



Research paper

Strengthening of RC slabs against punching shear in theory and practice

Tadeusz Urban¹, Michał Gołdyn², Łukasz Krawczyk³

Abstract: In the paper the problem of strengthening of flat slabs against punching shear was discussed. Selected methods verified on the basis of experimental tests such as increasing size of the support, applying post-installed shear reinforcement or increasing the main reinforcement by installing additional steel flat bars, were presented. The previous studies demonstrated, that the last method allows for an increase in punching shear resistance of up to 90%, depending on the longitudinal reinforcement ratio. The example of the application of such strengthening technique in the real structure was described. The use of steel flat bars located in the vicinity of the columns and additionally anchored to the slab made possible to compensate for the load capacity deficiencies that occurred due to execution errors (lowering of the main reinforcement within the support zones).

Keywords: strengthening, punching shear, flexural reinforcement, post-installed reinforcement, RC slab

¹Prof., PhD., Eng., Lodz University of Technology, Department of Concrete Structures, al. Politechniki 6, 93-590 Łódź, Poland, e-mail: tadeusz.urban@p.lodz.pl, ORCID: 0000-0001-7494-3747

²PhD., Eng., Lodz University of Technology, Department of Concrete Structures, al. Politechniki 6, 93-590 Łódź, Poland, e-mail: michal.goldyn@p.lodz.pl, ORCID: 0000-0002-7791-1940

³PhD., Eng., Lodz University of Technology, Department of Concrete Structures, al. Politechniki 6, 93-590 Łódź, Poland, e-mail: lukasz.krawczyk@p.lodz.pl, ORCID: 0000-0002-0406-7750

1. Introduction

The monolithic column-and-slab systems are commonly used, what cause that making mistakes is highly possible. The critical areas in these structures are the support zones, in which most often emergency states occur due to insufficient punching shear capacity. In drastic cases, they lead to progressive collapse of the entire buildings. The biggest disaster for this reason took place in Seoul (Sampoong Gallery) in 1995, in which 502 people died and several hundred were injured [8]. Examples of other structural failures in Switzerland are: the shopping centre Serfontana (1970), underground parking garages in Geneva (1976), Bluche (1981) and Gretznebach (2004). [7, 11].

2. Experimental basis of strengthening slabs against punching shear

2.1. The state of art

The need to strengthen existing slabs may be caused by errors at the design or construction stage. It can result also from a change in use of a building, as well as the occurrence of the accidental actions (e.g., fire) or operating wear resulting from many years of use. One possibility is increasing of the dimensions of the support. This procedure requires access from the bottom surface of slab. The authors of the paper [9] suggest three possibilities of increasing support area, see Figure 1: increasing diameter of column at the whole height of the storey using shotcrete (a), enlarging the column head using shotcrete (b) and using steel supporting structure (c). In the paper [9] the experimental verification of the second method was presented.

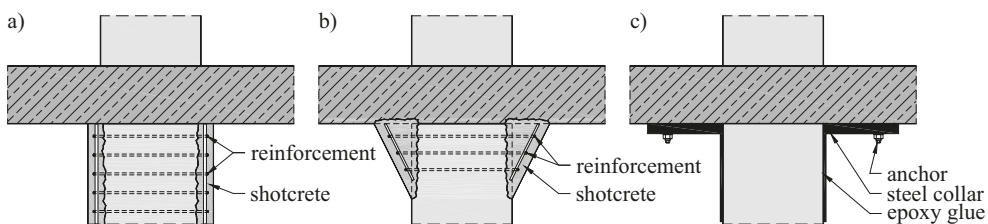


Fig. 1. Methods for increasing the transverse dimensions of columns proposed in paper [9]

The other possibility of increasing punching shear resistance is described in papers [3, 4]. Effect of this type of strengthening is similar to increasing area of support. In Figure 2 steel plates and bolts used to increase load carrying capacity are presented. The steel plates (6.35 mm thick) were bonded to concrete by epoxy resin and connected by 8÷16 bolts (diameter 19 mm). For the elements, which were strengthened by two steel welded L-shaped plates at each surface, an increase in punching shear capacity of about 50% was observed. Using four separate steel plates resulted in 36% higher load capacity.

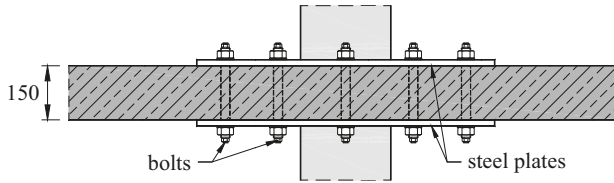


Fig. 2. Cross section of element tested by Ebead and Marzouk [4]

Steel collar was used by Noakowski [12] in order to strengthen the flat slab in an office building. Details of the strengthening are shown in Figure 3. The strengthening was realized in the following stages:

- preparing of concrete surface of the column and the slab,
- bonding of steel collar to bottom surface of the slab,
- bonding of steel accessories to column and tightening to column by prestressing bolts,
- installing vertical bolts and tightening them to the steel plates bonded to the slab.

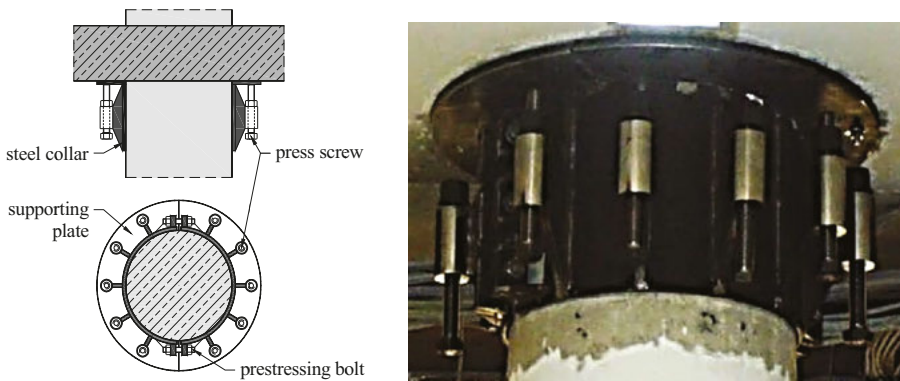


Fig. 3. Strengthening of the support zone applied by Noakowski [12]

The other option is to install shear reinforcement in the existing slab. Depending on the method of applying the reinforcement, anchoring can be performed by the adhesive forces (Fig. 4a) or mechanically, by stud's head and nut (Fig. 4b). The authors of the paper [9] were the first who applied this method. In the paper [11] a similar approach (see Fig. 4a) is presented. In case of the slabs with the main reinforcement ratio of 1.5%, a strengthening ratio of about 30÷70% was obtained compared to the element without transverse reinforcement.

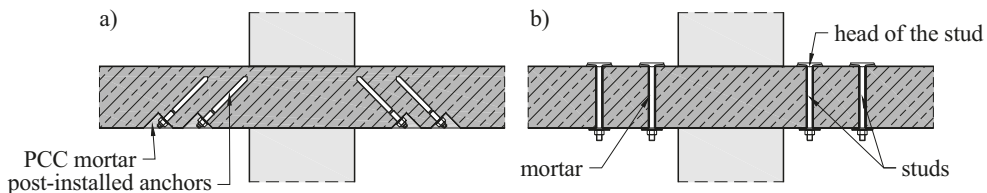


Fig. 4. Strengthening of the punching zone with additional transverse reinforcement, according to [1,9,11]

The Polak approach [1, 13] of punching shear strengthening by using post-installed transverse reinforcement is differently. In this concept, the strengthening operation consists in drilling holes through the entire thickness of the slab and embedding in them screws mechanically anchored on both sides of the slab. An example of the strengthened support zone is shown in Fig. 4b. The results of laboratory tests confirmed the high effectiveness of this method.

