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ANALYSIS OF STRUCTURAL FLOORING SCHEMES USING COMPUTER MODELING OF THEIR STRESS-STRAIN STATE

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Abstract. *The study examines how deviations in reinforcement placement affect strength and deflection of monolithic slabs. Numerical modeling in LIRA-SAPR shows that a 22% reduction in effective depth decreases strength by 31–42%. Contour prestressed reinforcement reduces deflections and improves strength utilization.*

Keywords: *monolithic reinforced concrete slabs, reinforcement placement deviations, concrete cover depth, effective cross-sectional height, load-bearing capacity, stress–strain state, numerical modeling, unbonded prestressed reinforcement, tendon layout.*

Introduction

The durability and operational reliability of reinforced concrete structures, particularly monolithic floor slabs, are determined not only by the quality of design solutions and the accurate accounting of material properties but also by the level of execution in construction and installation works. Violations of the coaxiality of frame elements and deviations from the design position of reinforcement significantly affect the load-bearing capacity and deformability of structures. The application of modern materials, including composites, enhances operational characteristics and reduces cross-sectional dimensions of elements, thereby increasing the demands for precision in reinforcement installation.

One of the most common technological violations during concreting is the increase in the concrete protective layer, which leads to a reduction in the working height of the section and, consequently, a decrease in the strength and stiffness of floor slabs. In the context of contemporary trends toward reducing material intensity, an effective structural solution for monolithic flat floor slabs is the use of post-tensioning with an expanded column grid.

Publication analysis

Despite the presence of normative requirements in DBN V.2.6-98:2009, DSTU-N B EN 1992-1-1:2010, and international standards, the impact of the actual reinforcement position on the performance of beamless slabs remains insufficiently studied in domestic practice. This necessitates a comprehensive analysis of the stress-strain state of slabs, accounting for real installation deviations and modern approaches to punching shear calculations.

In study [1, 15], the influence of the actual reinforcement position on the load-bearing capacity of beamless reinforced concrete slabs is investigated within the framework of ACI 318-19 provisions. It is demonstrated that an increase in the concrete protective layer leads to a decrease in the working height of the section d and, accordingly, a reduction in punching shear resistance. Experimental investigations of "slab-column" joints indicate a 20–40% decrease in ultimate punching shear force when allowable installation deviations are exceeded. It is also established that for post-tensioned slabs, the contour arrangement of cables is more effective compared to the diagonal scheme in terms of stress uniformity and limiting deflections in near-support zones.

In works [2–6, 16], a modern scientific-methodological foundation for the provisions of EN 1992-1-1 and Model Code 2010 regarding punching shear calculations is formed. Primary attention is given to the failure mechanism, considering crack formation, nonlinear concrete deformation, and reduced sectional stiffness. The authors note that an increase in the concrete protective layer not only reduces the slab's working height but also alters crack development patterns, potentially leading to more brittle failure.It is established that the critical slab thickness significantly depends on column dimensions, concrete class, and normal stress levels. The controlled critical shear crack theory (CSCT) method provides a more substantiated prediction of load-bearing capacity compared to traditional empirical dependencies.

In study [7, 17], nonlinear numerical modeling was applied to analyze the stress-strain state of slabs in the punching shear zone. It is demonstrated that even an increase in the concrete cover by 10–15 mm in thin slabs can lead to a significant increase in deflections and premature crack formation. The authors emphasize the importance of considering the combined influence of column geometric parameters, prestressing level, and concrete crack resistance. It is proven that a diagonal arrangement of post-tensioned reinforcement leads to local stress concentrations and is less effective in deformation control compared to the contour scheme.

The necessity for conducting this research is driven by current trends in the development of monolithic construction, particularly the extensive use of flat slabs. Despite their structural efficiency, these slabs exhibit heightened sensitivity to factors affecting their load-bearing capacity and operational reliability. Synthesizing the stated objectives allows for the formulation of the following points substantiating the work's relevance.

Accounting for Deviations in the Actual Position of Reinforcement. Variations in the concrete cover, changes in the effective depth of the section, and a reduction in the efficiency of reinforcement can lead to a significant deterioration in the strength and deformation characteristics of slabs. Current regulatory documents do not fully govern these aspects, creating a need for their detailed investigation.

