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THE INFLUENCE OF CONCRETE COVER ON THE BEARING CAPACITY AND RELIABILITY OF THE REINFORCEMENT CONCRETE SLAB – COLUMN SYSTEM

The reliability of building structures is a very important design criterion. The required level of security depends not only on the function and purpose of the facility, but also on the parameters considered at the design stage. There are factors to that with some probability they increase the uncertainty of the adopted parameters in the calculations, as a result of which the structure fails. It happens that execution errors are made during the process of creating an object. One of the significant disadvantages is the increase in the reinforcement cover thickness compared to the designed value, and hence the reduction of the effective depth of cross-section. The purpose of the analyzes presented in the article was to estimate the influence of the top reinforcement cover thickness on the reliability and bending and punching resistance in the reinforced concrete slab being part of the slab-column system. The reliability index βc was determined by the Cornell method. The analysis focused on a fragment of a cross reinforced slab, which is an element of the column-slab structure of a shopping mall. The ARSA 2019 program was used for the analysis of the structure. The impact of the coefficient of variation of the effective depth on the reliability index was also analyzed. The obtained values were compared with the target value of the reliability index read from EN 1990.

Keywords: reinforced concrete, flat plate, shear, bending, reliability index

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1. Introduction

The reliability of building structures is a very important design criterion. The required level of security depends not only on the function and purpose of the facility, but also on the parameters considered at the design stage. There are factors to that with some probability they increase the uncertainty of the adopted parameters in the calculations, as a result of which the structure fails. It happens that execution errors are made during the process of creating an object. One of the significant disadvantages is the increase in the reinforcement cover thickness compared to the designed value, and hence the reduction of the effective depth of cross-section. This fact quite often occurs in the construction of slab-column system. In the case of an incorrectly made element, the question arises whether it is safe or what is the probability of its failure.

The article considers the influence of the top reinforcement cover thickness on the reliability and bending and punching resistance in the reinforced concrete slab being part of the slab-column system.

2. Analyzed structure

2.1. The general concept of the building

The building of the shopping mall was designed in the frame technology: the supporting structure is a slab-column structure and a reinforced concrete core. The brick wall of the ground floor is a cover and does not transmit loads. The building has no basement. The building consists of 3 overground storeys and one above-ground storey of the height of the 3.8 m. The building is designed with dimensions in the axes 33.5 x 51.8 m. Loads from the columns are transferred to the ground through the foundational footings.

The slab was made of 25 cm thick concrete class C25 / 30 with a compressive strength $f_{ck} = 25$ MPa. The reinforcing steel B500SP was used with the yield strength $f_{yk} = 500$ MPa, the reinforcement cover is 3 cm. The reinforcement for bending and punching was selected based on the values of the forces obtained from the FEM model. In the analyzed fragment of the floor slab, the bending reinforcement was made of 16 mm diameter bars, while the Halfen HDB-14 / 195-6 / 917 [6] was used as punching shear reinforcement.

2.2. FEM model

In order to determine the internal forces in the slab, a computer model was made. For this purpose, the plate type available in the ARSA 2019 [7] program was used, in which a model of the floor slab was created. After creating the outline and giving the panels the features of the slab, the supports were modelled:

- a column as a nodal support with actual dimensions, i.e. 40 cm x 40 cm;
- a brick wall as a linear support 25 cm wide;
- and a reinforced concrete wall as a linear support 25 cm wide.

The resulting model is shown in Fig. 1.

