Assessment of Design Concepts for Post-installed Punching Shear Retrofitting

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Abstract Punching shear is a brittle form of failure observed in reinforced concrete slab structures and occurs without any visible signs before failure. This phenomenon typically arises around the slab-column connections, due to transverse forces being highly concentrated in these areas and can cause that the column punches through the slab. This type of failure is very brittle. The unpredictability of its occurrence makes it a particularly critical and dangerous phenomenon. Several methods have been developed for retrofitting and strengthening existing flat slabs against punching shear failure using different reinforcement-types, like shear bolts, screw anchors or bonded anchors. These methods are called post-installed shear reinforcement for existing flat slab systems. This study aims to assess the safety and economic performance of the Eurocode 2 (EC2) design method for the design of post-installed reinforcement in an existing flat slab structure endangered by punching shear, using probabilistic analysis. The probabilistic analysis was conducted based on the Monte Carlo simulation technique implemented using a MATLAB code developed in the study. The reliability indices obtained for EC2 design procedure were found to be close to the EN 1990 target reliability level.

Keywords Punching shear · Flat slab · Eurocode · Reinforced concrete · Retrofitting · Probabilistic analysis

1 Introduction

Flat slabs are one of the most widely used concrete floor systems. The most critical aspects of a flat slab system are the column support joints. These areas are considered the starting point of a brittle and sudden failure caused by shear and flexural tension [1]. Generally, flat slabs are prone to punching failure, which occurs when a flat slab

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system is overloaded, and the slab fails within a distance around the column. Novacek and Zich [2] described punching shear as a type of failure of reinforced concrete slabs due to shear forces. These forces are highly localised at column support points. Punching shear failure is very brittle and occurs without any visible signs before failure, which makes it a critical high phenomenon. Another significant issue is that the redistribution of the inner forces is minimal during the failure. This may lead to a progressive collapse of the structure [2], such as the collapse reported in Beutel [1] and Kunz et al. [3]. Many of such failures could be prevented if the concrete slabs were adequately retrofitted. Therefore, the design of flat slabs must be accorded proper and adequate attention to avoid failures.

Retrofitting existing structures is an important topic that has been given serious attention in the last decade. The older a building gets, the higher the need for inspections, health monitoring and maintenance. Nowadays built flat slabs are strengthened with shear reinforcement according to the existing code requirements to assure that their strength is sufficient to prevent failures. Existing older flat slabs supported by columns, however, must be strengthened against punching shear failure, in the case of insufficient existing strength. The reason for this can be attributed to higher code requirements as a result of increased knowledge gained in the past years on the topic. Other reasons are the change of use of the building and thus the increasing loads during the lifetime of the structure, but also construction and design errors [4].

Different methods have been proposed to improve and strengthen existing structures. One of these methods is called post-installed shear reinforcement for existing flat slab systems, where the used reinforcement-type is installed in the critical slab-column area of the slab. The methods use different reinforcement-types such as shear bolts, screw anchors or bonded anchors [5]. The estimation of the punching shear resistance depends on several variables (geometry, mechanical, material properties, etc.) with some degree of uncertainty. This study aims to investigate the safety performance of the European Eurocode 2 (EC2) design code [6, 7] for the design of post-installed reinforcement in flat slab structure endangered by punching shear, using probabilistic analysis. The probabilistic analysis is conducted based on the Monte Carlo simulation technique implemented using a MATLAB code developed in this study. This contribution intends to present a safety efficient method for the design of post-installed reinforcement in existing structures.

2 Punching Shear Resistance Formulations for Flats Slabs with Post-installed Retrofitting

The equations presented in this section are used to determine the punching shear capacity of a flat slab without shear reinforcement, and the contribution of the anchors used as post-installed reinforcement. These equations allow the characterization of the total resistance of the system.

