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A NUMERICAL STUDY OF THE PERFORMANCE OF A COMBINED FLOOR PANEL

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The purpose of the study is to analyze the proposed calculation models of a structure to take into account the nonlinear properties of reinforced concrete. The paper studies a reinforced concrete lightweight structure of a solid floor using light concrete packaged units. The numerical study is based on the use of combined floor panel models and certain theories of material deformation. The numerical study allowed us to obtain and compare structural indicators using various model theories. It has been established that the design model of the combined floor structure based on the finite element method (FEM) allows us to account for the nonlinear properties of concrete and reinforced concrete, as well as the influence on the distribution of bending moments and thrust forces between two-way beams. We proposed design models which allow us to account for the uneven distribution of bending moments and thrust forces in the limit equilibrium method while analyzing the floor according to the limit state. Based on the limit equilibrium method, we developed recommendations on determining the distribution of bending moments along linear plastic centroids. The practical value of the study is to identify the features of the stress-strain state and establish the compliance between the numerical results and the data of physical and mechanical experiments.

Keywords: combined lightweight reinforced concrete structure, stress-strain state, nonlinear deformation, stress redistribution, thrust influence, methods of finite elements and limit states

Introduction

The purpose of the study is to establish the influence of the nonlinear deformation of concrete and reinforced concrete on changes in the stress-strain state of the floor elements. The numerical study is based on structural models and certain theories of material deformation. We obtained and compared structural indicators using various models and theories. This paper studies a reinforced concrete lightweight structure of a solid floor. A beam and grinder construction was formed from heavy structural concrete. The weight of the floor structure was reduced to 40–60% through the use of light concrete packaged units, wood fiber concrete units, or foam polystyrene inserts laid out on the formwork with certain gaps, in which the bottom and top reinforcement are installed.

Significance of the study

Numerical studies allow us to identify the features of the stress-strain state and to establish the compliance between the numerical results and the data of physical and mechanical experiments. Accounting for nonlinear deformation gives a better understanding of the stress-strain state, which reduces the consumption of steel reinforcement and increases reliability. These studies are useful for practical design engineering.

Analytical research

An analysis of possible schemes to account for the nonlinearity of reinforced concrete

The research covered a section of a floor structure consisting of four columns with a 40×40 cm cross-section and a 20 cm plate attached by a mount which transmits the bending moment. The distance between the axes of the columns is 6×6 m; the length of the columns below and above the plate is 3 m. The columns and the plate are constructed from B25-concrete. Waffle-slab floors are analogs to this structure. Two calculation options were most often used in practical design engineering: the limit equilibrium method [1] and the finite element method [2–5]. The results of calculations for various options are shown below to apply design methods for the first limit state (destruction).

<u>Option I.</u> The finite element method based on the theory of elasticity. Shell-type finite elements (six degrees of freedom) were used. The presence of lightweight units was not taken into account; the plate was considered to be homogeneous (the most common design in design engineering practice).

<u>Option II.</u> The finite element (FE) method using volumetric rectangular FEs. The ratio of moments is shown in Fig. 1. The presence of lightweight concrete units was not taken into account; the plate was considered to be homogeneous.

