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REINFORCED CONCRETE CRANE BEAM DESIGN

Teaching aid

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Павліков А.М., Гарькава О.В. Проектування залізобетонної підкранової балки прольотом 12 м : навчальний посібник / А.М. Павліков, О.В. Гарькава. – Полтава : ПолтНТУ, 2017. – 82 с.

The calculation and construction of reinforced concrete crane beams with a span of 12 m for general purpose cranes with a capacity of up to 32 tons are stated. The calculation of the beams is made to provide the strength and crack resistance at loads in the stages of operation, manufacture, transportation and assembling. The design of the crane beam was performed in accordance with the current normative documents for the design of reinforced concrete structures.

The teaching aid is intended for students who study in the field of "Construction" in higher education.

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CONTENT

1 DATA

To design a single span reinforced concrete I-section crane beam with 12 m span.

The beam is designed for normal type bridge crane with driver's cab control, load capacity Q=32/5 t. Crane span L=16,5 m. Mode of operation – 1К – 7К.

Crane base B = 4400 mm, crane breadth A = 5600 mm, crane mass m_c = 34 t, trolley mass m_t = 10 t.

Maximum load on the crane wheel $F_{\max}$ = 270 kN (Table A.1).

Minimum load on the crane wheel

$$F_{\min} = \frac{10}{n}\left(m_c + Q\right) - F_{\max} = \frac{10}{2}\left(34+32\right) - 270 = 60\,\text{кН},$$

where n – wheel number from one side of the crane;

Q – crane load capacity, t.

Concrete C40/50.

The physical and mechanical characteristics of concrete are given in the Table 1.1 (from Table. A.2).

Table 1.1 – Characteristics of strength and deformability concrete

Concrete class	$f_{ck,cube}$, MPa	$f_{ck,prism}$, MPa	f_{cd}, MPa	f_{ctm}, MPa	E_{cm}, GPa	E_{cd}, GPa	$\varepsilon_{c1,cd}$, ‰	$\varepsilon_{cu1,cd}$, ‰
C50/60	60	43	33	4,1	40	34	1,91	2,29

The required design parameters for concrete:

$$K = \frac{1,05 E_{cd} \varepsilon_{c1,cd}}{f_{cd}} = \frac{1,05 \cdot 34 \cdot 10^3 \cdot 1,91 \cdot 10^{-3}}{33} \approx 2 \text{ [1; 3.1.5];}$$

η_u = 1,268; ω = 0,732; χ = 0,539 (Table A.5).

Prestressed reinforcement is Bp1200 class. The physical and mechanical characteristics of the wire are given in Table 1.2 (from Table A.3).

Table 1.2 – Characteristics of strength and deformability prestressed reinforcement

Reinforcement class	f_{pk}, MPa	$f_{p0,1k}$, MPa	f_{pd}, MPa	E_p, MPa	ε_{ud}, ‰
Bp1200	1260	1145	916	190000	0,014

Unprestressed reinforcement is A400C class. The physical and mechanical characteristics of the valve are given in Table 1.3 (from Table A.4).

Table 1.3 – Characteristics of strength and deformability unprestressed reinforcement

Reinforcement class	f_{yk}, MPa	f_{yd}, MPa	f_{ywd}, MPa	E_s, MPa	ε_{ud}
A400C	400	364	285	210000	0,025

Formwork drawings of the crane beam are shown in Figure 1.1 and Annex B.

