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INNOVATION OF PARAMETER α_{cc} FOR DESIGN RESISTANCE OF HIGH-STRENGTH CONCRETE COLUMNS

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Original scientific paper

The usage of high-strength concrete has been increased significantly in recent decades. By increasing the compressive strength of concrete, stress-strain features of concrete are changed. In contemporary technical regulations specific provisions for high-strength concrete are proposed. EN 1992-1-1 comprises concretes to the class C90/105. According to EN 1992-1-1 reduction factor for value of concrete compressive strength for design α_{cc}, depends on two parameters, none of which compressive strength of concrete. Test results have shown that calculation of bearing capacity of columns, with recommended α_{cc} and curved-rectangular stress block is non-conservative for high-strength concrete. It was shown that coefficient α_{cc} should be redefined by introducing of new parameter which will cover differences of less design strength for high strength concrete. In many countries value of 0,85 is used for α_{cc}, as proposed in their National Documents. Correlation between calculated capacity of columns and experimental results was used for its redefining.

Keywords: *column; design compressive strength; EN 1992-1-1; high-strength concrete; parameter α_{cc}*

Inoviranje parametra α_{cc} za proračun otpornosti stupova od betona visoke čvrstoće

Izvorni znanstveni članak

Uporaba betona visoke čvrstoće znatno je porasla u posljednjih nekoliko decenija. Povećanjem tlačne čvrstoća betona, naponsko-deformacijska svojstva betona se mijenjaju. U suvremenim tehničkim propisima predlažu se posebne odredbe za beton visoke čvrstoće. EN 1992-1-1 obuhvaća betone do klase C90/105. Prema EN 1992-1-1 faktor redukcije za vrijednost projektne tlačne čvrstoće betona α_{cc}, ovisi o dva parametra, od kojih niti jedan nije tlačna čvrstoća betona. Rezultati ispitivanja su pokazali da izračun nosivosti stupova, s preporučenom α_{cc} i naponsko-deformacijska veza oblika parabola-pravokutnik je ne-konzervativna za beton visoke čvrstoće. Pokazalo se da koeficijent α_{cc} treba redefinirati uvođenjem novog parametra, koji će pokriti razlike za manju proračunsku čvrstoću betona visoke čvrstoće. U mnogim zemljama koristi se vrijednost 0,85 za α_{cc}, kako je predloženo u njihovim nacionalnim dokumentima. Korelacija između izračunatog kapaciteta stupova i eksperimentalnih rezultata korištena je za njegovo redefiniranje.

Ključne riječi: *beton visoke čvrstoće; EN 1992-1-1; parametar α_{cc}; projektna tlačna čvrstoća; stup*

1 Introduction

The usage of high-strength concrete (>60 MPa) has increased in the last decades, due to the progress of the concrete technology, and also to the adoption of the design codes including concrete classes more than C50/60. Higher strength concrete has influence on dimensions decrease, and at the same time on the prices reduction of the embedded materials. Also, the important factor favoring the usage of high-strength concrete is better durability, which has influence on the prolongation of the exploitation time. The adoption of EN 1992-1-1 (EC2) is the prerequisite for the usage of high-strength concrete. This standard prescribes the usage of concrete with compressive strength to 90 MPa [1].

The special issue is the definition of particular design parameters referring to the high-strength concrete. According to EN 1992-1-1 (2004) design parameters for the concrete with strength up to C50/60 and for concrete class more than C50/60 are mostly different. Most of them include correction in dependence on the compressive strength of concrete. However, there are parameters where identical dependence is adopted between particular design values, no matter the concrete class. Such is the same case with design compressive strength of concrete.

The high-strength concrete is the most frequently used for the compressed structural elements, for columns of high-rise buildings, or structural bridge elements. It becomes an attractive alternative to other construction materials. Various concrete stress blocks have been proposed in the literature for designing cross-sections of HSC elements under compression and the adoption of different stress blocks can lead to quite different theoretical axial load-bending moment interaction diagrams.

Results of many experiments have shown the important differences in behavior between the high-strength concrete and normal-strength concrete. Stress-strain diagrams for uniaxial compression are quite different among high and normal strength concrete. The shape of the ascending branch becomes steeper and longer for concretes with higher strength. Strain at peak stress increases with the increasing of the concrete strength, while the ultimate strain of concrete will become a bit lesser for higher strength. The shape of softening branch becomes steeper, too. It is known very well that the stress-strain relationship of concrete has an important effect on design parameters, [2÷5]. Therefore, the idealized stress-strain diagrams established for normal-strength concretes may not be applicable to high-strength concrete, especially for columns in compression. Representative stress block requires for high-strength concrete to reflect concrete stress-strain characteristics reasonably accurately [6÷8].

Differences among concrete compressive strength measured on cylinder and strength in-place were registered for high-strength concrete. Experimental data are available for in-place strength of high-strength concrete in columns, obtained from tests of plain concrete columns, and they show a scatter of values between 0,87 and 0,97 of concrete compressive strength. The design issues for normal-strength concrete must be verified and extended for high-strength concrete [6÷9].

2 Design of load carrying capacity of columns

The basic parameters of design of reinforced-concrete and pre-stressed structures, exposed to compression or combination compression-bending, are characteristic as well as design concrete compressive strength, stress–strain relationship, ultimate strain of concrete and idealized design diagrams. The compressive strength of concrete directly affects the load carrying capacity of reinforced concrete members subjected to compression and combined compression and bending.

