KAPITEL 4 / *CHAPTER 4* [4]
EFFECTIVE PRESTRESSED SLABS FOR AIRFIELD PAVEMENTS AND ROADS

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Introduction

The efficiency and economy of construction largely depends on the successful introduction of new construction technologies and materials, the use of new equipment and the development of calculation and design methods. Modern materials and technologies make it possible to develop new constructive solutions for buildings and structures, reduce the duration of their construction, reduce labor costs for construction work, and also extend the duration of their operation. Basically, this is achieved due to a qualitative increase in the strength characteristics of materials [5…8].

Reinforced concrete is one of the most common structural materials used in construction, but it has two main disadvantages - high self-weight and low crack resistance. The main ways to reduce self-weight and increase the crack resistance of reinforced concrete structures are: the use of higher-strength concrete and prestressing in structures. Reinforced concrete works especially effectively when prestressed in two directions [5…8].

One of the ways to increase the strength and stiffness properties of concrete is the introduction of various composite admixtures into its composition. In this quality, steel fibers are becoming more and more widespread. This material was named - steel fiber concrete (SFC). Thanks to the introduction of steel fibers into the concrete matrix, its structure changes, as internal reinforcing elements appear, which make it possible to increase the tensile strength during bending - by 1.5...2.5 times, and most importantly, to reduce the deformability of the material by 10...20 times in comparison with the corresponding characteristics of the concrete matrix. No less significant is the improvement of a number of other characteristics of the material, such as frost resistance, waterproofness, corrosion resistance, heat resistance, resistance to abrasion and impact, etc.

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4.1. Methodology for calculating bending elements of rectangular cross-section, reinforced with ordinary and pre-stressed reinforcement, as well as steel fiber

Consider a bending element of rectangular cross-section, reinforced with steel fiber and rod ordinary and pre-stressed reinforcement in the compressed and stretched sections of the cross-section. The stress-strain state of a rectangular combined-reinforced section is shown in Fig. 1.

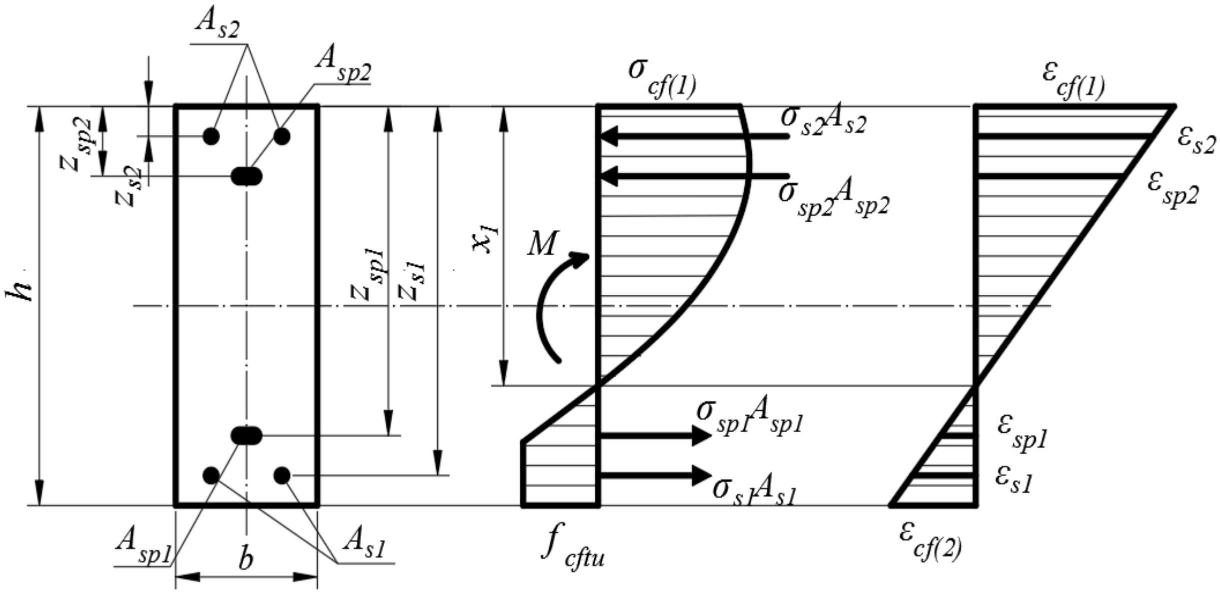

Figure 1 - Stress-deformed state of a rectangular combined-reinforced bending element

The criterion for the exhaustion of the load-bearing capacity along the normal section of the steel fiber concrete (SFC) of the element is the destruction of the SFC when the fiber deformations reach the limit values $\varepsilon_{cftu} = -2f_{cftu}/E_{cf}$. Then the value of the ultimate bending moment in the direction of one of the axes M_u for SFC bending elements of rectangular section with pre-stressed reinforcement, it is recommended to determine according to the formulas (Fig. 1):

$$\frac{bf_{cf}k_c}{\bar{\aleph}}\sum_{k=1}^{5}\frac{a_k}{k+1}\gamma^{k+1} - \frac{3}{4}bf_{cft}(h-x_1) + \sum_{i=1}^{n}\sigma_{si}A_{si} = 0; \quad (1)$$

$$\frac{bf_{cf}k_c}{\bar{\aleph}^2}\sum_{k=1}^{5}\frac{a_k}{k+2}\gamma^{k+2} - \frac{11}{24}bf_{cft}(h-x_1)^2 + \sum_{i=1}^{n}\quad\sigma_{si}A_{si}(x_1 - z_{si}) - M = 0. \quad (2)$$

In formulas (1), (2) according to [4]:

$\aleph = \left(\frac{1}{r}\right)$ - curvature of the curved axis in the section (1/m):

$$\aleph = \left(\frac{1}{r}\right) = \frac{\varepsilon_{c(1)} - \varepsilon_{c(2)}}{h}; \quad (3)$$

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$\varepsilon_{c(1)}$ - relative deformations of steel fiber concrete in the compressed cross-sectional area;

$\varepsilon_{c(2)}$ - relative deformations of steel fiber concrete in the stretched cross-sectional area;

γ - ratio of relative compression strains $\varepsilon_{c(1)}$ to the limit ε_{cf1}:

$$\gamma = \frac{\varepsilon_{c(1)}}{\varepsilon_{cf1}}; \tag{4}$$

x_1 - height of compressed zone (m):

$$x_1 = \frac{\varepsilon_{c(1)}}{\aleph}; \tag{5}$$

$\bar{\aleph}$ - relative curvature:

$$\bar{\aleph} = \frac{\aleph}{\varepsilon_{cf1}}; \tag{6}$$

σ_{si} - stress in the reinforcement;

z_{si} - the distance from the center of gravity of the reinforcement to the extreme compressed face of the section;

a_k - coefficients of the polynomial, which are determined depending on the experimental value of the prism strength of SFC according to the methodology [8].

We present equations (1), (2) in the form

$$N_{cf} - N_{cft} + N_s = 0; \tag{7}$$

$$M_{cf} + M_{cft} + M_s = M, \tag{8}$$

N_{cf}, M_{cf} - forces in the compressed zone of the SFC;

N_{cft} , M_{cft} - forces in the stretched zone of the SFC;

N_s, M_s - total forces in the reinforcement.

The value of internal forces

$$N_{cf} = \frac{bf_{cf}k_c}{\bar{\aleph}} \sum_{k=1}^{5} \frac{a_k}{k+1} \gamma^{k+1}; \tag{9}$$

$$N_{cft} = \frac{3}{4} bf_{cft}(h - x_1); \tag{10}$$

$$N_s = \sigma_{s2}A_{s2} + \sigma_{sp2}A_{sp2} - \sigma_{s1}A_{s1} - \sigma_{sp1}A_{sp1}; \tag{11}$$

$$M_{cf} = \frac{bf_{cf}k_c}{\bar{\aleph}^2} \sum_{k=1}^{5} \frac{a_k}{k+2} \gamma^{k+2}; \tag{12}$$

$$M_{cft} = \frac{11}{24} bf_{cft}(h - x_1)^2; \tag{13}$$

$$M_s = A_{s1}E_{s1}\aleph(x_1 - z_{s1})^2 + A_{sp1}E_{sp1}\big(\aleph(x_1 - z_{sp1}) - \varepsilon_{p01}\big)(x_1 - z_{sp1}) +$$
$$+A_{s2}E_{s2}\aleph(x_1 - z_{s2})^2 + A_{sp2}E_{sp2}\big(\aleph(x_1 - z_{sp2}) - \varepsilon_{p02}\big)(x_1 - z_{sp2}), \tag{14}$$

ε_{p0i} - relative deformations caused by the prestressing of the reinforcement, taking into account all losses.

Stresses in normal and pre-stressed reinforcement:

$$\sigma_{si} = E_{si}\aleph(x_1 - z_{si}); \tag{15}$$

$$\sigma_{spi} = E_{spi}\left(\aleph\left(x_1 - z_{spi}\right) - \varepsilon_{p0i}\right). \tag{16}$$

Substituting expressions (5), (6), (15), (16) into equations (9)...(11), we obtain:

$$N_{cf} = \frac{bf_{cf}\varepsilon_{cf1}k_c}{\aleph}\sum_{k=1}^{5}\frac{a_k}{k+1}\gamma^{k+1}; \tag{17}$$

$$N_{cft} = \frac{3}{4}bf_{cft}\left(h - \frac{\varepsilon_{cf(1)}}{\aleph}\right); \tag{18}$$

$$N_s = A_{s2}E_{s2}\aleph(x_1 - z_{s2}) + A_{sp2}E_{sp2}\left(\aleph(x_1 - z_{sp2}) - \varepsilon_{p02}\right) -$$
$$-A_{s1}E_{s1}\aleph(x_1 - z_{s1}) - A_{sp1}E_{sp1}\left(\aleph(x_1 - z_{sp1}) - \varepsilon_{p01}\right); \tag{19}$$

By substituting equations (17)...(19) into (7) and after transformations, we obtain the dependence for curvature

$$\aleph = \frac{-b_{\Sigma} + \sqrt{b_{\Sigma}^2 - 4a_{\Sigma}c_{\Sigma}}}{2a_{\Sigma}}, \tag{20}$$

$$a_{\Sigma} = A_{s1}E_{s1}z_{s1} + A_{s2}E_{s2}z_{s2} + A_{sp1}E_{sp1}z_{sp1} + A_{sp2}E_{sp2}z_{sp2}; \tag{21}$$

$$b_{\Sigma} = \frac{3}{4}bhf_{cft} - \varepsilon_{cf(1)}\left(A_{s1}E_{s1} + A_{s2}E_{s2} + A_{sp1}E_{sp1} + A_{sp2}E_{sp2}\right) +$$
$$+A_{sp1}E_{sp1}\varepsilon_{p01} + A_{sp2}E_{sp2}\varepsilon_{p02}; \tag{22}$$

$$c_{\Sigma} = -\frac{3}{4}bf_{cft}\varepsilon_{cf(1)} - bf_{cf}\varepsilon_{cf1}k_c\sum_{k=1}^{5}\frac{a_k}{k+1}\gamma^{k+1}. \tag{23}$$

After determining the curvature $\aleph$ its values are substituted into formulas (12)...(14) to determine moments M_{cf}, M_{cft} and M_s. After that, using formula (8), we determine the bending moment *M*, which corresponds to the curvature $\aleph$. The calculation is performed step by step for each value of the relative deformations in the compressed cross-sectional area $\varepsilon_{c(1)}$, which successively increases in value $\Delta\varepsilon_{c(1)}$.

At each step of the calculation, it is necessary to control the tension in the pre-stressed reinforcement, which is located in the stretched zone of the section. For this, we use the «σ-ε» diagram for stressed steel (Fig. 2) [3].

When the stress value is reached $\sigma_{sp} \geq f_{pd}$ in the following steps, the stress in the pre-stressed reinforcement must be determined according to the formula [8]

$$\sigma_{sp} = f_{pd} + \left(\frac{f_{pk}}{\gamma_s} - f_{pd}\right)\frac{\varepsilon_{sp} - \varepsilon_{p0}}{\varepsilon_{ud} - \varepsilon_{p0}}, \tag{24}$$

$$f_{pd} = \frac{f_{p0,1k}}{\gamma_s};\ \varepsilon_{p0} = \frac{f_{pd}}{E_p};\quad \varepsilon_{ud} = 0{,}9\varepsilon_{uk};\ \varepsilon_{sp} = \aleph\left(x_1 - z_{sp}\right) - 0{,}0021.$$

To determine the load-bearing capacity of the SFC with pre-stressed reinforcement, an algorithm was developed, which is presented in the form of a block diagram (Fig. 3).

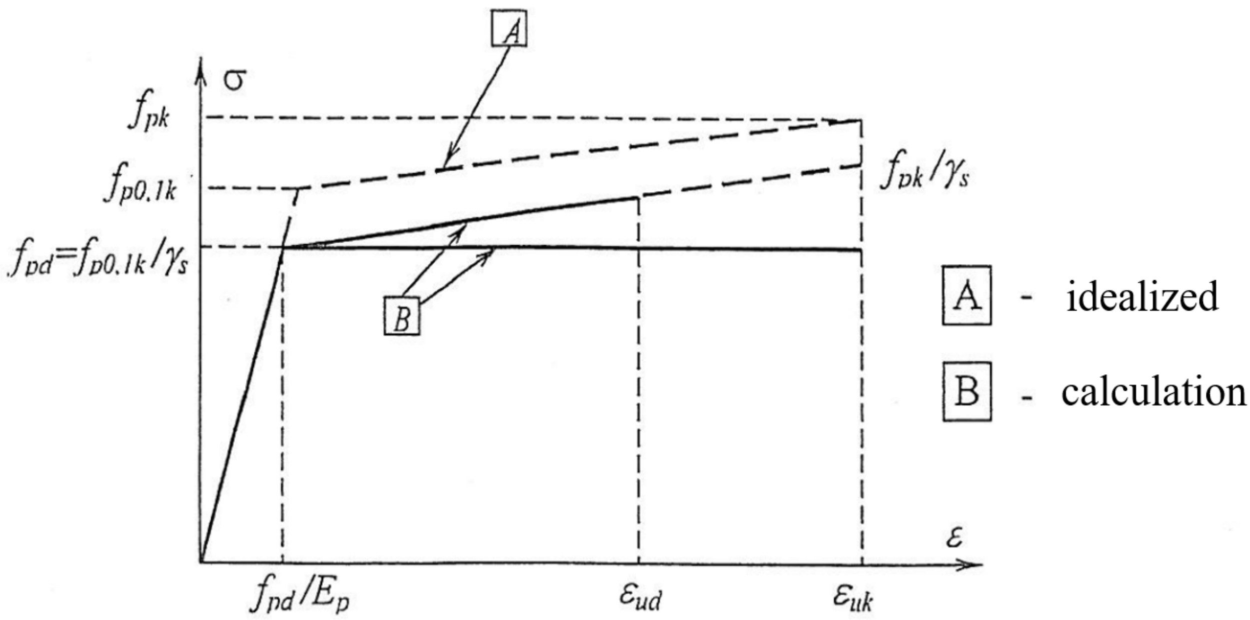

Figure 2 - Idealized and calculated «σ-ε» diagram for stressed steel [8]

1. We enter the input data: parameters of the section b, h; parameters of the deformation diagram of reinforced concrete f_{cf}, f_{cft}, E_{cf}, ε_{cfu}, ε_{cftu}, ε_{cf1}, ε_{cft1}; polynomial coefficients a_k ; reinforcement parameters E_{spi}, A_{spi}, ε_{p0i}, E_{si}, A_{si}; reinforcement step S along the axis perpendicular to what is being calculated; the distance from the upper compressed face of the section to the center of gravity of the reinforcement z_{spi} and z_{spi}.