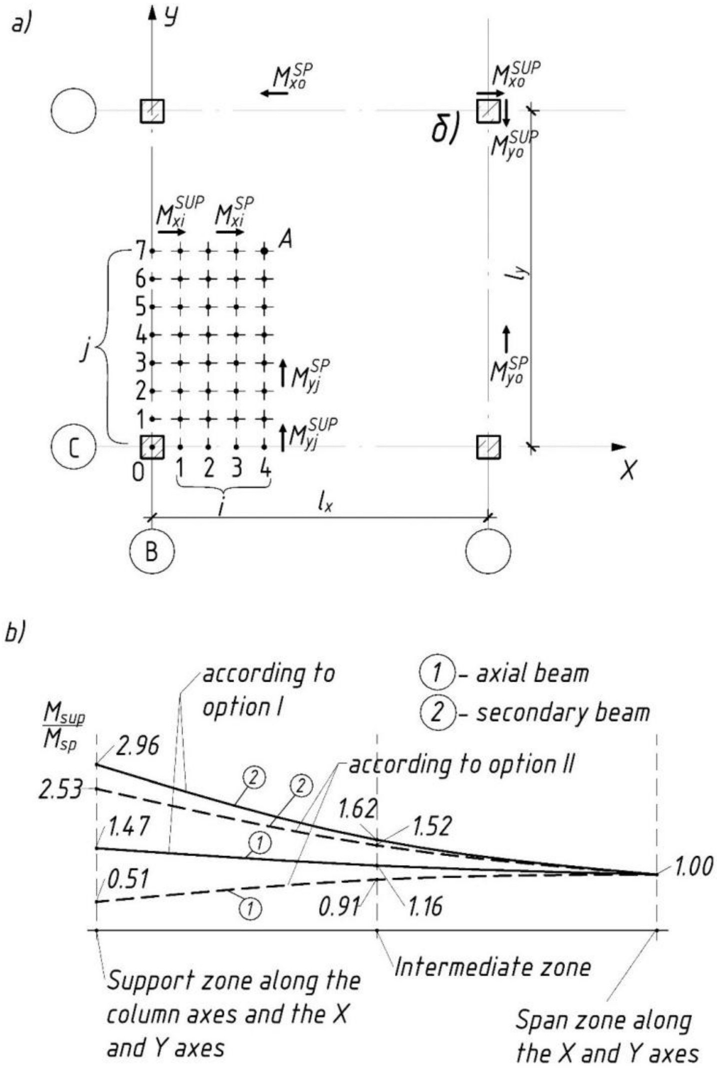

Fig. 1. Designation (a) and distribution (b) of bending moments

Option III. The finite element method using the following design model.

Axial beams are modeled by volumetric FEs (prism) since the width of the beam section is comparable to its height: $b_{\text{ax}} = 40$ cm, $h_{\text{ax}} = 24$ cm. The calculation allowed us to determine the stresses in the FEs and displacements of the axial and secondary beams (deflections). The calculations were completed with a stepwise load increase. The following features of the stress state of the beams were established:

- torsion appears in the axial beams, which can be seen from the uneven distribution of stresses across the width of the section. For example, near the columns (support sections), there was a 2.17 times difference in tension longitudinal stresses;
- no tension stresses appeared in the secondary beams located in the middle of the span of the axial beams where they are supported by the axial beams (support sections); tension stresses appeared in the remaining beams;
- in all the beams (axial and secondary), stress diagrams were asymmetric and slightly curvilinear; the asymmetry of the diagram was 1.5...2.5 times, which indicated the presence of normal forces apart from the bending moments.

Option III-(a). The design scheme for this option is similar to that of option III. The task account for inelastic deformations of compressed concrete and the appearance of cracks in the stretched zones of beams. Algorithms using the FE method have been proposed to solve this problem [6–8]. The FE method was combined with the theories of plasticity, creep, and deformation of cracked reinforced concrete.

Software has been developed for particular cases of structural systems, but they cannot be used in design engineering. The Lira software suite has a nonlinear block; however, practical recommendations are inaccurate, and the influence of cracking on the redistribution of stresses and forces is not taken into account. Besides, there is no experience of using the nonlinear component of Lira in practical design engineering. This study proposes an algorithm to account for the nonlinear resistance using the elastic component of Lira. The calculation consists of two stages (steps).

At the first stage, we calculate the design load and determine the amount of steel reinforcement. At this stage, we select finite elements, in which the compressive stresses have reached the values $\sigma_{bc} \geq 0.5R_b$ and the tension stresses have reached the values $\sigma_{bt} \geq R_{bt}$. Thus, we identify zones with inelastic compressive deformations and cracks. In these zones, the initial values of the elastic module of concrete decrease according to the following dependencies:

- for compressed concrete FE:

$$E_{b.red} = \frac{R_b}{\varepsilon_{b1.red}},$$

- for tensile concrete FE, the reduced module is determined from the conditions of the equality of deformation and tension forces of the concrete FE and its replacement by a section of steel reinforcement:

$$E_{bt.red} = E_s \frac{A_s}{A_{bt}} = E_s \mu_s \eta,$$

where μ_s is the steel ratio and η is the correction coefficient. If $\mu_s = 1\%$ and $\eta \cong 1$ for B25 concrete, the reduced module is $E_{bt.red} = 2 \cdot 10^4$ kgs/cm^2, which is 15 times less than the initial module $E_b = 30 \cdot 10^4$ kgs/cm^2.

At the second stage, the calculation is repeated according to the same design model with corrected elastic moduli.

The results of comparing the two calculations are shown below:

1) with initial elastic moduli (conditionally "elastic");

2) with corrected elastic moduli (conditionally "nonlinear").

In our analysis, we calculated the stresses in the FEs as well as the bending moments and longitudinal forces in the axial and secondary beams in two directions (*X*) and (*Y*) (Fig. 2). Due to the different sizes of the lightweight concrete units in plan view, the difference in the directions (*X*) and (*Y*) resulted in a different number of secondary beams in these directions: in the direction (*X*) – $i = 13$ pcs, in the direction (*Y*) – $j = 7$ pcs. Due to symmetry, we considered ¼ of the floor section in plan view.

Results

The results of our analysis are shown below.