Feix et al. [6, 19], strengthened 200 mm thick slabs with a reinforcement ratio of $\rho_l = 2.24\%$ (see Fig. 5). By using 32 special threaded bolts, evenly spaced on 8 circumferences, they obtained strengthening ratio $29\div 53\%$, depending on the diameter of the bolts, as well as whether they were screwed directly into the concrete or additionally installed with a special mortar.

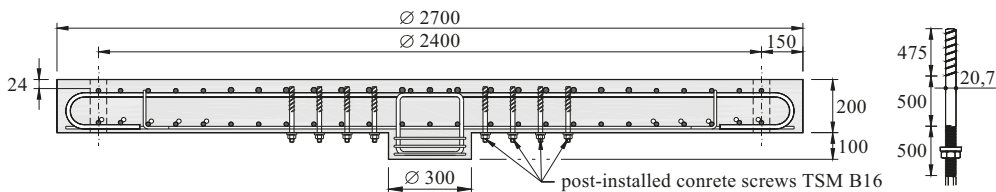


Fig. 5. Cross-section of the slab strengthened with TSM B16 bolts in tests [6]

However, strengthening methods that require drilling of holes cause a lot of controversy. Some researchers consider them risky because the structure of concrete can be damaged. By using diamond drilling rebars can be cut, because the upper and lower reinforcement mesh in the existing structure do not have to coincide with each other in the plan. In addition, the strengthening method proposed by Polak [13] requires access to the slab at the top and at the bottom.

2.2. Strengthening slabs by increasing longitudinal reinforcement

It is well known that punching shear capacity is dependent on flexural resistance. Eurocode 2 [5] included this fact by introducing the longitudinal reinforcement ratio in calculations of punching shear resistance $v_{Rd,c}$, according to formula (2.1):

$$(2.1) \quad v_{Rd,c} = C_{Rd,c} k (100\rho_l f_{ck})^{\frac{1}{3}} \quad [\text{MPa}]$$

where: $C_{Rd,c}$ – empirical coefficient [–], k – scale coefficient [–], ρ_l – average ratio of main reinforcement [–], f_{ck} – characteristic compressive cylinder strength of concrete at 28 days [MPa].

Figure 6 shows the effect of the main reinforcement ratio within the support area on the punching shear resistance for the two commonly used concrete strengths $f_{ck} = 25$ and 35 MPa. By increasing the reinforcement ratio from 0.5% to 2.0% an increase in shear resistance of 60% can be observed.

Eurocode 2 is dedicated to currently designed concrete structures reinforced with the steel of yield strength in the range of $400 \leq f_{yk} \leq 600$ MPa. However, in the existing structures it is possible to encounter steel with different parameters also outside this range. According to the authors, a more precise parameter is the mechanical reinforcement ratio $\omega = \rho_l f_{yk} / f_{ck}$ which, in addition to the geometric reinforcement ratio ρ_l , also takes into account the influence of the

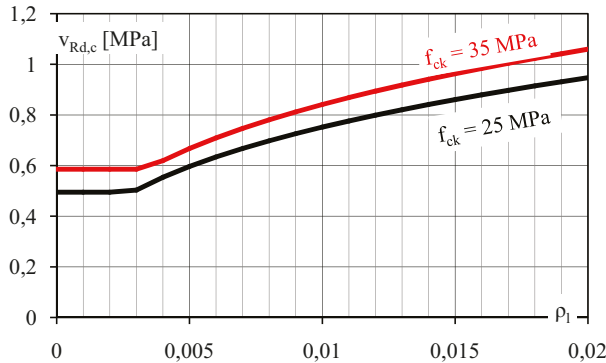


Fig. 6. The effect of the slab reinforcement ratio on the shear stress in the basic control perimeter u_1 [5]

yield strength f_{yk} and concrete compressive strength f_{ck} on the punching shear capacity. This problem is discussed in more detail in [15]. According to the Urban proposal, the punching shear resistance can be described as a function of the mechanical reinforcement ratio (see Fig. 7). Three principal failure mechanisms of column-slab connections can be distinguished:

- flexure for $\rho_l f_{yk} / f_{ck} \leq 0.15$,
- combined flexure and shear for $0.15 < \rho_l f_{yk} / f_{ck} \leq 0.3$,
- shear for $\rho_l f_{yk} / f_{ck} > 0.3$.

The flexure mechanism is characterized by yielding of the main reinforcement in a large area close to the support, which is accompanied by a considerable opening of the cracks. The ultimate limit state is clearly signaled. In the compression zone of the slab close to column edge, concrete strains are significant and as a result crushing can be observed. At the high mechanical reinforcement ratios (above 0.3) approaching of punching shear capacity is a result of pure shear, without yielding of the main reinforcement. Failure is not signaled

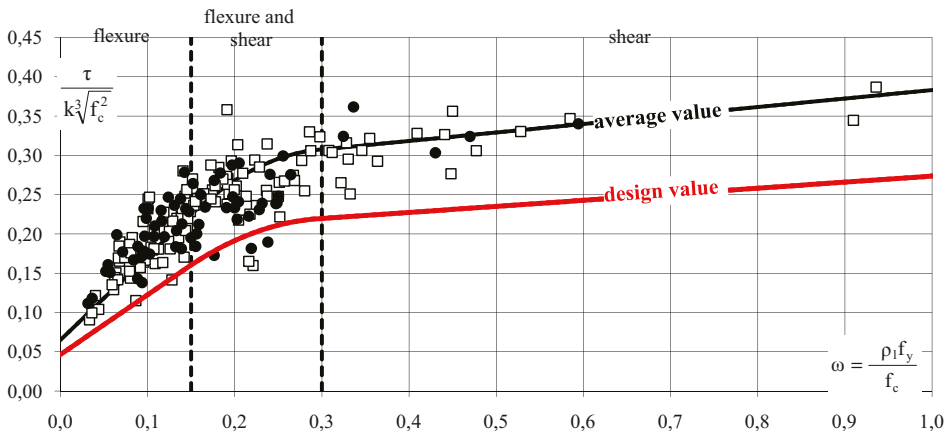


Fig. 7. Standardized critical stresses on the control perimeter at a distance $0.5d$ from the edge of the column as a function of the mechanical reinforcement ratio according to [15]

by any significant increase in the crack widths. This mechanism is characteristic for high reinforcement ratios and steel with a high yield strength. It can also take place in slabs with average reinforcement ratios (0.5÷1.0%), made from low strength concrete. Such situations may also occur during the disassembly of the formwork, when the concrete has not yet reached its full design strength.