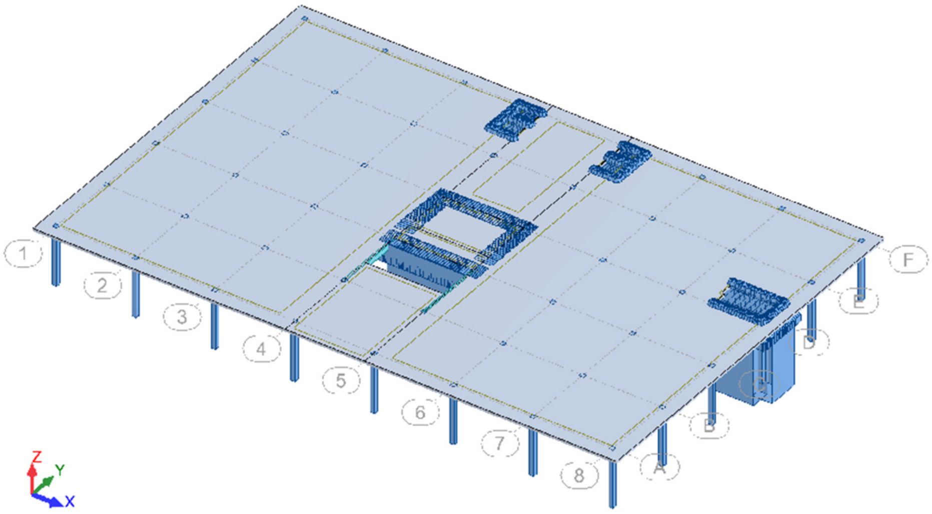

Fig. 1. View of the slab model created in ARSA 2019

In order to generate a mesh of finite elements, the Delaunay meshing method with a square shape of a finite element was used. The basic mesh size was 0.5 m. Additionally, the mesh was doubled in the spans and four times over the columns. The applied mesh of finite elements is shown in Figure 2.

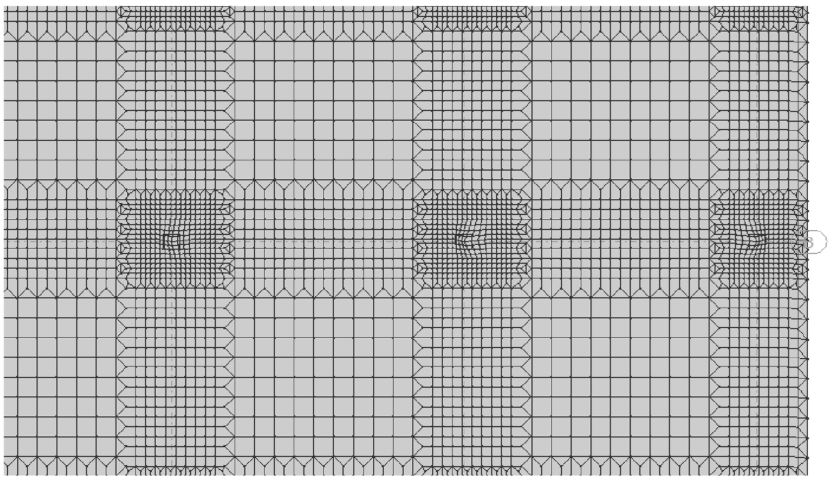

Fig. 2. View of mesh created in ARSA 2019

The following loads were taken into account in the design:
- self-weight of finishing layers 2.32 kN / m^2;
- imposed loads 5.0 kN / m^2;
- self-weight of movable partitions 0.8 kN / m^2;
- load from the glass facade 2.84 kN / m.

3. Assumptions adopted for the reliability analysis

The aim of the analysis was to estimate the Cornell reliability index βc [1] as a measure of structure reliability. The analysis focused on the fragment of the cross reinforced slab shown in Figure 3. The ARSA 2019 program was used for the structure analysis. In the model created for the purpose of determining the reinforcement in the slab, the values of loads were converted into average values, considering them as random variables with the following distributions and coefficients of variation:
- constant loads: normal distribution (N), $\nu = 0.08$;
- long-term variable loads: gamma distribution (Γ), $\nu = 0.25$.