1. Axial beams:

- in the inelastic analysis, the uneven distribution of the axial stresses σ_t and σ_c over the width of its section increased, which indicates an increase in torques in the axial beams due to the influence of the secondary beams;
- in the inelastic analysis, we observe a redistribution of the stresses σ_t and σ_c in the sections: on the support, σ_t decreases by 1.3 ... 1.4 times, σ_c increases by 1.7 ... 1.8 times; in the span σ_t and σ_c decrease by 1.3…1.5 times. This indicates a redistribution of internal forces from the main beams to the secondary beams;
- in the elastic analysis, the ratio of the moments M^{SUP}/M^{SP} in the main beams was 1.20...1.30; in the inelastic analysis, it was 1.70...1.80, respectively, which indicates that the forces are redistributed from the main beams to the secondary beams differently in the support and span sections; accounting for the changes in the distribution of the bending moments allows us to reinforce the floor structure more efficiently;
- in the inelastic analysis, we observe an increase in the deflection of the main beam by 1.6...2.1 times, which corresponds to the experimental data and results of calculations according to the Reference Book [9, 10]

2. Secondary beams

In this section of floor, there are seven beams along the (*X*) axis and four beams along the (*Y*) axis per ¼ of the section. Fig. 2 shows the stresses in the sections of these beams where they are supported by the main beams and in the span sections. We compared the stresses obtained in the elastic analysis with the inelastic analysis (Figs. 3–5).

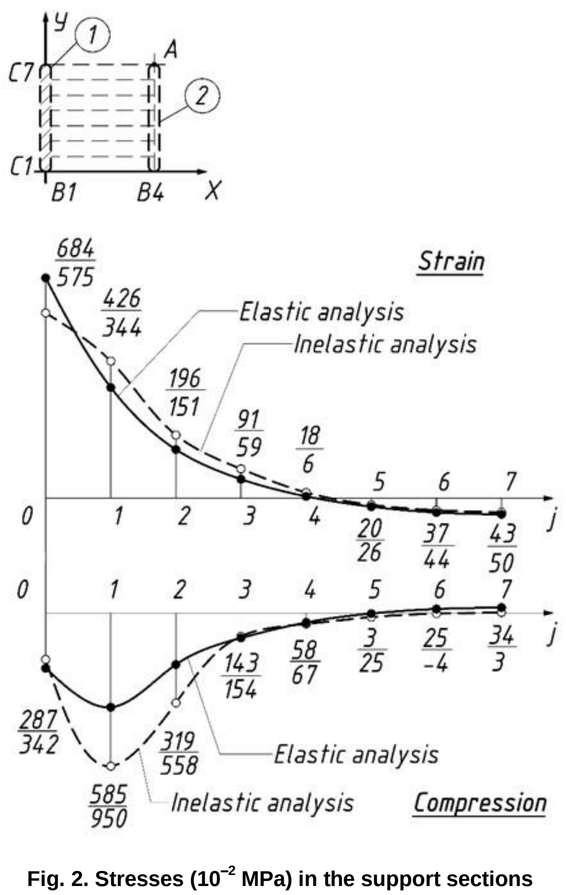

Fig. 2. Stresses (10^{-2} MPa) in the support sections of the secondary beams along the (X) axis where they are supported by the axial beams

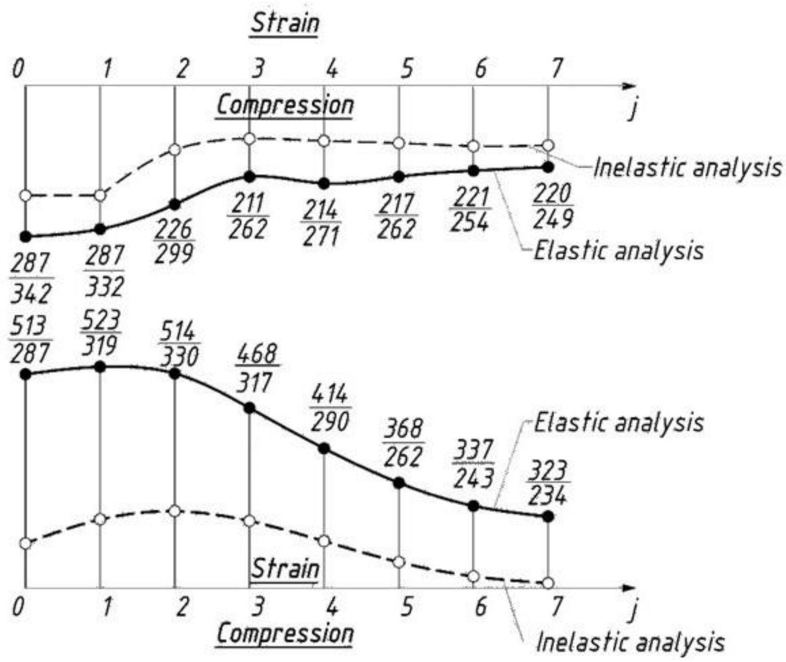

Fig. 3. Stresses (10^{-2} MPa) in the span sections of the secondary beams along the (X) axis