According to method [15], the critical shear stress ($v_{u,d}$) corresponding to each failure mechanism are described by the following empirical formulas:

$$(2.2) \quad \text{for } \frac{\rho_l \cdot f_{yk}}{f_{ck}} \leq 0.15: \\ v_{u,d} = \frac{1}{\gamma_c} \left(0.065 + 1.064 \frac{\rho_l \cdot f_{yk}}{f_{ck}} \right) \cdot \sqrt[3]{f_{ck}^2} \text{ [MPa]}$$

$$(2.3) \quad \text{for } 0.15 < \frac{\rho_l \cdot f_{yk}}{f_{ck}} \leq 0.30: \\ v_{u,d} = \frac{1}{\gamma_c} \left[1.97 \cdot \frac{\rho_l \cdot f_{yk}}{f_{ck}} - 3.15 \cdot \left(\frac{\rho_l \cdot f_{yk}}{f_{ck}} \right)^2 \right] \cdot \sqrt[3]{f_{ck}^2} \text{ [MPa]}$$

$$(2.4) \quad \text{for } \frac{\rho_l \cdot f_{yk}}{f_{ck}} > 0.3: \\ v_{u,d} = \frac{1}{\gamma_c} \left(0.275 + 0.108 \cdot \frac{\rho_l \cdot f_{yk}}{f_{ck}} \right) \cdot \sqrt[3]{f_{ck}^2} \text{ [MPa]}$$

The design punching shear resistance of the slab without transverse reinforcement is given by the formula:

$$(2.5) \quad V_{u,d}(c) = k_{d/c} \cdot k_s \cdot v_{u,d} \cdot u_p \cdot d \text{ [kN]}$$

where: $k_{d/c}$ – coefficient depending on the ratio of the effective depth (d) to effective dimension (c) [for a square column, equal to the side length, and in other cases equal to the value $A_c^{0.5}$ (A_c is the cross-sectional area of the support) [-], $k_{d/c} = 0,6 + 0,889d/c$ [-], k_s – size effect factor, $k_s = 0.5 + (50/d)^{0.5} \leq 1,0$ [-], $v_{u,d}$ – design limit stresses in the slab control section according to the formulas (2.2, 2.3 and 2.4) [MPa], u_p – length of the control perimeter located at a distance $d/2$ from the column edge [mm], d – average effective depth of the slab [mm], for different reinforcement power, it can be taken as: $d = (A_{sx} \cdot d_x + A_{sy} \cdot d_y) / (A_{sx} + A_{sy})$, A_{sx} and A_{sy} denote the cross-sectional areas of the tensioned reinforcement in the directions x and y respectively, d_x and d_y denote effective depths for these directions.

The methods of increasing the longitudinal reinforcement are shown in Fig. 8. The strengthening with flat bars (without concrete overlay) was experimentally tested in the laboratory of the Department of Concrete Structures at the Lodz University of Technology. Figure 9 shows one of the tested specimens after installing the flat bars and before the test. In the Figure 10 the saw-cut along the side of the column after the test was presented.

A very common error encountered on construction sites consists in insufficient stabilization of the top reinforcement meshes, which move downwards, reducing the effective depth of the cross-section d . This results in a limitation of the bending resistance. In the case of support

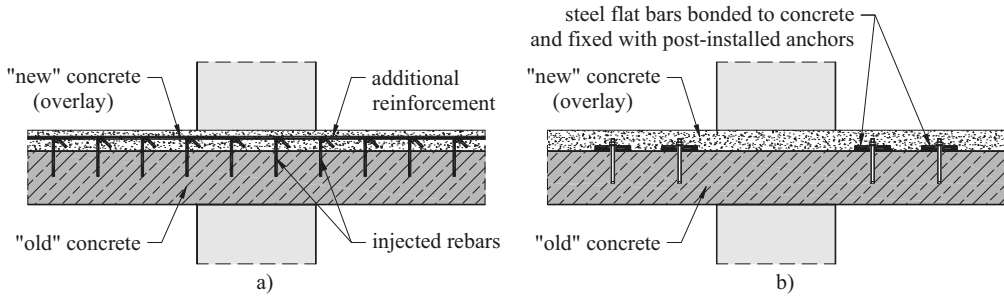


Fig. 8. Strengthening of the support zone with: a) external reinforcement placed in the additional top layer of concrete, b) bonded flat bars anchored with bonded screws

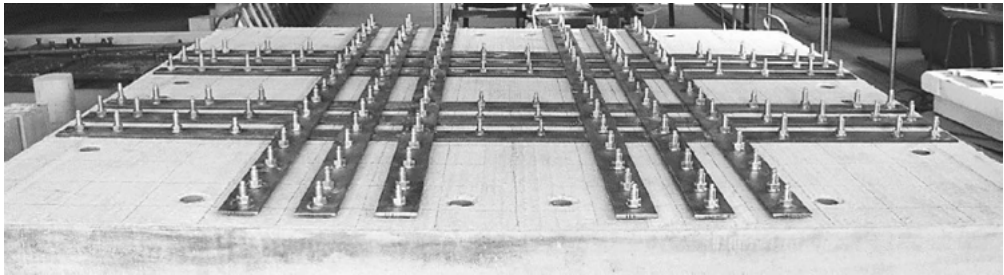


Fig. 9. An example of a slab strengthened with steel flat bars tested at the Department of Concrete Structures at the Lodz University of Technology [17]



Fig. 10. View of the saw-cut of the specimen after failure [17]

zones the additional problem of decrease in punching shear resistance can occur. In order to verify the possibility of repairing such type of errors, experimental tests were carried out on models of the support zones. All of the specimens were made from concrete from the same batch. Two of them were control elements – the S-3 model with a nominal cover equal to 20 mm, while the S-4 – with a cover increased to 50 mm. The other two slabs, in which the upper main reinforcement was also lowered, were strengthened with steel flat bars (8 pieces

for each slab) in order to compensate for losses in load capacity resulting from increasing the concrete cover. The flat bars were bonded to concrete over the entire contact surface and additionally tightened with screws. One of the models (WPSK-8') was strengthened before the load was applied, while the other one (WPSK-8'') was strengthened under load. The mean concrete strengths were determined on cubes (with 150 mm side length) and cylinders (with a diameter of 150 mm and height of 300 mm) and were equal to 46.1 MPa and 39.8 MPa, respectively. The main reinforcement, with an average yield strength of 573.2 MPa, was made in the form of an orthogonal mesh of $\varnothing 12$ mm bars at 150 mm in both directions. As a result, the reinforcement ratio was 0.51% for the slab with the nominal cover and 0.64% for the reference model with lowered main reinforcement. Steel flat bars with a cross-section of 80×8 mm, from steel of an average yield strength of 316 MPa (S235 grade steel) were used to strengthen the test specimens. The results of these tests are summarized in Table 1. It was found that the displacement of the top reinforcement by 30 mm from the nominal location, which meant an increase in cover of 250%, resulted in decrease in the punching shear resistance by over 20%. The use of external reinforcement in the form of steel flat bars allowed to compensate the error resulting from the reduction in the effective depth. The increase in load capacity in relation to the control specimen S-3 was 47% for the WPSK-8' and 42% for the WPSK-8'' element. The detailed results of these studies are presented in the paper [18].

