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Andriy Pavlikov
National University «Yuri Kondratyuk Poltava Polytechnic»
https://orcid.org/0000-0002-5654-5849

Olha Harkava *
National University «Yuri Kondratyuk Poltava Polytechnic»
https://orcid.org/0000-0003-2214-3128

Precast flat plate frame of buildings for renewal of the housing stock of Ukraine

The suggested precast flat slab system is composed of columns and flat plates that connect to the columns without the use of consoles or capitals. This structural system is intentionally designed in such a way that lines of arrangement of plastic hinges between precast plates are artificially created. Consequently, an analysis of precast flat plate floor systems is proposed based on the yield line method. The utilization of the yield line method facilitates the development of straightforward engineering design schemes, potential fracture scenarios, and the determination of the load at which destructive failure occurs for all types of precast flat plates. By employing the yield line method within the ultimate equilibrium state, utilizing a nonlinear deformation model with an extreme criterion, design equations to address the bearing capacity of flat plate of the frame structural system are derived. These formulas are well-suited for practical engineering use and account for the plate's installation conditions as part of a precast flat floor.

Keywords: bearing capacity, building frame, flat plate, reinforced concrete, slab

*Corresponding author E-mail: olga-boiko@ukr.net

Introduction. The restoration of the housing stock for today and in the future is one of the urgent problems of construction in Ukraine. In Poltava, the solution to this problem is implemented by constructing residential buildings and infrastructure facilities based on the flat plate frame structural system [1 – 3], which ensures a fast pace of their construction with minimal labour costs for builders. The flat plate frame structural system of buildings consists of prefabricated precast reinforced concrete elements that are mounted directly on the construction site. This system demonstrates versatility, which allows it to be used for buildings of various types: from multi-storey buildings up to 16 floors high to cottages 2-3 floors high in any climatic conditions of Ukraine. It has found successful application in the construction of above-ground multi-storey parking lots, residential and public buildings. In Poltava, a precast the flat plate frame structural system is being widely introduced into residential construction. On its basis, multi-storey buildings were built in the Sadovyi microdistrict, Family Park residential complex, Baronivskyi residential complex, Yevropeiskyi kvartal residential complex, and construction is being completed in the Peliustkovyi residential complex (Fig. 1, *a*, *b*). Interest in the flat plate frame is due to a number of its advantages when used in construction: freedom of architectural and planning solutions; wide possibilities for redevelopment of premises; use of energy-efficient materials in enclosing structures; rapid resumption of industrialization of construction, etc. By its design, the frame consists of a minimum number of standard sizes of precast elements, namely: vertical multi-storeyed columns without consoles and floor slabs [2 – 3]. The slabs are divided into over-columned, inter-columned and middle (Fig. 1, *c*). The operation of each slab depends on the method of its connection with other slabs and columns.

As shown in [7 – 9], for calculating the strength of two-way slabs, it is quite convenient to use the yield line method, which allows taking into account the actual conditions of fastening, reinforcement and the nature of the destruction of elements. In this case, one should take into account the prerequisites of the calculation using the nonlinear deformation model [10], which is regulated by current regulatory documents in the field of designing reinforced concrete structures.

Figure 1 – Precast flat plate frame structural system of buildings:
***a*) – new building in the residential complex “Pelyustkovy” (Poltava); b) – building under construction;**
c) – general view of the structural system: 1 – column; 2 – over-columned slab;
3 – inter-columned slab; 4 – middle slab; 5 – linear brace

Problem statement. In this article, based on the yield line method using a nonlinear deformation model with an extreme strength criterion, calculation dependencies for solving the problems of the strength of slabs of the flat plate structural system of buildings are obtained.

Main material and results. As a result of the analysis of the ultimate limit state of the flat plate floor, as a system of kinematically interconnected individual disks, it was established that the load on the columns is transferred in the following sequence (Fig. 2): the middle slab transfers the load to four adjacent inter-columned slabs; inter-columned slabs transfer the load to the over-columned slab; the over-columned slab transfer the load to the column. This load distribution scheme is due to the floor failure scheme, on the basis of which the calculation of its bearing capacity is implemented.

The established load distribution scheme allows to consider each floor element separately from each other in the calculations, but taking into account their interaction. The connection of the slabs with each other in practice is carried out by arranging a loop joint, which is an elastic-plastic joint capable of perceiving a fixed value of the bending moment [2]. The bending bearing capacity of the joint of the slabs 6 m long is 51 kN·m [1].