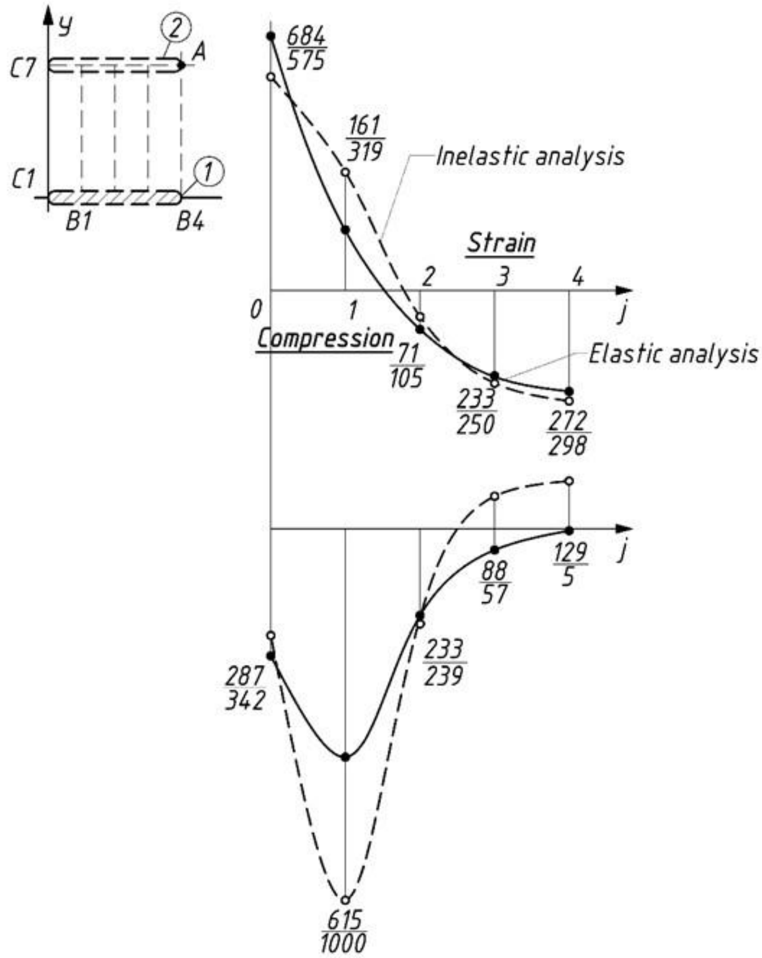

Fig. 4. Stresses (10^{-2} MPa) in the support sections of the secondary beams along the (Y) axis where they are supported by the axial beams

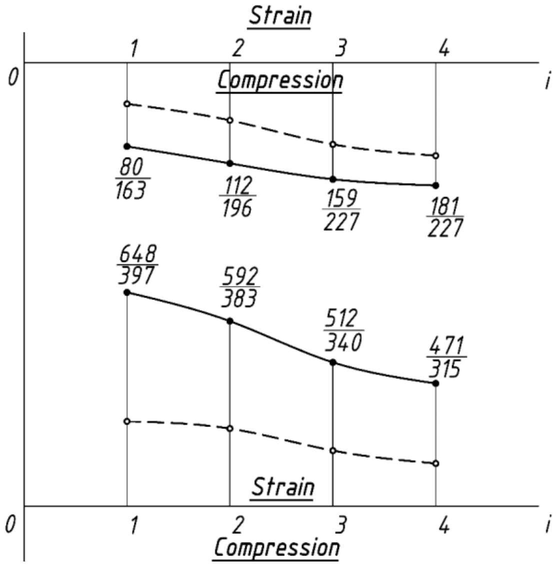

Fig. 5. Stresses (10^{-2} MPa) in the span sections of the secondary beams along the (*Y*) axis

We revealed a complex stress redistribution scheme. Notably, changes to the stresses (increase – decrease) ranged from 1.2 to 2.0 times. In the inelastic analysis, differences in the compression-tension stresses increased in the sections, as did the curvature of the diagram along the height of the section. A change in the longitudinal stresses in the sections leads to a change in the bending moments and, accordingly, changes in the steel reinforcement in the strength and deformability analyses.

In the inelastic analysis, the deflection in the center of the plate (point A in Figs. 3 and 4) increased by 14.5/6.87 = 2.1 times. This coincides with the experimental data and calculation results.

Determination of the thrust

The presence of the thrust in floors (bent plate structures) is noted in [9, 11]. The authors affirmed that the longitudinal tension reinforcement can be reduced by 5 ... 20% due to thrust.

Thrust may appear for two reasons:

- Due to the resistance to bending of the columns supporting the floors; the transverse force arising in the column is transferred to the floor plate in the form of a thrust;
- Due to the formation of cracks in the tension areas of the floor plate, an arch (dome) of tension concrete supported by contour structures appears.