Table 1. Results of punching shear tests on the slabs strengthened with steel flat bars

Model	Measured effective depth d [mm]	V_{exp} [kN]	Ratio $V_{\text{exp}}/V_{\text{exp S-3}}$	Ratio $V_{\text{exp}}/V_{\text{exp S-4}}$
S-3	147	475	1,000	1,293
S-4	118	367	0,773	1,000
WPSK-8'	118	700	1,474	1,907
WPSK-8''	119	675	1,421	1,839

It is even more advantageous to compare the load capacities of the strengthened models with the S-4 control element (50 mm cover). In this case, the achieved increase in load capacity is as much as 91% and 84% for WPSK-8' and WPSK-8'' specimens, respectively. Strengthening the slab directly under the load resulted in only a slightly lower experimental load (of about 6%) than for the model with the external reinforcement installed before the load was applied. This proves the high effectiveness of the applied strengthening method. Table 2 presents the comparison of theoretical calculations with the experimental results. Theoretical calculations were made according to the Urban's method, presented earlier. Average values of material strength parameters were included in the calculations (f_{ym} , f_{cm}) and obviously the partial safety factor $\gamma_c = 1$ in formula (2.2). As can be seen, only slight differences between theoretical predictions and experimental loads in range of $1 \div 10\%$ were stated. All of the theoretical predictions were safe.

The obtained results confirmed the high effectiveness of the proposed concept of strengthening of reinforced concrete slabs against punching by increasing the flexural reinforcement. Strengthening with steel flat bars bonded to the slabs with simultaneous using bolts proved to

Table 2. Comparison of experimental and theoretical load carrying capacities

Model	d_{eff} [mm]	$\rho_{l,eff}$ [%]	$\rho_{l,eff} \cdot f_{ym}/f_{cm}$	$A_{s,er}$ [mm ²]	V_{exp} [kN]	V_{cal} [kN]	V_{exp}/V_{cal}
S-3	147	0.51	0.072	–	475	442.3	1.07
S-4	118	0.63	0.087	–	367	332.9	1.10
WPSK-8'	149	0.97	0.140	357	700	667.7	1.05
WPSK-8''	149	0.96	0.139	357	675	668.2	1.01

be very effective. This method should be recommended especially for reinforced concrete slabs with low mechanical reinforcement ratios ($\rho_l f_{yk}/f_{ck} \leq 0.15$).

3. Case study

The presented case concerns a 280 mm thick reinforced concrete composite floor (consisted of 220 mm thick in-situ concrete cast on filigree precast slabs), in a two-story production facility with dimensions 55 × 64 m in plane – see Fig. 11.

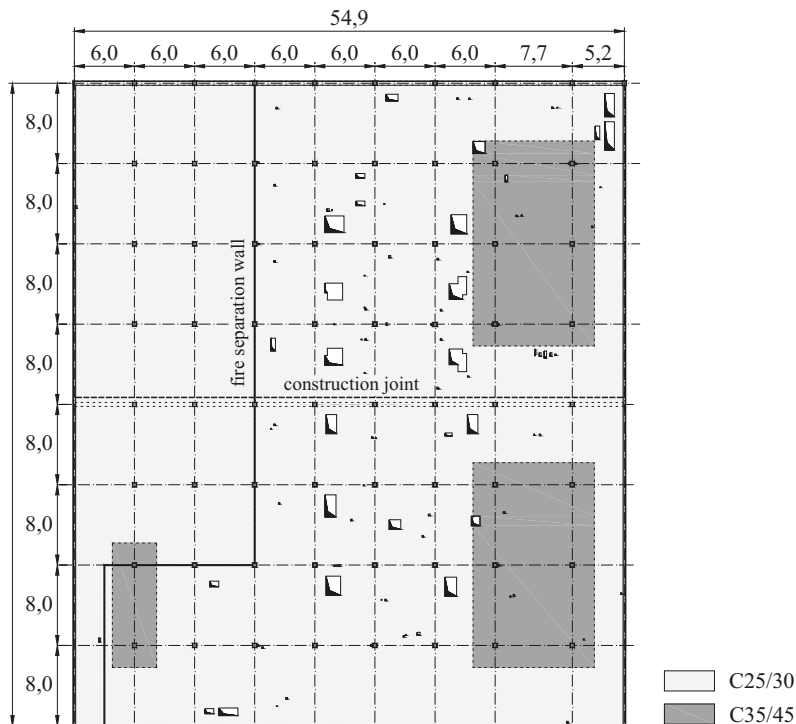


Fig. 11. Floor plan

The structure was designed as column-and-slab system, supported on 400×400 mm internal columns and 240 mm thick external wall from silicate blocks on a cement-lime mortar. A regular grid of columns was adopted with an axial spacing of 6.0/7.7 m × 8.0 m. In the middle of the building, a construction joint was designed. The connection of the two separated parts of floor slab was made by means of Halfen CRET shear dowels. A technical ceiling was suspended to the floor slab, in order to create a space of about 2 m for technical installations and equipment – see Fig. 12.



Fig. 12. View of the space between the floor slab and a suspended ceiling with technical equipment

On the floor slab the fire separation wall was made inconsistently with the detailed design, from silicate blocks. As part of the extemporary recommendations, the heavy masonry wall was replaced with a light partition wall made from plasterboards.

During the inspection, numerous cracks were found on the upper surface of the slab. The cracks with a width of 0.3 ÷ 0.4 mm, occurred mainly in the vicinity of almost all of the internal columns (Fig. 13) and in the span area where their widths were greater and reached up to



Fig. 13. Crack pattern in the upper surface of the floor slab close to column

1.5 mm. The location of cracks in the span section coincided with location of the shrinkage reinforcement (rebar mesh with a size of 150×150 mm was used). Most of the cracks were injected with a resin some months before visit of the authors as a result of the recommendations of other experts performing technical inspection.

The owner of the facility became concerned about the revealed cracks, which occurred despite the load was highly below the design value. The authors of the paper were asked to assess the situation, after about 9 years of using the facility. During this time the ceiling was only slightly loaded – distributed live load did not exceed on average 1 kN/m^2 because on part of the surface of floor slab the only load resulted from the suspended ceiling, however the design provided for the live load of 5 kN/m^2 as well as 100 mm thick floor screed.

While analyzing the documents of the investment process stage, significant changes in the detailed design in relation to the construction design were found. The construction design provided for a monolithic 380 mm thick flat slabs with polystyrene void formers. During the construction stage the solution of the ceiling was changed to a RC composite, 280 mm thick slab. The replacement design provided for a 60 mm thick precast filigree slabs made from concrete of C25/30 strength class. The structural overlay was designed from two types of concrete: C25/30 on most of the ceiling and C35/45 within selected area (marked with dark grey in Fig. 11), where higher internal forces were expected. The design provided for a nominal cover of the reinforcement equal to 25 mm (from the bottom) and 20 mm (from the top). The on-site inspection showed a certain difference between the realized slab and the design assumptions, consisting in the division of the precast slabs in the support strips – see Fig. 14.