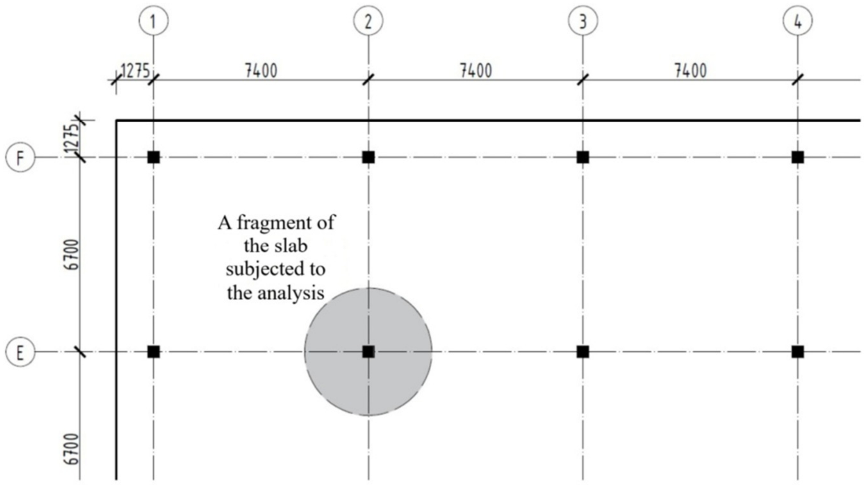

Fig. 3. A fragment of the slab subjected to the analysis

Thus, average values of the bending moments $M_x = 125.72$ kNm and $M_y = 114.74$ kNm and the average value of the punching force $V_{Ed} = 839.94$ kN were obtained. The strength of the reinforcing steel was assumed as a random variable with an average value of $f_{ym} = 544.81$ MPa and a coefficient of variation of 5%, which corresponds to the characteristic strength of B500SP steel with $f_{yk} = 500$ MPa. Based on [3], the average compressive strength of the concrete was determined $f_{cm} = 33$ MPa, the coefficient of variation was assumed to be $\nu = 15\%$. In order to compare the results, the main considerations were made by adopting the useful cross-section height d as a random variable with a coefficient of variation equal to 20%, which corresponds to the standard deviation $\sigma_d = 0.2d$, and treating it as a deterministic value, i.e. a constant. The values of the coefficients of variation for loads and materials are based on [5].

The following values, obtained in [4] were used in the calculations:

- constant loads: normal distribution (N), $\nu = 0.08$ bending reinforcement area As = 25.13 cm^2 and the average useful height of the cross-section d = 204 mm;
- bending reinforcement area A_s = 25.13 cm^2 and the average effective depth of the cross-section d = 204 mm;
- punching area A_{sw} = 18.84 cm^2, radial spacing of reinforcement perimeters s_r = 128 mm, column size c = 400 mm, coefficient β = 1.148, degree of reinforcement for bending ρ_l = 1.1% and the average effective depth of the cross-section d = 204 mm.

4. Analysis of the influence of top reinforcement cover on the bending resistance

4.1. Effective depth of the cross-section as a deterministic value

In the analysis below, the random variables were: loads, compressive strength of concrete and yield strength of reinforcing steel. The limit condition of the bending resistance of the analyzed floor slab is the difference between the resistance of the structural element R and the load effect E, it has a nonlinear form:

$$\Delta = R - E \tag{1}$$

where: E= 125.72 kNm – value of the bending moment due to mean loads obtained from Autodesk Robot Structural Analysis,

R – resistance.

The bending resistance of the element is given by formula 2.

$$R = A_s \cdot f_y \cdot d - \frac{A_s^2 \cdot f_y^2}{2 \cdot b \cdot f_c} \tag{2}$$

where: A_s – cross sectional area of reinforcement,

d – effective depth of a cross-section, $d = h - c$,

h – height of plate,

c – concrete cover,

b – overall width of a cross-section,

f_y – yield strength of reinforcement,

f_c – compressive strength of concrete.

After substituting:

$$\Delta = A_s \cdot f_y \cdot d - \frac{A_s^2 \cdot f_y^2}{2 \cdot b \cdot f_c} - 125.72$$

The expected value $\bar{\Delta}$ and the standard deviation σ_Δ necessary to calculate the Cornell reliability index βc were approximated by expanding into the Taylor series. Only linear unfolding terms are left.

In order to determine the expected value $\bar{\Delta}$, the mean values of the random variables were substituted for the limit state function:

$$\bar{\Delta} = A_s \cdot f_{ym} \cdot d - \frac{A_s^2 \cdot f_{ym}^2}{2 \cdot b \cdot f_{cm}} - 125.72$$

$$\bar{\Delta} = 2513 \cdot 544.81 \cdot 204 - \frac{2513^2 \cdot 544.81^2}{2 \cdot 1000 \cdot 33} - 125.72 \cdot 10^6 = 125.17 \cdot 10^6 \text{ Nmm}$$

Then, in order to determine the standard deviation σ_Δ, individual partial derivatives were calculated from the boundary state function.