2.1 Proposing of the design compressive strength of concrete

EN 1992-1-1 uses the characteristic compressive strength of concrete (f_{ck}) as the basis of design calculations. It is defined on the basis of compressive strength results, measured on cylinders 15/30 cm or cubes of 15 cm. It is a characteristic compressive strength of concrete, defined as that strength below which 5% of all possible strength measurements for the specified concrete may be expected to fall, [10]. Design compressive strength of concrete (f_{cd}), according to EN 1992-1-1, is defined using Eq. (1):

$$f_{cd} = \alpha_{cc} \frac{f_{ck}}{\gamma_C}, \tag{1}$$

where: α_{cc} is the coefficient taking account of long term effects on the compressive strength and of unfavorable effects resulting from the way the load is applied; γ_C is the partial safety factor for concrete.

EN 1992-1-1 proposed that coefficient α_{cc} should lie between 0,8 and 1,0. In addition, the recommended value is 1,0. This suggestion is based on the argument that there are no important long-term effects which are not included over the data which stress-strain diagrams are made from [11].

Studies have shown that the reduction coefficient for design strength of concrete should be analyzed in dependence on some other parameters which are not included according to EN 1992-1-1. The new correction factor should be included in α_{cc}, and should comprise other relevant parameters. The actual compressive strength in the structure is considered to be less than that obtained by testing standard 2:1 cylinders. This difference is more expressed for concretes with higher strength. These should be included in innovated factor, too.

The introduction of reduction factor should be related to the idealization of shape of stress-strain curve, so that the area under the real curve is approximately equal to the area under the idealized curve, Fig. 1. The maximum stress level for the idealized curve must be below the maximum stress of the real curve, [10].

Moreover, analysis of the data on the behavior of compressed zone at failure, suggests that the use of 1,0 as value of coefficient α_{cc} is un-conservative. The coefficient α_{cc} may be changed by amendments in a National Annex when adopted by an individual country. Accordingly, design compressive strength of concrete in compressed construction elements should be adjusted by correction of parameter α_{cc}. For that reason, many countries in Europe have adopted the value of α_{cc} as 0,85. According to the National Annex UK (United Kingdom) for α_{cc} the proposed value is 0,85, as is proposed by CEB Model Codes, [2, 10, 11]. National Annex of Belgium recommends the use of the value α_{cc} = 0,85 for axial load, bending and combined axial force with bending. For other types of load (shear and torsion), α_{cc} =1,0 should be used [12]. Some other European members, however, did not recognize the importance of coefficient α_{cc} in that sense. The National Annex of France prescribes the value 1,0 for all concrete structures, [13].

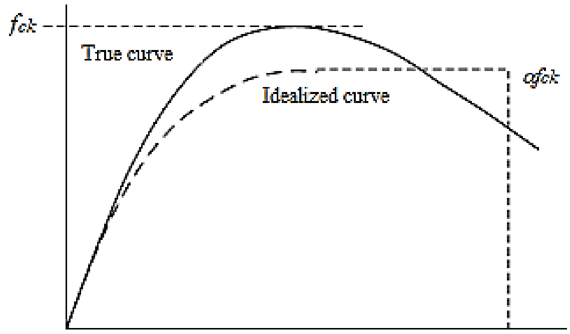

Figure 1 Real and idealized stress-strain diagram [11]

2.2 Stress-strain relationship of concrete

EN 1992-1-1:2004 permits the usage of high-strength concrete of the class to C90/105, where 90 is characteristic concrete compressive strength measured on cylinder, and 105 presents the characteristic compressive strength measured on the cube, in N/mm^2. The application of concrete of higher class than C50/60 requires modification of idealized stress-strain diagrams. The basic reason for such approach is more expressed brittle behavior for the high-strength concrete.

EN1992-1-1(2004) permits curved-rectangular idealized concrete stress blocks. Idealized stress-strain relationship is the parabola – rectangle diagram. Its general form, which is valid for all concrete classes, is:

$$\sigma_c = f_{cd}\left[1-\left(1-\frac{\varepsilon_c}{\varepsilon_{c2}}\right)^n\right], \text{ for } 0 \le \varepsilon_c < \varepsilon_{c2} \tag{2}$$

$$\sigma_c = f_{cd}, \text{ for } \varepsilon_{c2} \le \varepsilon_c \le \varepsilon_{cu2} \tag{3}$$

where: n − exponent which varies from 2,0 to 1,4, depending on the concrete strength (it is reduced with the strength increase); ε_{c2} − compression strain at reaching the maximal stress; ε_{cu2} − ultimate strain, which is also reduced with the increase of concrete strength.

EN 1992-1-1 prescribes the usage of the bilinear or rectangular diagrams. Idealized stress – strain diagrams are presented in Fig. 2.

Numerical parameters which are necessary for definition of diagrams have the constant values for concretes to C50/60, and for the concrete of more classes they are changed in dependence on the class. Numerical values for main classes are given in Tab. 1. Ultimate strains ε_{cu2} and ε_{cu3} for diagrams parabola – rectangle and bilinear diagram have the same values.

