SHEAR PROVISIONS FOR CONCRETE STRUCTURES ACCORDING TO EN 1992-1-1: OPEN ISSUES

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Summary: *The design of concrete members for shear without stirrups, except for the minimum required, has become a major issue worldwide. It was noticed that the shear capacity of concrete element according to the Eurocode 2 often gives significant smaller values than the one predicted by former codes. This fact brought into focus the assessment of the existing structures which were built with minimum shear reinforcement. Most of the Code provisions for shear for members without shear reinforcement are based on empirical relationships. In general, Eurocode 2 doesn't make a difference between reinforced and prestressed elements. Several clauses in EN1992-1-1 which deal with the shear design are critically reviewed in the paper.*

Key words: *Concrete structures, shear design, Eurocode 2.*

1. INTRODUCTION

Accurate prediction of the shear failure of concrete element is a challenging task. Shear transfer in the concrete structure is complex phenomenon affected by numerous parameters. To avoid potential safety concerns, design codes provide simplified rules which are usually based on conservative assumptions.
Five mechanisms of the shear transfer are identified in ACI 445R-99 [1]: 1) shear force component in the uncracked concrete pressure zone; 2) aggregate interlock or shear friction; 3) dowel action of the longitudinal reinforcement; 4) residual tensile stresses transmitted directly across cracks and 5) arch action or direct struts (in the area of supports).
The contribution of the particular mechanism depends on the specific member (beam or column) and loading conditions. For example, the quantity of the shear stress carried

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across the uncracked concrete can be significant in columns under compressive axial load, while it is relatively small in beams without axial compression since the depth of the compression zone is smaller. Dowel action should be considered when the element has high reinforcement ratios. However, it may be also significant in elements with lower reinforcement ratios, when the longitudinal reinforcement is distributed in layers. The mechanism of shear transfer at the cracks depends on the size of the element. The residual tensile stresses are important for smaller sized elements. For larger elements the friction across the cracks has a more important contribution.
Shear transfer mechanisms 1) to 4) are presented in the Figure 1. The vertical force in steel (stirrups) and the vertical component of prestressing are also shown.

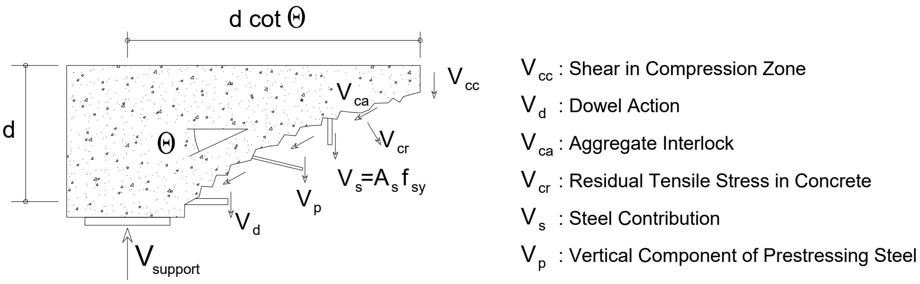

Figure 1. Shear transfer mechanisms (adjusted from [1])

2. SHEAR RESISTANCE MODELS

Since the early 1900s, engineers have used truss system made out of concrete struts and reinforcement ties to ensure equilibrium of internal forces in structural concrete elements. The original 45^0 truss model of Ritter (1899) and Mörsch (1920, 1922) has been adopted by most former international codes as the basis for shear design specifications. Mörsch pointed out that, in the parallel chord truss model, it was not possible to determine the angle of diagonal concrete strut as for there were four unknowns and only three equilibrium equations, so that the angle of diagonal (θ) was pre-set as 45^0. This selection has been shown to produce conservative results when compared with test values. Also, "it was observed through experimental research that the shear capacity of beams was greater than that predicted by this truss model by nearly a constant amount. Thus, the idea of a concrete contribution to shear resistance was introduced and linked to the diagonal cracking strength", [1].
Since the mid-1950's a large number of studies have been conducted on the shear resistance of concrete elements that lead to refined analytical models.
CEB-FIP Model Code 1978 [2] utilised the so-called "Variable Strut Inclination Method" approach with the nominal shear strength of reinforced or prestressed concrete beams with shear reinforcement as $Vn = Vs + Vc$ where Vs was strength provided by the shear reinforcement and Vc represented an additional concrete contribution which was a function of the shear stress level. This approach was implemented in the former Yugoslav/Serbian code for concrete structures PBAB 87 [3]. It was also applied in ENV 1992-1-1:1991 [4], but with constant value of Vc.