$$\sigma_\Delta = \sqrt{\left(\sigma_y \cdot \frac{\delta\Delta}{\delta f_y}|_{f_y = f_{ym}}\right)^2 + \left(\sigma_c \cdot \frac{\delta\Delta}{\delta f_c}|_{f_c = f_{cm}}\right)^2}$$

$$\frac{\delta\Delta}{\delta f_y}|_{f_y = f_{ym}} = A_S d - \frac{2A_s^2 \cdot f_{ym}}{2b \cdot f_{cm}}$$

$$\frac{\delta\Delta}{\delta f_y}|_{f_y = f_{ym}} = 2513 \cdot 204 - \frac{2 \cdot 2513^2 \cdot 544.81}{2 \cdot 1000 \cdot 33} = 408392.26 \text{ mm}^3$$

$$\frac{\delta\Delta}{\delta f_c}|_{f_c = f_{cm}} = \frac{A_s^2 \cdot f_{ym}^2}{2b \cdot f_{cm}^2}$$

$$\frac{\delta\Delta}{\delta f_c}|_{f_c = f_{cm}} = \frac{2513^2 \cdot 544{,}81^2}{2 \cdot 1000 \cdot 33^2} = 860633.65 \text{ mm}^3$$

The coefficient of invariance was calculated using the previously determined mean values and the assumed coefficients of variation.

$$\nu = \frac{\sigma}{X_m} \rightarrow \sigma = \nu \cdot X_m$$

$$\sigma_c = \nu_c \cdot f_{cm} = 0.15 \cdot 33 = 4.95 \text{ MPa}$$

$$\sigma_s = \nu_s \cdot f_{ym} = 0.05 \cdot 544.81 = 27.24 \text{ MPa}$$

The standard deviation is σ_Δ:

$$\sigma_\Delta = \sqrt{(27.24 \cdot 408392.26)^2 + (4.95 \cdot 860633.65)^2}$$

$$\sigma_\Delta = 11912617.41 \text{ Nmm} = 11.91 \text{ kNm}$$

Calculation of the Cornell Reliability Index β:

$$\beta_c = \frac{\bar{\Delta}}{\sigma_\Delta} = \frac{125.17}{11.91} = 10.51$$

The value of the determined reliability index βc is 10.51 and is a value greater than the minimum value contained in Annex B to the standard [2]. For the RC2 reliability class and the 50 years working life, the minimum reliability index is 3.8. The design can be considered reliable.

4.2. Effective depth of the cross-section as a random variable

In the analysis carried out at this point, the effective depth of the cross-section d was taken into account as a random variable. In this situation, the limit state condition will also have a non-linear form as in point 4.1. The size of the expected value $\bar{\Delta}$ is the same as in point 4.1.

In order to calculate the standard deviation σ_Δ, an additional partial derivative was determined from the limit state function after d.

$$\sigma_\Delta = \sqrt{\left(\sigma_y \cdot \frac{\delta\Delta}{\delta f_y}|_{f_y=f_{ym}}\right)^2 + \left(\sigma_c \cdot \frac{\delta\Delta}{\delta f_c}|_{f_c=f_{cm}}\right)^2 + \left(\sigma_d \cdot \frac{\delta\Delta}{\delta d}|_{d=d_m}\right)^2}$$

$$\frac{\delta\Delta}{\delta f_y}|_{f_y=f_{ym}} = A_S d_m - \frac{2A_s^2 f_{ym}}{2bf_{cm}}$$

$$\frac{\delta\Delta}{\delta f_y}|_{f_y=f_{ym}} = 2513 \cdot 204 - \frac{2 \cdot 2513^2 \cdot 544.81}{2 \cdot 1000 \cdot 33} = 408392.26 \text{ mm}^3$$

$$\frac{\delta\Delta}{\delta f_c}|_{f_c=f_{cm}} = \frac{A_s^2 \cdot f_{ym}^2}{2b \cdot f_{cm}^2}$$

$$\frac{\delta\Delta}{\delta f_c}|_{f_c=f_{cm}} = \frac{2513^2 \cdot 544.81^2}{2 \cdot 1000 \cdot 33^2} = 860633.65 \text{ mm}^3$$

$$\frac{\delta\Delta}{\delta d}|_{d=d_m} = A_S \cdot f_{ym}$$

$$\frac{\delta\Delta}{\delta d}|_{d=d_m} = 2513 \cdot 544.81 = 1369109.23 \text{ N}$$

Using the coefficient of invariance determined in point 4.1, the standard deviation σ_Δ was calculated:

$$\sigma_\Delta = \sqrt{(27.24 \cdot 408392.26)^2 + (4.95 \cdot 860633.65)^2 + (40.8 \cdot 1369109.23)^2}$$

$$\sigma_\Delta = 571155774.6 \text{ Nmm} = 57.12 \text{ kNm},$$

Calculation of the Cornell Reliability Index β:

$$\beta_c = \frac{\bar{\Delta}}{\sigma_\Delta} = \frac{125.17}{57.12} = 2.19$$

In this case, the value of the determined reliability index βc is 2.19 and is a value less than the minimum value of 3.8. The above calculations and considering the effective depth of the section d as a random variable with a coefficient of variation equal to 20%, showed the unreliability of the structure.