Influence of Geometric Parameters on Punching Shear Resistance. Punching shear represents the critical limit state for flat slab systems. Consequently, the application scope of standard calculation methods (such as EN 1992-1-1) requires refinement concerning slab thickness, column dimensions, and load levels to enhance the reliability of design calculations.

Determination of Critical Slab Thickness Parameters. Justifying the minimum allowable slab thickness for specified loading conditions is a necessary prerequisite for optimizing structural designs and preventing brittle failure in the column punch zone.

Investigation of Mixed Reinforcement Using Post-Tensioning Tendons. Combining conventional reinforcement with unbonded monostrands creates conditions for reducing deflections, increasing crack resistance, and forming a more uniform stress-strain state.

The insufficient understanding of their composite action necessitates a comprehensive analysis.

Application of Numerical Modeling for Selecting a Rational Reinforcement Layout. The configuration of post-tensioning tendon placement (banded, distributed) significantly influences the distribution of forces and deformations. This necessitates a comparative assessment to determine the optimal layout and level of prestressing force.

Refinement of Calculation Methods for Mixed Reinforcement. The interaction specifics between conventional and high-strength reinforcement are not fully addressed in current code provisions. Establishing the patterns of internal force redistribution will improve the accuracy of predicting the load-bearing capacity of such slabs.

Research Aim

To develop and substantiate optimized structural solutions for cast-in-situ reinforced concrete floor slabs based on multi-modeling of their stress-strain state. This aims to enhance the load-bearing capacity, reliability, and constructability of structures under conditions of real construction deviations and operational influences. Specifically, the research establishes the influence level of unbonded prestressed reinforcement and its layout on the strength of a monolithic floor slab, as well as compares the results concerning tendon stresses in slabs with contour and diagonal arrangements of prestressing reinforcement.

Objectives

To analyze the influence of deviations in the actual position of reinforcement from the design position, particularly the increase in concrete cover, on the effective depth, strength, and deformability of monolithic flat slabs.

To assess the influence of geometric parameters of structural elements (column dimensions and slab thickness) on the punching shear capacity of reinforced concrete slabs, using normative methodologies from EN 1992-1-1, Model Code 2010, ACI 318-19, DSTU-N B EN 1992-1-1:2010, and DBN V.2.6-98:2009 [10–14].

To establish critical slab thickness values for ensuring the required punching shear capacity depending on the magnitude of external loads and the cross-sectional dimensions of the column.

To perform numerical modeling of slab behavior using different layouts of post-tensioned reinforcement (contour and diagonal) and determine their impact on: the stress level in the tendons, slab deformations, and the overall stress-strain state of the floor.

Presentation of the main material

Assessment of the Influence of Concrete Cover Thickness Change on Strength.

A fragment of a monolithic slab from a multi-story building was adopted as the model. The slab thickness is 220 mm, column grid is 6x6 m, column cross-section is 400x400 mm. Column length equals half the story height. The lower column nodes are fixed in X, Y, and Z directions, the upper nodes only in X and Y. The connection between columns and slab is rigid. The slab is subjected to its self-weight and a uniformly distributed live load of 6 kPa. Slab reinforcement is class A400. series

A total of 4 types of models, differing in concrete class (C12/15, C16/20, C20/25, C25/30), were calculated. In each models, the distance from the tension face to the centroid of the reinforcement varied at five levels (*a* = 4, 5, 6, 7, and 8 cm), corresponding to effective depth values: d = 18, 17, 16, 15, and 14 cm, respectively.

Each models comprised 9 specimens: a base specimen with symmetrical top and bottom reinforcement (*a* = *a*' = 4 cm), four specimens for the support section (a' = 4, 5, 6, 7, and 8 cm with *a* = 4 cm), and four specimens for the span section (*a* = 4, 5, 6, 7, and 8 cm with *a*' = 4 cm).

For each effective depth value, the required area of support reinforcement A's was determined. Subsequently, the ratios of these obtained A's values to the area corresponding to an effective depth of 18 cm were calculated.