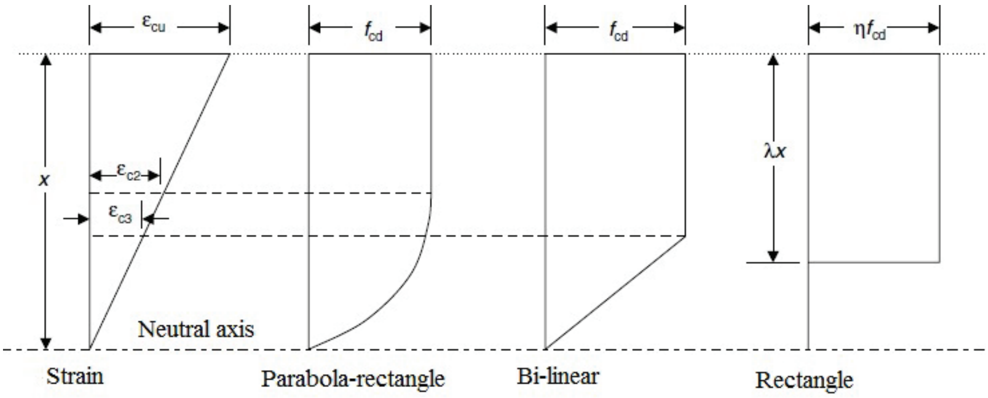

Figure 2 Idealized stress – strain diagrams according to EN 1992-1-1 [1, 11]

Table 1 Parameters for definition of stress-strain diagrams in Fig. 2 [1, 11]

Concrete class	ε_{cu2}, ε_{cu3}	ε_{c2}	ε_{c3}	n	λ	η
≤C50/60	0,0035	0,0020	0,00175	2,0	0,800	1,000
C55/67	0,0031	0,0022	0,00180	1,75	0,788	0,975
C60/75	0,0029	0,0023	0,00190	1,60	0,775	0,950
C70/85	0,0027	0,0024	0,00200	1,45	0,750	0,900
C80/95	0,0026	0,0025	0,00220	1,40	0,725	0,850
C90/105	0,0026	0,0026	0,00230	1,40	0,700	0,800

EN 1992-1-1 prescribes the reduction of the design compressive strength of concrete using coefficient η in the case of application of the rectangular stress block diagram. However, for curved-rectangular stress block diagram this reduction is not proposed. According to that, for greater values of f_{ck}, the theoretical strengths given by the rectangular stress block are smaller than the ones obtained using the curved-rectangular concrete diagram of the same code, [9]. In this case calculation of carrying capacity of concrete elements with axial compression may be un-conservative. In reference [9] it was showed that coefficient N_{exp}/N_{teo} decreases with increasing f_{ck}, if curved-rectangular stress block was used. N_{exp} represents the axial load carrying capacity of elements subjected to pure axial load or to combined axial load and bending moment. N_{teo} represents the theoretical values of axial load force obtained considering those compressive stress diagrams for the concrete. If rectangular stress block was adopted, N_{exp}/N_{teo} tends to increase with increasing f_{ck}. The analysis of that coefficient gives an idea about the level of safety related to the different design procedures. It may be concluded that the level of safety depends on the value adopted for the coefficient α_{cc} since it can lie between 0,8 and 1,0 [9].

According to ACI 318-11 the design strength refers to the nominal strength multiplied by the strength reduction factor (ϕ), which is always less than 1. This factor ϕ comprises effects of variations in material strength and dimensions, in accuracies in design equations, degree of ductility and the importance of the member in the structure [14].

CEB-FIP Model Code 90 proposed use of concretes up to cylinder characteristic strength of 80 MPa. It permits curved-rectangular idealized diagram for concrete stress distribution and simplified rectangular one. Stress-strain curves for concrete, which have been proposed according to CEB-FIP MC90 are similar to current EN1992-1-1, as shown by Eq. (2). In CEB-FIP MC90 0,85 f_{ck} is used, instead of f_{cd}. Coefficient 0,85 is described as parameter that takes account of unfavorable effects of long term loads. The strain in the concrete at the peak stress, the ultimate concrete strain, considered in this curve, varies with f_{ck}. Proposed values of the stresses in cross section, for simplified rectangular diagram of stresses, are adopted depending on the f_{ck}.

FIB Model Code for Concrete Structures 2010 covers concretes up to a characteristic strength of 120 MPa. That is normal strength concrete (NSC, $f_{ck} \leq 50$ MPa) and high strength concrete (HSC, $f_{ck} > 50$ MPa). The value of design compressive strength in FIB Model Code 2010 is defined same like in EN 1992-1-1, as is shown by Eq. (1). It is proposed that coefficient α_{cc} should be equal to 1,0 because increase of compressive strength after 28 days compensates the effects of sustained loading, [15].

2.3 Ultimate bearing capacity of the cross-section

Distribution of ultimate strains in concrete cross section according to EN 1992-1-1:2004 is presented in Fig. 3. Values of ultimate compression strain of concrete vary between ε_{cu2} and ε_{c2}, as it is given in Tab. 1. Domain 2a corresponds to the simple bending or bending in combination with compressive force, while domain 2b presents the case when all reinforcement is compressed, and neutral axis is still in section. Diagrams for strain in the concrete turn around point B, where ultimate compression strain is equal to ε_{cu2}. In the domain 3 the whole cross-section is in compression. In this case ultimate limit state strain diagram turns around point C.