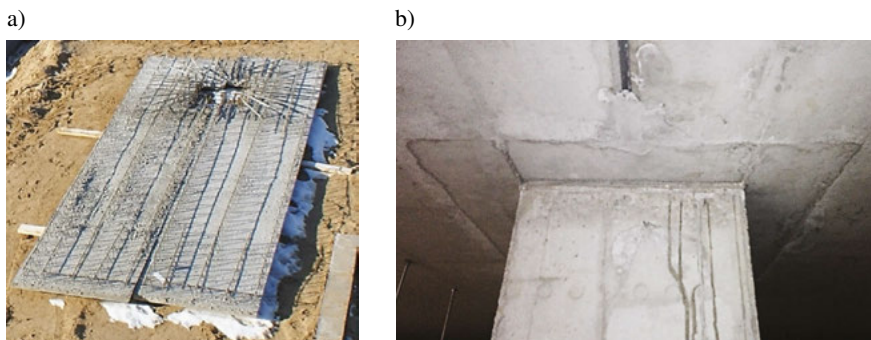


Fig. 14. Precast Filigree slabs: a) elements provided to cast within support strip, b) bottom view of the support zone with a monolithic insert of the width of 150 mm around the column

Tests on cores taken from the structure after about nine years of completion have shown that the concrete can be assigned to strength class C30/37, however, significant differences in strength of individual samples were found (characterized by the coefficient of variation of about 20.2%). The test results did not indicate that concrete of two different strength classes was built in, as it was assumed in the detailed design. Based on the inspection of the collected cores, it was possible to determine the actual position of the top reinforcement in the floor slab, which was shown in Fig. 15. It was found that the assumed cover of the upper reinforcement ($c_{\text{nom}} = 20$ mm) was exceeded several times.

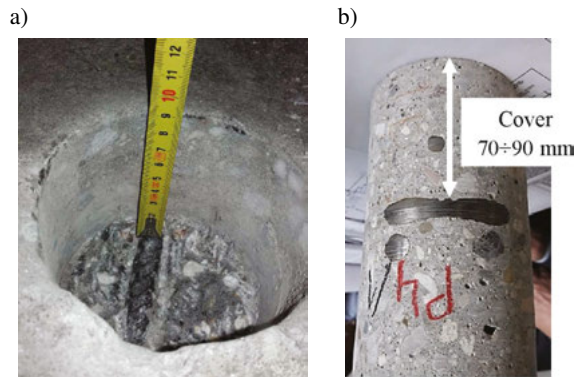


Fig. 15. Uncover of the main reinforcement in the supports area: a) hole after drilling core with visible rebar, b) concrete core with cut top reinforcement

The design provided for punching shear reinforcement in the support zones. According to the design and as-built documentation, the shear reinforcement consisted of C-shaped inserts embedded in the precast slabs, made of $\varnothing 12$ or $\varnothing 14$ rebars. The executive drawings did not contain any sections through the slab, therefore location of the reinforcement, presented in Fig. 16, should be regarded as a presumed. In order to meet the requirements regarding to minimum concrete cover, the C-type bars would have to be placed above the main reinforcement of the precast slabs. It does not follow from the design documentation that additional longitudinal bars were placed in the bent-ups of the inserts.

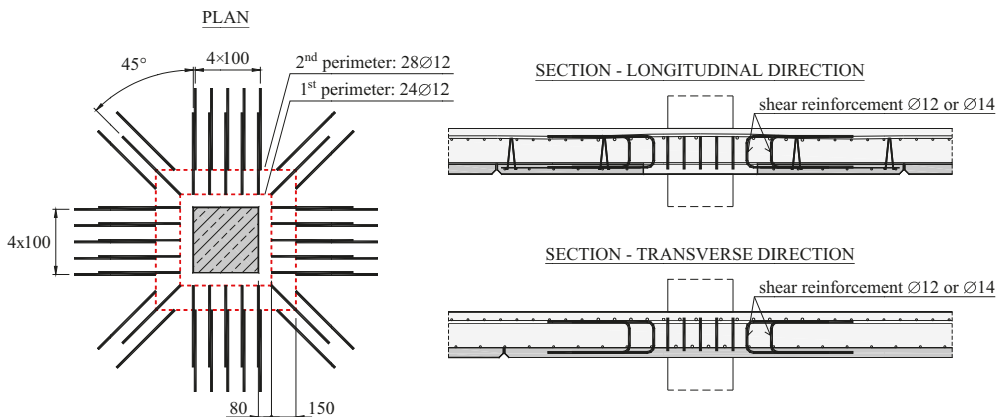


Fig. 16. Assumed location of the punching shear reinforcement according to the as-built documentation

An analysis of the photographic documentation from the construction stage, provided by the customer, clearly showed that the main longitudinal reinforcement was tied outside of the C-shaped inserts (see Fig. 17). No additional bars were placed in the corners which means that the requirements [5, 10, 14] for anchoring of the shear reinforcement in the form of links, which should enclose the longitudinal reinforcement, were not met.



Fig. 17. Detail of the reinforcement in the vicinity of support

Taking into account the existing deficits in the load-carrying capacity of the support zones, as well as the inability to strengthen the slab from the bottom, attempts were made to strengthen it by increasing the longitudinal reinforcement within the support areas. The advantage of this method consists in providing the construction works only at the upper surface of the floor slab. In order to assess the theoretical effectiveness of the designed strengthening the authors of the paper provided independent calculations according to two procedures, allowing to determine the punching shear resistance: Urban's approach (previously presented) and Eurocode 2 [5]. Due to the applied strengthening method by gluing flat bars on the upper surface of the slab, the effective depth of the slab changes – see Fig. 18. The modified effective depth can be calculated according to equation (3.1), including the locations of the primary and additional reinforcement, as well as the difference in the strength parameters of the reinforcing and structural steel:

$$(3.1) \quad d_{\text{eff}} = \frac{A_{sy} \cdot d_y + A_{sz} \cdot d_z + \alpha_s (A_{sa,y} \cdot d_{a,y} + A_{sa,z} \cdot d_{a,z})}{A_{sy} + A_{sz} + \alpha_s (A_{sa,y} + A_{sa,z})} \quad [\text{MPa}]$$

where: d_y and d_z – effective depth of the primary reinforcement, in the direction y and z respectively [mm], A_{sy} and A_{sz} – cross-section of the primary reinforcement [mm²], $d_{a,y}$ and $d_{a,z}$ – effective depth of the additional reinforcement (flat bars), in the direction y and z respectively [mm], $A_{sa,y}$ and $A_{sa,z}$ – cross-section of the additional reinforcement [mm²], α_s – coefficient reflecting the ratio of the yield strength of structural steel f_{yad} and primary reinforcement f_{ysd} ($\alpha_s = f_{yad}/f_{ysd}$) [–].

By introducing the external reinforcement in form of 4 or 6 flat bars in each direction, the increase of design punching shear resistance of the support zones from 42 to 94% compared to the existing situation was achieved. In order to provide effective strengthening the additional reinforcement had to be located as close to the column as possible, taking into account possible obstacles in form of openings as well as the location of the existing reinforcement (to avoid possible collisions of post-installed anchors with the existing reinforcement). The length of the flat bars was designed in such a way that they extend beyond the area of the radial bending moments i.e. about 1/5 of the floor span with respect to the support axis. The number of flat bars in each direction depended on the identified deficit of the load-carrying capacity. When selecting the cross-section of the flat bars, the assembling considerations were taken into