The values indicating the reduction in the slab's load-bearing capacity depending on the effective depth, obtained in this manner, are presented in Fig. 1. Moments were taken from the analysis performed in the LIRA software. The effective depth changes from 18 to 14 cm; this relationship is represented by a straight line on the graph (Fig. 1). The dependence of the relative increase in the required reinforcement area on the effective depth is non-linear and depends on the concrete strength.

Thus, for concrete class C12/15, a reduction in the effective section height by 20.2% leads to an increase in the required reinforcement area by 56%, which, for slabs with reinforcement area strictly corresponding to the design but installed such that the effective section height is 14 cm instead of 18 cm, will correspond to a reduction.

As can be seen, with an increase in concrete class, the relative reduction in required reinforcement (load-bearing capacity) decreases, and for concrete class C25/30, it amounts to approximately 36%. That is, a 20% reduction in the effective section height leads to a strength reduction of more than one-third. For slabs made of concrete C15/20 and C20/25, intermediate values apply.

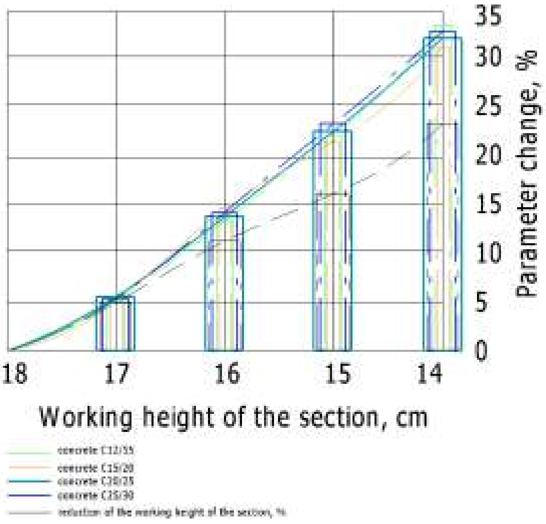

Fig. 1. Analytical dependence of support reinforcement A's on d

A similar pattern of dependence is observed in the comparison of the relative increase in the required reinforcement area obtained from calculations in the LIRA software environment. The increase in values obtained in this way regarding the change in reinforcement area compared to the analytical solution is explained by the influence of the LIRA software's calculation for the second group of limit states. The behavior of the graph corresponding to concrete class C12/15 deviates from the general pattern.

The comparison of numerical values of the relative increase in required span reinforcement, obtained from the analytical dependence and the results of calculations in the LIRA software environment, indicates that the results depend little on the calculation method – the difference is 1–2%. For this reason, only one graph is shown.

It is evident that in sections located in the span, as in tensioned elements [10], the relative increase in required reinforcement, and accordingly, the reduction in load-bearing capacity, is significantly lower than in support sections. The influence of concrete strength here has the same qualitative character but quantitatively affects to a much lesser extent. Thus, with a 22% reduction in the effective section height, 30% more reinforcement is required for concrete C25/30 and 33% more for concrete C12/15.

The analysis of the obtained results indicates varying levels of influence of deviations from the design reinforcement position in span sections and sections located above columns – in support sections. It has been established that support sections are more sensitive to reinforcement displacement relative to the design position. For the most commonly used concrete classes in floor slabs, C20/25, a 22% reduction in the effective section height will lead to a strength reduction of 38% and 42% in support sections and 31% in span sections, respectively.

The slab-column connection is one of the most critical in buildings with a monolithic reinforced concrete load-bearing system. This is due to a high concentration of forces and, consequently, the densest (within the floor slab) reinforcement. In the design process of such structures, it is important to assign correct element parameters, and one of the defining conditions here is the punching shear capacity. It should be noted that domestic norms for punching shear calculations account for the fewest parameters compared to European EN 1992 Eurocode 2, Model Code 2010, and American ACI 318-19. Researchers highlight the necessity of considering the cross-sectional shape and column dimension ratios, performance under high temperatures and dynamic impacts, punching shear behavior at the ends of shear walls, as well as the reduction of punching force.

However, even acknowledging the imperfection of the calculation methodology and the need to refine load-bearing capacity considering a broader range of factors, the issue of initially assigning dimensions to structural elements (columns and slab thickness) remains, and for these purposes, the calculation methodology is most convenient due to its simplicity. This work is dedicated to determining the limit values of thickness for flat reinforced concrete floor slabs based on punching shear conditions, depending on acting loads and column dimensions. Thus, for the considered cases, the condition for the minimum required effective slab height is derived.