2. At the first step, we determine the amount of lateral compression of the SFC from the pre-stressed reinforcement located along the axis Y $\sigma_2 = E_{sp}\varepsilon_{p0}\frac{A_{sp}}{h\cdot S}$.

3. We determine the coefficient of increase in the strength of SFC under biaxial compression $k_c = 1{,}0 + 1{,}38\frac{\sigma_2}{f_{cf}} - 1{,}15\left(\frac{\sigma_2}{f_{cf}}\right)^2$.

4. We set the value of the initial deformations in the compressed cross-sectional area $\varepsilon_{c(1)} = \Delta\varepsilon_{c(1)} = 0{,}1\varepsilon_{cf1}$.

5. We determine the coefficient $= \frac{\varepsilon_{cf(1)}}{\varepsilon_{cf1}}$.

6. We define a polynomial
$\sum_{k=1}^{5}\frac{a_k}{k+1}\gamma^{k+1} = \frac{a_1}{2}\gamma^2 + \frac{a_2}{3}\gamma^3 + \frac{a_3}{4}\gamma^4 + \frac{a_4}{5}\gamma^5 + \frac{a_5}{6}\gamma^6$.

7. We define the parameters
$a_\Sigma = A_{sp}E_{sp}z_{sp}$;
$b_\Sigma = \frac{3}{4}bhf_{cft} - A_{sp}E_{sp}\varepsilon_{cf(1)} + A_{sp}E_{sp}\varepsilon_{p0}$;
$c_\Sigma = -\frac{3}{4}bf_{cft}\varepsilon_{cf(1)} - bf_{cf}\varepsilon_{cf1}k_c\sum_{k=1}^{5}\frac{a_k}{k+1}\gamma^{k+1}$.

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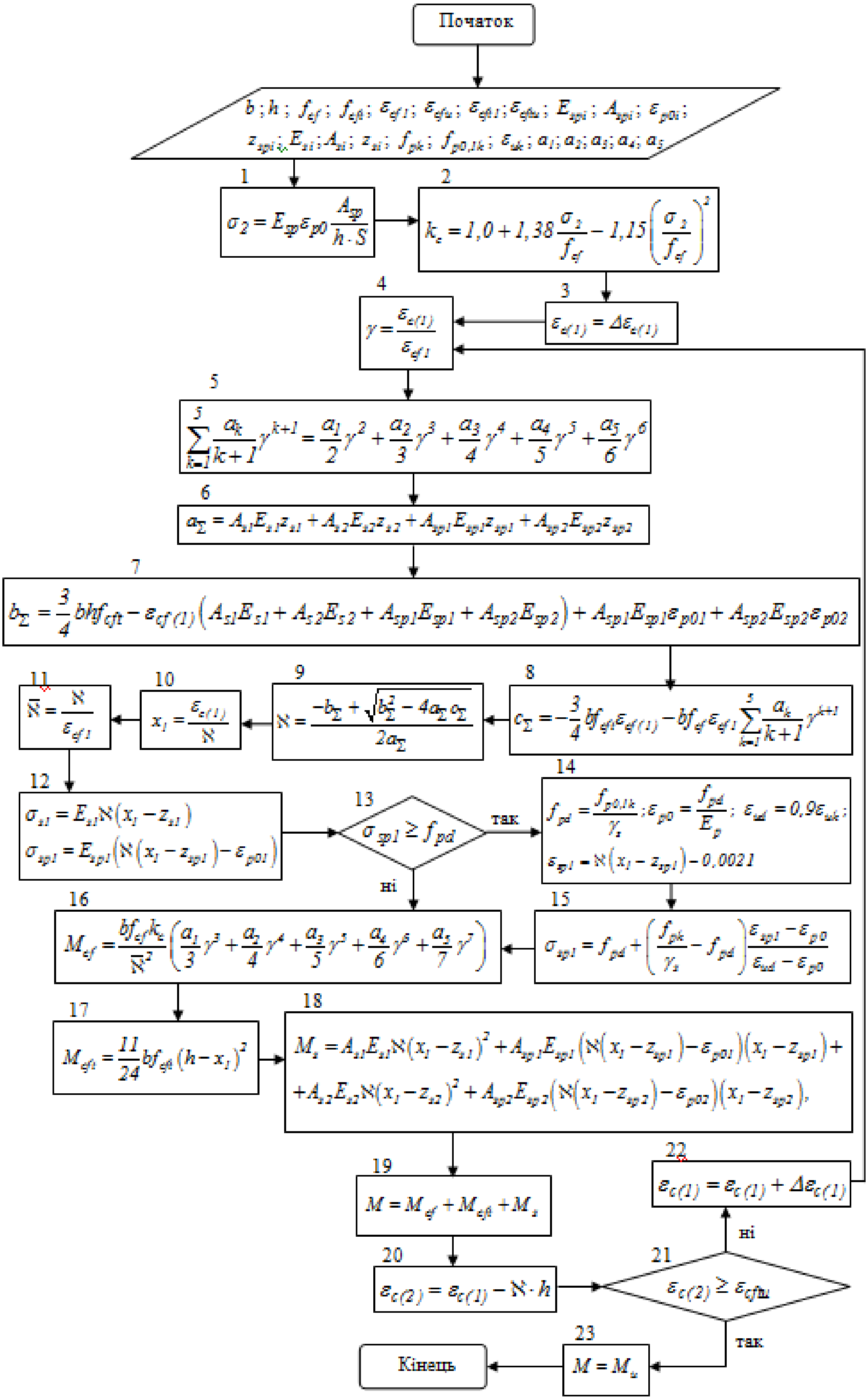

Figure 3 - Block diagram for calculating the bearing capacity of bending SFC elements with pre-stressed reinforcement located in two directions

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8. We determine the amount of curvature $\aleph = \frac{-b_\Sigma + \sqrt{b_\Sigma^2 - 4a_\Sigma c_\Sigma}}{2a_\Sigma}$.

9. We determine the height of the compressed cross-sectional area $x_1 = \frac{\varepsilon_{c(1)}}{\aleph}$.

10. We determine the value of the relative curvature $\overline{\aleph} = \frac{\aleph}{\varepsilon_{cf1}}$.

11. We determine the amount of tension in the armature

$\sigma_{sp} = E_{sp}\big(\aleph_i(x_1 - z_{sp}) - \varepsilon_{p0}\big)$.

12. We check the condition $\sigma_{sp1} \geq f_{pd}$. If the condition is fulfilled, then the stress in the prestressed armature is determined by the formula

$\sigma_{sp} = f_{pd} + \left(\frac{f_{pk}}{\gamma_s} - f_{pd}\right)\frac{\varepsilon_{sp} - \varepsilon_{p0}}{\varepsilon_{ud} - \varepsilon_{p0}}$, we: $f_{pd} = \frac{f_{p0,1k}}{\gamma_s}$; $\varepsilon_{p0} = \frac{f_{pd}}{E_p}$; $\varepsilon_{ud} = 0{,}9\varepsilon_{uk}$;

$\varepsilon_{sp} = \aleph\big(x_1 - z_{sp}\big) - 0{,}0021$ and we go to paragraph 13. If the condition $\sigma_{sp1} \geq f_{pd}$ is not performed, then the tension is not recalculated and we proceed to paragraph.

13. We determine the moment perceived by the compressed cross-sectional area $M_{cf} = \frac{b f_{cf} k_c}{\overline{\aleph}^2}\left(\frac{a_1}{3}\gamma^3 + \frac{a_2}{4}\gamma^4 + \frac{a_3}{5}\gamma^5 + \frac{a_4}{6}\gamma^6 + \frac{a_5}{7}\gamma^7\right)$.

14. We determine the moment perceived by the stretched cross-sectional area $M_{cft} = \frac{11}{24} b f_{cft}(h - x_1)^2$.

15. We determine the moment perceived by the reinforcing bars

$M_s = A_{sp}E_{sp}\big(\aleph(x_1 - z_{sp}) - \varepsilon_{p0}\big)\big(x_1 - z_{sp}\big)$.

16. We determine the moment in the section $M = M_{cf} + M_{cft} + M_s$.

17. We determine the magnitude of the relative deformations in the stretched cross-sectional area $\varepsilon_{c(2)} = \varepsilon_{c(1)} - \aleph \cdot h$ and compare its values with the limit deformations $\varepsilon_{c(2)} \geq \varepsilon_{cftu}$. If the condition is not met, then we proceed to the second step of the calculation. If the condition is fulfilled, that is, the relative tensile deformations are equal to or exceed the limit values, then the calculation is completed and the moment value is taken as the bearing capacity $M = M_u$.

18. In the second step, we increase the value $\varepsilon_{c(1)}$ by $\Delta\varepsilon_{c(1)} = 0{,}1\varepsilon_{cf(1)}$ and we perform the calculation according to p.p. 5...17.

19. Value *M* and $\aleph$ at each step, we put the "moment-curvature" on the graph.

The block diagram is implemented in the program Mathcad.

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4.2. Effective pre-stressed slabs of airfield pavements

For the arrangement of airfield surfaces and helipads, prefabricated reinforced concrete prestressed PAG-14 slabs are used, which correspond to the current DSTU B B.2.6-136:2010. Structures of buildings and structures. PAG-14 prestressed reinforced concrete slabs for airfield pavement [1]. The slabs have plan dimensions of 6.0× 2.0 m and a thickness of 140 mm (Fig. 4).

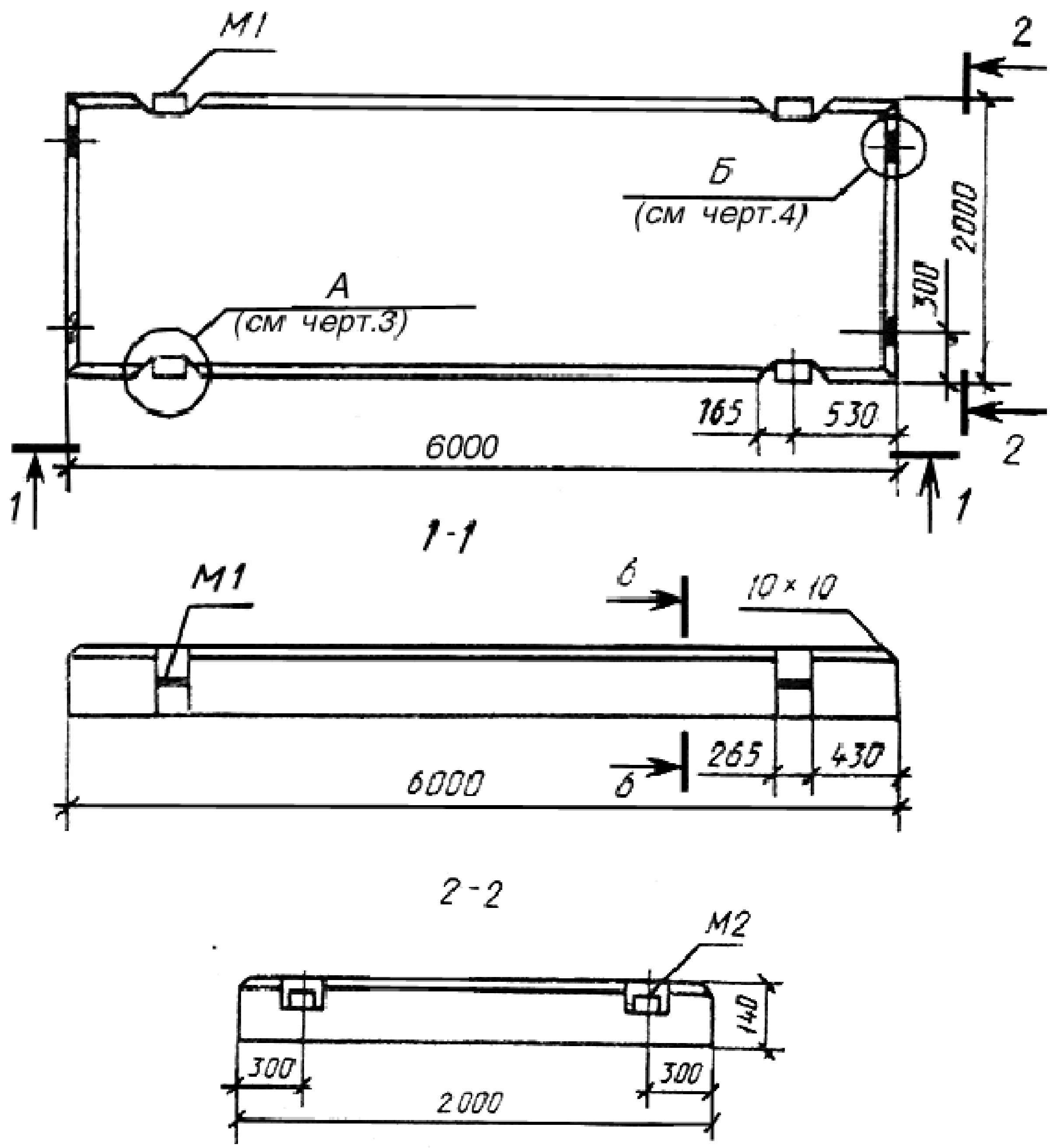

Figure 4 - Precast reinforced concrete airfield plate PAG-14

Slabs are made of C20/25 grade concrete and reinforced with longitudinal pre-stressed reinforcement 5Ø14Aт-V in two levels and C2 reinforcing meshes in two levels with reinforcement Ø5Bp-I with a step of 100 mm, located in the transverse direction of the slab, and 4Ø5Bp-I, located in the longitudinal direction of the plate (Fig. 5). In the end parts, grids C1 are arranged in two levels with reinforcement 4Ø8A400C, located in the transverse direction of the plate, and 2Ø5Bp-I+2Ø8A400C, located in the longitudinal direction of the plate.