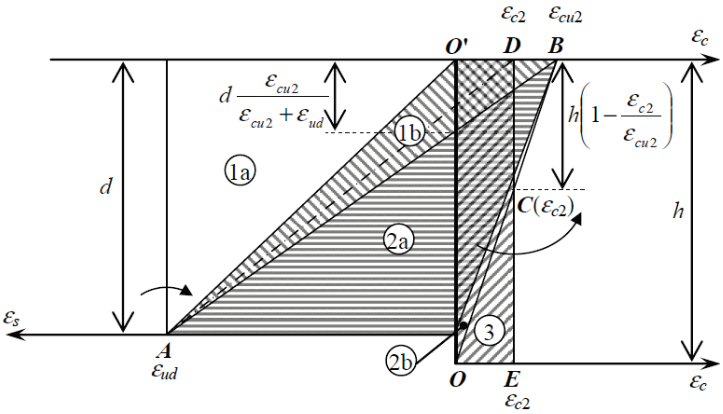

Figure 3 Ultimate limit state strain diagrams [12]

According to the design scheme for the analysis of bending with the compression in the domain 3, in Fig. 4, equations of translation equilibrium (4) and rotation equilibrium (5) can be written. The given equations refer to cross section of experimentally tested columns, which are presented in Fig. 5, which beside the reinforcement A_{s1} and A_{s2} have the reinforcement A_{s3}, placed in the half-height of cross section.

$$N_d = \psi b h f_{cd} + A_{s1}\sigma_{s1} + A_{s2}\sigma_{s2} + A_{s3}\sigma_{s3} \tag{4}$$

$$N_d e_2 = \psi b h f_{cd}(\delta_G h - d_2) + A_{s1}\sigma_{s1}(d - d_2) + A_{s3}\sigma_{s3}\left(\frac{h}{2} - d_2\right) \tag{5}$$

where: N_d − design value of imposed internal load (axial compression force); ψ − degree of filling-up of the rectangle ($f_{cd}\cdot x$) by the parabola-rectangle diagram; δ_G − the coefficient of the centre of gravity position.

Other parameters are explained in Fig. 4. Value ψ can be calculated by computing the ratio of areas under curve-rectangular diagram and rectangle $f_{cd}\cdot x$. or $f_{cd}\cdot h$, depending on the position of neutral axis.

In the domain 3 the filling coefficient ψ is increased with the increase of parameter $\xi_t = x/h$. Minimal value for ξ_t = 1,0, is dependent on concrete class (for the concrete to C50/60 is ψ = 0,8095, and for the concrete C90/105 is ψ = 0,5831). Maximal value is ψ = 1,0 for $\xi_t = \infty$, for all concrete classes.

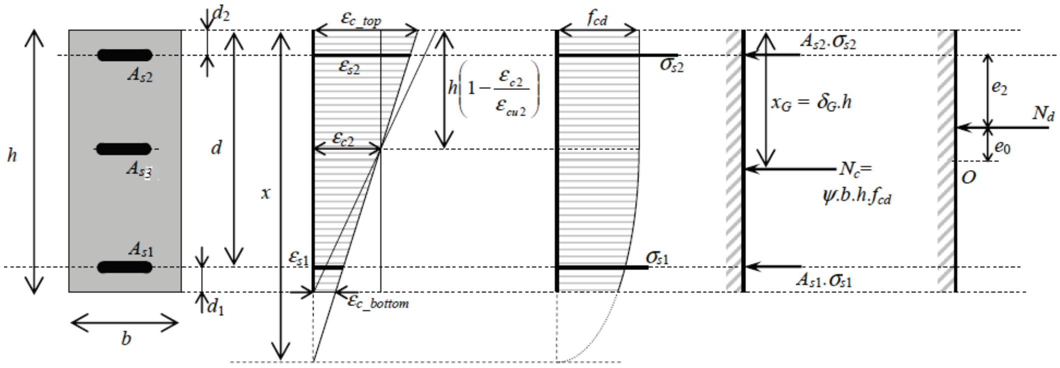

Figure 4 Figure for the analysis of bending with the compression in the domain 3 [12]

The right limit of the domain 3 presents the case of pure compression. Distribution of strains is uniform for the whole section and they are equal to ε_{c2}. Concrete stress is equal to the value of design concrete strength f_{cd}. The design force N_d can be solved by means of the translation equilibrium:

$$N_d = A_c \cdot f_{cd} + (A_{s1} + A_{s2} + A_{s3})\cdot\sigma_{sd} \tag{6}$$

A_c is area of the clear cross section of the concrete. It is the difference of gross area of cross section and area of cross section of longitudinal reinforcement.

The design stress in reinforcement is equal to the design value of steel yield point, $\sigma_{sd} = f_{yd}$, where is $f_{yd} = f_y/\gamma_s$.

Because of equality of strains in concrete and steel, $\varepsilon_s = \varepsilon_{c2}$, the given equality is valid if $f_{yd} \le \varepsilon_{c2}\cdot E_s$. If $f_{yd} > \varepsilon_{c2}\cdot E_s$, it is adopted $\sigma_{sd} = \varepsilon_{c2}\cdot E_s$.