In the calculations of slabs using the yield line method, the following assumptions are made:

– it is assumed that the slab breaks into flat disks that are interconnected along the yield lines by plastic hinges;

– the smallest possible displacement for a given fracture pattern and slab loading pattern is arbitrarily set;

– an equation is drawn up that expresses the equality of the work of external and internal forces on the specified displacement;

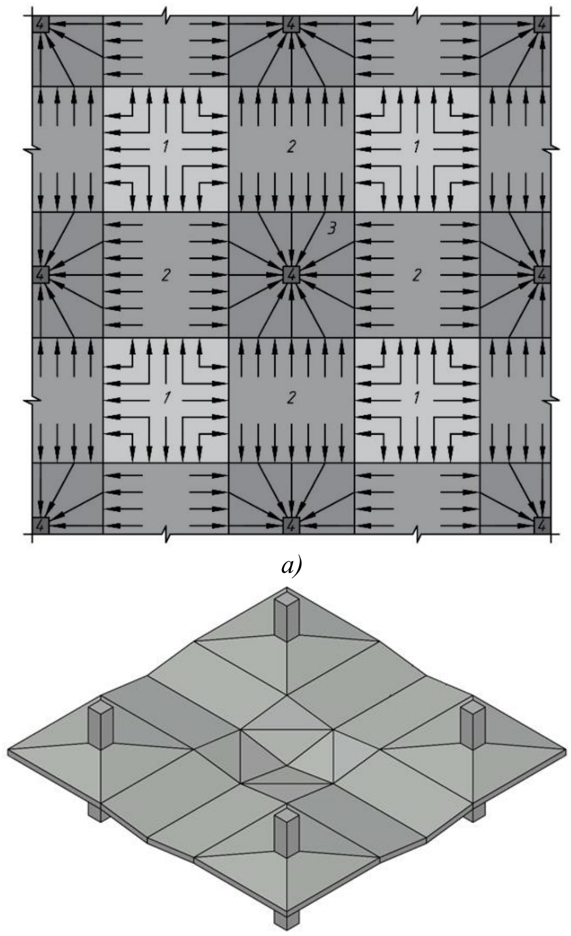
– the value of the external load that satisfies the obtained equation is the bearing capacity of the slab.

The fracture scheme of the slab must comply with the conditions of its support and the loading scheme, and also ensure a single kinematic variability of the system. In order to determine the degree of kinematic variability, it is convenient to use the analogy of the fracture scheme with a truss: the degree of kinematic variability of the fracture scheme of the slab is equal to the kinematic variability scheme of the truss, composed of all (positive and negative) yield lines and support hinges of the slab.

For each slab, a set of fracture schemes is accepted that meets the specified requirements and is confirmed experimentally. The acceptable (with a certain degree of idealization) will be the one for which the bearing capacity of the slab has the smallest value.

The equilibrium equation, which expresses the equality between the virtual works of external W_{Ed} and internal W_{Rd} forces on the corresponding possible movements of the slab in the direction of action of the forces, is generally presented as follows:

$$W_{Ed} = W_{Rd} \tag{1}$$



a)

b)

Figure 2 – Load distribution between slabs of a precast flat plate floor (a) and yield line model of floor deformation in the limit state (b): 1 – middle slab; 2 – inter-columned slab; 3 – over-columned slab; 4 – column

The equation of virtual work (1) for the case of calculating slabs loaded with loads distributed over the area and along the yield lines is presented as follows:

$$\int_A q y_q dA + \int_l p(y) y_p dl = \sum_{i=1}^{n} M_i \varphi_i l_i \,, \tag{2}$$

where q is the design limit value of the load uniformly distributed over the area A; y_q is the virtual displacement of the slab in the direction of the load q; A is the area of the part of the slab on which the load q acts; $p(y)$ is the design limit value of the load distributed along a line of length l; y_p is the virtual displacement of the slab in the direction of the load $p(y)$; l is the length of the plate line along which the load $p(y)$ acts; M_i is the ultimate value of the bending moment perceived by the i-th plastic hinge (yield line) per unit of its length; φ_i is the mutual angle of rotation of the disks in the i-th plastic hinge of the design section; l_i is the length of the i-th plastic hinge, n is the number of linear plastic hinges.