Common truss models adopt that the concrete compression struts are parallel to the direction of cracking and that no stresses are transferred through the cracks.
The latest models attempt to fulfil the equilibrium, the compatibility conditions and stress-strain relationship for materials. This concept yields to a set of nonlinear equations intended to determine the angle of the compression struts θ, named the modified compression field theory (MCFT). The angle θ at failure depends on the cross-sectional dimensions, the amount of reinforcement (both transverse and longitudinal) and the bending moment related with the shear force acting at the considered section. MCFT accounts for the tensile stresses carried by cracked concrete and can predict shear behaviour even for elements without shear reinforcement.
MCFT accounts for the concrete contribution as the vertical component of the shear stress transferred across the crack, while the traditional model (as in the CEB-FIP Model Code 1978 [2]) applies the diagonal cracking strength to account for the concrete contribution.
MCFT model is rather complex to be implemented in a code of practice. Model Code 2010 [5] offers simplified options with different levels of complexity.
Simpler models omit factors that are considered to be of minor impact. But, due to complexity of the shear transfer, a factor that is secondary in one case may be of major impact in another. The simple traditional truss model is an oversimplification of a complex problem as it neglects key variables.
The models from various codes sometimes seem different, but the basic difference is that they have been based on the different simplifying assumptions. In general, most of the code provisions for shear are based on empirical relationships (for members without shear reinforcement), and on the truss model or a combination of truss and empirical models (for members with shear reinforcement). Empirical equations for members without shear reinforcement typically involve the following parameters: the concrete tensile strength, the depth of the element (to account for size effect), the longitudinal reinforcement ratio and the axial force or amount of prestress.
Considerable differences exist in empirical equations in various codes. That is generally result of the uncertainty in assessing the influence of particular parameter in a simple equation. The problem arise from interpretation of experiments which are performed, in most cases, on the scaled specimens that do not reflect entirely properties of the actual structures.
The truss model with variable inclination angle of concrete compressive struts is adopted in the current European Code EN 1992-1-1 [6]. The designer is allowed to select the inclination angle within the range from 21.8° to 45°. This model does not apply to elements without shear reinforcement and empirical equations are provided in such case. Also, the shear capacity of elements without shear reinforcement $V_{Rd,c}$ may be evaluated from stress analysis, but this clause (6.2.2(2)) applies only to the prestressed elements which are uncracked in the ultimate limit state. 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 for any more.
In the chapter 3, differences between former Yugoslav/Serbian code PBAB 87 and EN1992-1-1 (EC2) regarding shear design are discussed. It is of interest when existing structures are evaluated. In the chapter 4 effects of the axial force on the shear capacity according to EC2 are analysed.

3. REINFORCED CONCRETE ELEMENTS WITHOUT AXIAL FORCE

Reinforced concrete elements without axial compression subjected to transverse loading are assumed to be cracked.
Former Serbian code PBAB 87 applied bigger values of partial safety factors for loads in comparison to those in Eurocodes (to account for absence of safety factors for material properties), with average ratio of 1.19 [7]. PBAB concrete compression grade **MB** fairly good corresponded to EC2 concrete class $Cf_{ck,cyl}/\boldsymbol{f_{ck,cube}}$ established on 150 mm cube, and steel grade had an equivalent definition [7]. Nominal shear stress at ULS τ_u was obtained dividing shear force by internal lever arm z and section width b_w. Shear design was 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, was resisted by concrete), and $3\tau_r < \tau_u \leq 5\tau_r$ (total shear was resisted by reinforcement). Limit $5\tau_r$ denoted capacity of diagonal compression which was not allowed to overcome. Shear limits τ_r were provided in relation to concrete grade MB. The procedure was generally relied upon CEB-FIP Model Code 1978 [2], with some modifications.

