2 \tag{13}$$

for high bond bars and long-term load. $\sigma_{s,cr}$ is the stress in the tensile reinforcement due to the crack-opening load (M_{cr}).
In EC2 the reduction factor is

$$\omega(\text{EC2}) \;=\; 1 - 0.4\,\frac{f_{ct,ef}\,A_{ct,eff}}{\sigma_s A_s}\left(1 + \frac{E_s A_s}{E_{cm} A_{ct,eff}}\right) \tag{14}$$

where $f_{ct,ef}$ is the mean value of the tensile strength of concrete at the time of crack formation.
The product $f_{ct,ef}\,A_{ct,eff}$ represents the ‘capacity’ of the tensioned concrete and the force transferred to the reinforcement when opening the cracks, i.e. $f_{ct,ef}\,A_{ct,eff} \approx \sigma_{s,cr}\,A_s$. Also, the term in parentheses is greater than 1 and its product with 0.4 may be roughly rounded to 0.5. With these approximations, expression (14) becomes

$$\omega(\text{EC2}) \;\approx 1 - 0.5\,\frac{\sigma_{s,cr}}{\sigma_s} \tag{15}$$

Comparing expressions (13) and (15), it can be seen that expression (15) gives a smaller value - the stress ratio is less than 1 and squaring in expression (13) reduces its value. Input data contribute to further widening the gap. EC2 uses the average tensile strength of concrete to determine $\sigma_{s,cr}$, while the procedure from [3,7] applies 70% of the value. Numerically, this means the following: while, according to [7], the coefficient ω was usually between 0.90 and 1, its value according to EC2 is generally considerably less than one (the limit is $\omega \geq 0.6$).

For the same element and the same load, the calculated crack width of the flexural elements according to the procedure from EC2 is smaller in comparison to the crack width according to the Manual [7] for application with BAB 87.

Basically, the reason is the higher level of tension stiffening of the concrete, which is prescribed by EC2, while the crack spacing is similar. Besides, EC2 requirements for crack width limitation are less rigorous than BAB 87, as shown in the chapter 2.5. On the whole, EC2 is less restrictive in terms of limiting the crack width of reinforced concrete elements compared to BAB 87. However, one should have in mind that a larger concrete cover is required for this.

Both codes have a procedure for indirect checking of the crack width, but neither of these has been widely used. In clause 114, BAB 87 provided a very simple, but also very conservative procedure. Combined with a relatively strict crack width limit, the procedure in most cases did not provide successful check. The Manual [7] offered an improved procedure for indirect check which, due to its noticeable complexity, has not been widely used. EC also provides simple criteria for indirect check of the crack width, shown in Table 11.

Table 11. EC2: Maximum bar diameters ∅ and maximum bar spacing $e_{\varnothing}$ for crack control

Steel stress (MPa)	w_k = 0.4 mm		w_k = 0.3 mm		w_k = 0.2 mm	
	max ∅ (mm)	max $e_{\varnothing}$ (mm)	max ∅ (mm)	max $e_{\varnothing}$ (mm)	max ∅ (mm)	max $e_{\varnothing}$ (mm)
160	40	300	32	300	25	200
200	32	300	25	250	16	150
240	20	250	16	200	12	100
280	16	200	12	150	8	50
320	12	150	10	100	6	-
360	10	100	8	50	5	-

For the selected crack width limitation and stress in the tensile reinforcement due to quasi-permanent load, the maximum bar ∅ and maximum bar spacing $e_{\varnothing}$ (Table 11) are determined. However, these criteria are applicable for rectangular cross-sections, while for T-beams with wide flange may be significantly on the unsafe side [9]. Thus, the application of Table 11 is limited, as most rectangular cross-sections refer to concrete slabs and EC2 does not require crack control for slabs up to 20 cm thick. Improved criteria from Table 11 of EC2 can be found in [9].

6. DEFLECTION CONTROL

Both codes provide simplified and refined deflection control procedures. Simplified procedures apply span/depth ratio limitation and do not require calculation of deflection. Refined procedures are based on the calculation of curvatures, taking into account cracking and long term properties of concrete, and the deflection is calculated.

BAB 87 defined in clause 118 an expression that limited the ratio of section height to span. The procedure was easily performed with the provided tabulated coefficients. However, this criterion appeared to be very conservative as it neglected the effect of tensile stiffening of concrete and the use of larger reinforcement than necessary, which is why it has not found wider application. The manual [7] offered a modified procedure that was no longer so simple as to be competitive with the calculation of deflections.

EC2 gives in clause 7.4.2 expressions for span/depth limits together with instructions for modifications concerning steel grade, additional reinforcement or large spans. The expressions were developed on the basis of a previously conducted extensive study. The procedure has been quite criticized in the previous period (for example [10]), so that, in this form, it will be omitted in the new generation of Eurocodes. However, the main objections are essentially remediable, so the procedure did not have to be abandoned. The first objection relates to the fact that the criterion may be significantly on the unsafe side. This is a direct consequence of the chosen quasi-permanent-to-ultimate-load ratio of 0.50. A more appropriate value is 0.6, which was also treated in the study, but the authors chose to present expressions for a value of 0.5 that underestimates long-term deflections, [11]. The second relates to the need to limit modification factors in order to prevent misuse of correction due to additional reinforcement. National documents of some countries have already set functional restrictions, [12].