The application of the kinematic method is considered for the over-columned slab of the flat plate floor. The over-columned slabs perceive the following loads: q uniformly distributed over the entire floor, as well as the load p from the reaction of the inter-columned slab supports uniformly distributed over all faces of the slab. Since each inter-columned slab rests on two sides on the over-columned slabs, one over-columned slab is subjected to a load from 1/2 of the area of the inter-columned slab on each side. This area is rectangular in shape, so the resulting linear load on the over-columned slab will be uniformly distributed with a maximum ordinate value of $0.75ql$. The over-columned slab is rigidly fixed in the middle part within the opening and works as a cantilever in four directions (Fig. 3).

According to the applied kinematic scheme (Fig. 3), the destruction of the over-column slab occurs as a result of its division into four disks by diagonal linear hinges with the formation of linear plastic hinges at the attachment points along the perimeter of the opening.

The work of external forces in the over-columned slab in (1) is determined by the formula

$$W_{Ed} = \int_A q y_q dA + \int_l p y_p dl = \frac{qf\left(2l^2 - lb - b^2\right)}{3} + 3qfl^2 \,. \tag{3}$$

The work of internal forces (moments) at the corresponding angles of rotation is determined by the expression

$$W_{Rd} = \sum_{i=1}^{n} M_i \varphi_i l_i = 4M_{12}\varphi_{12}\left(l-b\right)\cos 45^0 + 4M_1\varphi b + 4M_{\sup}\varphi l \,, \tag{4}$$

where M_{12} is the internal bending moment that occurs when the plate breaks into adjacent disks 1 and 2 per unit length of the plastic hinge between these disks;

φ_{12} is the mutual angle of rotation of adjacent disks 1 and 2;

M_1 is the internal bending moment that occurs when disk 1 breaks off from the support along the face of the opening per unit length of the plastic hinge formed along the break-off line;

M_{sup} = 8.5 kN·m/m is the internal bending moment that occurs at the loop joint;

φ is the angle of rotation of the slab disks about the design position.

In formula (4), the angles of rotation of the disks can be expressed as follows:

$$\varphi \approx \mathrm{tg}\varphi = \frac{2f}{l-b} \,, \tag{5}$$

$$\varphi_{12} \approx \mathrm{tg}\varphi_{12} = \frac{2f}{(l-b)\cos 45^0}. \qquad (6)$$

After substituting expressions (5) – (6) into equation (4), it is obtained that

$$W_{Rd} = 8M_{12}f + 8M_1 f\frac{b}{l-b} + 8M_{\sup} f\frac{l}{l-b}. \qquad (7)$$

Equation (7) contains the unknown bending moment M_{12}, the value of which per unit length of the plastic hinge depends on the cross-sectional area of the principal reinforcement intersected by this hinge. The hinge under consideration intersects the principal reinforcement of both directions, while the over-columned slab is uniformly reinforced, i.e. the amount of reinforcement in both directions is the same, and therefore the bending moments perceived by the slab in these directions are also the same.

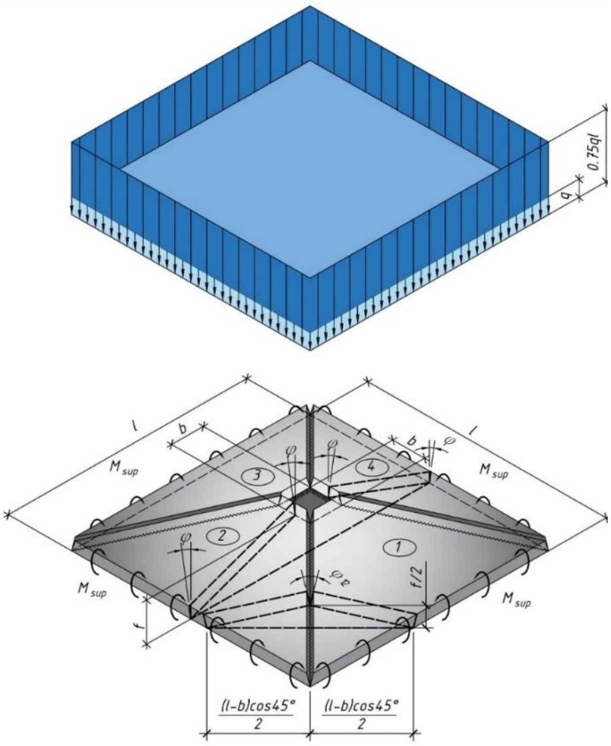


Figure 3 – Design diagram of the fracture (yield lines formation) of the over-columned slab