For the case that strains in section are in the domain 2a in ultimate limit state, the equilibrium equations are:

$$N_d = \psi b x f_{cd} - A_{s1}\sigma_{s1} + A_{s2}\sigma_{s2} + A_{s3}\sigma_{s3} \tag{7}$$

$$N_d e_1 = \psi b x f_{cd}(d - \delta_G x) + A_{s2}\sigma_{s2}(d - d_2) + A_{s3}\sigma_{s3}\left(\frac{h}{2} - d_1\right) \tag{8}$$

In this case coefficient of the position of the neutral axis, ξ, is expressed in relation to the effective depth d. The filling coefficient ψ and coefficient of the center of gravity δ_G, which are calculated for the complete curve – rectangular diagram, have the constant values for the concrete to C50/60 (ψ = 0,810; δ_G = 0,416). For the concrete class >C50/60 the values are reduced and they are dependent on the class. For the concrete class C90/105 values are: ψ = 0,583 and δ_G = 0,353.

3 Analysis and processing of experimental results
3.1 Introduction

Bearing capacity of columns under the centric and eccentric pressure is main topic of more experimental researches. Use of high strength concrete makes significant contributions to discussion of this topic, because the stress-strain relationship of concrete varies with the strength. Various stress blocks in concrete have been proposed for the design of concrete elements in contemporary codes. Experimental results did not confirm enough accuracy of proposed design rules.

Experimental researches which have been used in analysis were studying the behavior of high-strength concrete columns under the centric and eccentric pressure. Subject of the research is deviation of testing results calculated according to rules proposed in EN 1992-1-1:2004. Consequently, corrections in design rules introduced through coefficient α_{cc} are defined.

In this solution long term loading has not been analyzed, and neither has been the unfavorable way of load application which EN 1992-1-1 uses for defining of α_{cc}. The results were used for a comparative analysis of the calculated capacity of columns and experimentally determined values. The influence of compressive strength of concrete on the behavior of the columns at centric and eccentric pressure has been researched.

The experiment showed that the calculated limit load capacity provides less security for high strength concrete than for normal strength concrete, if the calculation has been done using curve-rectangular diagram, as proposed according to EN 1992-1-1.

3.2 Testing

Results of experimental investigation of bearing capacity of columns, shown in [3, 4], was used for the analysis. Design bearing capacity of tested columns in centric and eccentric compression was calculated according to EN 1992-1-1:2004 and is shown in Tab. 2. Effects of the using of high strength concrete on bearing capacity of the columns are shown by correlation of tested and calculated values of bearing capacity.

The variables in experimental tests were: the concrete strength, specimen size, longitudinal and transverse reinforcement ratios, as well as eccentricity of applied load for eccentric loading. Columns are constructed of concrete with the compressive strength ranging from 54 to 114 MPa. The analysis includes 32 columns, 24 of them were tested under the centric compression, and the remaining 8 were tested under the eccentric compression. Twenty columns, of all 32, had dimensions of cross section 22,9 × 30,5 cm, and 101,6 cm in height. The rest of 12 columns had cross section dimensions of 17,8 × 22,9 cm, and 91,4 cm in height.

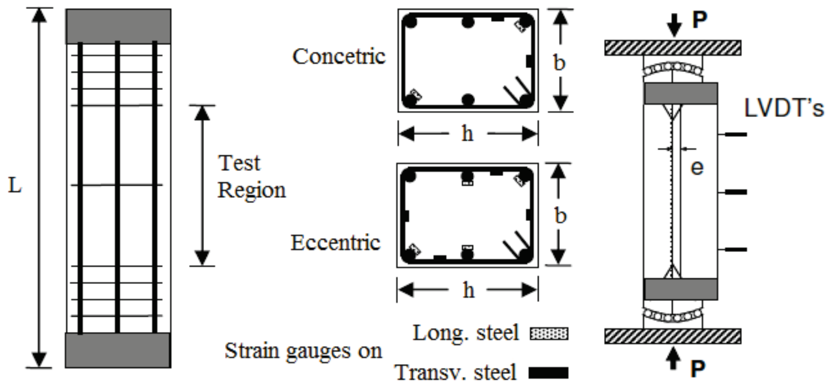

Figure 5 Review of geometry, position of strain gauges and eccentric loading [3, 4]

All columns are reinforced with per 6 longitudinal bars, and reinforced transversely with stirrups ∅13mm. Diameters of longitudinal reinforced bars are adopted according to different ratios of longitudinal reinforcement, which were approximately 1%, 2,5% and 4% for the columns with 22,9 × 30,5 cm section, and 2%, 3% and 4% for the columns with smaller section. The protective concrete cover over the stirrups is 13 mm. Both ends of columns are reinforced with closely spaced stirrups, and confined with external steel tubes which prevented premature failure of the ends of column. At eccentrically loaded samples of larger specimens, the longitudinal reinforcement ratio is 4%, and transversal reinforcement ratio is 0,91%. At the smaller-dimension specimens the longitudinal reinforcement ratio is 4,2%, and transversal reinforcement ratio is 1,55%.

3.3 Results of experiment and calculation

Data about columns tested on centric load have been given in Tab. 2, as well as measured pick load (P_{max}) and the average measured axial concrete strain corresponding to the peak load, (ε_c). Correlations of test load resistance P_{max}, with design bearing capacity according to EN 1992-1-1, N_d, and nominal axial load carrying capacity of a column, $N_{u,\ nom}$ are shown in Tab. 2, too. The values of design force N_d are calculated according to the equation (6), where the adopted coefficient is α_{cc} =1,0, safety factor for concrete γ_c = 1,5, and for steel γ_s = 1,15, while the yield point of longitudinal reinforcement f_y is adopted according to the experimental data. Results of the experimental research have shown that the measured strains of longitudinal reinforcement exceeded the yield strain of the reinforcement, at failure.