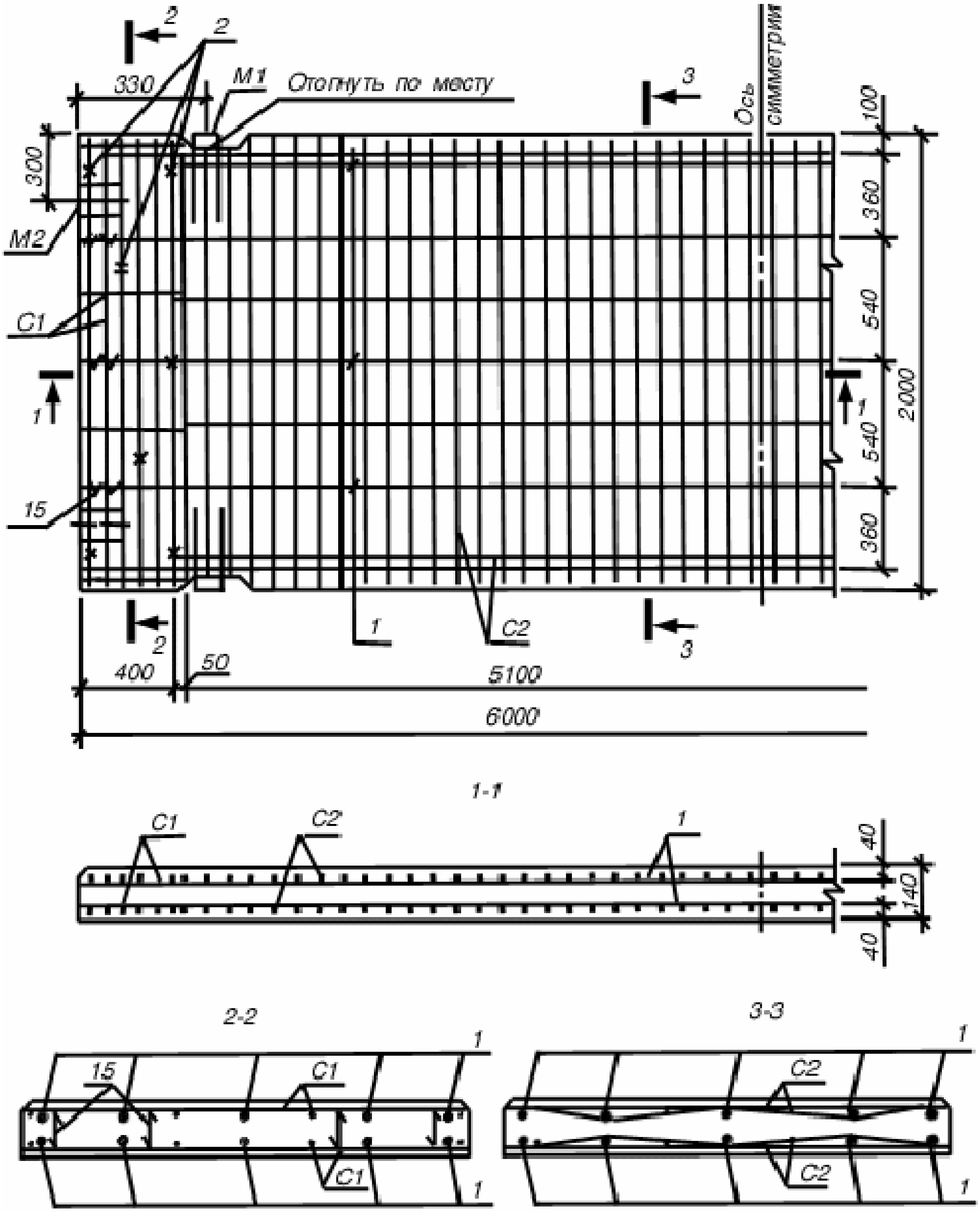

Figure 5 - Scheme of PAG-14 airfield plate reinforcement

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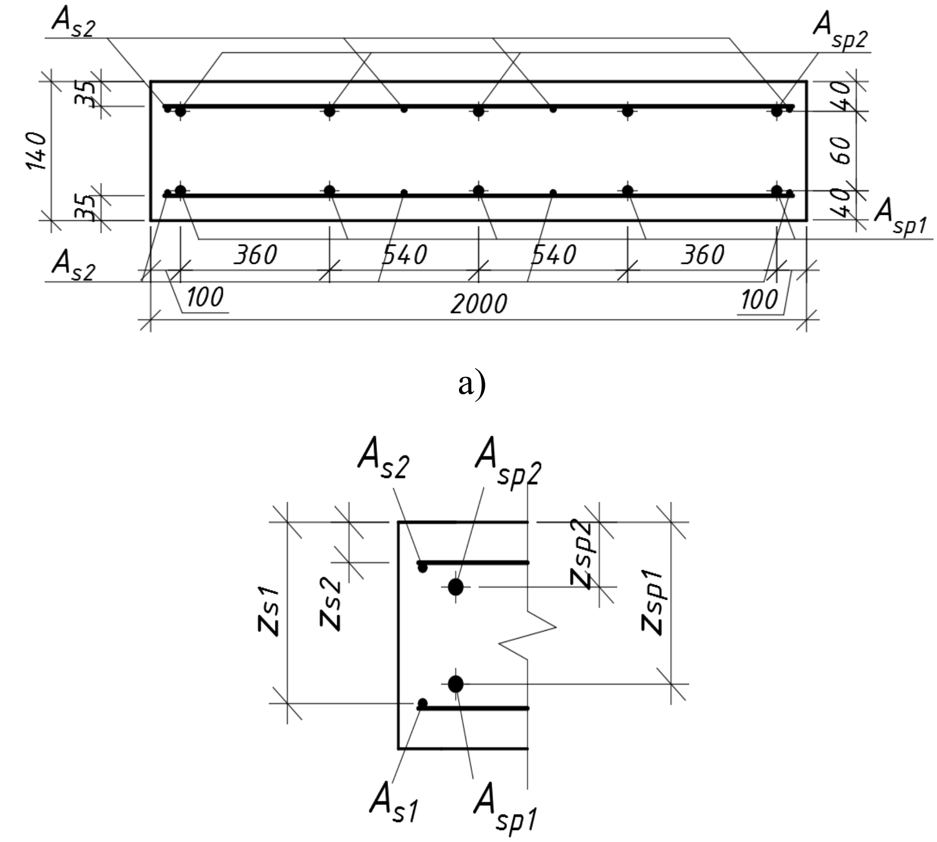
4.2.1. Calculation of the bearing capacity of the PAG-14 plate without steel fiber

Cross-sectional dimensions b=2000 mm, h=140 mm (Fig. 6).



a)

б)

Figure 6 - Estimated section of PAG-14 plate (a) and calculation parameters (b)

Pre-stressed reinforcement in two levels of 5Ø14At-V: A_{sp1}=A_{sp2}=769 mm^2; E_{sp}=190000 MPa; f_{pk}=840 MPa; $f_{p0,1k}$=765 MPa; ε_{uk}=0,018. Unstressed reinforcement in two levels according to 4Ø5Bp-I: A_{s1}=A_{s2}=79 mm^2; E_{sp}=170000 MPa. Elongation of reinforcement, taking into account all losses, is ε_{sp0}= -0,002. The distance from the upper compressed face of the section to the center of gravity of the reinforcing bars z_{sp1}=100 mm, z_{sp2}=40 mm, z_{s1}=105 mm, z_{s2}=35 mm.

Class of concrete C20/25: f_{cd}=14,5 MPa, E_{cd}=23000 MPa, ε_{c1}=0,00165, ε_{cu}=0,00344.

Polynomial coefficients:

a_1	a_2	a_3	a_4	a_5
2,8785	-3,1586	1,7454	-0,52904	0,06374

We perform the calculation according to the block diagram. The calculation results are summarized in Table 1. The "moment-curvature" graph is shown in Fig. 7. The bearing capacity of the plate is M_u=*71,49 kNm*.

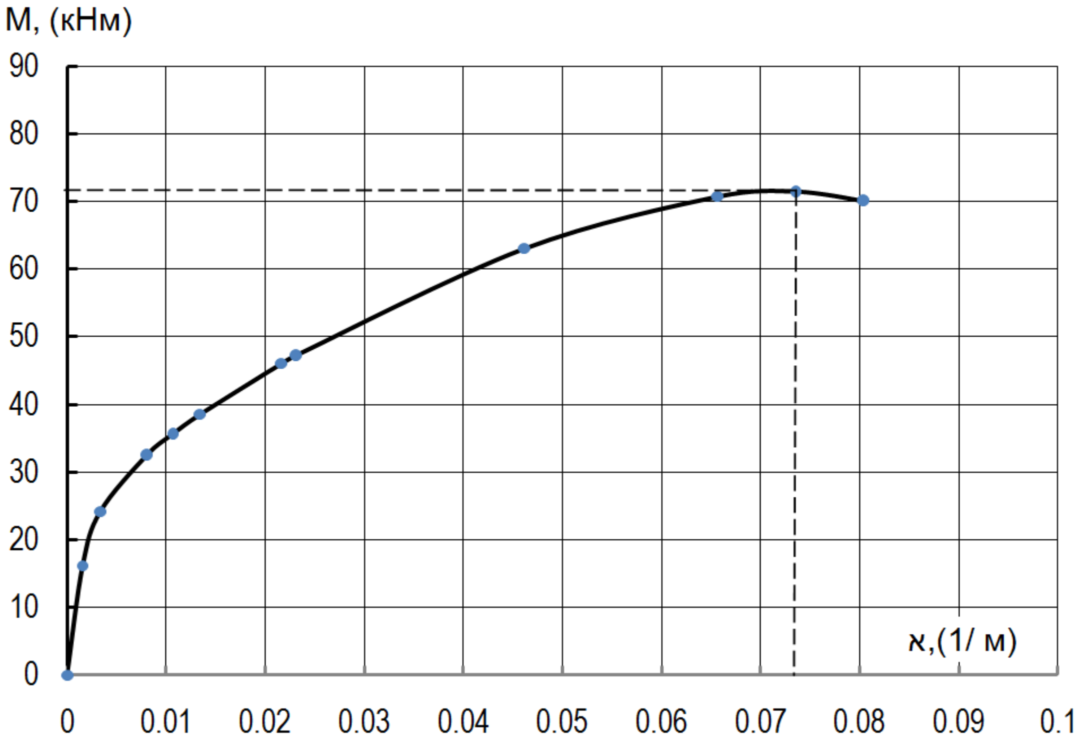


Figure 7 - "Moment-curvature" graph when calculating the PAG-14 plate without metal fiber

Table 1 - Results of calculation of bearing capacity of PAG-14 plate without steel fiber

Point	$\varepsilon_{c(1)}$	$\varepsilon_{c(2)}$	$\aleph$ 1/m	γ	x_1 m	$\overline{\aleph}$	σ_{s1} MPa	σ_{sp1} MPa	M MNm
1	0.0002	$-2.079 \cdot 10^{-5}$	$1.577 \cdot 10^{-3}$	$1.212 \cdot 10^{-1}$	$1.268 \cdot 10^{-1}$	$9.558 \cdot 10^{-1}$	5.849	$-3.91 \cdot 10^{2}$	$1.615 \cdot 10^{-2}$
2	0.0003	$-1.698 \cdot 10^{-4}$	$3.356 \cdot 10^{-3}$	$1.818 \cdot 10^{-1}$	$8.939 \cdot 10^{-2}$	2.034	-8.905	$-4.058 \cdot 10^{2}$	$2.419 \cdot 10^{-2}$
3	0.0005	$-6.263 \cdot 10^{-4}$	$8.045 \cdot 10^{-3}$	$3.03 \cdot 10^{-1}$	$6.215 \cdot 10^{-2}$	4.876	$-5.861 \cdot 10$	$-4.569 \cdot 10^{2}$	$3.253 \cdot 10^{-2}$
4	0.0006	$-8.958 \cdot 10^{-4}$	$1.068 \cdot 10^{-2}$	$3.636 \cdot 10^{-1}$	$5.616 \cdot 10^{-2}$	6.475	$-8.871 \cdot 10$	$-4.88 \cdot 10^{2}$	$3.566 \cdot 10^{-2}$
5	0.0007	$-1.176 \cdot 10^{-3}$	$1.34 \cdot 10^{-2}$	$4.242 \cdot 10^{-1}$	$5.223 \cdot 10^{-2}$	8.123	$-1.202 \cdot 10^{2}$	$-5.207 \cdot 10^{2}$	$3.852 \cdot 10^{-2}$
6	0.001	$-2.027 \cdot 10^{-3}$	$2.162 \cdot 10^{-2}$	$6.061 \cdot 10^{-1}$	$4.626 \cdot 10^{-2}$	$1.31 \cdot 10$	$-2.159 \cdot 10^{2}$	$-6.197 \cdot 10^{2}$	$4.603 \cdot 10^{-2}$
7	0.0010552	$-2.18 \cdot 10^{-3}$	$2.311 \cdot 10^{-2}$	$6.395 \cdot 10^{-1}$	$4.567 \cdot 10^{-2}$	$1.4 \cdot 10$	$-2.33 \cdot 10^{2}$	$-6.375 \cdot 10^{2}$	$4.727 \cdot 10^{-2}$
8	0.0002	$-4.46 \cdot 10^{-3}$	$4.614 \cdot 10^{-2}$	1.212	$4.334 \cdot 10^{-2}$	$2.796 \cdot 10$	$-4.836 \cdot 10^{2}$	$-6.441 \cdot 10^{2}$	$6.303 \cdot 10^{-2}$
9	0.0003	$-6.185 \cdot 10^{-3}$	$6.561 \cdot 10^{-2}$	1.818	$4.573 \cdot 10^{-2}$	$3.976 \cdot 10$	$-6.611 \cdot 10^{2}$	$-6.487 \cdot 10^{2}$	$7.071 \cdot 10^{-2}$
10	0.0035	$-6.8 \cdot 10^{-3}$	$7.357 \cdot 10^{-2}$	2.121	$4.757 \cdot 10^{-2}$	$4.459 \cdot 10$	$-7.183 \cdot 10^{2}$	$-6.502 \cdot 10^{2}$	$7.149 \cdot 10^{-2}$
11	0.004	$-7.245 \cdot 10^{-3}$	$8.032 \cdot 10^{-2}$	2.424	$4.98 \cdot 10^{-2}$	$4.868 \cdot 10$	$-7.537 \cdot 10^{2}$	$-6.51 \cdot 10^{2}$	$7.012 \cdot 10^{-2}$

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4.2.2. Calculation of the bearing capacity of the PAG-14 plate with steel fiber

Cross-sectional dimensions b=2000 mm, h=140 mm (Fig.8).

Pre-stressed reinforcement in two levels of 5Ø14At-V: A_{sp1}=A_{sp2}=769 mm^2; E_{sp}=190000 MPa; f_{pk}=840 MPa; $f_{p0,1k}$=765 MPa; ε_{uk}=0,018. Unstressed reinforcement in two levels according to 4Ø5Bp-I: A_{s1}=A_{s2}=79 mm^2; E_{sp}=170000 MPa. Elongation of reinforcement, taking into account all losses, is ε_{sp0}= -0,002. The distance from the upper compressed face of the section to the center of gravity of the reinforcing bars z_{sp1}=100 mm, z_{sp2}=40 mm, z_{s1}=105 mm, z_{s2}=35 mm.

Class of concrete C20/25: f_{cd}=14,5 MPa, E_{cd}=23000 MPa, ε_{c1}=0,00165, ε_{cu}=0,00344.

Steel fiber STAFIB 50/1.0: f_f=1000 MPa; l_f=50 mm; d_f=1 mm; μ_{fv}=0,01.

The calculated compressive strength of steel fiber concrete is determined according to DSTU [4]:

We define the ratio (h/l_f)=140/50=2,8 and (b/l_f)=2000/50=40 and determine the coefficient k_n=0,534.