5. Analysis of the influence of top reinforcement cover on the resistance to punching

5.1. Effective depth of the cross-section as a deterministic value

In the analysis of the resistance to punching carried out at this point, the random variables were: loads, yield strength of the reinforcing steel and compressive strength of concrete. The ultimate state condition of the punching resistance of the analyzed floor slab has the form of formula (1). The punching resistance of the element is determined by the formula (3).

$$R = 0.75\nu_{Rd,c} + 1.5\frac{d}{s_r}A_{sw}f_y\frac{1}{u_1 d}\sin\alpha \tag{3}$$

where: A_{sw} – cross sectional area of shear reinforcement,
$\nu_{Rd,c}$ – design punching shear resistance,
s_r – radial spacing of perimeters of shear reinforcement,
u_1 – basic control perimeter,
α – angle between the shear reinforcement and the plane of the slab.

The effect of the interactions is given by the formula (4):

$$E = \beta\frac{V_{Ed}}{u_1 d} \tag{4}$$

where: β – coefficient,
V_{Ed} – design value of the applied shear force.

After substituting into formula (1):

$$\Delta = 0.75\nu_{Rd,c} + 1.5\frac{d}{s_r}A_{sw}f_y\frac{1}{u_1 d}\sin\alpha - \beta\frac{V_{Ed}}{u_1 d}$$

where:

$$\nu_{Rd,c} = \frac{0.18}{\gamma_c}\cdot k\cdot(100\rho_l f_{ck})^{\frac{1}{3}}$$

$$u_1 = 4c + 4\pi d \quad \text{and} \quad k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{204}} = 1.99$$

Similarly to the bending resistance, the expected value $\bar{\Delta}$ and the standard deviation σ_Δ necessary to calculate the Cornell reliability index βc were approximated by expanding into the Taylor series. Only linear unfolding terms are left. The mean values of random variables were substituted to the limit state function in order to determine the expected value $\bar{\Delta}$:

$$\bar{\Delta}= 0.75 \cdot 0.18k(100\rho_l f_{cm})^{\frac{1}{3}} + 1.5\frac{d}{s_r}A_{sw}f_{ym}\frac{\sin\alpha}{4cd + 4\pi d^2} - \beta\frac{V_{Ed}}{4cd + 4\pi d^2}$$

$$\bar{\Delta}= 0.75 \cdot 0.18 \cdot 1.99 \cdot (100 \cdot 0.01103 \cdot 33)^{\frac{1}{3}} + \frac{1.5 \cdot \frac{204}{128} \cdot 1848 \cdot 544.81}{4 \cdot 400 \cdot 204 + 4 \cdot \pi \cdot 204^2} -$$

$$- \frac{1{,}148 \cdot 839940}{4 \cdot 400 \cdot 204 + 4 \cdot \pi \cdot 204^2} = 2{,}59 \text{ MPa}$$

In order to determine the standard deviation σ_Δ, individual partial derivatives from the limit state function were calculated.

$$\sigma_\Delta = \sqrt{\left(\sigma_y \cdot \frac{\delta\Delta}{\delta f_y}|_{f_y=f_{ym}}\right)^2 + \left(\sigma_c \cdot \frac{\delta\Delta}{\delta f_c}|_{f_c=f_{cm}}\right)^2}$$

$$\frac{\delta\Delta}{\delta f_y}|_{f_y=f_{ym}} = 1.5\frac{d}{s_r}A_{sw}\frac{\sin\alpha}{4cd + 4\pi d^2}$$

$$\frac{\delta\Delta}{\delta f_y}|_{f_y=f_{ym}} = \frac{1{,}5 \cdot \frac{204}{128} \cdot 1848 \cdot 1}{4 \cdot 400 \cdot 204 + 4 \cdot \pi \cdot 204^2} = 0.00520\ [-]$$

$$\frac{\delta\Delta}{\delta f_c}|_{f_c=f_{cm}} = 0.75 \cdot 0.18k\frac{(100\rho_l f_{cm})^{\frac{1}{3}}}{3 \cdot f_{cm}}$$

$$\frac{\delta\Delta}{\delta f_c}|_{f_c=f_{cm}} = 0.75 \cdot 0.18 \cdot 1.99 \cdot \frac{(100 \cdot 0.01103 \cdot 33)^{\frac{1}{3}}}{3 \cdot 33} = 0.00899\ [-]$$