In this theoretical study of a section of a floor, the thrust value was determined by the stress values in the cross sections of the axial and secondary beams. Columns were considered as supporting structures of the axial beams, while the axial beams were considered as supporting structures of the secondary beams.

In our previous analysis we found that the stress diagrams in the beam sections are asymmetric throughout the height of the section and differ in elastic and inelastic analyses. Ignoring the curvilinearity of the stress diagrams, the thrust value was calculated from the system of equations:

- for compression stress

$$\sigma_c = -\frac{M}{W} - \frac{H}{A};$$

- for tension stresses

$$\sigma_t = +\frac{M}{W} - \frac{H}{A};$$

where σ_c, σ_t are the known stresses; W, A are the known geometric characteristics of the beam sections.

Figs. 6 and 7 show the results of calculating the thrust values in the beams of the analyzed section of the floor.

After comparing the results, we found that:

- The maximum thrust value appears in the areas near the axle beams;
- The distributions of the thrust values along the beams differ in the support and span sections;
- In the inelastic analysis, the thrust value changes as compared to the elastic analysis; on average thrust value changes 1.4…2.1 times.

The use of the limit equilibrium method

The limit equilibrium method [3, 12] used to design bending plate structures assumes the appearance of linear plastic centroids where bending moments reach the limit values. There are recommendations for the distribution of moments between the support and span sections. Elastic analyses and testing of the plates supported by individual columns along the outline showed that the difference in bending moments along the linear plastic centroids can be significant. For example, according to calculations, the moments in the direction of the (*X*) axis along the (*Y*) axis differ by 0.51… 2.96 times, and when testing a section of a girderless structure, cracks near the columns (support zone)

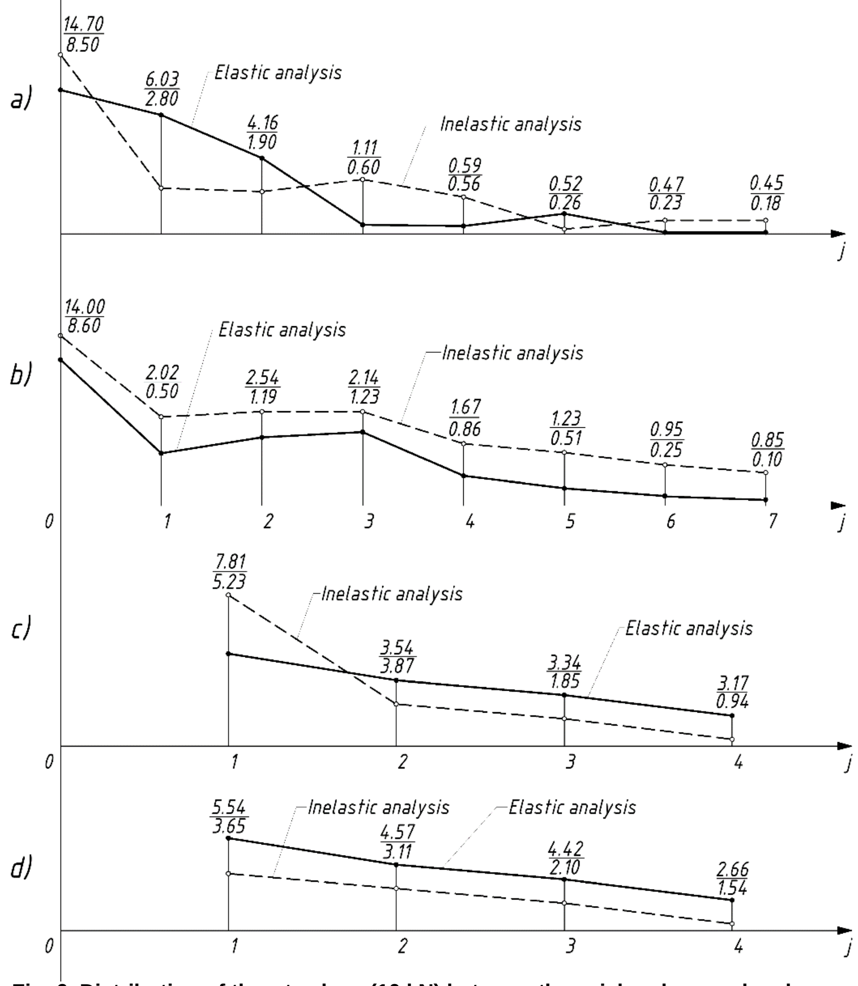

Fig. 6. Distribution of thrust values (10 kN) between the axial and secondary beams:
a) on supports in the direction of the (*X*) axis; b) in spans in the direction of the (*X*) axis;
c) on supports in the direction of the (*Y*) axis; d) in spans in the direction of the (*Y*) axis