We determine the coefficient of efficiency of indirect fiber reinforcement:

$$\varphi_f = \frac{5+L}{1+4{,}5L} = \frac{5+0{,}1967}{1+4{,}5\cdot 0{,}1967} = 2{,}767,$$

$$L = \frac{k_n^2 \cdot \mu_{fv} \cdot f_f}{f_{cd}} = \frac{0{,}534^2 \cdot 0{,}01 \cdot 1000}{14{,}5} = 0{,}1967.$$

$$f_{cf} = f_{cd} + \left(k_n^2 \cdot \varphi_f \cdot \mu_{fv} \cdot f_f\right) = 14{,}5 + \left(0{,}534^2 \cdot 2{,}767 \cdot 0{,}01 \cdot 1000\right) = \\ = 22{,}36 \quad MPa.$$

Coefficient

$$K_T = \sqrt{1-\left(1{,}2-80\mu_{fv}\right)^2} = \sqrt{1-(1{,}2-80\cdot 0{,}01)^2} = 0{,}9165.$$

$$f_{cft} = 1{,}1 f_{cd}\left(K_T \frac{k_{or}^2 \cdot \mu_{fv} \cdot l_f}{0{,}8 \cdot \eta_f \cdot d_{f,red}} + 0{,}08 - 0{,}5\mu_{fv}\right) =$$

$$= 1{,}1 \cdot 14{,}5\left(0{,}9165 \frac{0{,}534^2 \cdot 0{,}01 \cdot 50}{0{,}8 \cdot 0{,}9 \cdot 1} + 0{,}08 - 0{,}5 \cdot 0{,}01\right) = 1{,}49 \quad MPa.$$

The modulus of elasticity of SFC is determined according to DSTU [4]:

$$E_{cf} = E_{cd}\left(1-\mu_{fv}\right) + E_f \mu_{fv} = 25000(1-0{,}01) + 190000 \cdot 0{,}01 \\ = 24940 \quad MPa.$$

Polynomial coefficients:

a_1	a_2	a_3	a_4	a_5
2,51816	-2,14804	0,71003	-0,04839	-0,03169

Relative deformations of SFC in compression $\varepsilon_{cf1} = 0{,}00176$;

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$\varepsilon_{cftu} = 0{,}00293$.

Relative deformations of SFC in tension $\varepsilon_{cft1} = 0{,}00018$; $\varepsilon_{cftu} = 0{,}00035$.

We perform the calculation according to the block diagram. The calculation results are summarized in Table 2. The "moment-curvature" graph is shown in Fig. 8. The bearing capacity of the plate is M_u=86,73 kNm.

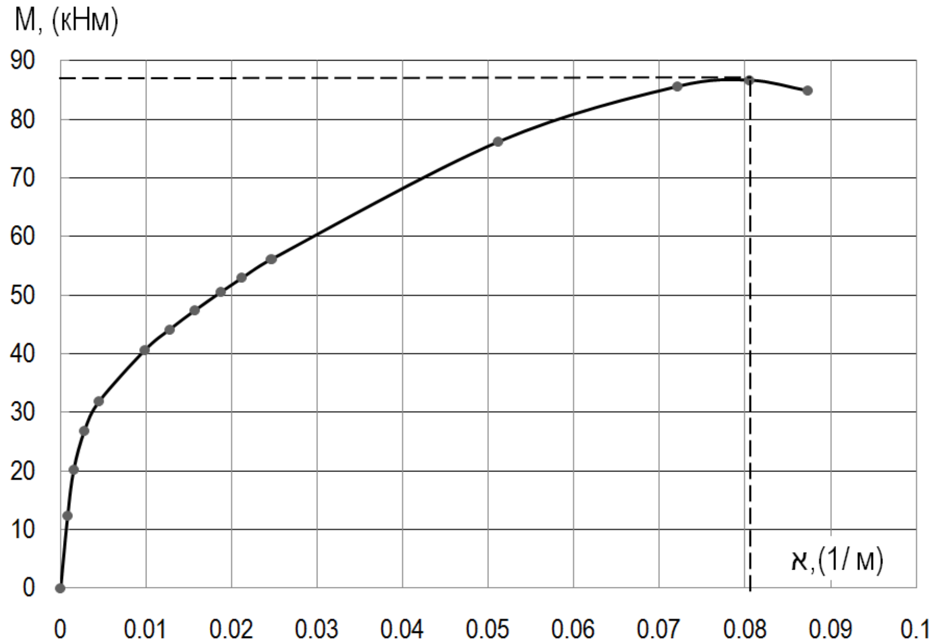

Figure 8 - "moment-curvature" graph when calculating the PAG-14 plate with steel fiber

As a result of the comparative calculation, it was established that the bearing capacity of the PAG-14 plate, in which the reinforcing meshes were replaced by steel fiber, was M_u=86,73 kNm. The bearing capacity of the standard PAG-14 plate was M_u=71,49 kNm, which is 21.3 % less.

The effectiveness of the steel fiber plate is that the steel fiber almost completely replaces the reinforcing mesh with a total weight of 72.0 kg. There are also no costs for the production of these grids. The comparative calculation showed that it is possible to reduce the amount of high-strength pre-stressed reinforcement to 10...15 %.

Due to the good anti-abrasion properties of steel-reinforced concrete, their service life is much longer than that of reinforced concrete.

Table 2 - Results of calculation of bearing capacity of PAG-14 plate with steel fiber

Point	$\varepsilon_{c(1)}$	$\varepsilon_{c(2)}$	$\aleph$ 1/m	γ	x_1 m	$\bar{\aleph}$	σ_{sp1} MPa	M MNm
1	0.0001	$-1.432\cdot10^{-5}$	$8.166\cdot10^{-4}$	$5.917\cdot10^{-2}$	$1.225\cdot10^{-1}$	$4.832\cdot10^{-1}$	$-1.865\cdot10^{2}$	$1.237\cdot10^{-2}$
2	0.00015	$-6.268\cdot10^{-5}$	$1.519\cdot10^{-3}$	$8.876\cdot10^{-2}$	$9.874\cdot10^{-2}$	$8.989\cdot10^{-1}$	$-1.904\cdot10^{2}$	$2.015\cdot10^{-2}$
3	0.0002205	$-1.7\cdot10^{-4}$	$2.789\cdot10^{-3}$	$1.305\cdot10^{-1}$	$7.905\cdot10^{-2}$	1.651	$-2.011\cdot10^{2}$	$2.689\cdot10^{-2}$
4	0.0003	$-3.351\cdot10^{-4}$	$4.536\cdot10^{-3}$	$1.775\cdot10^{-1}$	$6.614\cdot10^{-2}$	2.684	$-2.192\cdot10^{2}$	$3.193\cdot10^{-2}$
5	0.0005	$-8.78\cdot10^{-4}$	$9.843\cdot10^{-3}$	$2.959\cdot10^{-1}$	$5.08\cdot10^{-2}$	5.824	$-2.82\cdot10^{2}$	$4.063\cdot10^{-2}$
6	0.0006	$-1.185\cdot10^{-3}$	$1.275\cdot10^{-2}$	$3.55\cdot10^{-1}$	$4.706\cdot10^{-2}$	7.545	$-3.183\cdot10^{2}$	$4.417\cdot10^{-2}$
7	0.0007	$-1.501\cdot10^{-3}$	$1.572\cdot10^{-2}$	$4.142\cdot10^{-1}$	$4.452\cdot10^{-2}$	9.304	$-3.557\cdot10^{2}$	$4.745\cdot10^{-2}$
8	0.0008	$-1.82\cdot10^{-3}$	$1.872\cdot10^{-2}$	$4.734\cdot10^{-1}$	$4.274\cdot10^{-2}$	$1.107\cdot10$	$-3.936\cdot10^{2}$	$5.052\cdot10^{-2}$
9	0.001	$-2.451\cdot10^{-3}$	$2.465\cdot10^{-2}$	$5.917\cdot10^{-1}$	$4.057\cdot10^{-2}$	$1.459\cdot10$	$-4.683\cdot10^{2}$	$5.616\cdot10^{-2}$
10	0.0014939	$-3.895\cdot10^{-3}$	$3.849\cdot10^{-2}$	$8.84\cdot10^{-1}$	$3.881\cdot10^{-2}$	$2.278\cdot10$	$-6.375\cdot10^{2}$	$6.763\cdot10^{-2}$
11	0.002	$-5.167\cdot10^{-3}$	$5.119\cdot10^{-2}$	1.183	$3.907\cdot10^{-2}$	$3.029\cdot10$	$-6.412\cdot10^{2}$	$7.626\cdot10^{-2}$
12	0.003	$-7.09\cdot10^{-3}$	$7.207\cdot10^{-2}$	1.775	$4.163\cdot10^{-2}$	$4.264\cdot10$	$-6.465\cdot10^{2}$	$8.567\cdot10^{-2}$
13	0.0035	$-7.765\cdot10^{-3}$	$8.046\cdot10^{-2}$	2.071	$4.35\cdot10^{-2}$	$4.761\cdot10$	$-6.482\cdot10^{2}$	$8.673\cdot10^{-2}$
14	0.004	$-8.224\cdot10^{-3}$	$8.732\cdot10^{-2}$	2.367	$4.581\cdot10^{-2}$	$5.167\cdot10$	$-6.491\cdot10^{2}$	$8.492\cdot10^{-2}$

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4.3. Effective pre-stressed road slabs

Pre-stressed concrete slabs of the P60.38, P60.35 and P60.30 brands are used for the construction of road surfaces, which correspond to the current DSTU B V.2.6-120:2010 [2]. Structures of buildings and structures. Reinforced concrete slabs for covering city roads. Plates of the P60.38 brand have dimensions in plan of 6.0×3.75 m, the P60.35 brand - 6.0×3.5 m, the P60.30 brand - 6.0×3.0 m and a thickness of 140 mm (Fig. 9).

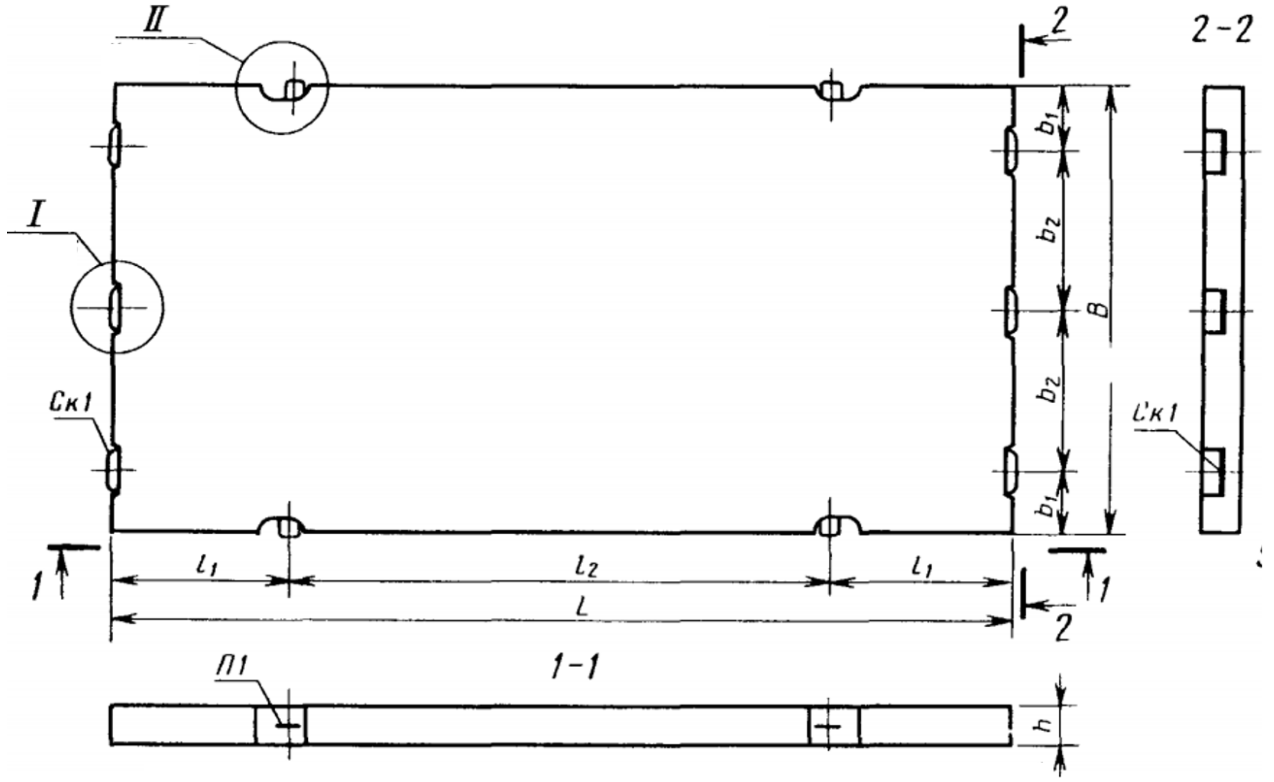


Plate brand	L, mm	B, mm	h, mm
P60.38	6000	3750	140
P60.35	6000	3500	140
P60.30	6000	3000	140

Figure - 9. Precast reinforced concrete slabs of brands P60.38, P60.35 and P60.30

The slabs are made of C25/30 grade concrete and reinforced with pre-stressed reinforcement in two directions (Fig. 10). The P60.38 slab is reinforced with 24Ø10At-V reinforcement, located in the longitudinal direction of the slab in two levels, and 18Ø12At-V, located in the transverse direction of the slab in the center. The P60.35 plate is reinforced with 22Ø10At-V reinforcement, located in the longitudinal direction of the plate in two levels, and 18Ø12At-V, located in the transverse direction of the plate in the center. The P60.30 slab is reinforced with 20Ø10At-V reinforcement, located in the longitudinal direction of the slab in two levels in two levels, and 18Ø12At-V, located in the transverse direction of the slab in the center.

4.3.1. Calculation of the bearing capacity of the P60.38 plate without steel fiber

Cross-sectional dimensions b=3750 mm, h=140 mm (Fig. 11).