Using the coefficients of variation determined in point 4.1, the standard deviation σ_Δ was determined:

$$\sigma_\Delta = \sqrt{(27.24 \cdot 0.00520)^2 + (4.95 \cdot 0.00899)^2}$$

$$\sigma_\Delta = 0{,}15 \text{ MPa}$$

Calculation of the Cornell Reliability Index β:

$$\beta_c = \frac{\bar{\Delta}}{\sigma_\Delta} = \frac{2.59}{0.15} = 17.43$$

The reliability index βc is 17.43 and is a value greater than the minimum value contained in Annex B to the standard [2], equal to 3.8. The design can be considered reliable.

5.2. Effective depth of the cross-section as a random variable

In the analysis carried out at this point, the effective depth of the cross-section d was taken into account as a random variable. In this situation, the limit state condition and the expected value $\bar{\Delta}$ will have the form as in point 5.1.

In order to calculate the standard deviation σ_Δ, an additional partial derivative was determined from the limit state function after d.

$$\sigma_\Delta = \sqrt{\left(\sigma_y \cdot \frac{\delta\Delta}{\delta f_y}|_{f_y=f_{ym}}\right)^2 + \left(\sigma_c \cdot \frac{\delta\Delta}{\delta f_c}|_{f_c=f_{cm}}\right)^2 + \left(\sigma_d \cdot \frac{\delta\Delta}{\delta d}|_{d=d_m}\right)^2}$$

$$\frac{\delta\Delta}{\delta f_y}|_{f_y=f_{ym}} = 1.5\frac{d_m}{s_r}A_{sw}\frac{\sin\alpha}{4cd + 4\pi d^2}$$

$$\frac{\delta\Delta}{\delta f_y}|_{f_y=f_{ym}} = \frac{1,5 \cdot \frac{204}{128} \cdot 1848 \cdot 1}{4 \cdot 400 \cdot 204 + 4 \cdot \pi \cdot 204^2} = 0.00520\ [-]$$

$$\frac{\delta\Delta}{\delta f_c}|_{f_c=f_{cm}} = 0.75 \cdot 0.18k\frac{(100\rho_l f_{cm})^{\frac{1}{3}}}{3 \cdot f_{cm}}$$

$$\frac{\delta\Delta}{\delta f_c}|_{f_c=f_{cm}} = 0.75 \cdot 0.18 \cdot 1.99 \cdot \frac{(100 \cdot 0.01103 \cdot 33)^{\frac{1}{3}}}{3 \cdot 33} = 0.00899\ [-]$$

$$\frac{\delta\Delta}{\delta d}|_{d=d_m} = -1.5\frac{A_{sw}}{s_r}f_y \sin\alpha\frac{\pi}{4(c + \pi d_m)^2} - \beta V_{Ed}\frac{-c - 2\pi d_m}{4d_m^2(c + \pi d_m)^2}$$

$$\frac{\delta\Delta}{\delta d}|_{d=d_m} = -1.5 \cdot \frac{1848}{128} \cdot 544.81 \cdot 1 \cdot \frac{\pi}{4 \cdot (400 + \pi \cdot 204)^2} - 1.148 \cdot 839940 \cdot \frac{-400 - 2 \cdot \pi \cdot 204}{4 \cdot 204^2(400 + \pi \cdot 204)^2} = 0.0175\ \frac{N}{mm^3}$$

Using the coefficients of variation determined in point 4.1, the standard deviation σ_Δ was determined:

$$\sigma_\Delta = \sqrt{(27.24 \cdot 0.00520)^2 + (4.95 \cdot 0.00899)^2 + (40.8 \cdot 0.0175)^2}$$

$$\sigma_\Delta = 0.73\ \text{MPa}$$

Calculation of the Cornell Reliability Index β:

$$\beta_c = \frac{\bar{\Delta}}{\sigma_\Delta} = \frac{2.59}{0.73} = 3.54$$

The value of the determined reliability index βc is 3.54 and is a value lower than the minimum value of 3.8. The above calculations and considering the effective cross-section height d as a random variable with a coefficient of variation equal to 20%, proved the failure of the structure.

The above analyzes showed that taking into account the useful height of the cross-section in calculations, as a random variable it leads to a lack of safety of the structure.