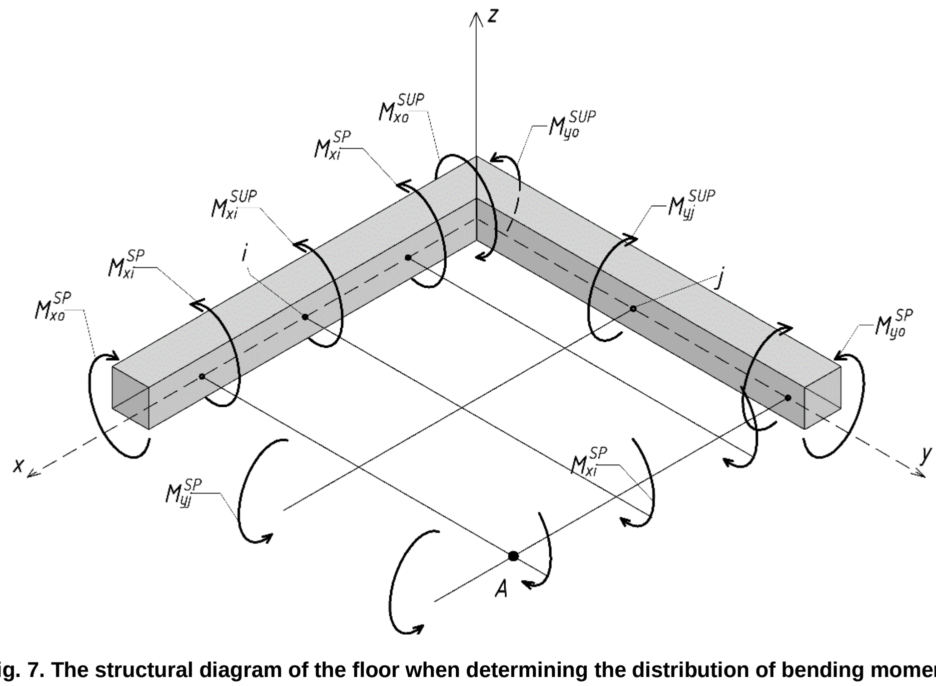

Fig. 7. The structural diagram of the floor when determining the distribution of bending moments in the two-way beams (point A is the center of the floor with maximum deflection)

were 1.5…3.0 times wider than in the span zone [6, 13]. A significant difference is observed along the proposed linear span plastic centroid.

The limit equilibrium method does not theoretically establish the distribution of bending moments acting perpendicular to a linear plastic centroid. The assumption that the distribution can be constant is improbable, since at the elastic stage the distribution is essentially inconstant (the difference is 1.50...3.00 times). Conversion of such unevenness into evenness has not been observed in the experiments. To this end, we have developed the following recommendations for determining the distribution of bending moments along linear plastic centroids.

For a plate with column supports, there may be two fracture schemes (strip and adjacent) and, accordingly, the equilibrium equations are written as follows:

$$q \cdot \text{А} = q\left[\frac{l_y(l_x-2c_x)2}{8}\right] = M_x^{SUP} + M_x^{SP}, \quad (1)$$

$$q \cdot \text{Б} = q\left[\frac{l_c \cdot l_y}{8}\left(\frac{l_x + l_y}{2} - 2c_x + \frac{4}{3}\cdot\frac{c^3}{l_x l_y}\right)\right] = = 0.5\left(M_x^{SUP} + M_y^{SP}\right) + 0.5\left(M_x^{SP} + M_y^{SP}\right). \quad (2)$$

Applying these equations to the design of the floor plate, we should write using one of the directions (*X*) or (*Y*):

$$q \cdot \text{А} = \sum_{i=0}^{n} M_{xi}^{SUP} + \sum_{i=0}^{n} M_{xi}^{SP}, \quad (3)$$

where *i* is the number of beams in the direction (X), of which M_{x0}^{SUP} and M_{x0}^{SP} are axial beams;

$$q \cdot \text{Б} = 0.5\left(\sum M_{xi}^{SUP} + \sum M_{yj}^{SP}\right) + +0.5\left(\sum M_{xi}^{SP} + \sum M_{yj}^{SP}\right), \quad (4)$$

where j is the number of beams in the direction (Y), of which M_{yj}^{SUP} and M_{yj}^{SP} are axial beams.

There are two options for determining the ratio of moments in each of their sums in equations (3) and (4).

Option A. Using the results of calculations according to the inelastic design model (option III-a).

Option B. Building a design model and a system of equations, in which the moments M_i and M_j are unknown.

The design models are based on the following notions:

1) The support conditions determining the appearance of M_{ij}^{SUP} in the secondary beams depend on the deformation (displacement) of the axial beams; possible displacements are angles of rotation, deflections, and horizontal displacements at points (*i*) and (*j*).