Most often, the concrete class for flat monolithic reinforced concrete floor slabs ranges from C20/25 to C35/40. This is due to the fact that as the concrete class increases, its workability and convenience decrease, but the load-bearing capacity for bending moments increases weakly. Having set the design resistance of concrete to axial tension for the first group of limit states, graphs were constructed for convenient selection of the effective height of floor slabs.

Investigation of the Influence of High-Strength Reinforcement Application on the Strength of Monolithic Floor Slabs.

The performance of floor slabs is examined, particularly the impact of prestressed high-strength reinforcement without bond to concrete and its arrangement on the strength of such slabs. A refinement of the strength calculation methodology for monolithic beamless floor slabs with mixed reinforcement is conducted, where reinforcement in a plastic sheath of the monostrand type without bond to concrete is utilized to maximize the utilization of the strength properties of the floor slab materials.

One of the drawbacks hindering the widespread adoption of floor slabs is the development of excessive deflections at the slab center.

For spans exceeding 7 m, norms recommend arranging capitals to reduce deflections or additionally applying high-strength prestressed reinforcement without bond to concrete, without calculation.

European and American norms distinguish between systems with bond to concrete and without it, but for the latter, coefficients limiting the prestress level are imposed. Detailed justification for the introduced coefficients is not provided in the norms.

Several contemporary works in this direction, accounting for the performance of tensioned reinforcement without bond to concrete, are dedicated to monolithic floor slabs with only orthogonal reinforcement arrangement.

When accounting for prestressed reinforcement without bond to concrete in strength calculations, it is necessary to evaluate its stress state, considering the initial stress level and their increments due to deflections.

Prestressed reinforcement in plan can be laid according to one of the schemes shown in Fig. 2. The initial arrangement of cables along the section height typically corresponds to the expected moment diagram and, under uniformly distributed loading, can be represented as part of a parabola or circle.

In this work, the sagging form of the cable is represented as an arc of a circumscribed circle passing through the vertices of an isosceles triangle, the base of which equals the strand span, and the height equals the given deflection (Fig. 2).

Certain issues regarding the application of diagonal reinforcement without bond to concrete in beamless floor slabs have been addressed by the authors in previously published works: constructive solutions for beamless capless floor slabs with prestressed reinforcement, determination of the stress-strain state of such slabs, and deflection calculations.

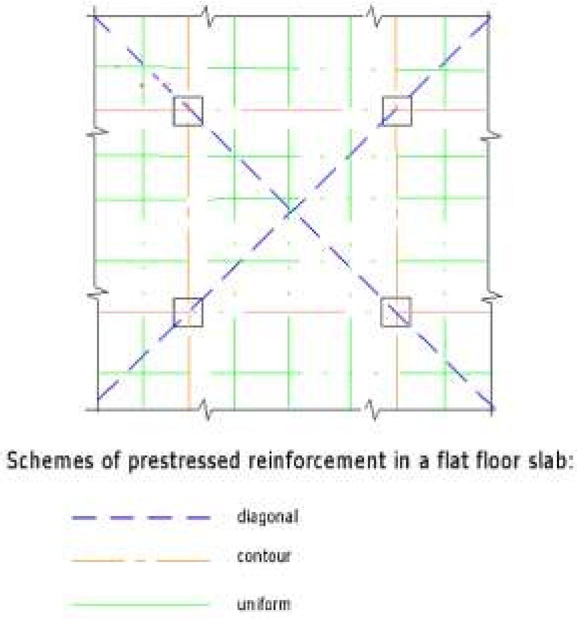

Fig. 2. Schemes of prestressed reinforcement in a flat floor slab

Scheme 1. A monolithic beamless floor slab of 9×9m with diagonal prestressed reinforcement is given, with slab thickness d = 200 mm and protective layer a = a' = 30 mm. The prestressed reinforcement consists of three cables of class K7 monostrand in each direction. The deflection magnitudes at the slab center varied within 1/250…1/125 of the clear distance between the inner faces of columns (8700 mm). Calculated stress increments in the cables Δσ and maximum values σ_{sp} at different prestress levels and deflections are presented in Table 2, 3.