6. Summary and conclusions

The aim of the analysis was to estimate the reliability index βc using the Cornell method and compare the results with the relevant target values adopted by international standards. The main considerations were made by adopting the effective depth of cross-section as a random variable with a coefficient of variation equal to 20% and treating it as a deterministic value. It was noticed that assuming a constant cover thickness and the related effective depth of the cross-section, the reliability indices for both bending and shear, respectively 10.51 and 17.43, are significantly higher than the minimum value of 3.8. This design is safe and reliable. Increasing the reinforcement cover thickness compared to the designed value, and thus reducing the effective depth of cross-section by 20%, leads to a significant decrease in the values of the reliability indexes, and thus the previously safe structure becomes unreliable.

In order to illustrate the impact of changing the top reinforcement cover on the load capacity and reliability of the structure, calculations were made for various coefficients of variation of the effective depth of cross-section. The Table 1 presents the results from the calculations performed.

Table 1. Influence of the coefficient of variation d on the reliability index βc

	Coefficient of variation of effective depth of cross-section d						
	0%	5%	7%	15%	20%	25%	30%
	The value of the reliability index β_c						
Bending	10.51	6.82	5.47	2.87	2.19	1.77	1.48
Punching	17.43	11.13	8.89	4.65	3.54	2.85	2.39

The obtained values of the reliability indicators βc depending on the different coefficient of variation d were compared with the standard value and presented in Fig. 4.

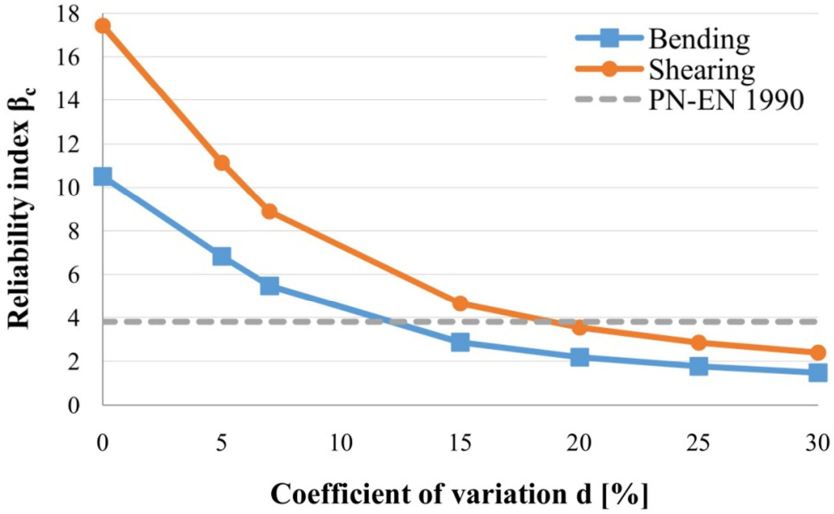

Fig. 4. Influence of the coefficient of variation d on the reliability index βc

In the case of typical properties of materials and reinforcement, the results of the analysis showed that there is a large variability of the reliability index depending on the value of the coefficient of variation of the effective depth of cross-section. It can be noticed that despite the implementation of a different thickness of the reinforcement cover than designed, there is a certain area in which the structure is still safe and reliable. Therefore, when assessing existing structures, it is possible to accept a certain change in the reinforcement cover thickness compared to the design value. However, it should be borne in mind that a significant increase in the thickness of the reinforcement cover will make the building structure, in this case the slab structure, unreliable and dangerous for the people using it.

The analysis also showed the legitimacy of using higher-level probabilistic methods. Level I probabilistic methods, where the measure of reliability are appropriate sets of partial factors, allow us only to evaluate the safety of the structure in two values. The application of the higher level method, in this case the Cornell reliability index method, allowed for an accurate, quantitative estimation of the structure reliability. The results of the analysis showed that, assuming a constant cover thickness, the reliability factors for both bending and shear are much higher than the minimum value. The use of the higher-level method allows for obtaining alternative measures allowing for the design of structures with a constant probability of exceeding certain limit states.

In the next stages, it is planned to perform experimental tests on the basis of which the numerical model will be validated. Extensive numerical analyzes with the use of Monte Carlo simulations will allow for a more detailed understanding of the slab behaviour in the event of a change in the cover. Analyzes are also planned of the impact of changing the useful height of the cross-section on the fulfilment of the serviceability limit state - the value of deflections.

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