The bending moments perceived by the reinforcement in one direction will be $M_{Rd}\sin 45°$, and in the other - $M_{Rd}\cos 45°$. The value of the bending moment M_{12} is the geometric sum of the moments acting in the two directions:

$$M_{12} = \sqrt{M_{Rd}\sin 45° + M_{Rd}\cos 45°} = M_{Rd}, \qquad (8)$$

where M_{Rd} is the bending moment experienced by the slab in one direction.

The bending moment experienced by the slab in one direction is calculated by the formula:

$$M_{Rd} = f_{yd}A_s z_s, \qquad (9)$$

Taking into account (8) and (9) equation (1) takes the form

$$\frac{q\left(11l^2 - lb - b^2\right)}{3} = 8\frac{l}{l-b}\left(M_{Rd} + M_{\sup}\right). \qquad (10)$$

The ultimate load that the over-columned slab can withstand according to the failure scheme (Fig. 3) with a given reinforcement can be determined from equation (10) by the formula:

$$q = \frac{24l\left(M_{Rd} + M_{\sup}\right)}{11l^3 - 12l^2 b + b^3}, \qquad (11)$$

in which

$$M_{Rd} = \frac{f_{yd}A_s}{l-b}\left(d - \chi\frac{f_{yd}A_s}{f_{cd}(l-b)}\right), \qquad (12)$$

where M_{Rd} is the value of the internal bending moment per unit length of the slab;

f_{cd} is the design value of the concrete compressive strength;

d is the effective depth of the slab section;

χ = 0.52 … 0.59 is a parameter that depends on the concrete grade [10].

The problem of selecting the cross-sectional area of the principal reinforcement per unit length in one of the directions of the slab at a given load is solved based on the condition of equality of the internal and external bending moments by the formula

$$A_s = \frac{f_{cd}}{f_{yd}}\left(\frac{1-\sqrt{1-4\chi\bar{\alpha}_m}}{2\chi}\right)d, \qquad (13)$$

in which

$$\bar{\alpha}_m = \frac{q\left(11l^3 - 12l^2 b + b^3\right) - 24lM_{\sup}}{24 f_{cd} l^2 d^2}. \qquad (14)$$

The inter-columned slabs of the flat plate floor perceive the following loads: uniformly distributed q over the entire floor, as well as the load $p(y)$ from the reaction of the supports of the middle slab distributed in the form of triangles on the opposite faces of the slab. This distributed load on the slab is transmitted from two neighbouring middle slabs. Since each middle slab rests along 4 sides on the inter-columned slabs, a load from 0.25 of the area of the middle slab on each side is transmitted to one inter-columned slab. This area is triangular in shape, therefore the reduced linear load on the inter-columned slab will be triangular with the maximum ordinate value $ql/2$ in the middle of the span. The intercolumn slab is actually a crossbar by the nature of its work and support schemes.

The ultimate load that the inter-columned slab with a given reinforcement can perceive is determined based on the yield line method by the formula:

$$q = \frac{24\left(M_{Rd} + M_{\sup}\right)}{5l^2}, \qquad (15)$$

in which M_{Rd} is the value of the internal bending moment per unit length of the slab is calculated by formula (12) under the condition $b = 0$.

The problem of selecting the cross-sectional area of the principal reinforcement per unit length in one of the directions of the inter-columned slab at a given load is solved based on the condition of equality of the internal and external bending moments by formula (13), in which

$$\bar{\alpha}_m = \frac{5ql^2 - 24M_{\sup}}{24 f_{cd} l d^2}. \quad (16)$$

The middle slab in the limit state from the action of a uniformly distributed load q over its area takes the shape of a pyramid with the apex at the bottom and a height equal to the deflection f. In the calculation of the bearing capacity, the middle slab is considered as hingedly supported. At the same time, in the linear hinges formed at the joints with the inter-columned plates, a uniformly distributed bending moment M_{sup} acts, which arises at the loop joint.