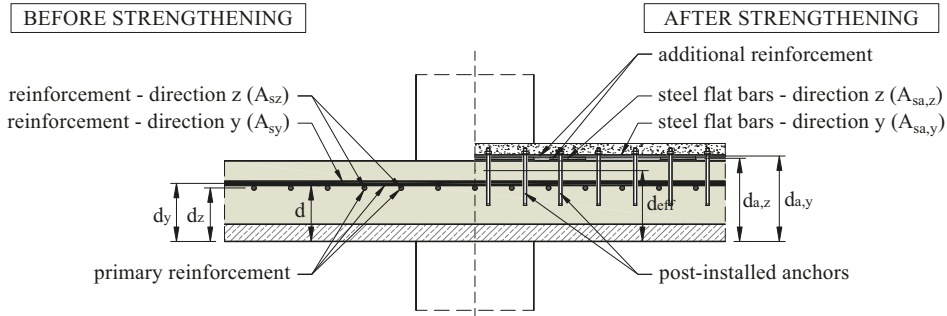


Fig. 18. Concept of strengthening of the floor slab by means of flat bars

account therefore, it was allowed to apply no more than 6 flat bars in each direction. Finally, flat bars with a cross-section of 120×8 mm made of S355 grade steel were designed. Due to the different support reactions as well as the number and location of openings, 4 to 6 flat bars in each direction were needed. It was proposed to fix the flat bars to the upper surface of the ceiling with Sikadur-30, a solvent-free, thixotropic, two-component adhesive based on epoxy resin and filler. In order to limit the undesirable failure mode, consisting in detaching the flat bars, additional anchoring with post-installed M12 threaded rods, injected with Hilti 200A adhesive were used. Figure 19 shows the examples of flat bars arrangement.

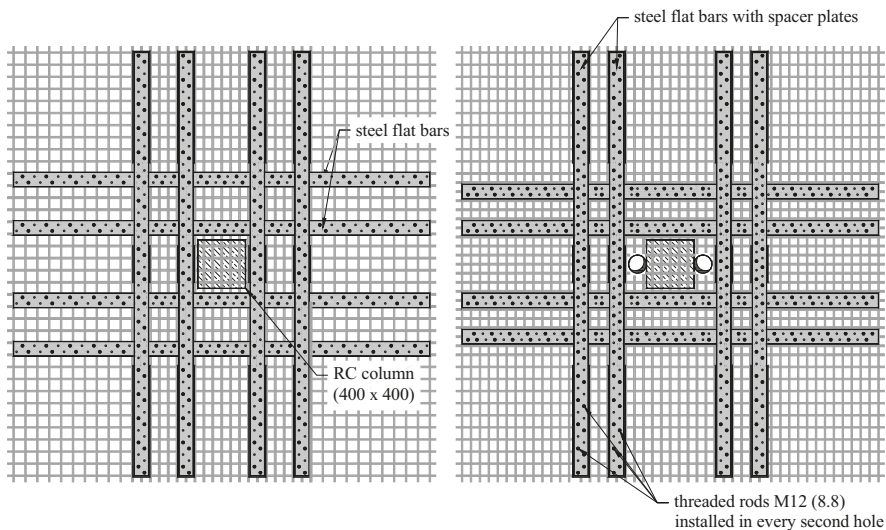


Fig. 19. Exemplary location of flat bars in the vicinity of the columns

The cruciform arrangement of flat bars required use of additional spacer plates, connected to the flat bars with fillet welds (in case of flat bars of the upper layer). As part of the manufactory preparations, in all of the elements holes were drilled, which number was twice as high as the number of anchor bolts. The decision to increase the perforation was due to concerns about

the possibility of correct installation of the anchors in all intended locations. Due to the high intensity of the primary reinforcement as well as the construction tolerances, it was to be expected that some of the openings could be located above the existing rebars. In such case, drilling had to be stopped and the adjacent hole was used for anchoring. The strengthening process was divided into the following stages:

- 1) preparation of the floor surface by removing the top layer of weaker concrete (bleeding which usually flows up in fresh concrete mix) in places where flat bars were bonded;
- 2) preparation of flat bars by cleaning the bonded surface and degreasing immediately before applying the resin and sticking the holes in the flat bars with tape to prevent flowing out the adhesive – see Fig. 20a;
- 3) dust removal and degreasing the concrete surface immediately before bonding of flat bars;
- 4) bonding the first layer of flat bars to concrete; the adhesive was spread on the surface of the concrete; and then a flat bar was applied and pressed; the excess of the adhesive was scraped off and formed into a fillet weld shape along the edge of the flat bar;
- 5) pressing the flat bars with weights and leaving for the setting time;
- 6) drilling holes in the floor slab (Fig. 20b); after the adhesive had hardened, the holes were drilled (after removing the masking tape); drilling was performed in stages – at each stage about 25% of the total number of holes provided were drilled; in case of

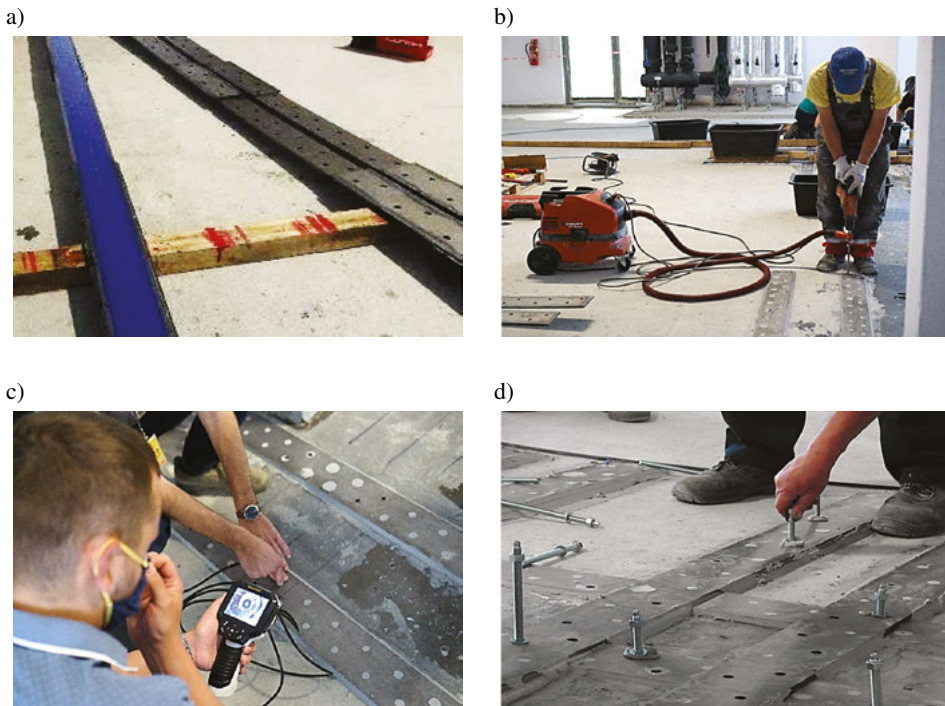


Fig. 20. Subsequent steps during the execution of the trial strengthening: a) preparation of flat bars for assembly (visible protection with tape), b) drilling the holes, c) inspection of holes using an endoscope camera, d) installing the anchors

encountering obstacle, drilling was stopped and an inspection with an endoscope camera (see Fig. 20c) was carried out – if a rebar was visible, drilling was interrupted and moved to the adjacent hole;

- 7) cleaning holes with compressed air and injecting threaded anchors (see Fig. 20d); the adhesive was applied to the holes filling about half of their volume and the anchors were installed in a rotary motion; after cross-linking of the resin, which lasted about 30÷60 minutes, the next holes were drilled and the remaining anchors were installed (the steps described in points 6 and 7);
- 8) installing of the second layer of flat bars (the steps described in points 6 and 7);
- 9) tightening of nuts and shortening of excessively protruding bolts.