Table 2 Data, results and design values for concentrically loaded columns

Mark	$b \times h \times L$ (mm)	f_{ck} (MPa)	ρ_l (%)	ρ_s (%)	P_{max} (kN)	ε_c at P_{max}	N_d (kN)	$N_{u, nom}$ (kN)	$N_{s, nom}$ (kN)	P_{max}/N_d	P_{max} $/N_{u, nom}$	$P_{c, max}$ $/f_{ck}A_c$
C57-ρ1-0.9	229×305×1016	57,0	1,11	0,91	4363,5	0,00242	2894,2	4231,2	293,4	1,51	1,03	1,03
C57-ρ1-1.8		57,0	1,11	1,82	4314,6	0,00257	2894,2	4231,2	293,4	1,49	1,02	1,02
C56-ρ2.5-0.9		56,3	2,44	0,91	5008,4	0,00259	3199,3	4496,2	659,6	1,57	1,11	1,13
C56-ρ2.5-1.8		56,3	2,44	1,82	4915,0	0,00267	3199,3	4496,2	659,6	1,54	1,09	1,11
C54- ρ4-0.9		54,3	4,04	0,91	5355,4	0,00244	3452,1	4732,5	1092,0	1,55	1,13	1,17
C54- ρ4-1.8		54,3	4,04	1,82	5386,5	0,0028	3452,1	4732,5	1092,0	1,56	1,14	1,18
C78-ρ1-0.9		77,9	1,11	0,91	5431,0	0,00211	3893,1	5732,8	351,1	1,40	0,95	0,94
C78-ρ2.5-0.9		78,6	2,44	0,91	6285,0	0,00234	4212,4	6094,0	737,8	1,49	1,03	1,04
C78-ρ4-0.9		77,9	4,04	0,91	6667,6	0,00267	4521,5	6418,4	1195,6	1,47	1,04	1,05
C106-ρ1		106,0	1,11	0,91	6747,6	0,00247	5146,4	7627,0	304,0	1,31	0,88	0,88
C106-ρ1		106,0	1,11	1,82	6881,1	0,00222	5146,4	7627,0	304,0	1,34	0,90	0,90
C104-ρ2.5		104,5	2,44	0,91	7681,7	0,00289	5389,0	7859,0	737,8	1,43	0,98	0,98
C104-ρ2.5		104,5	2,44	1,82	8282,2	0,00275	5389,0	7859,0	737,8	1,54	1,05	1,06
C111-ρ4	178×229×914	96,5	4,04	3,10	8411,2	0,00253	6001,0	8637,6	1195,6	1,40	0,97	0,97
C111-ρ4		97,2	4,04	3,10	8353,3	0,00239	6001,0	8637,6	1195,6	1,39	0,97	0,96
C96-ρ2		98,6	1,90	3,10	3794,1	0,00259	2878,8	4211,3	351,1	1,32	0,90	0,89
C97-ρ3		104,8	2,95	1,55	3909,8	0,00224	3002,9	4350,6	505,2	1,30	0,90	0,89
C98-ρ4		104,8	4,19	3,10	4421,3	0,0028	3209,2	4589,3	737,8	1,38	0,96	0,96
C105-ρ2		105,0	1,90	1,55	3571,7	0,00246	3059,2	4496,2	304,0	1,17	0,79	0,78
C105-ρ2		105,0	1,90	3,10	3745,2	0,00272	3059,2	4496,2	304,0	1,22	0,83	0,82
C105-ρ3		100,7	2,95	1,55	3860,9	0,00264	3222,2	4674,8	520,8	1,20	0,83	0,80
C105-ρ3		102,5	2,95	3,10	3696,3	0,00289	3222,2	4674,8	520,8	1,15	0,79	0,76
C101-ρ4		96,5	4,19	3,10	3963,2	0,00233	3263,9	4671,3	737,8	1,21	0,85	0,82
C102-ρ4		97,2	4,19	3,10	4181,1	0,00259	3310,8	4741,7	737,8	1,26	0,88	0,86

The values of nominal axial load carrying capacity of column, $N_{u, nom}$, are calculated and both partial factors for materials (concrete and steel) were 1,0, according to relation (9).

$$N_{u, nom} = A_c \cdot f_{ck} + \left(A_{s1} + A_{s2} + A_{s3}\right) \cdot f_y \tag{9}$$

Maximum load resistance of columns, P_{max}, is divided on two parts: one which is bearing by concrete, $P_{c, max}$, and the other which is bearing by steel, $N_{s, nom}$.

$$P_{c, max} = P_{max} - N_{s, nom} \tag{10}$$

$N_{s,nom}$ is nominal resistance of longitudinal reinforcement, and it is equal to second addend of right side of Eq. (9).

In Tab. 2 there are values of correlation $P_{c, max}$ and nominal bearing capacity of concrete that is calculated as $f_{ck}A_c$.