3.1. Elements not requiring design shear reinforcement

The design value for the shear resistance in EC2 is given by (numeration of formulas from EC2 is according to [5]):

$$V_{Rd,c} = \left[C_{Rd,c}\, k\, (100 \rho_l\, f_{ck})^{1/3} + k_1\, \sigma_{cp} \right] b_w d \qquad \text{EC2(6.2a)}$$

but not less than:

$$V_{Rd,c} = \left[v_{\min} + k_1\, \sigma_{cp} \right] b_w d \qquad \text{EC2(6.2b)}$$

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. The tensile reinforcement that can be included into area A_{sl} (mm^2) should extend beyond the section considered for a specified distance; 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, $\sigma_{cp} = N_{Ed}/A_c < 0.2 f_{cd}$. The recommended value for is $C_{Rd,c} = 0.18/\gamma_c = 0.18/1.5 = 0.12$

$$v_{\min} = 0.035\, k^{3/2} f_{ck}^{1/2}. \qquad \text{EC2(6.3N)}$$

Shear resistance depends on concrete class, structural depth and longitudinal reinforcement ratio. The range of the shear resistance by Eqs. (6.2a,b) is evaluated for concrete classes C25/30, C35/45, C50/60, structural depth d from 200 to 600 mm, and reinorcemet ratio ρ_l from 0.001 to 0.02, without axial force ($\sigma_{cp} = 0$). Obtained values of

$V_{Rd,c}/b_w d$ are presented in the Table 1 ($EC2_{min}$: d=600 mm, ρ_l = 0.001; $EC2_{max}$: 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 Serbian code PBAB (for a corresponding concrete grade, 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 1: Shear resistance of an element not requiring design shear reinforcement (the limit shear stress)

Concrete class EC2 (1)	$EC2_{min}$ (MPa) $V_{Rd,c}/(b_w d)$ (2)	$EC2_{max}$ (MPa) $V_{Rd,c}/(b_w d)$ (3)	$\tau_r\times0.9/1.19$ (MPa) (4)	(5)=(4)/(2) (5)	(6)=(4)/(3) (6)	Concrete grade PBAB 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 1 shows that, in case of a low reinforcement ratio, EC2 requires shear reinforcement at a significantly lower stress level compared to PBAB (column (5)). Shear resistance of the concrete of ligthtly reinforced elements can be 2.5 times smaller than the one that was allowed according to former Serbian code, i.e. there may be a problem in verification of the load-bearing capacity of previously designed structures, if required. In case of a high reinforcement ratio of longitudinal reinforcement, the shear resistance is similar, col. (6).

3.2. 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:

$$V_{Rd,max} = 0.5\alpha_{cc}\, b_w\, z\, \nu f_{ck} / \gamma_c \qquad \text{EC2(6.9)}$$

where:

$$\nu = 0.6\left[1 - \frac{f_{ck}}{250}\right], \quad f_{ck} \text{ in MPa} \qquad \text{EC2(6.6)}$$

The recommended value for α_{cc} is 1, for non-prestressed structures ((6.11aN) of EC2), while Serbian NA to EC2 [8] states α_{cc} =0.85. Comparison of $V_{Rd,max}/zb_w$ (EC2) with $5\tau_r$ (PBAB, weighted by the ratio of ULS shear forces (1.19)) is presented in Table 2.

Table 2: Maximum shear resistance (MPa)

Concrete class (1)	$V_{Rd,max}/(b_w z)$ (2)	$5\tau_r/1.19$ (3)	(4) = (3)/(2) (4)	Concrete grade PBAB (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 2 shows that EC2's maximum shear resistance is lower than one allowed by code PBAB. However, it is consequence of the reduced value of α_{cc} =0.85. Values for C50/60 match as a result of reduced strength parameters for MB60 in PBAB.

3.3. Minimum area of shear reinforcement

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

$$\rho_{w,\min} = \frac{0.08\sqrt{f_{ck}}}{f_{yk}} \qquad \text{EC2(9.5N)}$$

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 3.