The Manual [7] introduced the "bilinear method" according to the CEB manual [3] in common practice for calculating deflections:

$$u = \zeta \cdot u_{II} + (1-\zeta)\cdot u_I \tag{16}$$

where u_I and u_{II} are the deflections calculated for the uncracked (“state I”) and fully cracked (“state II”) conditions. Deflections u_I and u_{II} are determined using *AAEM* (*Age Adjusted Effective Modulus*) method with effective modulus of concrete $E_{c,eff}$:

$$E_{c,eff} = \frac{E_c}{1+\chi\cdot\varphi} \tag{17}$$

where

- φ is the creep coefficient relevant for the load and the time interval
- E_c is the modulus of elasticity of concrete related to the creep coefficient

and the adopted value of the ageing coefficient χ is 0.8

Manual [3] states an explicit value of the interpolation coefficient $\zeta = \zeta_b$ in the bilinear method

$$\zeta_b = 1 - 0.5\cdot\frac{M_{cr}}{M_D} \tag{18}$$

where M_D is the maximum bending moment in the span of the element. M_{cr} is the cracking moment, calculated using the mean tensile strength f_{ctm} or the flexural tensile strength $f_{ctm,fl}$, when appropriate.

EC2 indicates that the calculation of the deflection from long-term curvatures at frequent sections along the span of the element is the most accurate method of predicting deflection. However, this approach is not applicable in all cases due to lack of necessary instructions, [12]. On the other hand, an approach following equation (16) is allowed, which can always be applied. The long-term effects due to sustained load may be calculated using *EM* (*Effective Modulus*) method with effective modulus for concrete

$$E_{c,eff} = \frac{E_c}{1+\varphi} \tag{19}$$

and the interpolation coefficient is

$$\zeta = 1 - 0.5\cdot\left(\frac{M_{cr}}{M}\right)^2 . \tag{20}$$

There is no explicit instruction for selecting the moment M in Expression (20). In more recent works, the authors recommend the use of the largest moment in the span, i.e. $M = M_D$.

It should be emphasized that both methods (EC2, BAB 87 manual [7] = CEB manual [3]) give identical values of the calculated deflections, for an element under sustained load, in case that:

- The same value of interpolation coefficient ζ is used for both methods (according to the Expression (18) or (20) – this issue is discussed below);
- The value of ageing coefficient χ is taken 1,0 for the bi-linear method according to CEB manual [3];
- The same values of E_c and φ are used for both methods.

Calculation of the interpolation coefficient ζ according to Expression (20) requires a representative value of the bending moment M. ζ quantifies the tension stiffening effects of the concrete in cracked part(s) of element’s span ($M \geq M_{cr}$) according to the strain of tensile reinforcement. Locally (on a short segment of the span), this strain is proportional to the acting moment and Expression (20) is applied with the local value of the moment. For the whole element, an average value of the tension stiffening should be used. Along the cracked part of the span M varies from M_{cr} to M_D ($M_{cr} \leq M \leq M_D$). To apply in Expression (16), ζ can be estimated from expression (20) using the average value of the bending moment such as the arithmetic mean $M = ½(M_{cr} + M_D)$. Bi-linear method (CEB 1985, [3]) applies the geometric mean

$$M = \sqrt{M_{cr} \cdot M_D} \tag{21}$$

resulting in ζ_b from Expression (18). This value underestimates the deflection to some extent, in comparison to the basic model, but it provides simple tool that was suitable for hand calculation. In recent publications some authors suggest to adopt $M = M_D$. This is on the safe side, but also inconsistent with the basic model. However, in most cases both M from Expression (21) and $M = M_D$ provide a suitable value of the interpolation coefficient ζ in Expression (20) for design purposes.

Since BAB 87 gives slightly higher values of the creep coefficient compared to EC2, but also that the *AAEM* ($\chi = 0.8$) method gives slightly lower calculated values of deflection compared to *EM* method ($\chi = 1$) for the same input, the deflections calculated by these two procedures differ very little. Therefore, structures that meet the requirements of BAB 87 regarding deflections also comply with EC2 recommendations.

7. CONCLUSIONS

BAB 87 and EC2 have many conceptual similarities in their fundamentals. The main reason for this is that BAB 87 is based on the principles proclaimed in the CEB-fip Model Code 1978, while EC2 follows the same tradition, but includes the advances in science contained in the CEB-fip Model Code 1990. This makes the transition from BAB 87 to EC2 much easier for designers.

There are two basic issues related to changing design regulations. The first is how much work and effort it will take for designers to adopt design procedures under the new regulations. The second is the extent to which structures designed according to previous regulations meet the requirements of the new regulations. The second item can also be observed through the question to what extent the previous experience in the conceptual design of structures is applicable with the new regulations.

The answer to the first question is that, despite the many similarities between BAB 87 and EC2, a lot of work is needed to adopt new regulations. The education of engineers is necessary for quick and successful adaptation to new standards. National regulatory authorities should make new regulations available with a credible translation. Qualified institutions should organize appropriate training programs and provide quality manuals with clear and precise instructions.

Regarding the second question, the answer is more complex. Constructions designed according to BAB 87 regarding bearing capacity (ULS) for basic forms of loading in most cases meet the requirements of EC2. The paper underlines when it may not be fulfilled, for example in a shear design. In terms of meeting the durability requirements (SLS), significantly larger concrete covers are required compared to BAB 87, but crack widths and deflections in all cases meet EC2 requirements. Nevertheless, it should be emphasized that countries have the opportunity to significantly mitigate the transition to new regulations through the choice of NDP, and the concrete cover is among them.

However, perhaps the most important conclusion is that previous experience in the conceptual design of structures is generally valid and that the differences are mainly in the required reinforcement, but much less in the required dimensions of concrete sections. Also, although this is not the topic of this paper, it should not be forgotten that the requirements of Eurocode 8 regarding the seismic resistance of structures may be governing for certain elements.

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