The stress increment $\Delta\sigma_{sp}$ when displacing the cable from the initial deflection to the final (1/125) amounted to 234 MPa, which at σ_{sp0} = 930 MPa equals 25%, at σ_{sp0} = 1116 MPa equals 21%. The prestress level accounts for elastic compression and all losses σsp0. The coefficient k was taken as 0.5; 0.6; 0.7. The initial cable sag arrow z = d – a – a' = 200 – 30 – 30 = 140 mm. Methods and conditions of anchoring can be selected in accordance with special information sheets, European technical norms 4, and others.

Scheme 2. Floor slab of 9×9 m, with prestressed reinforcement arranged along the contour. Characteristics of concrete and reinforcement, as well as section dimensions, are as in scheme a. The calculated strand span equals the distance between column axes, 9000 mm.

When determining stresses in the cables, it should be considered that the ratio of the maximum deflection in the over-column strip to the deflection in the span center, according to the data, equals 0.75 f. This circumstance should be accounted for when evaluating the efficiency of stressing in beamless floor slabs with contour cable arrangement. (Fig. 3)

Table 2 – Stresses increment in cables at different prestress levels and deflections

Deflection			Stress increment Δσ, MPa
additional		general $f_1 = z + \delta f$, m	
relative deflection	absolute δf, m		
0	0,14	0,14	0
1/250	0,035	0,175	105,5
1/225	0,039	0,179	118,6
1/200	0,044	0,18	135,4
1/175	0,05	0,19	157,7
1/150	0,06	0,20	188,6
1/125	0,07	0,21	234,1

Table 3 – Tension Stresses in cables at different prestress levels

Relative deflection	Tension, MPa, in ropes at $\sigma_{sp} = kf_{yd}$ (taking into account losses)		
	$k = 0{,}5$	$k = 0{,}6$	$k = 0{,}7$
0	930	1116	1302
1/250	1036	1221	1408
1/225	1049	1235	1421
1/200	1065	1251	1437
1/175	1088	1274	1460
1/150	1119	1305	1491
1/125	1164	1350	1536

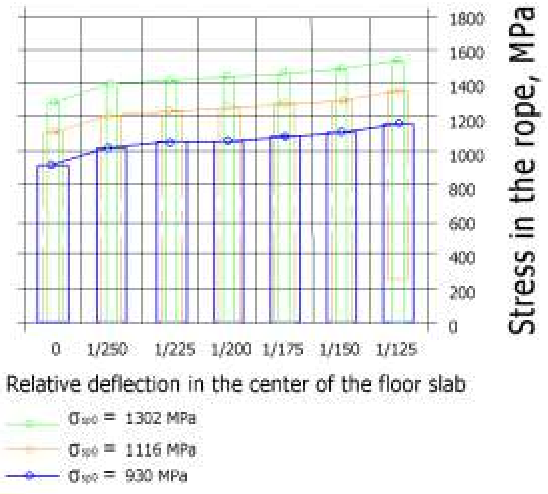

Fig. 3. Increase in Stresses in Cables with Increasing Deflections

The stress increment $\Delta\sigma_{sp}$ upon cable displacement from the initial deflection to the final

(1/125) amounted to 333 MPa, which at σ_{sp0} = 930 MPa equals 36%, at σ_{sp0} = 1116 MPa equals 30%, and at σ_{sp0} = 1302 MPa equals 26%.

Conclusions

Analysis indicates a varying sensitivity to reinforcement positioning deviations: support sections are more sensitive than span sections. A 22% decrease in effective depth leads to strength reductions of 38–42% in support sections and 31% in span sections.

The use of contour high-strength unbonded post-tensioned reinforcement reduces slab deflections. Deflection reduction ranges from 5% to 16% depending on the panel size.

It is recommended to use high-strength unbonded post-tensioned tendons as supplementary reinforcement, as this reduces slab deflections and lowers the consumption of conventional reinforcement.

From the perspective of ensuring load-bearing capacity, the contour arrangement of tendons is preferable due to more complete utilization of the high-strength reinforcement's strength.

For variable concrete strength through the thickness, deflections increase by 12.5%; the presence of cracks in the floor slab increases deflections by 70% under the condition of applying a coefficient of 0.3.