2) The support conditions determining the appearance of M_0^{SP} in the axial beams depend on the deformation of the columns.

The systems of equations for determining the distribution of M_{ij}^{SUP} and M_0^{SP} have the form (method of forces):

$$\begin{Bmatrix} \delta_{11}x_1 + \delta_{12}x_2 + \cdots = \Delta_{1n} \\ \delta_{21}x_1 + \delta_{22}x_2 + \cdots = \Delta_{2n} \\ \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \end{Bmatrix}. \quad (5)$$

The general form is:

$$[\delta_{ij}][x_i] = [\Delta_{in}], \quad (6)$$

where $x_i = M^{SUP}\ or\ M^{SP}$, $\delta_{ij,}$ and Δ_{in} are the angles of rotation of the beam sections, accounting for their horizontal and vertical displacements.

Systems (5) and (6) are not designed to determine the values of M^{SP} and M^{SUP}, which is used to determine the amount of steel reinforcement. They are designed to determine the ratio (distribution) of moments between the beams in the support and span sections. The necessary amount of steel reinforcement is calculated by jointly solving the system of equations (4) and (6). The calculation and, accordingly, reinforcement can be simplified by dividing all the beams in each direction into two groups:

- A group near the columns, including the axial beam and part of the secondary beams located near the axial beam;
- A group between the columns, including the remaining secondary beams.

Then, system (6) consists of two equations:

$$\left.\begin{matrix} \delta_{11}x_1 + \delta_{12}x_2 = \Delta_1 \\ \delta_{21}x_1 + \delta_{22}x_2 = \Delta_2 \end{matrix}\right\}. \quad (7)$$

The equations can be calculated by the following formulas:

$$\delta_{ij} = \varphi_0 + \varphi_{sec} = \frac{1}{B_{ax}} + \frac{1}{B_{sec}} + \frac{1}{C_{ax}}, \quad (8)$$

where B_{ax} and B_{sec} are the bending stiffnesses of the axial and secondary beams; C_{0ax} is the torsional stiffness of the axial beam; and Δ_i is the reciprocal angle of rotation of the sections in the main system (Fig. 6) caused by an external load. In this case, system (7) can be solved "manually". System (7) is compiled for the support and span sections on the (*X*) and (*Y*) axes, ignoring their mutual influence.

Accounting for the thrust values in the limit equilibrium method

Apart from recommendations to account for thrust to reduce the amount of steel reinforcement by 5–20%, the literature [9, 11] presents formulas to calculate the thrust value and adjust the reinforcement. The essence of these proposals is that the moment ΔM, accounting for the thrust, is added to the right side of the limit equilibrium equation (3) or (4).

The above studies of a section of a floor have shown that the thrust is distributed unevenly between the axial and secondary beams, and the nonlinear deformation influences this distribution. To this end, we propose two options to account for thrust in the limit equilibrium method.

Option A. Use the recommendations contained in [9, 11, 14, 15] and distribute the thrust between the beams, accounting for the results obtained in option III-a.

Option B. Create a simple design model to determine the thrust distribution. We propose an arched diagram shown in Figs. 7, 8 and built according to the following provisions:

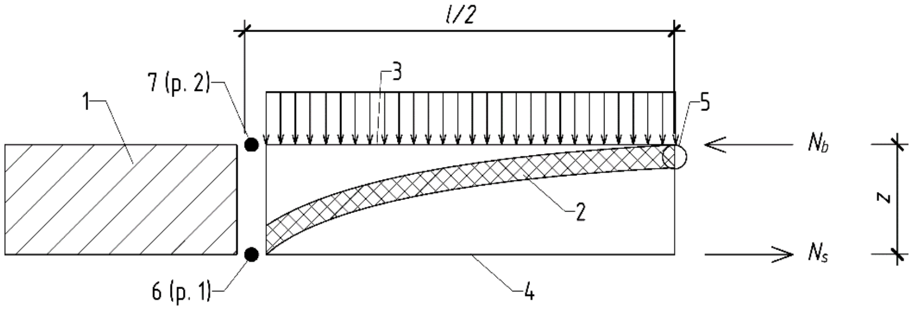

Fig. 8. The structural diagram for determining the distribution of thrust between the secondary beams: *1* axial beam; *2* solid (without cracks) section of the secondary beams; *3 and 4* top and bottom reinforcement of the secondary beams; *5* plastic centroid in the compressed area of the span section; *6 and 7* bonds in the compressed and stretched zones of the support section

• The thrust appears due to a rigid contour in the form of axial beams with a bending stiffness B_{SUP} in the horizontal direction;

• The secondary beam in the limiting state is divided into three sections: a solid compressed sections and adjacent tensioned sections with cracks;

• In tensioned sections of steel reinforcement connected to bonds, B_{SH} and B_{SB} are the stiffnesses of the bottom and top reinforcement, respectively.