Table 3: Minimum shear reinforcement ratio (EC2)

Concrete class (1)	B500 (2)	RA 400/500 (3)	GA 240/360 (4)	Concrete grade PBAB (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 PBAB so no problems should be expected here when evaluating existing structures. The amount of shear force (stress) that can be resisted by the minimum shear reinforcement is

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

This value should be multiplied by the ratio $z/d \approx 0.9$ for comparison with the values of $V_{Rd,c}/(b_w d)$.

Table 4: Shear resistance $V_{Rd,s,min}$ of minimum 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

It is apparent (Table 4) 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.

3.4. Elements requiring design shear reinforcement

In case that the shear stress (ULS according to PBAB) exceeded value 3 τ_r, the required area of vertical links was slightly bigger than one by EC2. Partial safety factor for steel γ_s = 1.15 (EC2) combined with the ultimate load ratio 1.19 gives the total ratio (EC2 : PBAB) = 1.15/1.19 = 0.966. But, with the shear ranging from τ_r to 3 τ_r, PBAB took into account the shear resistance of concrete, while EC2 accounts for the resistance of reinforcement only, since $V_{Ed} > V_{Rd,c}$. Due to reduced value of the shear stress sustained by the reinforcement, PBAB 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 2a over the shear stress $V_{Ed}/(b_w z)$. Figure 2b shows the ratio of required shear reinforcement by EC2 and PBAB. That ratio is independent of steel grade. Figure 2b also refers to the class C35/45.The ratio is similar for other classes.

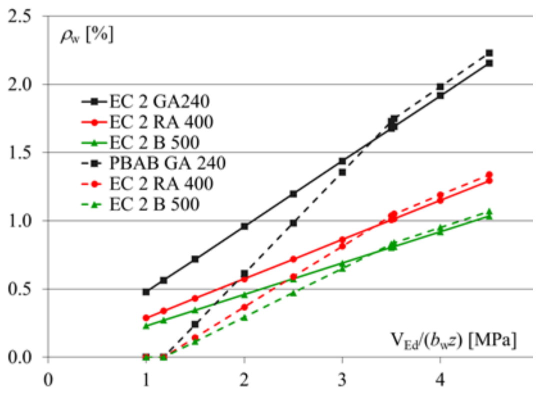

Figure 2a. Required reinforcement ratio ρ_w for C35/45 (MB45) according to EC2 and PBAB as a function of the shear stress

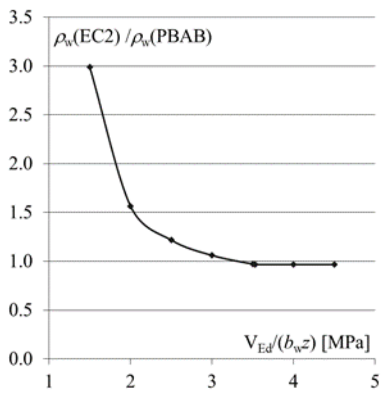

Figure 2b. Ratio $\rho_w(EC2)/\rho_w(PBAB)$ for C35/45 (MB45) as a function of the shear stress

Figure 2b shows that elements designed for shear according to PBAB, for lower and medium levels of the shear stress, can exibit large deficiency of links when evaluated according to EC2.

4. DESIGN OF CONCRETE ELEMENTS WITH AXIAL COMPRESSION

EC2 provides two diferent approaches for elements not requiring design shear reinforcement. The first approach is intended for cracked elements and follows the Eqs. (6.2a,b) to calculate the shear resistance. The second one is restricted to single span prestressed elements that remain uncracked due to bending (in shear zone). The second approach is given by Eq. (6.4)

$$V_{Rd,c} = \frac{Ib_w}{S}\sqrt{(f_{ctd})^2 + \sigma_{cp} f_{ctd}} \qquad \text{EC2(6.4)}$$

where: I is the second moment of area; b_w is the width of the cross-section at the centroidal axis; S is the first moment of area above and about the centroidal axis; σ_{cp} is the concrete compressive stress at the centroidal axis due to axial loading and/or prestressing.
Both the principal tensile stress and the flexural tensile stress are limited to the value $f_{ctd} = 0.7 f_{ctm}/1.5$, where f_{ctm} is the mean tensile strength of concrete. (Eq. (6.4) calculates the shear resistance from the condition that the principal tensile stress at centroid of the section equals f_{ctd}.
No explanation has been provided why this approach is not permitted for columns under compression. It will be shown below that this approach, in some cases, can provide a significantly a higher shear resistance compared to that of the Eqs. (6.2a,b).
For elements (with axial compression) requiring design shear reinforcement, both codes follow the truss model, and conclusions presented in chapters 3.2 and 3.4 remain in force. Certain modifications are provided in EC2 for Eq.(6.9) in case of prestressed elements, but with minor effect on the conclusions. Further consideration of these elements is not presented below.