2.1 Shear Resistance Formulations of the Eurocode [6, 7]

The characteristic resistance of a slab without punching shear reinforcement in $\left(\frac{MN}{m^2}\right)$ is expressed by Eq. (1).

$$\nu_{Rdc} = C_{Rd,c}k(100\rho_l f_{cm})^{\frac{1}{3}} \quad (1)$$

where f_{cm} = mean compressive concrete strength; $k = 1+\sqrt{\frac{200}{d}} \leq 2$ with d in (mm); $\rho_l = \sqrt{\rho_{ly}\rho_{lz}} \leq 0.02$ = longitudinal reinforcement ratio for the y- or z-axis; $C_{Rdc} = 0.18$ [6].

The equations of the anchors in tension according to the EC2 Part 4 [7] are expressed below.

The yield strength of an anchor (in kN) is expressed by Eq. (2).

$$N_{Rm,s} = A_i f_{yvm} \quad (2)$$

where A_i = cross-sectional area of the anchor in mm^2; f_{yvm} = mean yield strength of the anchor in $\frac{N}{mm^2}$.

The pull-out failure of an anchor in tension (in kN) is expressed by Eq. (3).

$$N_{Rm,p} = k_2 A_{brg} f_{cm} \quad (3a)$$

where $k_2 = 10.5$ (for anchors in uncracked concrete); d_h = head diameter in mm; d_a = shaft diameter in mm

$$A_{brg} = 0.25\left(d_h^2 - d_a^2\right) \quad (3b)$$

The bond strength of an anchor in tension (in kN) is expressed by Eq. (4).

$$N_{Rm,a}^0 = \frac{h_{ef}\pi\Phi_1\tau_{bm}}{\alpha_1\alpha_2} \quad (4)$$

where h_{ef} is the effective length of the anchor in mm. Φ_1 is the concrete cover in mm. τ_{bm} is the strength of the capacity in $\frac{\text{N}}{\text{mm}^2}$. $\alpha_1 = 1.0$, $\alpha_2 = 1 - 0.15 * \frac{(c_d - \varphi_1)}{\varphi_1}$ [6]. $c_d = \min(\Phi_1, 0.5 * s)$. s is the distance of the anchors.

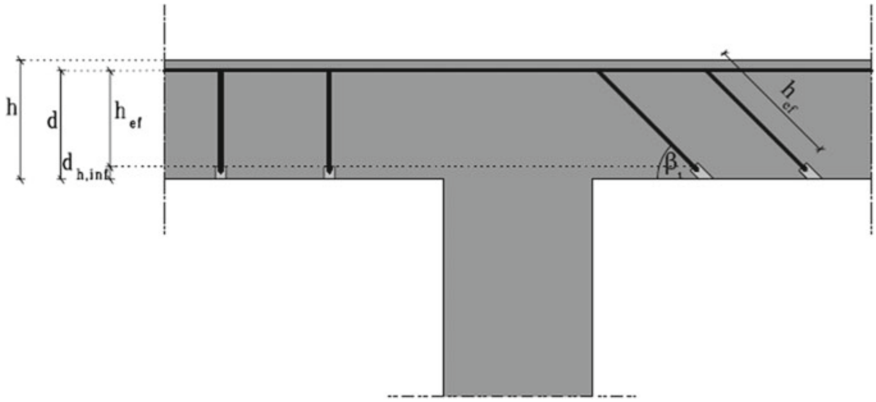

Fig. 1 Geometrical properties of a typical test case, also exemplarily indicating the anchor configuration

3 Probabilistic Analysis

3.1 Parameters for the Punching Shear Test Cases

Six scenarios of flat slabs with post-installed bonded anchors are chosen as test cases for the investigation. The post-installed bonded anchors in each test cases have a diameter of 20 mm and an anchoring plate with a diameter of 60 mm. The longitudinal reinforcement is chosen as B500 reinforcing steel bars. The material characteristics, the amount of longitudinal reinforcements, the height of the slab, the number of anchors and the angle of the anchors are varied in the chosen test cases, in order to assess their impact on the strength of the members (see Fig. 1). The material characteristics considered are the compressive strength of the concrete, the yield strength of both the transverse and longitudinal reinforcements and the bond strength of the adhesive. Table 1 details the parameters of the chosen test case. Figure 1 provides an overview of the geometrical properties of the test cases and explains the variables.