Figure 21 shows the support zones after completion of the strengthening. Taking into account the experience from the trial realization, the owner decided to strengthen the ceiling in stages, depending on the current needs and technical possibilities, without intervention within interceiling space. In the first stage, documentation covering the area intended for offices was prepared. The authors pointed to the need to secure steel flat bars with 60 mm thick reinforced concrete plinths. This solution resulted from the need of anti-corrosion and fire protection. In order to ensure an even surface in the room, it was proposed to build a raised (technical) floor, supported on the existing ceiling. The floor consisting of ready-made gypsum fiber tiles will be supported on steel pedestals with adjustable height. Such solution allows for a free arrangement of the necessary installations – see Fig. 22.

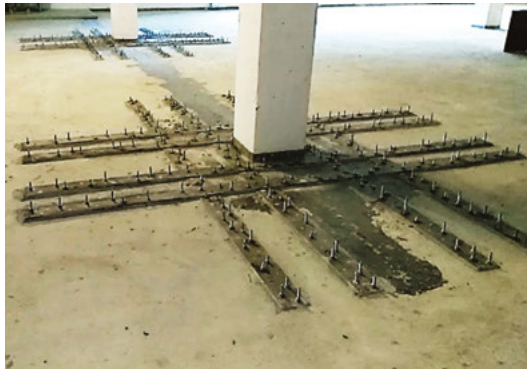


Fig. 21. View of the strengthened support zone

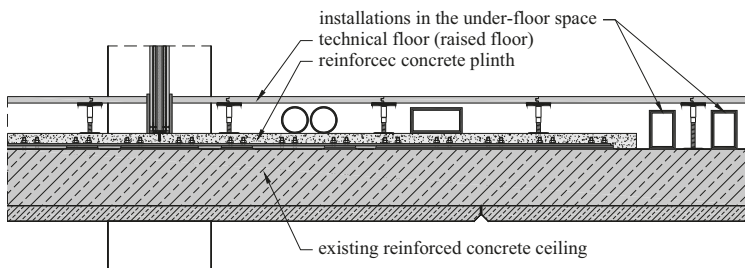


Fig. 22. Protection the external reinforcement with a concrete plinth and leveling the levels with a technical floor

4. Conclusions

The issue of strengthening the existing reinforced concrete flat slabs is still relevant today. Each of the methods presented in the paper has some advantages and disadvantages, and the choice of a specific solution depends on the conditions and application possibilities in a given facility. The biggest advantage of strengthening with external reinforcement is the need for access to the ceiling from one side only – at the top surface. Experimental studies demonstrated high efficiency of this method of strengthening of flat slabs against punching shear, especially in cases of low longitudinal reinforcement ratios, when the top reinforcement in the support zones was moved downward.

The significant increase in the reinforcement ratio is possible using steel flat bars and difficult to achieve using CFRP strips. The comparison of the effectiveness of steel flat bars and CFRP strips was presented in [16]. The example of flat bars application in another facility can be found in the paper of Buda-Ożóg and Kujda [2]. A difficulty in the application of the flat bars results from existing reinforcement, which exact location can be difficult to determine, especially in case of a large cover and high reinforcement ratio. The experience from the implementation has shown that this difficulty can be overcome by providing twice as many holes in flat bars, assuming that not all of them will be used.

References

- [1] B. Adetifa and M.A. Polak. “Retrofit of slab column interior connections using shear bolts”. *ACI Structural Journal*, vol. 102, no. 2, pp. 268–274, March–April, 2005, DOI: [10.14359/14278](https://doi.org/10.14359/14278).
- [2] L. Buda-Ożóg and J. Kujda, “Analiza numeryczna wzmocnienia strefy przysłupowej istniejącego stropu słupowo-płytowego”. *Inżynieria i Budownictwo*, vol. 74, no. 6, pp. 310–312, 2018.
- [3] U. Ebead and H. Marzouk. “Strengthening of two-way slabs subjected to moment and cyclic loading”. *ACI Structural Journal*, vol. 99, no. 3, pp. 435–444, July–August, 2002, DOI: [10.14359/12112](https://doi.org/10.14359/12112).
- [4] U. Ebead and H. Marzouk. “Strengthening of two-way slabs using steel plates”. *ACI Structural Journal*, vol. 99, no. 1, pp. 23–31, January–February, 2002, DOI: [10.14359/11032](https://doi.org/10.14359/11032).
- [5] EN 1992-1-1:2004, Eurocode 2. Design of concrete structures. Part 1-1: General rules – Rules for buildings.
- [6] J. Feix, P. Worle, and A. Gerhard. “Ein neuer Ansatz zur steigerung der durchstantragfähigkeit bestehender stahlbetonbauteile”. *Bauingenieur*, vol. 87, no. 4, pp. 149–155, April, 2012.
- [7] M. Fernandez Ruiz, Y. Mirzaei, and A. Muttoni. “Post-punching behavior of flat slabs”. *ACI Structural Journal*, vol. 110, no. 3, pp. 801–811, September–October, 2013, DOI: [10.14359/51685833](https://doi.org/10.14359/51685833).
- [8] N.J. Gardner, J. Huh, and L. Chung. “Lessons from the Sampoong department store collapse”. *Cement and Concrete Composites*, vol. 24, no. 6, pp. 523–529, 2002, DOI: [10.1016/S0958-9465\(01\)00068-3](https://doi.org/10.1016/S0958-9465(01)00068-3).
- [9] G. Hassanzadeh and H. Sundqvist. “Strengthening of bridge slabs on columns”. *Nordic Concrete Research. Publication*, no. 21, p. 12, 1998.
- [10] Model Code 2010, Final draft, fib Bulletin 66, Fédération internationale du béton, 2012.
- [11] A. Muttoni, M. Fernandez Ruiz, and J. Kunz. “Nachträgliche durchstanzbewehrung zur verstärkung von stahlbetonflachdecken”. *Bauingenieur*, vol. 83, pp. 503–511, December, 2008.
- [12] P. Noakowski. “Wzmocnienie stropów biurowca. Duże ugięcia i niedostateczna nośność na przebicie”. in *Problemy remontowe w budownictwie ogólnym i obiektach zabytkowych*, Wrocław, DWE, 2006.
- [13] M.A. Polak and W. Bu. “Design considerations for shear bolts in punching shear retrofit of reinforced concrete slabs”. *ACI Structural Journal*, vol. 110, no. 1, pp. 15–25, January–February, 2013, DOI: [10.14359/51684325](https://doi.org/10.14359/51684325).
- [14] EN 1992-1-1:2020-11-16 Eurocode 2: Design of concrete structures – Part 1-1: General rules – Rules for buildings, bridges and civil engineering structures, European Committee for Standardization, Brussels, 2020.