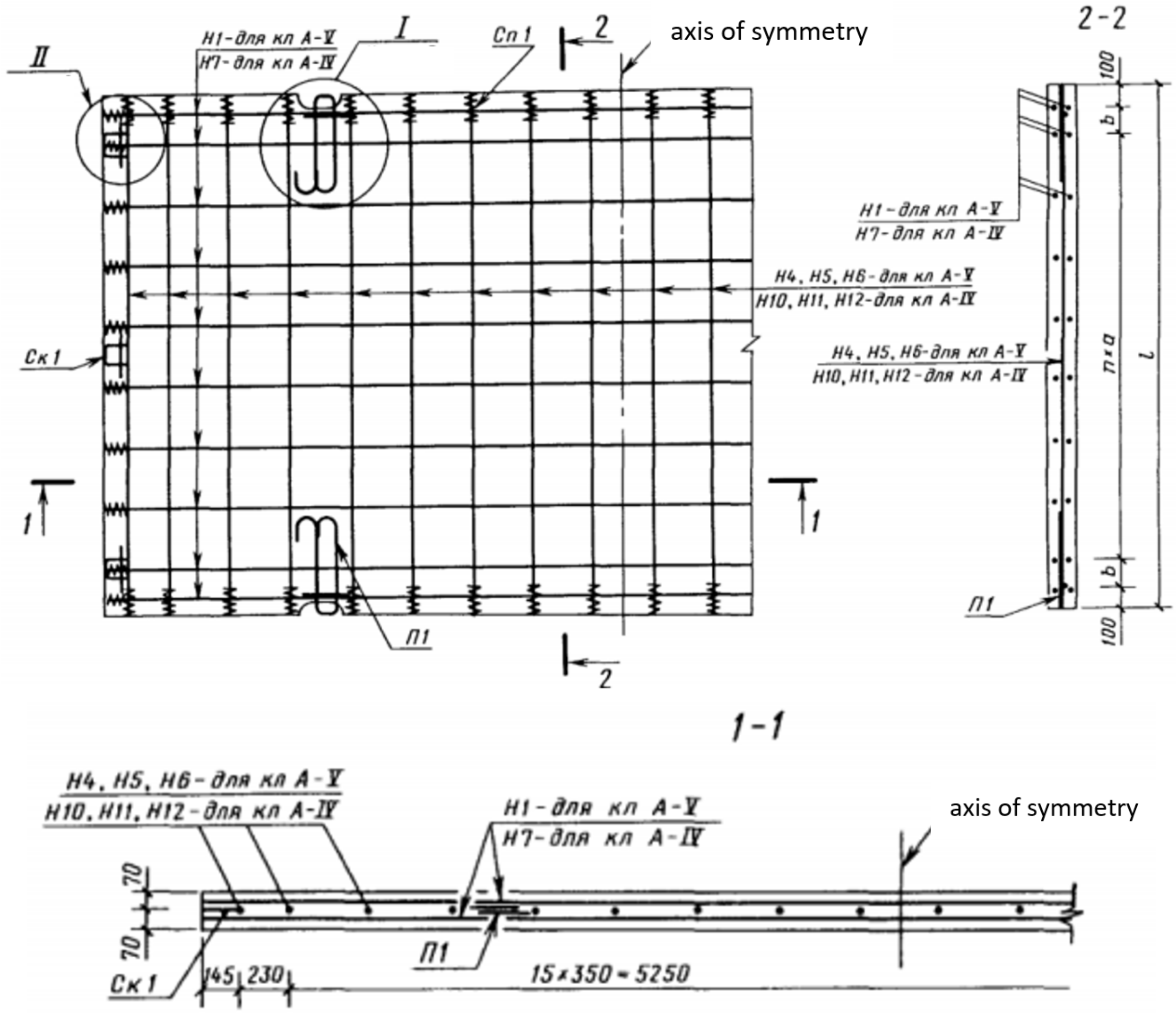
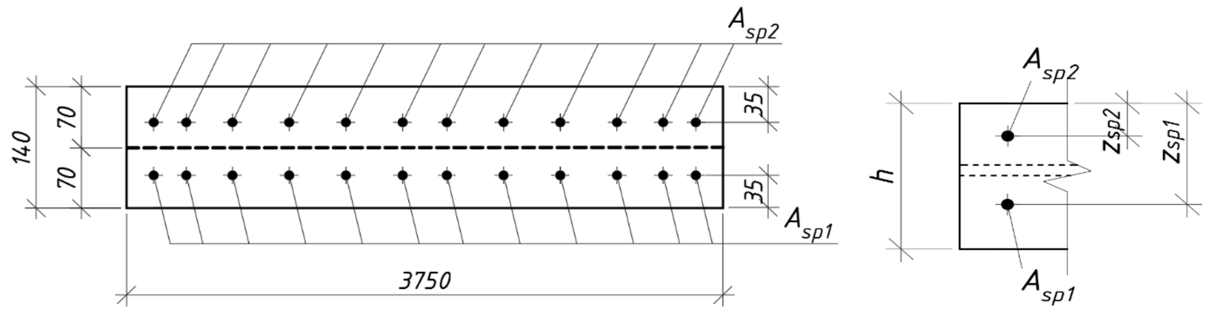


Plate	size, mm			
	a	*b*	*n*	*l*
P60.38	350	200	9	3750
P60.35	350	250	8	3500
P60.30	350	175	7	3000

Figure 10 - Scheme of plate reinforcement P60.38, P60.35 and P60.30

Pre-tensioned reinforcement in two levels of 12Ø10At-V: A_{sp1}=A_{sp2}=942 mm^2; E_{sp}=190000 MPa; f_{pk}=840 MPa; $f_{p0,1k}$=765 MPa; ε_{uk}=0,018. Elongation of reinforcement, taking into account all losses, is ε_{sp0}= -0,002. Distance from the upper compressed face of the section to the center of gravity of the reinforcing bars z_{sp1}=105 mm, z_{sp2}=35 mm. The step of reinforcement along the axis perpendicular to the one that is calculated S=350 mm.



a) b)

Figure 11 - Estimated section of plate P60.38 (a) and calculation parameters (b)

Class of concrete C25/30: f_{cd}=17,0 MPa, E_{cd}=25000 MPa ε_{c1}=0,00169, ε_{cu}=0,00328. Polynomial coefficients:

a_1	a_2	a_3	a_4	a_5
2,7404	-2,7649	1,3416	-0,35004	0,03295

We perform the calculation according to the block diagram. The calculation results are summarized in Table 3. The "moment-curvature" graph is shown in Fig. 12. The bearing capacity of the plate is M_u= *144,9 kNm*.

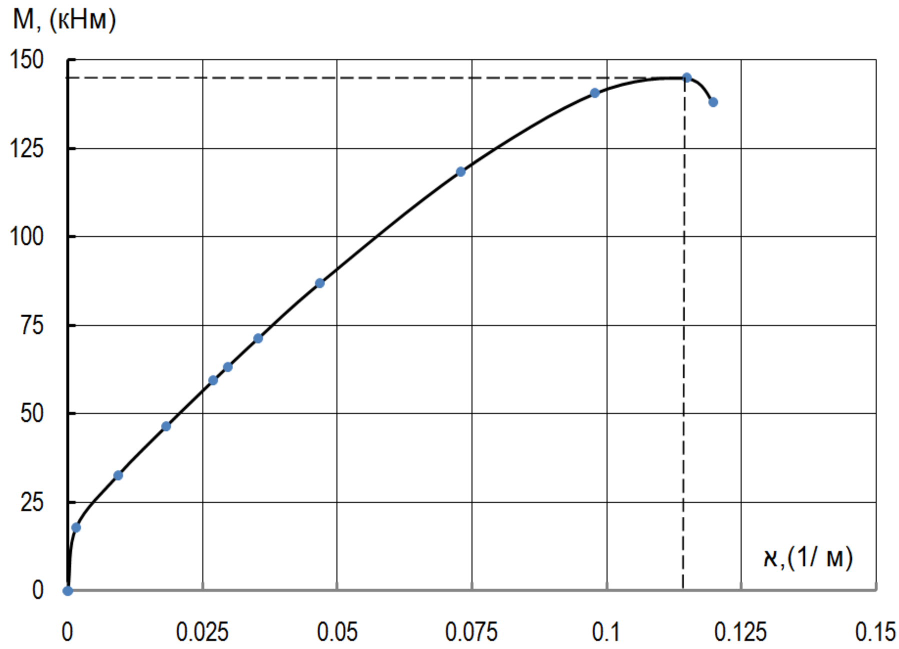


Figure 12 - The "moment-curvature" graph when calculating the P60.38 plate without steel fiber

Table 3 - Results of calculation of bearing capacity of P60.38 plate without steel fiber

Point	$\varepsilon_{c(1)}$	$\varepsilon_{c(2)}$	$\aleph$ 1/m	γ	x_1 m	$\overline{\aleph}$	σ_{sp2} MPa	σ_{sp1} MPa	M MNm
1	0.0001	$-1.124\cdot10^{-4}$	$1.517\cdot10^{-3}$	$5.917\cdot10^{-2}$	$6.592\cdot10^{-2}$	$8.977\cdot10^{-1}$	$-1.811\cdot10^{2}$	$-2.013\cdot10^{2}$	$1.788\cdot10^{-2}$
2	0.0003	$-1.011\cdot10^{-3}$	$9.365\cdot10^{-3}$	$5.917\cdot10^{-2}$	$3.204\cdot10^{-2}$	5.541	$-1.953\cdot10^{2}$	$-3.198\cdot10^{2}$	$3.273\cdot10^{-2}$
3	0.0005	$-2.063\cdot10^{-3}$	$1.83\cdot10^{-2}$	$2.959\cdot10^{-1}$	$2.732\cdot10^{-2}$	$1.083\cdot10$	$-2.167\cdot10^{2}$	$-4.602\cdot10^{2}$	$4.656\cdot10^{-2}$
4	0.0007	$-3.08\cdot10^{-3}$	$2.7\cdot10^{-2}$	$4.142\cdot10^{-1}$	$2.592\cdot10^{-2}$	$1.598\cdot10$	$-2.366\cdot10^{2}$	$-5.957\cdot10^{2}$	$5.946\cdot10^{-2}$
5	0.0007644	$-3.395\cdot10^{-3}$	$2.971\cdot10^{-2}$	$4.523\cdot10^{-1}$	$2.573\cdot10^{-2}$	$1.758\cdot10$	$-2.423\cdot10^{2}$	$-6.375\cdot10^{2}$	$6.338\cdot10^{-2}$
6	0.0009	$-4.036\cdot10^{-3}$	$3.526\cdot10^{-2}$	$5.325\cdot10^{-1}$	$2.553\cdot10^{-2}$	$2.086\cdot10$	$-2.535\cdot10^{2}$	$-6.397\cdot10^{2}$	$7.126\cdot10^{-2}$
7	0.0012	$-5.348\cdot10^{-3}$	$4.677\cdot10^{-2}$	$7.101\cdot10^{-1}$	$2.566\cdot10^{-2}$	$2.767\cdot10$	$-2.73\cdot10^{2}$	$-6.441\cdot10^{2}$	$8.694\cdot10^{-2}$
8	0.002	$-8.198\cdot10^{-3}$	$7.284\cdot10^{-2}$	1.183	$2.746\cdot10^{-2}$	$4.31\cdot10$	$-2.944\cdot10^{2}$	$-6.535\cdot10^{2}$	$1.184\cdot10^{-1}$
9	0.003	$-1.069\cdot10^{-2}$	$9.777\cdot10^{-2}$	1.775	$3.068\cdot10^{-2}$	$5.785\cdot10$	$-2.702\cdot10^{2}$	$-6.614\cdot10^{2}$	$1.406\cdot10^{-1}$
10	0.004	$-1.207\cdot10^{-2}$	$1.148\cdot10^{-1}$	2.367	$3.484\cdot10^{-2}$	$6.794\cdot10$	$-1.935\cdot10^{2}$	$-6.652\cdot10^{2}$	$1.449\cdot10^{-1}$
11	0.0045	$-1.225\cdot10^{-2}$	$1.197\cdot10^{-1}$	2.663	$3.761\cdot10^{-2}$	$7.08\cdot10$	$-1.307\cdot10^{2}$	$-6.653\cdot10^{2}$	$1.382\cdot10^{-1}$

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4.3.2. Calculation of the bearing capacity of the P60.38 plate with steel fiber

Cross-sectional dimensions b=3750 mm, h=140 mm (Fig. 11).

Pre-tensioned reinforcement in two levels of 12Ø10At-V: $A_{sp1}=A_{sp2}$=942 мм^2; E_{sp}=190000 MPa; f_{pk}=840 MPa; $f_{p0,1k}$=765 MPa; ε_{uk}=0,018. Elongation of reinforcement, taking into account all losses, is ε_{sp0}= -0,002. Distance from the upper compressed face of the section to the center of gravity of the reinforcing bars z_{sp1}=105 mm, z_{sp2}=35 mm. The step of reinforcement along the axis perpendicular to the one that is calculated S=350 mm.

Class of concrete C20/25: f_{cd}=14,5 MPa, E_{cd}=23000 MPa ε_{c1}=0,00165, ε_{cu}=0,00344.

Steel fiber STAFIB 50/1.0: f_f=1000 MPa; l_f=50 mm; d_f=1 mm; μ_{fv}=0,01.

The calculated compressive strength of steel fiber concrete is determined according to DSTU [4]:

We define the ratio (h/l_f)=140/50=2,8 and (b/l_f)=2000/50=40 and determine the coefficient k_n=0,534.

We determine the coefficient of efficiency of indirect fiber reinforcement:

$$\varphi_f = \frac{5+L}{1+4{,}5L} = \frac{5+0{,}1967}{1+4{,}5\cdot 0{,}1967} = 2{,}767,$$

$$L = \frac{k_n^2 \cdot \mu_{fv} \cdot f_f}{f_{cd}} = \frac{0{,}534^2 \cdot 0{,}01 \cdot 1000}{14{,}5} = 0{,}1967.$$

$$f_{cf} = f_{cd} + \left(k_n^2 \cdot \varphi_f \cdot \mu_{fv} \cdot f_f\right) = 14{,}5 + \left(0{,}534^2 \cdot 2{,}767 \cdot 0{,}01 \cdot 1000\right) = \\ = 22{,}36 \quad MPa.$$

Coefficient

$$K_T = \sqrt{1-\left(1{,}2-80\mu_{fv}\right)^2} = \sqrt{1-(1{,}2-80\cdot 0{,}01)^2} = 0{,}9165.$$

$$f_{cft} = 1{,}1 f_{cd}\left(K_T \frac{k_{or}^2 \cdot \mu_{fv} \cdot l_f}{0{,}8\cdot \eta_f \cdot d_{f,red}} + 0{,}08 - 0{,}5\mu_{fv}\right) =$$

$$= 1{,}1\cdot 14{,}5\left(0{,}9165 \frac{0{,}534^2 \cdot 0{,}01 \cdot 50}{0{,}8\cdot 0{,}9\cdot 1} + 0{,}08 - 0{,}5\cdot 0{,}01\right) = 1{,}49 \quad MPa.$$

The modulus of elasticity of SFB is determined according to DSTU [4]:

$$E_{cf} = E_{cd}\left(1-\mu_{fv}\right) + E_f \mu_{fv} = 25000(1-0{,}01) + 190000\cdot 0{,}01 = \\ = 24940 \quad MPa.$$

Polynomial coefficients:

a_1	a_2	a_3	a_4	a_5
2,51816	-2,14804	0,71003	-0,04839	-0,03169

Relative deformations of SFC in compression $\varepsilon_{cf1} = 0{,}00176$;

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$\varepsilon_{cftu} = 0{,}00293.$

Relative deformations of SFB in tension $\varepsilon_{cft1} = 0{,}00018$; $\varepsilon_{cftu} = 0{,}00035$.

We perform the calculation according to the block diagram. The calculation results are summarized in Table 4. The "moment-curvature" graph is shown in Fig. 13. The bearing capacity of the plate is M_u=*180,3 kNm*.

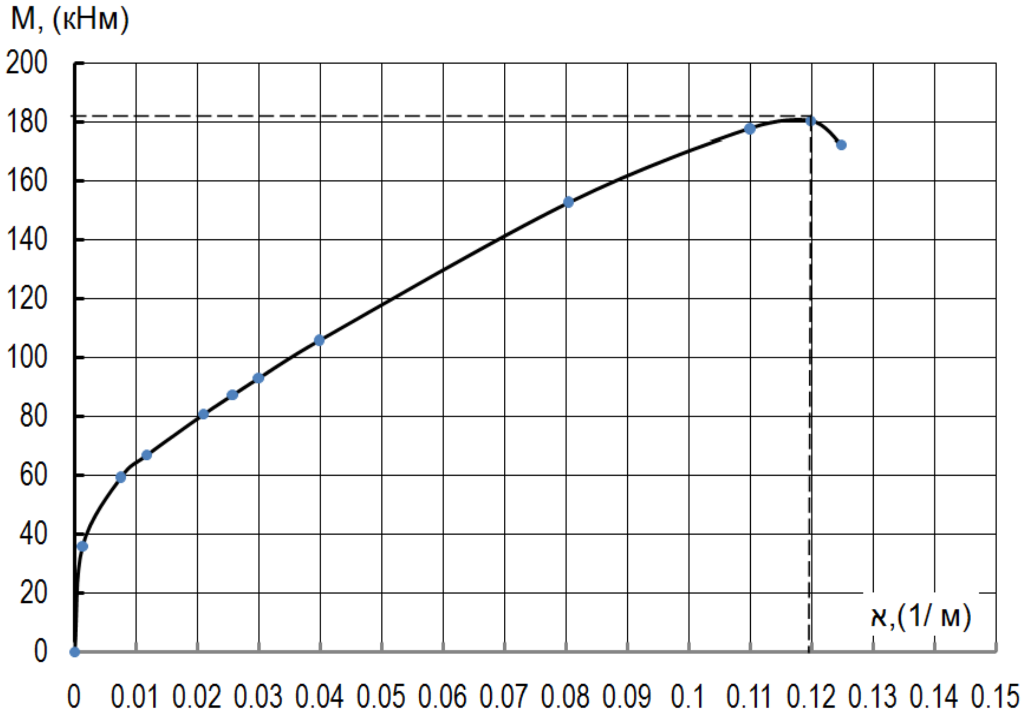


Figure 13 - The "moment-curvature" graph when calculating the P60.38 plate with steel fiber

As a result of the comparative calculation, it was established that the bearing capacity of the P60.38 plate with metal fiber was M_u=180,3 kNm, which is more than the bearing capacity of a standard slab (M_u=144,9 kNm) by 24.4%.