3.2 Probability Models for the Basic Random Variables

The variables of the test cases 1–6 presented in Table 1 is used in the probabilistic analysis. The variables are set as random variables that are normally distributed. The uncertainties in the variables (caused by time-dependent effects, inaccuracies or human errors) should be taken into account. Given this, the characteristic values of the variables (presented in Table 1) are converted to mean values. The values of the compressive strength of the concrete and the bond strength of the adhesive are

Table 1 Parameters for the test case (1–6)

	h (m)	d (m)	f_{ck} (MPa)	n (−)	$f_{yv,k}$ (MPa)	β_i (°)	$\tau_{b,k}$ (MPa)	A_s (cm^2)	$f_{yt,k}$ (MPa)
Case 1	0.35	0.32	C20/25	20	500	90	10	20	500
Case 2	0.35	0.32	C30/37	15	500	90	10	20	500
Case 3	0.30	0.27	C20/25	15	500	90	10	30	500
Case 4	0.30	0.27	C40/45	20	500	45	10	30	500
Case 5	0.25	0.22	C25/30	26	500	45	10	35	500
Case 6	0.25	0.22	C30/37	26	500	45	10	35	500

calculated according to EN 1990 [8]. The EN 1990 recommend values for calculating the mean values of normally distributed factors depending on the number of samples that should be used (Eqs. 5–7). For sample sizes larger than 30, the factor k_n shall be 1.64, according to EN 1990. This factor represents that the distance from the mean value of a normal distribution to the 5%-quantile of the distribution is 1.64 times the standard deviation, as shown in the equations below. This represents the overlap between the distribution of the load and the distribution of the resistance of a member (see Fig. 2).

$$X_m = X_k + k_n.\sigma \tag{5}$$

$$X_m = X_k + 1.64.\text{cov}.X_m \tag{6}$$

$$X_k = X_m - 1.64.\text{cov}.X_m \tag{7}$$

X_m is the mean value of the distribution. X_k represents the characteristic value, which equals the 5%-quantile of a normal distribution [8].

The coefficient of variation (cov) for the bond strength of an adhesive anchor τ_b is 0.10 [9]. The coefficient of variation for the yield strength of steel, according to the Federal International Federation for Structural Concrete [10] is 0.05. This could lead to higher variations in the resistance. The variation of the angle of the reinforcement depended on the chosen angle and was taken as 3° in this study. The cov used for the flexural reinforcement was taken as 0.05, and the concrete compressive strength was taken as 0.15, based on consideration of the values proposed by [11, 12]. The inaccuracies of the placing of the reinforcement are usually not higher than 10 mm, according to the Probabilistic Model Code [13]. The height of the specimens and the number of post-installed anchors are considered constant and deterministic. Table 2

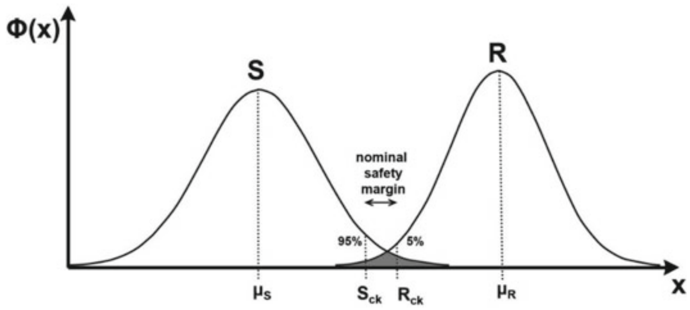

Fig. 2 Definition of safety margin, characteristic values, and basis of reliability concept

Table 2 Coefficients of variation and mean values of the variables used in the probabilistic calculation