4.1. Effects of axial compression on design of cracked concrete elements not requiring design shear reinforcement

Shear resistance is enlaged when the axial compression σ_{cp} is introduced into Eqs. (6.2a,b). Comparisons of the shear resistance of elements with and without axial compression, for various depths are presented on the Figures 3a,b. The maximum axial compression to be used in Eqs. (6.2a,b) of $0.2 f_{cd}$ is applied, so that maximum shear resistance is obtained for concrete class C35/45. The maximum relative effect of axial compression to the shear resistance by Eqs. (6.2a,b) is presented on Figure 3b. It is apparent that the resistance can be about twice as large as the one without axial compression.

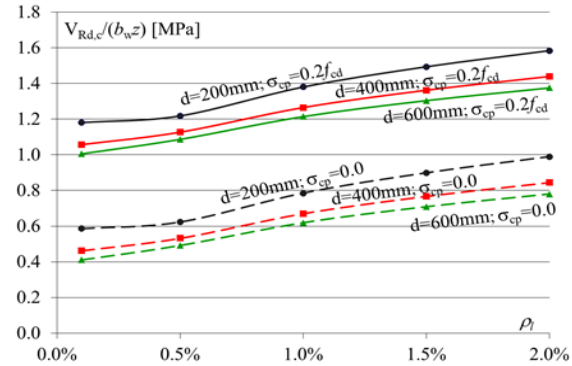

Figure 3a. Shear resistance of the element not requiring shear reinforcement for C35/45 (MB45) according to EC2 as a function of the longitudinal reinforcement ratio

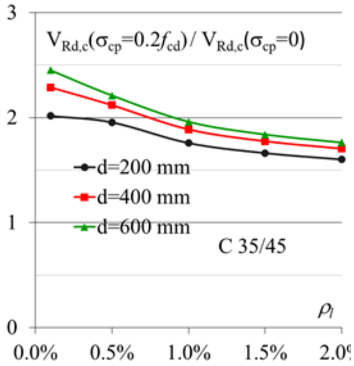

Figure 3b. Ratio $V_{Rd,c}(\sigma_{cp}=0.2f_{cd})$/ $V_{Rd,c}(\sigma_{cp}=0.0)$ as a function of the longitudinal reinforcement ratio

4.2. Effects of axial compression on design of uncracked concrete elements not requiring design shear reinforcement

The second EC2's model for evaluation of the shear resistance, presented by Eq. (6.4) and intended for uncracked prestressed elements, enables to account for bigger levels of axial compression than $0.2f_{cd}$. Figures 4a,b demonstrate differences in the shear resistance by two models (uncracked (2) vs. cracked (1)). Figure 4a shows the impact of model change only: ratio $V_{Rd,c}(2)/V_{Rd,c}(1)$ is calculated for σ_{cp} = $0.2f_{cd}$ in both models. The uncracked model provides about 1.5-2 times higher shear resistance than the cracked model, for the same data. Figure 4b shows the impact of the higher levels axial compression σ_{cp} = $(0.2\div0.8)f_{cd}$ on the shear resistance: $V_{Rd,c}(2)$ increase with the axial stress, while $V_{Rd,c}(1)$ remains at the level corresponding to σ_{cp} = $0.2f_{cd}$ (the reinforcement ratio ρ_l is set to 1%). The ratio $V_{Rd,c}(2)/V_{Rd,c}(1)$ goes up to 3. However, as previously mentioned, this beneficial effect is not intended for use with columns.