The efficiency of the plate with steel fiber is that the steel fiber makes it possible to reduce the amount of high-strength pre-stressed reinforcement from 24Ø10At-V to 16Ø10At-V. At the same time, the bearing capacity of a plate with steel fiber is much higher. Also, the amount of pre-stressed reinforcement in the transverse direction is reduced by 15...20 %.

Due to the good anti-abrasion properties of steel-reinforced concrete, their service life is much longer than that of reinforced concrete.

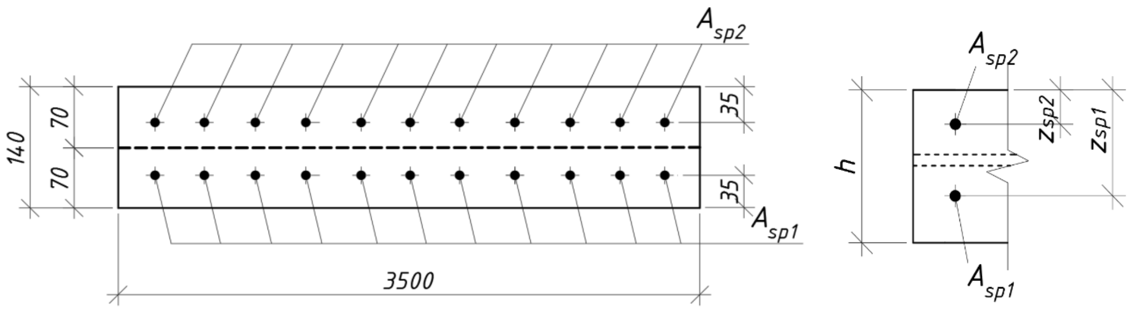
Table 4 - Results of calculation of bearing capacity of P60.38 plate with steel fiber

Point	$\varepsilon_{c(1)}$	$\varepsilon_{c(2)}$	$\aleph$, 1/m	γ	x_1, m	$\overline{\aleph}$	σ_{sp1}, MPa	σ_{sp2}, MPa	M, MNm
1	0.0001	$-7.816\cdot10^{-5}$	$1.273\cdot10^{-3}$	$5.682\cdot10^{-2}$	$7.858\cdot10^{-2}$	$7.231\cdot10^{-1}$	$-1.964\cdot10^{2}$	$-1.795\cdot10^{2}$	$3.597\cdot10^{-2}$
2	0.0003	$-7.567\cdot10^{-4}$	$7.548\cdot10^{-3}$	$1.705\cdot10^{-1}$	$3.974\cdot10^{-2}$	4.289	$-2.836\cdot10^{2}$	$-1.832\cdot10^{2}$	$5.946\cdot10^{-2}$
3	0.0004	$-1.246\cdot10^{-3}$	$1.176\cdot10^{-2}$	$2.273\cdot10^{-1}$	$3.403\cdot10^{-2}$	6.679	$-3.485\cdot10^{2}$	$-1.922\cdot10^{2}$	$6.695\cdot10^{-2}$
4	0.0006	$-2.337\cdot10^{-3}$	$2.098\cdot10^{-2}$	$3.409\cdot10^{-1}$	$2.86\cdot10^{-2}$	$1.192\cdot10$	$-4.945\cdot10^{2}$	$-2.155\cdot10^{2}$	$8.079\cdot10^{-2}$
5	0.0007	$-2.902\cdot10^{-3}$	$2.573\cdot10^{-2}$	$3.977\cdot10^{-1}$	$2.721\cdot10^{-2}$	$1.462\cdot10$	$-5.703\cdot10^{2}$	$-2.281\cdot10^{2}$	$8.738\cdot10^{-2}$
6	0.0007889	$-3.403\cdot10^{-3}$	$2.994\cdot10^{-2}$	$4.482\cdot10^{-1}$	$2.635\cdot10^{-2}$	$1.701\cdot10$	$-6.375\cdot10^{2}$	$-2.392\cdot10^{2}$	$9.306\cdot10^{-2}$
7	0.001	$-4.57\cdot10^{-3}$	$3.979\cdot10^{-2}$	$5.682\cdot10^{-1}$	$2.513\cdot10^{-2}$	$2.261\cdot10$	$-6.415\cdot10^{2}$	$-2.646\cdot10^{2}$	$1.059\cdot10^{-1}$
8	0.002	$-9.251\cdot10^{-3}$	$8.037\cdot10^{-2}$	1.136	$2.489\cdot10^{-2}$	$4.566\cdot10$	$-6.574\cdot10^{2}$	$-3.444\cdot10^{2}$	$1.528\cdot10^{-1}$
9	0.003	$-1.238\cdot10^{-2}$	$1.099\cdot10^{-1}$	1.705	$2.73\cdot10^{-2}$	$6.243\cdot10$	$-6.676\cdot10^{2}$	$-3.507\cdot10^{2}$	$1.777\cdot10^{-1}$
10	0.0035	$-1.327\cdot10^{-2}$	$1.198\cdot10^{-1}$	1.989	$2.922\cdot10^{-2}$	$6.806\cdot10$	$-6.702\cdot10^{2}$	$-3.216\cdot10^{2}$	$1.803\cdot10^{-1}$
11	0.004	$-1.347\cdot10^{-2}$	$1.248\cdot10^{-1}$	2.273	$3.206\cdot10^{-2}$	$7.09\cdot10$	$-6.703\cdot10^{2}$	$-2.598\cdot10^{2}$	$1.721\cdot10^{-1}$

4.3.3. Calculation of the bearing capacity of the P60.35 plate without steel fiber

Cross-sectional dimensions b=3500 mm, h=140 mm (Fig. 14).

Pre-tensioned reinforcement in two levels of 11Ø10Aт-V: $A_{sp1}=A_{sp2}$=864 mm^2; E_{sp}=190000 MPa; f_{pk}=840 MPa; $f_{p0,1k}$=765 MPa; ε_{uk}=0,018. Elongation of reinforcement, taking into account all losses, is ε_{sp0}= -0,002. Distance from the upper compressed face of the section to the center of gravity of the reinforcing bars z_{sp1}=105 mm, z_{sp2}=35 mm. The step of reinforcement along the axis perpendicular to the one that is calculated S=350 mm.



a) б)

Figure 14 - Estimated section of plate P60.35 (a) and calculation parameters (b)

Class of concrete C25/30: f_{cd}=17,0 MPa, E_{cd}=25000 MPa ε_{c1}=0,00169, ε_{cu}=0,00328.

Polynomial coefficients:

a_1	a_2	a_3	a_4	a_5
2,7404	-2,7649	1,3416	-0,35004	0,03295

We perform the calculation according to the block diagram. The calculation results are summarized in Table 5. The "moment-curvature" graph is shown in Fig. 15. The bearing capacity of the plate is M_u= *134,4 kNm*.

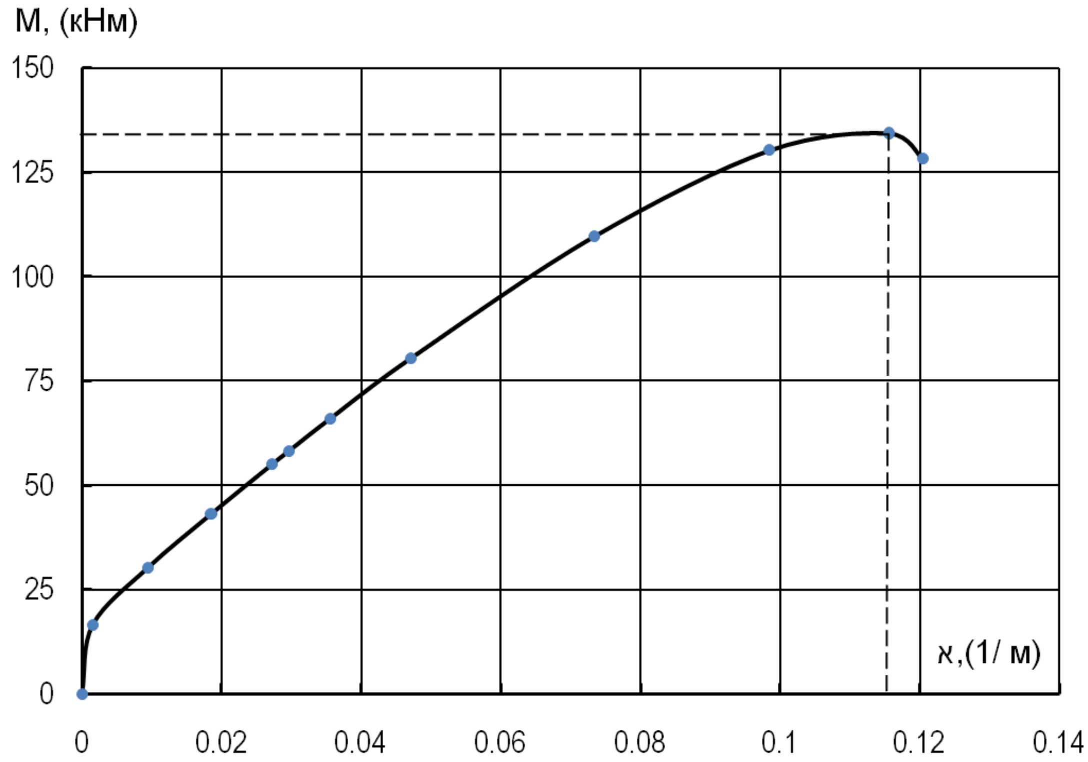


Figure 15 - The "moment-curvature" graph when calculating the P60.35 plate without steel fiber

Table 5 - Results of calculation of bearing capacity of P60.35 plate without metal fiber

Point	$\varepsilon_{c(1)}$	$\varepsilon_{c(2)}$	$\aleph$ 1/m	γ	x_1 m	$\overline{\aleph}$	σ_{sp2} MPa	σ_{sp1} MPa	M MNm
1	0.0001	$-1.158 \cdot 10^{-4}$	$1.541 \cdot 10^{-3}$	$5.917 \cdot 10^{-2}$	$6.489 \cdot 10^{-2}$	$9.119 \cdot 10^{-1}$	$-1.812 \cdot 10^{2}$	$-2.017 \cdot 10^{2}$	$1.655 \cdot 10^{-2}$
2	0.0003	$-1.027 \cdot 10^{-3}$	$9.475 \cdot 10^{-3}$	$1.775 \cdot 10^{-1}$	$3.166 \cdot 10^{-2}$	5.607	$-1.96 \cdot 10^{2}$	$-3.22 \cdot 10^{2}$	$3.028 \cdot 10^{-2}$
3	0.0005	$-2.089 \cdot 10^{-3}$	$1.849 \cdot 10^{-2}$	$2.959 \cdot 10^{-1}$	$2.704 \cdot 10^{-2}$	$1.094 \cdot 10$	$-2.18 \cdot 10^{2}$	$-4.639 \cdot 10^{2}$	$4.309 \cdot 10^{-2}$
4	0.0007	$2.726 \cdot 10^{-2}$	$2.726 \cdot 10^{-2}$	$4.142 \cdot 10^{-1}$	$2.568 \cdot 10^{-2}$	$1.613 \cdot 10$	$-2.383 \cdot 10^{2}$	$-6.008 \cdot 10^{2}$	$5.505 \cdot 10^{-2}$
5	0.0007644	$-3.393 \cdot 10^{-3}$	$2.963 \cdot 10^{-2}$	$4.473 \cdot 10^{-1}$	$2.551 \cdot 10^{-2}$	$1.753 \cdot 10$	$-2.434 \cdot 10^{2}$	$-6.375 \cdot 10^{2}$	$5.822 \cdot 10^{-2}$
6	0.0009	$-4.08 \cdot 10^{-3}$	$3.557 \cdot 10^{-2}$	$5.325 \cdot 10^{-1}$	$2.53 \cdot 10^{-2}$	$2.105 \cdot 10$	$-2.556 \cdot 10^{2}$	$-6.398 \cdot 10^{2}$	$6.6 \cdot 10^{-2}$
7	0.0012	$-5.403 \cdot 10^{-3}$	$4.717 \cdot 10^{-2}$	$7.101 \cdot 10^{-1}$	$2.544 \cdot 10^{-2}$	$2.791 \cdot 10$	$-2.756 \cdot 10^{2}$	$-6.443 \cdot 10^{2}$	$8.053 \cdot 10^{-2}$
8	0.002	$-8.278 \cdot 10^{-3}$	$7.341 \cdot 10^{-2}$	1.183	$2.724 \cdot 10^{-2}$	$4.344 \cdot 10$	$-2.982 \cdot 10^{2}$	$-6.538 \cdot 10^{2}$	$1.097 \cdot 10^{-1}$
9	0.003	$-1.079 \cdot 10^{-2}$	$9.848 \cdot 10^{-2}$	1.775	$3.046 \cdot 10^{-2}$	$5.827 \cdot 10$	$-2.749 \cdot 10^{2}$	$-6.618 \cdot 10^{2}$	$1.303 \cdot 10^{-1}$
10	0.004	$-1.218 \cdot 10^{-2}$	$1.156 \cdot 10^{-1}$	2.367	$3.46 \cdot 10^{-2}$	$6.84 \cdot 10$	$-1.987 \cdot 10^{2}$	$-6.656 \cdot 10^{2}$	$1.344 \cdot 10^{-1}$
11	0.0045	$-1.236 \cdot 10^{-2}$	$1.204 \cdot 10^{-1}$	2.663	$3.737 \cdot 10^{-2}$	$7.126 \cdot 10$	$-1.358 \cdot 10^{2}$	$-6.657 \cdot 10^{2}$	$1.282 \cdot 10^{-1}$

4.3.4. Calculation of the bearing capacity of the P60.35 plate with steel fiber

Cross-sectional dimensions b=3750 mm, h=140 mm (Fig. 11).

Pre-tensioned reinforcement in two levels of 7Ø10Ат-V: $A_{sp1}=A_{sp2}$=550 mm^2; E_{sp}=190000 MPa; f_{pk}=840 MPa; $f_{p0,1k}$=765 MPa; ε_{uk}=0,018. Elongation of reinforcement, taking into account all losses, is ε_{sp0}= -0,002. Distance from the upper compressed face of the section to the center of gravity of the reinforcing bars z_{sp1}=105 mm, z_{sp2}=35 mm. The step of reinforcement along the axis perpendicular to the one that is calculated S=350 mm.

Class of concrete C20/25: f_{cd}=14,5 MPa, E_{cd}=23000 MPa ε_{c1}=0,00165, ε_{cu}=0,00344.

Steel fiber STAFIB 50/1.0: f_f=1000 MPa; l_f=50 mm; d_f=1 mm; μ_{fv}=0,01.