- [15] T.S. Urban. "Przebiecie w żelbecie. Wybrane zagadnienia". Zeszyty Naukowe, nr 959, Politechnika Łódzka, Łódź, p. 348, 2005.
- [16] T. Urban and M. Gołdyn. "How to strengthen flat slabs on punching shear – traditionally or innovative, by using fiber composite materials?". *Engineering Structures and Technologies*, vol. 11, no. 2, pp. 57–65, 2019, DOI: [10.3846/est.2019.10657](https://doi.org/10.3846/est.2019.10657).
- [17] T. Urban, M. Sitnicki, and J. Tarka. "Investigation of slab-column connections externally strengthened on punching (Badania połączeń płyta-słup wzmocnionych zewnętrznie na przebiecie)". z. 18, Wydawnictwo Katedry Budownictwa Betonowego WBAiŚ, Łódź, p. 134, 2003, DOI: [10.34658/kbb.2003.18](https://doi.org/10.34658/kbb.2003.18).
- [18] T. Urban, M. Sitnicki, and J. Tarka. "Wzmacnianie na przebiecie płyt żelbetowych z błędnie usytuowanym zbrojeniem". *Inżynieria i Budownictwo*, vol. 69, no. 9, pp. 479–483, 2013.
- [19] R. Walkner and M. Spiegl, J. Feix. "Concrete screws as a post-installed punching shear reinforcement". *Structural Concrete*, vol. 22, no. 3, pp. 709–725, 2021, DOI: [10.1002/suco.202100073](https://doi.org/10.1002/suco.202100073).

Wzmacnianie żelbetowych płyt na przebiecie w teorii i praktyce

Słowa kluczowe: wzmacnianie, przebiecie, zbrojenie na zginanie, zbrojenie dodatkowe, płyta żelbetowa

Streszczenie:

W artykule omówiono tematykę związaną ze wzmacnianiem płyt płaskich na przebiecie. Przedstawiono przegląd dotychczasowych prac badawczych jak również przykład realizacji wzmocnienia zaprojektowanego przez autorów. Wśród przytoczonych badań zaprezentowano prace, w których nośność na przebiecie zwiększono poprzez:

- zwiększenie zbrojenia głównego nad podporą,
- zainstalowanie zbrojenia poprzecznego w postaci prętów klejanych, specjalnych śrub do betonu a także trzpieni umieszczanych w otworach przewierconych przez całą grubość stropu,
- zwiększanie wymiaru poprzecznego podpory poprzez obetonowanie słupa, zainstalowanie akcesorium stalowego lub wykonanie głowicy,
- zainstalowanie płyt stalowych dookoła słupa.

Szczegółowo omówiono metodę dotyczącą wzmacniania poprzez zwiększenie zbrojenia głównego. Metoda ta została zweryfikowana eksperymentalnie przez Urbana, który zaproponował stosowanie płaskowników stalowych, mocowanych na górnej powierzchni płyty. W artykule omówiono procedurę obliczeniową, pozwalającą określać nośność na przebiecie płyt wzmacnianych zewnętrznym zbrojeniem. W metodzie Urbana nośność uzależniona jest od mechanicznego stopnia zbrojenia ($pl f_{yk}/f_{ck}$), co w łatwy sposób pozwala przeprowadzać obliczenia dla elementów z zastosowanym zbrojeniem o różnej granicy plastyczności. Przedstawione w pracy badania eksperymentalne wykazały wysoką skuteczność zaproponowanej metody wzmacniania płyt płaskich na przebiecie za pomocą stalowych płaskowników. W zależności od stopnia zbrojenia głównego możliwe było zwiększenie nośności od 40 do 80%. Rozwiązanie to pozwala na dość dużą swobodę doboru przekroju elementów wzmocnienia, co stanowi niewątpliwą zaletę względem m.in. taśm kompozytowych CFRP. Metoda ta okazuje się szczególnie skuteczna w przypadku płyt charakteryzujących się niskim stopniem zbrojenia głównego a także w sytuacji zmniejszenia wysokości użytecznej na skutek obniżenia zbrojenia głównego, spowodowanej na przykład niewłaściwym ustabilizowaniem tego zbrojenia przed betonowaniem.

W artykule przedstawiono przykład realizacji wzmocnienia płaskiego stropu zespolonego, formowanego na płytach typu filigran, w dwukondygnacyjnym budynku produkcyjnym o wymiarach w rzucie 55×64 m. W trakcie przeglądu konstrukcji stwierdzono występowanie licznych rys na górnej powierzchni stropu. Jak wykazały późniejsze oględziny, były one następstwem niedostatecznej nośności w strefach

podporowych, mimo że strop był obciążony głównie ciężarem własnym i urządzeniami, których ciężar nie przekraczał średnio 20% projektowanego obciążenia użytkowego. W trakcie późniejszych prac eksperckich stwierdzono, że taki stan rzeczy był wynikiem błędów popełnionych na etapie wykonywania konstrukcji. Polegały one głównie na przemieszczeniu zbrojenia górnego głowicowego do dołu, skutkiem czego wysokość użyteczna uległa obniżeniu nawet o 20÷30%. Ze względu na znaczne planowane obciążenia w projekcie przewidziano zbrojenie poprzeczne w strefach podporowych. Składało się ono z wkładek typu „C” (tzw. bigli) osadzonych w płytach filigran. Analiza dokumentacji z okresu budowy wykazała niezbicie, iż zbrojenie główne zostało dowiązane od zewnętrznej strony wkładek, co wiązało się z niespełnieniem wymagań norm projektowych dotyczących zakotwienia zbrojenia poprzecznego. Wskazane błędy przełożyły się na niedostateczną nośność na przebicie stref podporowych i wymusiły potrzebę realizacji wzmocnienia w związku z planami zwiększenia obciążeń przez właściciela obiektu.

Biorąc pod uwagę istniejące deficyty nośności stref podporowych a także brak możliwości realizacji prac od spodu, podjęto próby wzmocnienia poprzez zwiększenie zbrojenia głównego nad podporami. Zdecydowano o zastosowaniu w tym celu płaskowników stalowych klejonych do powierzchni stropu i dodatkowo kotwionych łącznikami wklejanymi. W celu zapewnienia skuteczności, płaskowniki zlokalizowano możliwie blisko podpór, uwzględniając przy tym ewentualne przeszkody w postaci otworów a także położenie istniejącego zbrojenia (możliwe kolizje kotew wklejanych ze zbrojeniem). Liczba płaskowników uzależniona została od stwierdzonego deficytu nośności i wynosiła od 4 do 6 w każdym kierunku. Krzyżowy układ płaskowników wymagał zastosowania dodatkowych blach dystansowych, połączonych z płaskownikami za pomocą spoin pachwinowych. Wszystkie elementy przewidziane do montażu miały wykonane, w ramach przygotowań warsztatowych, otwory, których liczba dwukrotnie przewyższała wymaganą liczbę śrub kotwiących, co wynikało z obaw odnośnie poprawnej instalacji śrub kotwiących ze względu na kolizje z pierwotnym zbrojeniem. Prace wzmocniające podzielono na etapy, obejmujące m.in. przygotowanie powierzchni stropu, klejenie płaskowników, wiercenie i czyszczenie otworów a także instalację śrub kotwiących.

Przeprowadzona próba poligonowa wykazała możliwość techniczną realizacji zaproponowanego wzmocnienia w szerszej skali. Mając na uwadze doświadczenia z realizacji próby poligonowej, właściciel obiektu podjął decyzję o etapowym wzmocnianiu stropu w zależności od bieżących potrzeb. W projekcie przewidziano zabezpieczenie płaskowników za pomocą żelbetowych cokołów, na których ustawiona zostanie podłoga podniesiona (techniczna).

Received: 2021-03-19, Revised: 2021-05-17