Parameters (mean values)	h (m)	d (m)	f_c (MPa)	f_{yv} (MPa)	β_i (°)	τ_b (MPa)	A_s (cm^2)	f_{yt} (MPa)
Case 1	0.35	0.32	26.53	500	90	7.45	20	500
Case 2	0.35	0.32	39.79	500	90	7.45	20	500
Case 3	0.30	0.27	26.53	500	90	7.45	30	500
Case 4	0.30	0.27	53.05	500	45	7.45	30	500
Case 5	0.25	0.22	33.16	500	45	7.45	35	500
Case 6	0.25	0.22	39.79	500	45	7.45	35	500
cov	–	–	0.15	0.15	a	0.10	0.05	0.15

[a]The deviation in the installation is assumed to be 3° regardless of the mean value, i.e. cov equals 0.03 and 0.06 for installation at 90° and 45° respectively

shows the mean values and the coefficients of variation of the variables (calculated using Eq. 5) used in the probabilistic analysis.

3.3 Reliability Verification

In order to assess the safety and economic performance of members designed using the Eurocode design procedure, reliability verification, as presented in this section, was conducted [14]. The adequacy of design is confirmed if the limit states are not reached when the design values are introduced into the analysis models. The Eurocode demands that the design value of the resistance must be equal or higher than the design value of the load [8] (Eq. 8).

$$R_d \geq L_d \tag{8}$$

The design load calculated according to the Eurocode 2 requirements has a connection with the resistance. The design value for resistance may be obtained directly by dividing the characteristic value of a material or product resistance by 1.5 [15]. In order to obtain the design value of the load, the characteristic load is divided by 1.35, according to DIN EN 1990. Therefore, Eq. (8) is further expressed as (9).

$$R_d = \frac{R_k}{1.5} \geq L_d = 1.35.L_k \tag{9}$$

The characteristic value of the resistance equals to 5%-quantile of the distribution [8], which means that only 5% of the resistances are lower than the characteristic value. On the contrary, the characteristic value of the load equals the 95%-quantile of the distribution, since only 5% of the loads should be lower than this value. This way, one can ensure realistic values for both the resistance and the load. The 5%-quantile of the resistance and the 95%-quantile of the load are calculated according to Eqs. (10) and (11), respectively.

$$R_k = R_{5\%} = R_m - 1.64\sigma_R \tag{10}$$

$$L_k = L_{95\%} = L_m + 1.64\sigma_L \tag{11}$$

The standard deviation can be calculated by the multiplication of the coefficient of variation and the mean value of the distribution i.e. $\sigma_R = \text{cov}_R * R_m$ and $\sigma_L = \text{cov}_L * L_m$. The coefficient of variation for resistance cov_R was calculated in this study using the MATLAB code and was obtained as 0.10. The International Federation of Structural Concrete (*fib*) bulletin 80 [10] provides values for the coefficient of variation for basic variables in probabilistic models. The *fib* bulletin 80 gives a coefficient of variation of 0.10 for shear loads. With $\text{cov}_R = \text{cov}_L = 0.10$, therefore, Eqs. (7) and (8) can be expressed by Eqs. (12) and (13).

$$R_k = R_m - 0.164.R_m \tag{12}$$

$$L_k = L_m + 0.164.L_m \tag{13}$$

Replacing the characteristic values of the load and resistance in (9), the equation is further expressed by Eq. (14).

$$\frac{R_m - 0.164 * R_m}{1.5} \geq 1.35 * (L_m + 0.164 * L_m) \tag{14}$$

In order to obtain the mean value of the load, Eq. (14) is further solved to obtain Eq. (15). In this way, we get an equation, based on the mean resistance of a member, to calculate the mean value of the load that can be applied to each member.

$$L_m = 0.354 R_m \tag{15}$$

Considering the fact that Eqs. (12)–(15) are based on the Eurocode 0 and the Eurocode 2 design provisions, the resistance R_m required in Eq. (15) to estimate the mean value of the applied load is taken as the resistance of the Eurocode design formulation. This way, we have a load that is calculated for each test case individually, considering every variable. Hence, the probabilistic model for the load is obtained with a mean value of L_m, coefficient of variation of 0.10 and a normal distribution.