The calculated compressive strength of steel fiber concrete is determined according to DSTU [4]:

We define the ratio (h/l_f)=140/50=2,8 and (b/l_f)=2000/50=40 and determine the coefficient k_n=0,534.

We determine the coefficient of efficiency of indirect fiber reinforcement:

$$\varphi_f = \frac{5+L}{1+4{,}5L} = \frac{5+0{,}1967}{1+4{,}5\cdot 0{,}1967} = 2{,}767,$$

$$L = \frac{k_n^2 \cdot \mu_{fv} \cdot f_f}{f_{cd}} = \frac{0{,}534^2 \cdot 0{,}01 \cdot 1000}{14{,}5} = 0{,}1967.$$

$$f_{cf} = f_{cd} + \left(k_n^2 \cdot \varphi_f \cdot \mu_{fv} \cdot f_f\right) = 14{,}5 + \left(0{,}534^2 \cdot 2{,}767 \cdot 0{,}01 \cdot 1000\right) = $$
$$= 22{,}36 \quad MPa.$$

Coefficient

$$K_T = \sqrt{1-\left(1{,}2-80\mu_{fv}\right)^2} = \sqrt{1-(1{,}2-80\cdot 0{,}01)^2} = 0{,}9165.$$

$$f_{cft} = 1{,}1 f_{cd}\left(K_T \frac{k_{or}^2 \cdot \mu_{fv} \cdot l_f}{0{,}8\cdot \eta_f \cdot d_{f,red}} + 0{,}08 - 0{,}5\mu_{fv}\right) =$$
$$= 1{,}1\cdot 14{,}5\left(0{,}9165\frac{0{,}534^2 \cdot 0{,}01\cdot 50}{0{,}8\cdot 0{,}9\cdot 1} + 0{,}08 - 0{,}5\cdot 0{,}01\right) = 1{,}49 \quad MPa.$$

The modulus of elasticity of SFB is determined according to DSTU [4]:

$$E_{cf} = E_{cd}\left(1-\mu_{fv}\right) + E_f\mu_{fv} = 25000(1-0{,}01) + 190000\cdot 0{,}01 =$$
$$= 24940 \quad MPa.$$

Polynomial coefficients:

a_1	a_2	a_3	a_4	a_5
2,51816	-2,14804	0,71003	-0,04839	-0,03169

Relative deformations of SFC in compression $\varepsilon_{cf1} = 0{,}00176$;

$\varepsilon_{cftu} = 0{,}00293$.

Relative deformations of SFB in tension $\varepsilon_{cft1} = 0{,}00018$; $\varepsilon_{cftu} = 0{,}00035$.

We perform the calculation according to the block diagram. The calculation results are summarized in Table 6. The "moment-curvature" graph is shown in Fig. 16. The bearing capacity of the plate is M_u=*152,2 kNm*.

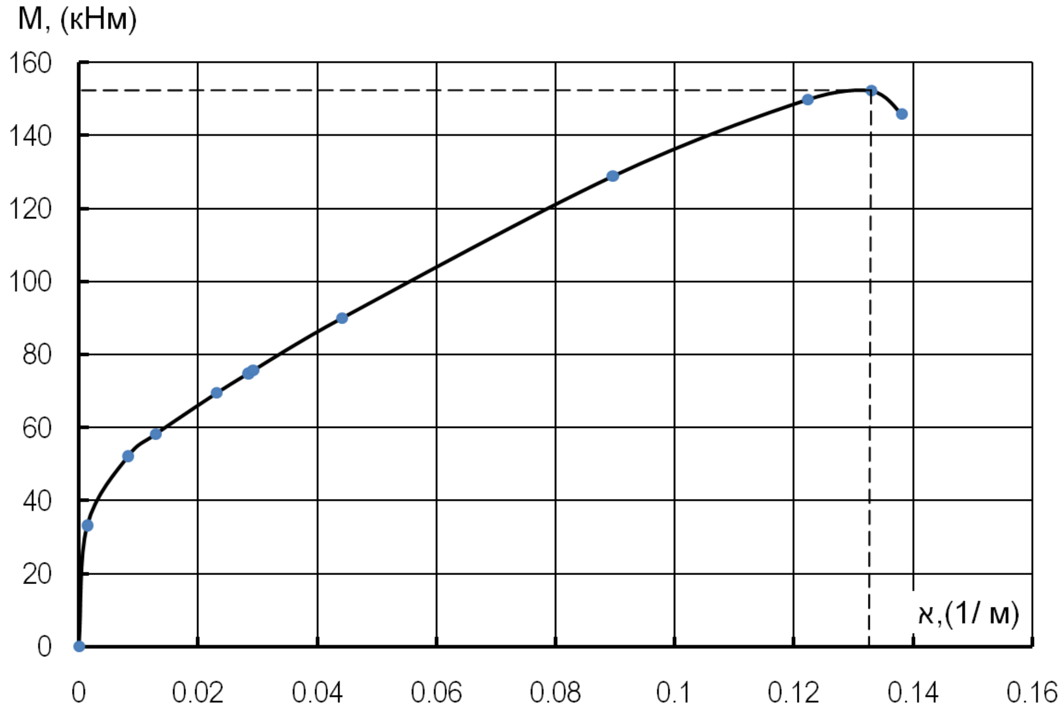


Figure 16 - The "moment-curvature" graph when calculating the P60.35 plate with steel fiber

As a result of the comparative calculation, it was established that the bearing capacity of the P60.35 plate with steel fiber was M_u=152,2 kNm, which is more than the bearing capacity of a standard slab (M_u=134,4 kNm) by 13,2%.

The efficiency of the plate with steel fiber is that the steel fiber makes it possible to reduce the amount of high-strength pre-stressed reinforcement from 22Ø10At-V to 14Ø10At-V. At the same time, the bearing capacity of a plate with steel fiber is much higher. Also, the amount of pre-stressed reinforcement in the transverse direction is reduced by 15...20 %.

Due to the good anti-abrasion properties of steel-reinforced concrete, their service life is much longer than that of reinforced concrete.

Table 6 - Results of calculation of bearing capacity of P60.35 plate with steel fiber

Point	$\varepsilon_{c(1)}$	$\varepsilon_{c(2)}$	$\aleph$ 1/m	γ	x_1 m	$\bar{\aleph}$	σ_{sp1} MPa	σ_{sp2} MPa	M MNm
1	0.0001	$-9.426\cdot10^{-5}$	$1.388\cdot10^{-3}$	$5.682\cdot10^{-2}$	$7.207\cdot10^{-2}$	$7.884\cdot10^{-1}$	$-1.987\cdot10^{2}$	$-1.802\cdot10^{2}$	$3.316\cdot10^{-2}$
2	0.0003	$-8.558\cdot10^{-4}$	$8.256\cdot10^{-3}$	$1.705\cdot10^{-1}$	$3.634\cdot10^{-2}$	4.691	$-2.977\cdot10^{2}$	$-1.879\cdot10^{2}$	$5.217\cdot10^{-2}$
3	0.0004	$-1.406\cdot10^{-3}$	$1.29\cdot10^{-2}$	$2.273\cdot10^{-1}$	$3.102\cdot10^{-2}$	7.328	$-3.713\cdot10^{2}$	$-1.998\cdot10^{2}$	$5.824\cdot10^{-2}$
4	0.0006	$-2.64\cdot10^{-3}$	$2.314\cdot10^{-2}$	$3.409\cdot10^{-1}$	$2.593\cdot10^{-2}$	$1.315\cdot10$	$-5.376\cdot10^{2}$	$-2.299\cdot10^{2}$	$6.948\cdot10^{-2}$
5	0.0007	$-3.281\cdot10^{-3}$	$2.844\cdot10^{-2}$	$3.977\cdot10^{-1}$	$2.462\cdot10^{-2}$	$1.616\cdot10$	$-6.243\cdot10^{2}$	$-2.461\cdot10^{2}$	$7.485\cdot10^{-2}$
6	0.0007889	$-3.379\cdot10^{-3}$	$2.924\cdot10^{-2}$	$4.064\cdot10^{-1}$	$2.446\cdot10^{-2}$	$1.662\cdot10$	$-6.375\cdot10^{2}$	$-2.486\cdot10^{2}$	$7.565\cdot10^{-2}$
7	0.001	$-5.184\cdot10^{-3}$	$4.417\cdot10^{-2}$	$5.682\cdot10^{-1}$	$2.264\cdot10^{-2}$	$2.51\cdot10$	$-6.437\cdot10^{2}$	$-2.937\cdot10^{2}$	$8.998\cdot10^{-2}$
8	0.002	$-1.054\cdot10^{-2}$	$8.958\cdot10^{-2}$	1.136	$2.233\cdot10^{-2}$	$5.09\cdot10$	$-6.621\cdot10^{2}$	$-4.057\cdot10^{2}$	$1.288\cdot10^{-1}$
9	0.003	$-1.412\cdot10^{-2}$	$1.223\cdot10^{-1}$	1.705	$2.453\cdot10^{-2}$	$6.948\cdot10$	$-6.739\cdot10^{2}$	$-4.332\cdot10^{2}$	$1.498\cdot10^{-1}$
10	0.0035	$-1.512\cdot10^{-2}$	$1.33\cdot10^{-1}$	1.989	$2.631\cdot10^{-2}$	$7.559\cdot10$	$-6.77\cdot10^{2}$	$-4.097\cdot10^{2}$	$1.522\cdot10^{-1}$
11	0.004	$-1.533\cdot10^{-2}$	$1.381\cdot10^{-1}$	2.273	$2.897\cdot10^{-2}$	$7.845\cdot10$	$-6.771\cdot10^{2}$	$-3.482\cdot10$	$1.459\cdot10^{-1}$

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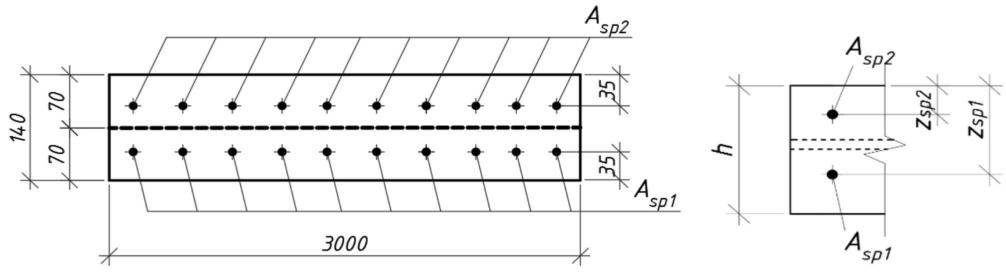
4.3.5. Calculation of the bearing capacity of the P60.30 plate without steel fiber

Cross-sectional dimensions b=3000 mm, h=140 mm (Fig. 17).

Pre-tensioned reinforcement in two levels of 10Ø10Aт-V: $A_{sp1}=A_{sp2}$=786 mm^2; E_{sp}=190000 MPa; f_{pk}=840 MPa; $f_{p0,1k}$=765 MPa; ε_{uk}=0,018. Distance from the upper compressed face of the section to the center of gravity of the reinforcing bars z_{sp1}=105 mm, z_{sp2}=35 mm. The step of reinforcement along the axis perpendicular to the one that is calculated S=350 mm.



a) б)

Figure 17 - Estimated section of plate P60.30 (a) and calculation parameters (b)

Class of concrete C25/30: f_{cd}=17,0 MPa, E_{cd}=25000 MPa ε_{c1}=0,00169, ε_{cu}=0,00328.

Polynomial coefficients:

a_1	a_2	a_3	a_4	a_5
2,7404	-2,7649	1,3416	-0,35004	0,03295

We perform the calculation according to the block diagram. The calculation results are summarized in Table 7. The "moment-curvature" graph is shown in Fig. 18. The bearing capacity of the plate is M_u=*117,8 kNm*.

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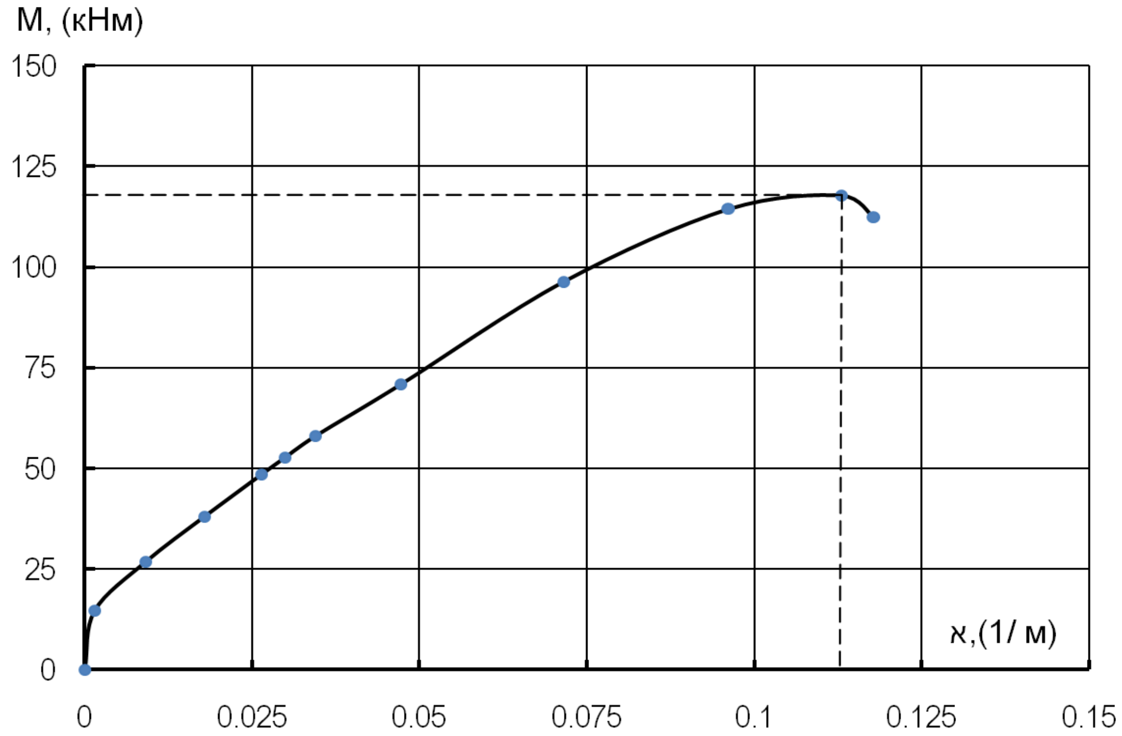

Figure 18 - The "moment-curvature" graph when calculating the P60.30 plate without steel fiber

4.3.6. Calculation of the bearing capacity of the P60.30 plate with steel fiber

Cross-sectional dimensions b=3000 mm, h=140 mm (Fig. 17).

Pre-tensioned reinforcement in two levels of 6Ø10Aт-V: $A_{sp1}=A_{sp2}=471$ mm^2; E_{sp}=190000 MPa; f_{pk}=840 MPa; $f_{p0,1k}$=765 MPa; ε_{uk}=0,018. Elongation of reinforcement, taking into account all losses, is ε_{sp0}= -0,002. Distance from the upper compressed face of the section to the center of gravity of the reinforcing bars z_{sp1}=105 mm, z_{sp2}=35 mm. The step of reinforcement along the axis perpendicular to the one that is calculated S=350 mm.

Class of concrete C20/25: f_{cd}=14,5 MPa, E_{cd}=23000 MPa ε_{c1}=0,00165, ε_{cu}=0,00344.