The multifactorial nature and interdependence of geometric, structural, technological, and operational parameters governing the behavior of flat slabs, along with existing discrepancies among current regulatory provisions, necessitate a comprehensive investigation of the specified issues.

The obtained results can be utilized to refine calculation models, improve design methodologies, and optimize structural solutions, thereby contributing to enhanced reliability and durability of reinforced concrete slabs in modern construction.

The practical significance of the work lies in improving the economic efficiency and service life of monolithic frame residential buildings. The research scope includes performing numerical simulations of the stress-strain state of a monolithic reinforced concrete floor slab with variable concrete strength through its thickness, conducting experimental tests on analogous specimens, evaluating the specifics of their performance, and formulating recommendations for the operation of slabs considering the differentiated concrete characteristics across the section depth.

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Аналіз конструктивних схем перекриттів із застосуванням комп’ютерного моделювання їх напружено-деформованого стану

Анотація. Актуальність. *Довговічність і надійність залізобетонних конструкцій, зокрема монолітних плит перекриття, залежать не лише від якості проєктування та властивостей матеріалів, а й від точності виконання будівельно-монтажних робіт. Порушення положення елементів каркаса й арматури знижують несну здатність і жорсткість конструкцій. Використання сучасних матеріалів підвищує вимоги до точності монтажу, а збільшення захисного шару бетону під час бетонування зменшує робочу висоту перерізу й погіршує міцнісні характеристики плит. За умов зниження матеріаломісткості ефективним рішенням для монолітних плоских перекриттів є застосування попереднього натягу з розширеною сіткою колон.* ***Мета.*** *Розробити й обґрунтувати оптимізовані конструктивні рішення для монолітних залізобетонних плит перекриття на основі багатоетапного моделювання їх напружено-деформованого стану. Мета – підвищити несну здатність, надійність і конструктивну придатність конструкцій в умовах реальних будівельних відхилень та експлуатаційних впливів. Зокрема дослідження встановлює рівень впливу незв’язаної попередньо напруженої арматури та її розташування на міцність монолітної плити перекриття, а також порівнює результати щодо напружень арматури в плитах із контурним і діагональним розташуванням попередньо напруженої арматури.* ***Результати.*** *Аналіз продемонстрував різну чутливість до відхилень розташування арматури: опорні перерізи більш чутливі, ніж прогонові. Зменшення ефективної глибини на 22 % призводить до зниження міцності на 38–42 % в опорних перерізах та на 31 % – у прогонових перерізах. Використання контурної високоміцної незв’язаної арматури з попереднім натягом зменшує прогини плити. Зменшення прогину коливається від 4,81 % до 16,46 % залежно від розміру панелі. Рекомендовано застосовувати високоміцні незв’язані арматурні елементи з попереднім натягом як додаткову арматуру, оскільки це зменшує прогини плити та знижує витрату звичайної арматури. Щодо забезпечення несної здатності контурне розташування арматури є кращим завдяки більш повному використанню міцності високоміцної арматури. У разі змінніої міцності бетону за товщиною прогини збільшуються на 12,42 %; наявність тріщин у плиті перекриття збільшує прогини на 70,2 % за умови застосування коефіцієнта 0,3.* ***Новизна дослідження.*** *Установлено кількісний вплив відхилень ефективної висоти перерізу, змінної міцності бетону й тріщиноутворення на напружено-деформований стан монолітних плит перекриття, а також досліджено ефективність контурного розташування високоміцної незв’язаної арматури з попереднім натягом. Досягнуті результати дають змогу уточнити вплив конструктивних і технологічних факторів на несну здатність і деформованість плит.* ***Практична цінність.*** *Можливість використання запропонованих рекомендацій у процесі проєктування монолітних перекриттів для зменшення прогинів, раціонального використання високоміцної арматури та зниження витрат звичайної арматури за умови забезпечення необхідної несної здатності та експлуатаційної надійності конструкцій.*

Ключові слова: *монолітні залізобетонні плити, відхилення розміщення арматури, глибина бетон-*

ного шару, ефективна висота поперечного перерізу, несна здатність, напружено-деформований стан, числове моделювання, LIRA-SAPR, незв'язана попередньо напружена арматура, розташування арматури.

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