3.4 Determination of the Failure Probability

In order to confirm if the resistances values obtained according to the design guideline is sufficient and reasonable, the probability of failures is calculated. This is done by subtracting the total resistance value from the load value for each iteration, in accordance with the typical Limit State Equation $G = R - L$ [14]. When the value is positive, the resistance is higher than the load, representing Eq. (8). When the iterations yield negative values, a failure instance is marked because the load is higher than the total resistance of the slab. To estimate the probability of failure P_f out of several samples, the number of failures is divided by the number of samples.

The assessment is based on a target probability of failure $P_{f,T} = 1 \times 10^{-6}$ according to the EN 1990 for reliability class two (RC2) structure for a one-year reference period. This can be further explained as one failure in a million samples. The civil engineering industry also works with reliability index β [14], which is related to the probability of failure by the expression in (16).

$$\mathrm{P}_f = \Phi(-\beta) \tag{16}$$

where Φ is the cumulative distribution function of the standardised normal distribution.

The target probability of failure $P_{f,T} = 1 \times 10^{-6}$ for RC2 structure is equivalent to the target reliability index $\beta_T = 4.7$ [8]. The probability of failure and the connected reliability index is calculated for every column slab configuration of Table 1. The results are compared to the performance requirements recommended by the basis of design standards EN 1990 for RC2 structures.

4 Results and Discussions

4.1 Discussion of Estimated Deterministic and Probabilistic Resistances

The design values and the characteristic values obtained by the deterministic and the probabilistic calculations are presented in Fig. 3. The 5%-quantiles of the Eurocode 2 are 18 to 36% higher than the respective characteristic resistances. By comparing the design resistances with the 5%-quantiles, the design resistances are significantly lower. In addition to that, the ratio between the resistances are almost the same (with small variation) for the different test cases—the 5%-quantiles of the Eurocode are nearly two times the design resistance in most cases.

Test case 1 has the highest shear resistance compared to the other test cases. The high shear resistance obtained for test case 1 can be attributed to the combination of its large member size ($d = 0.32$) and the high number of anchors ($n = 20$) when compared to the other test cases. This is expected as the larger size of the member, and the high number of anchors implies a higher contribution from the concrete and anchor reinforcement, respectively, to total shear resistance.

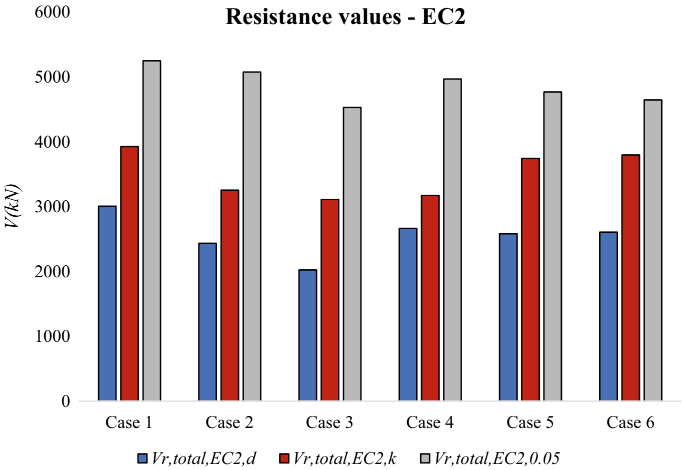

Fig. 3 Estimated total resistances in terms of design (Vr, total, EC2, d), characteristics (Vr, total, EC2, k) and 5%-quantiles values (5%-quantile values)

Table 3 Mean values μ and standard deviations σ of the total resistances $V_{r,total}$ according to EC2 codes (kN)

Case	K_n sample size	$V_{r,total,EC2}$ (kN)	
		μ	σ
1	10 Mio	6585.7	812.6
2	10 Mio	6299.9	744.8
3	10 Mio	5675.5	696.3
4	10 Mio	6420.9	882.4
5	10 Mio	6346.3	958.7
6	10 Mio	6196.6	942.4

4.2 Assessment of the Mean Values and Standard Deviations for the Resistance

The calculated mean values and standard deviations of the total resistances for the six scenarios are presented in Table 3. The total resistances include the contribution of the concrete V_c and the contribution of the shear reinforcement anchors V_{anchor}. Test cases 2 and 5 have the lowest and the highest coefficient of variations, respectively. The test cases differ majorly in terms of the flexural reinforcement ratio, the member size and number of anchors. Thus, it can be assumed that the variation in the various input parameters resulted in the variation in the resistance of the anchors.