The data for eccentric loaded columns are given in Tab. 3. Eccentricity of compressive force e_0 during the experiment was ~0,1h (mark E1) or ~0,2h (mark E2). The value of coefficient α_{cc} = 0,85 is adopted for the design in this case. Column P_{exp} in Tab. 3 contains data with values of maximum loads that were measured during the testing. Values of design compressive force, N_d, were calculated using equilibrium equations.

Reduced axial compression force and reduced moment were calculated with the experimental and designed values of these parameters. Correlations ν_{exp}/ν_d and μ_{exp}/μ_d are used for the analysis of acceptability coefficient α_{cc} if it has constant value. Formulas (11) and (12) are used for the calculation of values presented in Tab. 3.

$$\frac{\nu_{exp}}{\nu_d} = \frac{\dfrac{P_{exp}}{f_{ck} \cdot b \cdot h}}{\dfrac{N_d}{f_{cd} \cdot b \cdot h}} = \frac{P_{exp}}{N_d} \cdot \frac{\alpha_{cc}}{\gamma_c} \tag{11}$$

$$\frac{\mu_{exp}}{\mu_d} = \frac{\dfrac{M_{exp}}{f_{ck} \cdot b \cdot h^2}}{\dfrac{M_d}{f_{cd} \cdot b \cdot h^2}} = \frac{M_{exp}}{M_d} \cdot \frac{\alpha_{cc}}{\gamma_c} \tag{12}$$

Table 3 Data, experimental results, and design values for eccentric loaded columns

Mark	$b \times h \times L$ (mm)	f_{ck} (MPa)	e_0 (cm)	P_{exp} (kN)	N_d (kN)	P_{exp}/N_d	ν_{exp}/ν_d	μ_{exp}/μ_d	Domain
C54-E1	229×305×1016	54	3,10	3972,1	2386,0	1,66	0,94	0,95	3
C54-E2		54	6,30	3126,7	1691,8	1,85	0,96	1,03	2a
C75-E1		75	2,82	4550,3	2881,3	1,58	0,90	0,85	3
C113-E1		113	3,10	6040,4	3740,6	1,61	0,91	0,92	3
C114-E2		114	6,25	4490,8	3049,0	1,47	0,90	0,81	2a
C97-E1	178×229×914	97	2,06	3411,6	2058,3	1,66	0,98	0,97	3
C108-E1		108	2,31	3509,5	2153,5	1,63	0,91	0,90	3
C108-E2		108	4,78	2562,1	1726,6	1,48	0,92	0,84	2a

4 Analysis of design parameters

Values of correlation of maximum measured load resistance of columns P_{max} and nominal forces $N_{u,\ nom}$ in the function of concrete compressive strength are shown in Fig. 6. It is clearly visible the trend of value decrease $P_{max}/N_{u,\ nom}$ with the increase of concrete strength. It is also noticeable the value $P_{max}/N_{u,\ nom}$ for the high-strength concrete is significantly less than 1,0. This suggests that the use of constant value for the reduction factor of concrete compressive strength design parameter may not be appropriate for the calculation of design concrete compressive strength for all concrete classes. This approach does not provide the same degree of the calculation safety of the design values of the axial compression force for elements made of normal and high-strength concrete if curve-rectangular or bilinear stress-strain diagram is used.

For this reason some corrective factor should be included through the parameter α_{cc}, only where other restrictive factor is not included, already. In this manner carrying capacity of concrete elements with axial compression might be assessed more accurately.

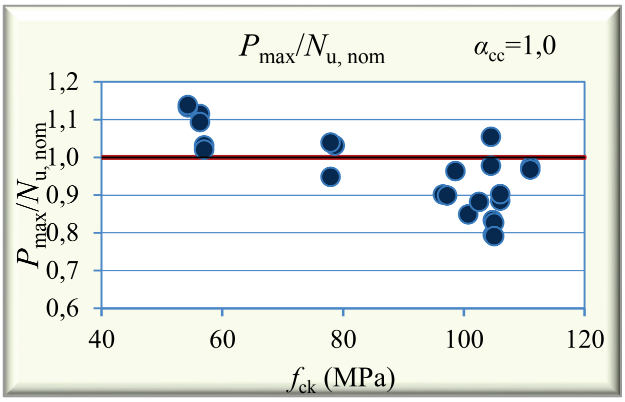

Figure 6 Correlation of experimentally established compressive force P_{max} and nominal ultimate force $N_{u,\ nom}$

5 Innovation of α_{cc} parameter

Innovated parameter α_{cc} is defined based on the results of tests so that it includes a new factor in function of the concrete compressive strength. This factor includes differences between normal and high-strength of concrete, for classes higher than C50/60. Equation for calculation of axial resistance for compression members is defined as:

$$N_{u} = \alpha_{cc} \cdot f_{ck} \cdot A_{c} + A_{s} \cdot f_{y}, \tag{13}$$

Coefficient α_{cc} was defined as the product of multiplication of two new coefficients, α_c and k_{cc}, so that $\alpha_{cc} = \alpha_c \cdot k_{cc}$.