The ultimate load that the middle plate can perceive with a given reinforcement can be determined based on the yield line method by the formula:

$$q = \frac{24\left(M_{Rd} + M_{\sup}\right)}{l^2}, \quad (17)$$

in which M_{Rd} is the value of the internal bending moment per unit length of the slab is calculated by formula (12) under the condition $b = 0$.

The problem of selecting the cross-sectional area of the principal reinforcement per unit length in one of the directions of the middle slab at a given load is solved based on the condition of equality of the internal and external bending moments using dependence (10) by formula (13), in which

$$\bar{\alpha}_m = \frac{ql^2 - 24M_{\sup}}{24 f_{cd} l d^2}. \quad (18)$$

Interpretation of results and their approval. Based on the developed methodology, the bearing capacity of full-scale samples of the flat plate floor slabs [2] was calculated. The characteristic values of the strength of materials were used in the calculation. The results of comparisons of the values of the destructive load without taking into account the self-weight are given in Table 1.

Table 1 – Comparison of experimental and theoretical values of the destructive load for the precast flat floor slabs

Slab type	Experimental destructive load, kN	Design destructive load, kN
Over-columned	266,70	240,07
Inter-columned	156,90	122,92
Middle	211,82	109,91

Conclusions. Based on the analysis, it was found that the flat plate floor of the precast frame combines separate elements connected by linear joints, which can perceive a certain value of the bending moment. Based on this, the possibility of calculating the bearing capacity of each slab in the floor composition based on the yield line method has been established. As a result, it is possible to consider the calculation of each slab separately, taking into account the nature of the load distribution on it, the support scheme and the interaction between the slabs in the floor composition. The developed methodology for calculating the bearing capacity of a slab of a precast flat plate floor has a clear algorithm and a substantiated experimentally physical content, which allows the desired solution to be obtained in the form of analytical dependencies, and therefore does not require the use of iterative or other approximate methods in calculations. All this will contribute to improving the design process of buildings with a precast flat plate frame, increasing the accuracy of calculating the bearing capacity, and will contribute to a faster implementation of such frames in the renovation of the housing stock of Ukraine.

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Павліков А.М.
Національний університет «Полтавська політехніка імені Юрія Кондратюка»
https://orcid.org/0000-0002-5654-5849

Гарькава О.В. *
Національний університет «Полтавська політехніка імені Юрія Кондратюка»
https://orcid.org/0000-0003-2214-3128

Збірний безбалковий каркас будівель для поновлення житлового фонду України

У сучасному будівництві широко застосовуються каркасні конструктивні системи будівель з плоскими безбалковими перекриттями. Реалізація таких систем у збірному варіанті з мінімальною кількістю елементів створює широкі можливості для прискорення термінів будівництва. Запропонована до використання безбалкова каркасна конструктивна система будівель складається з колон і плоских плит, які з'єднуються з колонами без використання консолей або капітелей.
Розрахунок несучої здатності перекриття безбалкової конструктивної системи пропонується здійснювати на основі кінематичного способу методу граничної рівноваги. Розглядається граничний стан перекриття, котрий характеризується появою пластичних шарнірів. При цьому в збірному перекритті утворення лінійних пластичних шарнірів наперед запрогнозовано вздовж ліній стиків збірних плит. Це дає змогу розглядати в розрахунку кожну плиту окремо з урахуванням умов її завантаження та закріплення у складі перекриття. У граничному стані плита внаслідок утворення пластичних шарнірів перетворюється в механізм з подальшим зростанням деформацій без збільшення навантаження. Навантаження, що відповідає такому станові плити, є несучою здатністю плити.
Використання кінематичного методу спрощує розроблення розрахункових схем для ймовірних випадків руйнування та визначення руйнівного навантаження для всіх типів збірних плоских плит безбалкового перекриття. Цей підхід характеризується чітко визначеним алгоритмом і підтверджується експериментальними даними.
На основі кінематичного способу виведені аналітичні залежності для визначення несучої здатності надколонної, міжколонної та середньої плит збірного перекриття, а також формули для розрахунку необхідної площі арматури цих плит. Ці рівняння зручні для інженерного застосування і усувають потребу в ітераційних або наближених методах розрахунку. Отримані залежності базуються на нелінійній деформаційній моделі залізобетонних елементів та враховують передумови розрахунку за діючими нормативними документами.

Ключові слова: залізобетон, каркас, безбалкове перекриття, плита, несуча здатність.

*Адреса для листування E-mail: olga-boiko@ukr.net