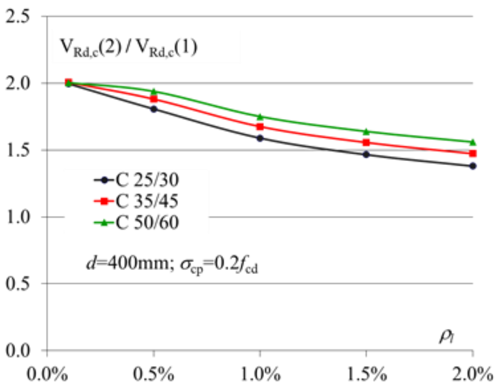

Figure 4a. Ratio of the shear resistance of uncracked/cracked element for elements not requiring shear reinforcement

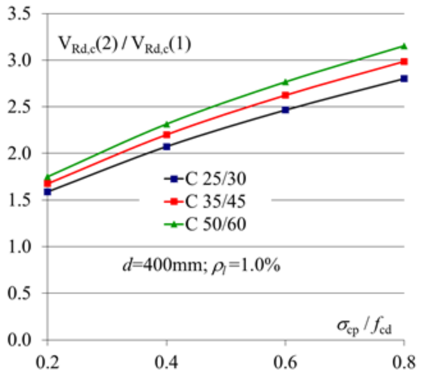

Figure 4b. Ratio of the shear resistance of uncracked/cracked element as a function of axial stress

5. CONCLUSION

The paper considers some issues related to the shear design of concrete elements according to EN 1992-1-1 (EC2). The development of the shear transfer models for concrete elements is briefly presented. Two main topics are discussed.
SRPS EN 1992-1-1 introduced significant changes in the shear design compared to the previous Serbian code PBAB 87. Differences in the shear design according to PBAB 87 and according to EC2 are discussed in relation to the shear stress level. The shear resistance of concrete elements without design shear reinforcement determined by EC2 can be significantly lower than the one calculated according to PBAB. Consequently, EC2 requires design shear reinforcement at the lower level of the shear stress.
Once the shear resistance of concrete is exceeded, EC2 requires that the whole shear force is resisted by the reinforcement, while PBAB accounted for the concrete resistance. As a result, PBAB required less shear reinforcement, for a medium level of the shear stress. It is pointed out in the literature [9,10] that this leads to very conservative results when compared with experiments on lightly shear-reinforced beams.
Both maximum shear resistance of concrete and required shear reinforcement at high levels of the shear stress are similar for two codes. Minimum shear reinforcement required by EC2 is lower than that set by PBAB, and does not reach the shear resistance of the concrete.
The assessment of existing structures designed according to PBAB may, in indicated cases, show a lack of shear reinforcement compared to that required by EC2.
EC2 accounts for the axial force when calculating the shear resistance. The effect of axial compression on shear resistance depends on the level of axial stress. Two models are provided for concrete elements not requiring shear reinforcement. The first model is general and intended for cracked elements. It has been shown that the shear resistance can be significantly increased in the case of a moderate compressive stress of $0.2f_{cd}$. However, the design shear capacity of the first model remains limited regardless of a further increase in axial compression. Unlike that, the second model can account for higher levels of the compressive stress. It can be applied to elements without cracks due to flexure in the shear zone. The shear resistance is determined based on the tensile strength of concrete and can be significantly larger than that calculated according to the first model. Despite the clear mechanical model, this approach is restricted to one span uncracked prestressed elements and cannot be used for columns.

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КОНТРОЛА НА СМИЦАЊЕ БЕТОНСКИХ КОНСТРУКЦИЈА ПРЕМА ЕН 1992-1-1: ОТВОРЕНА ПИТАЊА

Резиме: *Доказ носивости бетонских елемената на смицање који имају само минималну попречну арматуру постао је учестао проблем. Примећено је да је носивост на смицање бетонског елемента без прорачунске арматуре за смицање према Еврокоду 2 може бити знатно мање вредности од оне предвиђене ранијим кодовима. То је створило проблем при процени носивости постојећих конструкција које су изведене без осигурања арматуром за смицање. Генерално, Еврокод 2 не прави суштинску разлику између армирано-бетонских и претнодно напрегнутих елемената и у већини питања третира их на исти начин. Неколико клаузула у ЕН1992-1-1 које се баве контролом смицања су критички разматране у раду.*

Кључне речи: *Бетонске конструкције, прорачун на смицање, Еврокод 2*