Steel fiber STAFIB 50/1.0: f_f=1000 MPa; l_f=50 mm; d_f=1 mm; μ_{fv}=0,01.

The calculated compressive strength of steel fiber concrete is determined according to DSTU [4]:

Table 7 - Results of calculation of bearing capacity of P60.30 plate without metal fiber

Point	$\varepsilon_{c(1)}$	$\varepsilon_{c(2)}$	$\aleph$ 1/m	γ	x_1 m	$\overline{\aleph}$	σ_{sp2} MPa	σ_{sp1} MPa	M MNm
1	0.0001	$-1.044\cdot10^{-4}$	$1.46\cdot10^{-3}$	$5.917\cdot10^{-2}$	$6.848\cdot10^{-2}$	$8.641\cdot10^{-1}$	$-1.807\cdot10^{2}$	$-2.001\cdot10^{2}$	$1.459\cdot10^{-2}$
2	0.0003	$-9.743\cdot10^{-4}$	$9.102\cdot10^{-3}$	$1.775\cdot10^{-1}$	$3.296\cdot10^{-2}$	5.386	$-1.935\cdot10^{2}$	$-3.146\cdot10^{2}$	$2.677\cdot10^{-2}$
3	0.0005	$-2.001\cdot10^{-3}$	$1.786\cdot10^{-2}$	$2.959\cdot10^{-1}$	$2.799\cdot10^{-2}$	$1.057\cdot10$	$-2.138\cdot10^{2}$	$-4.513\cdot10^{2}$	$3.8\cdot10^{-2}$
4	0.0007	$-2.996\cdot10^{-3}$	$2.64\cdot10^{-2}$	$4.142\cdot10^{-1}$	$2.652\cdot10^{-2}$	$1.562\cdot10$	$-2.326\cdot10^{2}$	$-5.837\cdot10^{2}$	$4.849\cdot10^{-2}$
5	0.0007852	$-3.402\cdot10^{-3}$	$2.991\cdot10^{-2}$	$4.646\cdot10^{-1}$	$2.625\cdot10^{-2}$	$1.77\cdot10$	$-2.397\cdot10^{2}$	$-6.375\cdot10^{2}$	$5.269\cdot10^{-2}$
6	0.0009	$-3.931\cdot10^{-3}$	$3.451\cdot10^{-2}$	$5.325\cdot10^{-1}$	$2.608\cdot10^{-2}$	$2.042\cdot10$	$-2.485\cdot10^{2}$	$-6.393\cdot10^{2}$	$5.808\cdot10^{-2}$
7	0.0012	$-5.216\cdot10^{-3}$	$4.583\cdot10^{-2}$	$7.101\cdot10^{-1}$	$2.619\cdot10^{-2}$	$2.712\cdot10$	$-2.667\cdot10^{2}$	$-6.436\cdot10^{2}$	$7.083\cdot10^{-2}$
8	0.002	$-8.009\cdot10^{-3}$	$7.149\cdot10^{-2}$	1.183	$2.798\cdot10^{-2}$	$4.23\cdot10$	$-2.854\cdot10^{2}$	$-6.528\cdot10^{2}$	$9.636\cdot10^{-2}$
9	0.003	$-1.045\cdot10^{-2}$	$9.609\cdot10^{-2}$	1.775	$3.122\cdot10^{-2}$	$5.686\cdot10$	$-2.59\cdot10^{2}$	$-6.605\cdot10^{2}$	$1.144\cdot10^{-1}$
10	0.004	$-1.182\cdot10^{-2}$	$1.13\cdot10^{-1}$	2.367	$3.541\cdot10^{-2}$	$6.685\cdot10$	$-1.813\cdot10^{2}$	$-6.643\cdot10^{2}$	$1.178\cdot10^{-1}$
11	0.0045	$-1.2\cdot10^{-2}$	$1.178\cdot10^{-1}$	2.663	$3.819\cdot10^{-2}$	$6.972\cdot10$	$-1.185\cdot10^{2}$	$-6.643\cdot10^{2}$	$1.123\cdot10^{-1}$

We define the ratio (h/l_f)=140/50=2,8 and (b/l_f)=2000/50=40 and determine the coefficient k_n=0,534.

We determine the coefficient of efficiency of indirect fiber reinforcement:

$$\varphi_f = \frac{5+L}{1+4{,}5L} = \frac{5+0{,}1967}{1+4{,}5\cdot 0{,}1967} = 2{,}767,$$

$$L = \frac{k_n^2 \cdot \mu_{fv} \cdot f_f}{f_{cd}} = \frac{0{,}534^2 \cdot 0{,}01 \cdot 1000}{14{,}5} = 0{,}1967.$$

$$f_{cf} = f_{cd} + \left(k_n^2 \cdot \varphi_f \cdot \mu_{fv} \cdot f_f\right) = 14{,}5 + \left(0{,}534^2 \cdot 2{,}767 \cdot 0{,}01 \cdot 1000\right) = \\ = 22{,}36 \quad MPa.$$

Coefficient

$$K_T = \sqrt{1-\left(1{,}2-80\mu_{fv}\right)^2} = \sqrt{1-(1{,}2-80\cdot 0{,}01)^2} = 0{,}9165.$$

$$f_{cft} = 1{,}1 f_{cd}\left(K_T \frac{k_{or}^2 \cdot \mu_{fv} \cdot l_f}{0{,}8 \cdot \eta_f \cdot d_{f,red}} + 0{,}08 - 0{,}5\mu_{fv}\right) =$$

$$= 1{,}1 \cdot 14{,}5\left(0{,}9165 \frac{0{,}534^2 \cdot 0{,}01 \cdot 50}{0{,}8 \cdot 0{,}9 \cdot 1} + 0{,}08 - 0{,}5 \cdot 0{,}01\right) = 1{,}49 \quad MPa.$$

The modulus of elasticity of SFB is determined according to DSTU [4]:

$$E_{cf} = E_{cd}\left(1-\mu_{fv}\right) + E_f\mu_{fv} = 25000(1-0{,}01) + 190000 \cdot 0{,}01 = \\ = 24940 \quad MPa.$$

Polynomial coefficients:

a_1	a_2	a_3	a_4	a_5
2,51816	-2,14804	0,71003	-0,04839	-0,03169

Relative deformations of SFC in compression $\varepsilon_{cf1} = 0{,}00176$;

$\varepsilon_{cftu} = 0{,}00293$.

Relative deformations of SFB in tension $\varepsilon_{cft1} = 0{,}00018$; $\varepsilon_{cftu} = 0{,}00035$.

We perform the calculation according to the block diagram. The calculation results are summarized in Table 8. The "moment-curvature" graph is shown in Fig. 19. The bearing capacity of the plate is M_u=*130,5 kNm.*

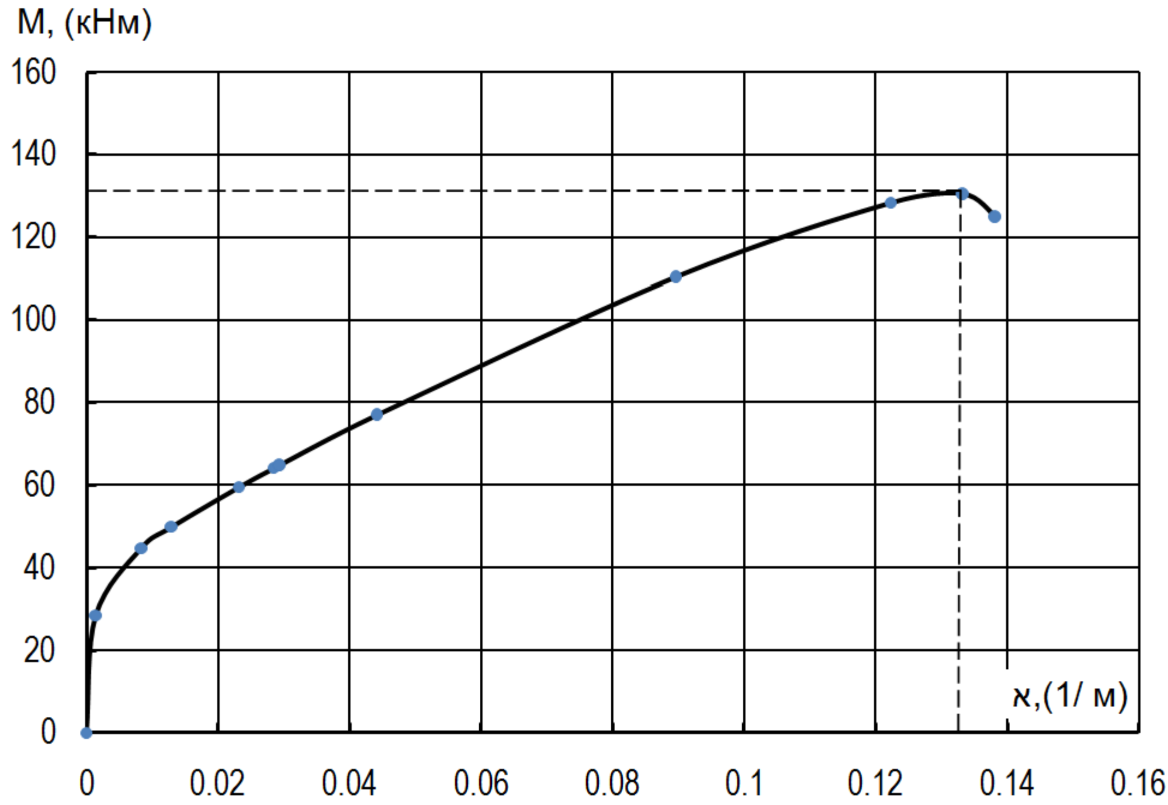

Figure 19 - The "moment-curvature" graph when calculating the P60.30 plate with steel fiber

As a result of the comparative calculation, it was established that the bearing capacity of the P60.30 plate with steel fiber was M_u=130,5 kNm, which is more than the bearing capacity of a standard slab (M_u=117,8 kNm) by 10,8 %.

The efficiency of the plate with steel fiber is that the steel fiber makes it possible to reduce the amount of high-strength pre-stressed reinforcement from 20Ø10At-V to 12Ø10At-V. At the same time, the bearing capacity of a plate with steel fiber is much higher. Also, the amount of pre-stressed reinforcement in the transverse direction is reduced by 15...20 %.

Due to the good anti-abrasion properties of steel-reinforced concrete, their service life is much longer than that of reinforced concrete.

Table 8 - Results of calculation of bearing capacity of P60.30 plate with steel fiber

Point	$\varepsilon_{c(1)}$	$\varepsilon_{c(2)}$	$\aleph$ 1/m	γ	x_1 m	$\overline{\aleph}$	σ_{sp1} MPa	σ_{sp2} MPa	M MNm
1	0.0001	$-9.431\cdot10^{-5}$	$1.388\cdot10^{-3}$	$5.682\cdot10^{-2}$	$7.205\cdot10^{-2}$	$7.886\cdot10^{-1}$	$-1.987\cdot10^{2}$	$-1.802\cdot10^{2}$	$2.842\cdot10^{-2}$
2	0.0003	$-8.561\cdot10^{-4}$	$8.258\cdot10^{-3}$	$1.705\cdot10^{-1}$	$3.633\cdot10^{-2}$	4.692	$-2.977\cdot10^{2}$	$-1.879\cdot10^{2}$	$4.471\cdot10^{-2}$
3	0.0004	$-1.406\cdot10^{-3}$	$1.29\cdot10^{-2}$	$2.273\cdot10^{-1}$	$3.101\cdot10^{-2}$	7.33	$-3.714\cdot10^{2}$	$-1.998\cdot10^{2}$	$4.991\cdot10^{-2}$
4	0.0006	$-2.641\cdot10^{-3}$	$2.315\cdot10^{-2}$	$3.409\cdot10^{-1}$	$2.592\cdot10^{-2}$	$1.315\cdot10$	$-5.378\cdot10^{2}$	$-2.299\cdot10^{2}$	$5.954\cdot10^{-2}$
5	0.0007	$-3.282\cdot10^{-3}$	$2.845\cdot10^{-2}$	$3.977\cdot10^{-1}$	$2.461\cdot10^{-2}$	$1.616\cdot10$	$-6.245\cdot10^{2}$	$-2.462\cdot10^{2}$	$6.414\cdot10^{-2}$
6	0.000715	$-3.379\cdot10^{-3}$	$2.924\cdot10^{-2}$	$4.063\cdot10^{-1}$	$2.445\cdot10^{-2}$	$1.661\cdot10$	$-6.375\cdot10^{2}$	$-2.486\cdot10^{2}$	$6.482\cdot10^{-2}$
7	0.001	$-5.186\cdot10^{-3}$	$4.418\cdot10^{-2}$	$5.682\cdot10^{-1}$	$2.263\cdot10^{-2}$	$2.51\cdot10$	$-6.437\cdot10^{2}$	$-2.938\cdot10^{2}$	$7.711\cdot10^{-2}$
8	0.002	$-1.055\cdot10^{-2}$	$8.961\cdot10^{-2}$	1.136	$2.232\cdot10^{-2}$	$5.092\cdot10$	$-6.621\cdot10^{2}$	$-4.059\cdot10^{2}$	$1.104\cdot10^{-1}$
9	0.003	$-1.413\cdot10^{-2}$	$1.223\cdot10^{-1}$	1.705	$2.452\cdot10^{-2}$	$6.951\cdot10$	$-6.739\cdot10^{2}$	$-4.335\cdot10^{2}$	$1.283\cdot10^{-1}$
10	0.0035	$-1.513\cdot10^{-2}$	$1.331\cdot10^{-1}$	1.989	$2.63\cdot10^{-2}$	$7.561\cdot10$	$-6.77\cdot10^{2}$	$-4.1\cdot10^{2}$	$1.305\cdot10^{-1}$
11	0.004	$-1.534\cdot10^{-2}$	$1.381\cdot10^{-1}$	2.273	$2.896\cdot10^{-2}$	$7.848\cdot10$	$-6.771\cdot10^{2}$	$-3.485\cdot10^{2}$	$1.25\cdot10^{-1}$

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Conclusions

1. The proposed general algorithm for calculating bending elements of a rectangular cross-section reinforced with conventional and pre-stressed reinforcement, as well as steel fiber.

2. The calculation method is based on the deformation theory of the calculation of reinforced concrete structures, taking into account the full "σ-ε" diagram for concrete and steel-reinforced concrete under compression.

3. The method makes it possible to calculate two-way prestressed slabs. At the same time, the increase in strength of concrete and reinforced concrete under biaxial compression conditions is taken into account.

4. As a result of a comparative calculation of the load-bearing capacity of the standard PAG-14 airfield coating board and a similar board in which the reinforcing meshes were replaced by steel fiber, it was established that the bearing capacity of the board with steel fiber is 21.3 % higher than the standard board.

5. As a result of a comparative calculation of the bearing capacity of standard road slabs P60.38, P60.35, P60.30 and similar slabs in which steel fiber was included and the amount of tension reinforcement was reduced by 30...40%, it was established that the bearing capacity of slabs with steel fiber is higher than the standard one by 10.8...24.4 %.

6. The effectiveness of plates with steel fiber is that the steel fiber almost completely replaces the reinforcing mesh. There are also no costs for the production of these nets. It is also possible to reduce the amount of high-strength pre-stressed reinforcement to 10...15 %.

7. Due to the good anti-abrasion properties of steel fiber concrete, the service life of airfield and road pavement slabs is much longer than that of reinforced concrete slabs.

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