• The main system of the method of forces is obtained by eliminating the bonds and replacing them with the forces x_1 and x_2.

The forces x_1 and x_2 are calculated from the system of equations compiled for different areas – (*i*) or (*j*) – of the connection of the secondary beams with the axial beams:

$$\left.\begin{aligned}\delta_{11}x_1 + \delta_{12}x_2 = \Delta_{1q}\\ \delta_{21}x_1 + \delta_{22}x_2 = \Delta_{2q}\end{aligned}\right\}. \quad (9)$$

The following formulas were used to calculate the coefficients in equation (9) (reciprocal horizontal displacements):

• Horizontal displacements at point 1 (6) from an external load to the secondary beam

$$\Delta_{1q} = q\frac{l^2}{8\cdot z}\cdot\frac{1}{B_{SB}}; \quad \text{where } H_1 = q\frac{l^2}{8\cdot z}$$

$$\Delta_{2q} = q\frac{\Delta_{1q}\cdot l}{l+\Delta_{1q}};$$

• Horizontal displacement of the secondary beam from the action $x_1 = 1$ and $x_2 = 1$

$$\Delta_1(x_1) = \frac{1}{B_{SH}}; \quad \Delta_2(x_2) = \frac{1}{B_{SB}};$$

• Horizontal displacements in the axial beam from the action $x_1 = 1$ and $x_2 = 1$

$$\Delta_1 = \Delta_2 = \frac{1}{6EI}(3a^2 l + 4a^3);$$

where a is the distance from the axis of the column to the point (*i*) or (*j*) where secondary beams are mounted to the axial beams.

Conclusions

1. The design model of the combined floor structure made up of two-way beams presented in the form of flat FEs allows us to account for the nonlinear properties of concrete and reinforced concrete through two-stage analysis (options III and III-a) and determine the bending moments and thrust forces.

2. A nonlinear two-stage analysis allows us to establish the influence of the nonlinear properties of concrete and reinforced concrete on the distribution of bending moments and thrust forces between the two-way beams.

3. We recommended using two methods to design the floor structure: the two-stage method and the limit equilibrium method. This provides control and reliability in determining the amount and distribution of steel reinforcement.

4. We proposed design models allowing us to take into account the uneven distribution of bending moments and thrust forces in the limit equilibrium method when calculating the limit state of the floor structure.

Designations, definitions, and symbols

The following designations are used in the text:

– axial beams are those along the axes of the columns (section h_{ax} x $в_{ax}$), where h_{ax} is the height of the section, equal to the thickness of the floor plate; $в_{ax}$ is the width;

– secondary beams are those between the axes of the columns (section h_{sec} x $в_{sec}$), where $в_{sec}$ is the width of the section.

The following designations are used for the values of bending moments arising in beams (Fig. 1):

M_X are moments arising in the direction of the X and Y axes;

M_{XO}, M_{YO}, M_{Xi}, M_{Yj} are moments in the axial beams, respectively;

M_X^{SUP}, M_Y^{SUP}, M_X^{SP}, M_Y^{SP} are moments in the beams at the supports (“SUP”) and in the span (“SP”).

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ЧИСЛЕННОЕ ИССЛЕДОВАНИЕ РАБОТОСПОСОБНОСТИ ПЛИТЫ ПЕРЕКРЫТИЯ КОМБИНИРОВАННОЙ КОНСТРУКЦИИ

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Цель исследования – анализ предложенных расчетных моделей конструкции для учета нелинейных свойств железобетона. В статье исследуется железобетонная облегченная конструкция монолитного перекрытия с использованием сборных блоков из легкого бетона. Численное исследование основано на применении моделей плиты перекрытия комбинированной конструкции и определенных теорий деформирования материалов. Результатом численного исследования является получение величин показателей конструкции и их сравнение при использовании различных теорий моделей. Установлено, что расчетная схема комбинированной конструкции перекрытия, разработанная на основе применения метода конечных элементов (МКЭ), позволяет учесть нелинейные свойства бетона и железобетона, а также влияние на распределение изгибающих моментов и распорных усилий между перекрестными балками. Предложены расчетные схемы, позволяющие в методе предельного равновесия учесть неравномерное распределение изгибающих моментов и распорных сил при расчете перекрытия по предельному состоянию. На основе метода предельного равновесия разработаны рекомендации по определению распределения изгибающих моментов вдоль линейных пластических шарниров. Практическое значение исследований состоит в выявлении особенностей напряженно-деформированного состояния и установлении соответствия численных результатов данным физико-механических опытов.

Ключевые слова: комбинированная облегченная железобетонная конструкция, напряженно-деформированное состояние, нелинейное деформирование, перераспределение напряжений, влияние распора, методы конечных элементов и предельных состояний

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