Changing of value of coefficient α_{cc} in the function of compressive strength of concrete is introduced using coefficient k_{cc}, as it is shown in Fig. 7. Coefficient k_{cc} was defined as:

$k_{cc} = 1{,}0$ for $f_{ck} \le 50$ MPa
$k_{cc} = 1{,}0 - 0{,}004(f_{ck} - 50)$, for $50 < f_{ck} \le 100$ MPa
$k_{cc} = 0{,}8$ for $f_{ck} > 100$ MPa

Coefficient k_{cc} was introduced in order to cover the differences in the calculation of the carrying capacity of concrete elements made of concretes of normal and high strength. Through parameter α_c should be included effects that were earlier proposed according to EN 1992-1-1. In this way coefficient α_{cc} will be defined in different manner for the use with different idealized stress-strain diagrams. For example, for simplified rectangular diagram, reduction factor for high strength concrete is already included. Reduction is not needed in this case. For other two, curve-rectangle and bi-linear diagrams, calculation of carrying capacity of concrete elements with axial compression should be revised and corrected by using of described manner.

Figs. 7 and 8 show that the change in the value of the coefficient k_{cc} corresponds to the trend of changes in values of resistance ratio obtained under test methods and theoretical calculations.

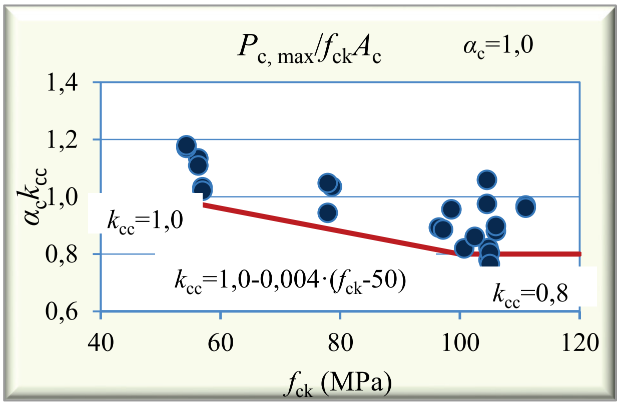

Figure 7 Compliance of new α_{cc} with ratio of experimentally established and nominal carrying capacity of concrete

According to Eq. (12) it follows that trend of changes in relationships μ_{exp}/μ_d is directly related to α_{cc}. In Fig. 8 it is shown that the trend of change of μ_{exp}/μ_d ratio corresponds to the proposed expressions for k_{cc}. It is clear that parameter α_{cc} should be defined as a function of the strength of concrete. Additional adjustment of this value can be made by adopting appropriate coefficient α_c. It may include other local parameters, as well as parameters required by EN 1992-1-1. Previous analysis was done with α_c = 1,0. It is necessary to further consider the conditions in which $\alpha_c < 1{,}0$ should be adopted.

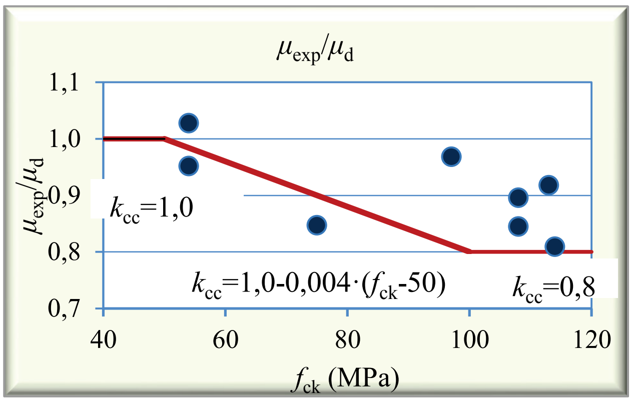

Figure 8 Compliance of trend of new α_{cc} with ratio of reduced moments

For comparing the experimental strength of eccentrically loaded columns and the theoretically obtained load capacity of them, the ratio μ_{exp}/μ_d was used. In Fig. 8 it is shown that the ratio of reduced values of experimental bending moment and designed bending moment tends to decrease with increasing of concrete strength. It can be concluded that constant value of α_{cc} parameter is not suitable, because there is obvious decline of values μ_{exp}/μ_d with increase of concrete compressive strength, if α_{cc} has a constant value.

6 Conclusion

The design of structural elements made of reinforced high-strength concrete, higher class than C50/60, according to EN 1992-1-1, should include mostly the specificity of high-strength concrete. Idealized design stress-strain diagrams and other design coefficients are prescribed in the function of compressive strength of concrete, which is justified and in concordance with the results of experimental researches.

In design elements in centric and eccentric compression, where the usage of high-strength concrete is indicated, one of the most important design parameters is the coefficient α_{cc}, which serves for calculation of the design concrete compressive strength. Special rules are not predicted for this coefficient for higher-class concrete than C50/60. Proposed value of this coefficient, which is 1,0, is mostly non-conservative and non-concordant with experimental results, especially for the high-strength concrete. However, as some other correction coefficient is not predicted for defining the design compressive strength it is justified that coefficient α_{cc} must include more parameters among the others and specific qualities of high-strength concrete.

Considering the fact that the considered coefficient is adopted by the National Annex, there is a possibility that their value is accustomed to the experimental results. Some countries have already done it by the National Annexes, but they are not specially connected to the high-strength concrete. It would be very useful to consider this coefficient also from that aspect, and to define its values in the function of concrete strength, as it is done in the case of some other design parameters.

The value of coefficient α_{cc} can be defined by the National Annex in accordance with the suggested equations which are shown in this paper in concordance with the local conditions of high-strength concrete production. Firstly the suggested equations should be verified by the application of the additional experimental researches.

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