The degree of fitting of the distributions of the obtained resistances was examined for unbounded normal and the lognormal distribution, taken as candidate distributions. A degree of the fitting is calculated based on the maximum likelihood estimation, in order to capture the appropriateness of the function mainly at the tails of the distributions. The fit characterisation parameter obtains values between 0 and 1, with an ideal fitting denoted by a value of 0.50. It was observed that neither the normal nor the lognormal curve fit with the distributions of the resistances for the 10 million samples.

4.3 Discussion of the Obtained Probability of Failure and β-Index

The derived probabilities of failure for the 10 million samples are presented in this section. The load applied on the samples is based on the approach of the Eurocode and was, therefore, kept the same for every sample. As shown in Table 4, the probabilities and the β-values differ for each test case. This trend of the result is attributed to the fact that different parameters influence and contribute to punching shear resistance, and thus have more considerable variation in the resistances. The failure probabilities of the first four cases of the Eurocode are smaller than 1×10^{-6} (the target probability of failure according to the EN 1990 RC2 structures). Reasonable consistency in the reliability of the Eurocode procedure is observed in Table 4. The highest reliability

Table 4 Probabilities of failure P_f and reliability index β for 10 Million samples

	EC2	
	P_f	β
Case 1	$5.0\ 10^{-7}$	4.89
Case 2	$1.0\ 10^{-7}$	5.19
Case 3	3.0×10^{-7}	4.99
Case 4	6.0×10^{-7}	4.85
Case 5	2.2×10^{-6}	4.59
Case 6	2.7×10^{-6}	4.54

is obtained for a test case with the largest member size and the lowest amount of reinforcements with $\beta = 5.19$ whereas the lowest reliability is obtained for a test case with the smallest size and highest amount of anchor reinforcement with $\beta = 4.54$.

Generally, EC2 has acceptable failure probabilities and thus, reliability indices above 4.7 target value (EN 1990 target reliability for reliability class RC2) in most cases. Considering the obtained results, the approach of the Eurocode has very reasonable and good results for post-installed anchors in flat slabs. Therefore, the Eurocode methods seem to be good design approximation of the design of retrofitting existing flat slabs and thus ensuring economy and enough safety against punching shear.

5 Conclusion

Flat slabs are slabs with low construction heights that are supported by columns or walls. These types of slabs are endangered by the risk of punching shear failure. This study aims to investigate the degree of the safety performance of EC2 method for the design of post-installed reinforcement in an existing flat slab structure endangered by punching shear, using probabilistic analysis.

- To assess the adequacy of the design concept, a probabilistic analysis was proposed. The investigation was conducted based on the Monte Carlo simulation technique implemented using a MATLAB code developed in this study. The main criterion considered to decide whether the resistances obtained from the design code is reasonable or not is the probability of failure or reliability index obtained for each test case. The obtained probability of failure is compared to the target probability of failure recommended by EN 1990 for RC2 structures.
- Assessment of the obtained results indicates that the reliability indices and probability of failure obtained for Eurocode method for post-installed anchors in flat slabs are in accordance with the target reliability requirement for Reliability Class 2 structures prescribed by the basis of design standards EN 1990, for most of the test cases considered in this study. Therefore, the Eurocode method seems to reflect reasonable design approximation of the design of retrofitting existing flat

slabs and thus ensuring enough safety against punching shear for the test cases investigated.
- This contribution established a safe method for the design of post-installed reinforcement in existing structures.

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