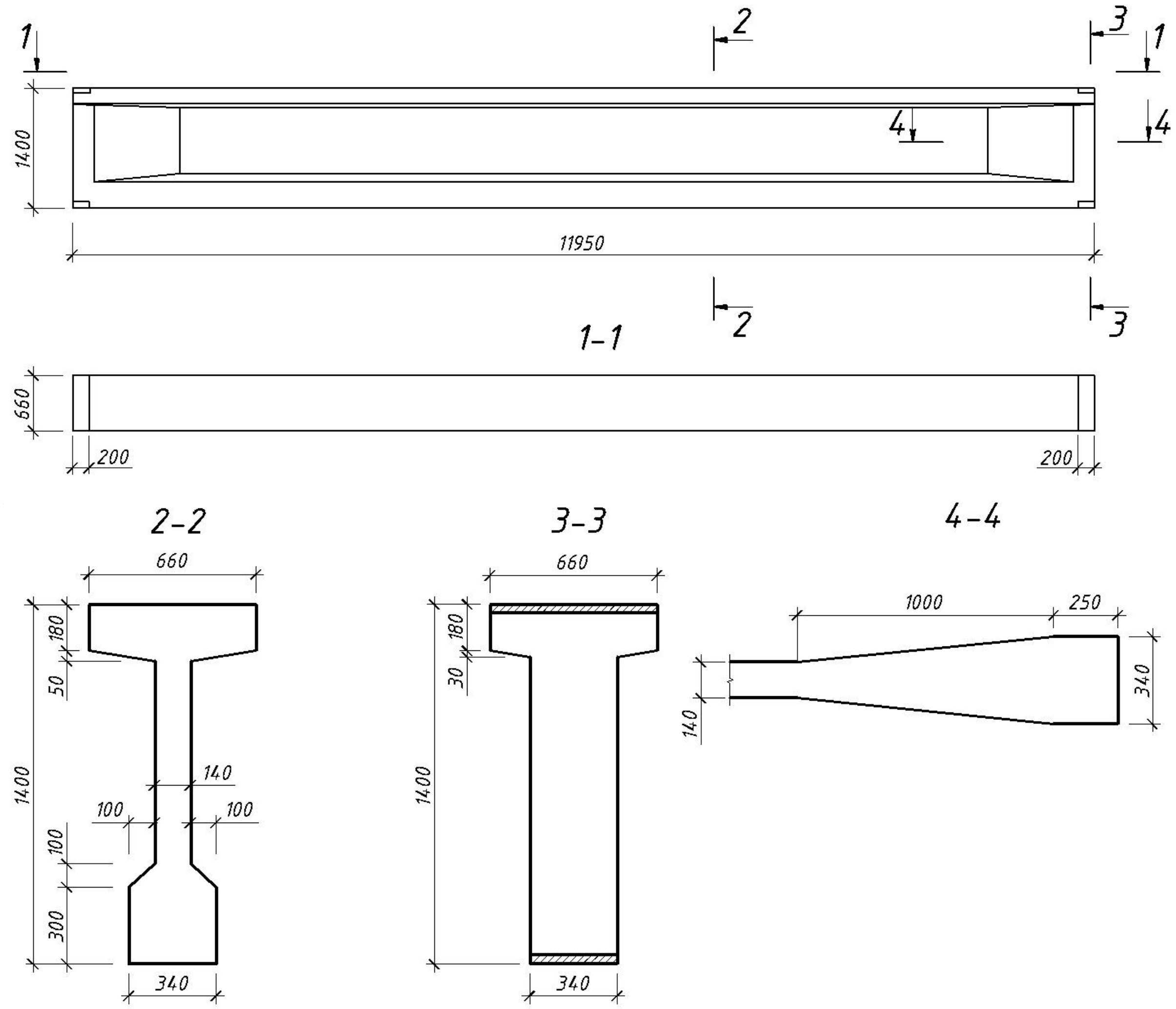


Figure 1.1 – Formwork drawings of the crane beam

2 STRENGTH CALCULATION OF BEAM IN NORMAL SECTION AT VERTICAL LOADS

2.1 Determination of design values of the span and external load

When the width of the supporting embedded item of the beam is 200 mm (Fig. 1.1), the design span is determined

$$l_0 = l_b - 2 \cdot \frac{a}{2} = 11950 - 2 \cdot \frac{200}{2} = 11750 \text{ mm=11,75 m},$$

where l_b – beam length;

a – the length of the supporting embedded item.

The total load on the crane beam consists of a dead load g from the mass of the beam and the crane rails acting in the vertical plane, as well as a live load: F (in the vertical plane) and H (in the horizontal plane) from the bridge crane. Load values are calculated in tabular form (Table 2.1).

Characteristic value of evenly distributed dead load for mass of crane beam (Fig. 1.1)

$$g_{0,b} = A\rho g = \left(0,66 \cdot 0,205 + 0,34 \cdot 0,35 + 0,845 \cdot 0,14\right) \cdot 2500 \cdot 9,81 =$$
$$= 9,14 \cdot 10^3 \text{ N/m=9,14 kN/ m},$$

where A – the cross-sectional area of the beam, m^2;

ρ – density of reinforced concrete, kg/m^3;

g – free fall acceleration, м/s^2.

Characteristic value of uniformly distributed dead load for crane rails mass

$$g_{0,cr} = g_{cr} g = 200 \cdot 9,81 = 1,96 \cdot 10^3 \text{ N/m} = 1,96 \text{ kN/ m},$$

where g_{cr} – the linear mass of the crane rail, kg/m.

The characteristic value of the bridge crane load in the vertical plane $F_0 = F_{max} = 270$ kN.

The characteristic value of the load from the crane in the horizontal plane, which arises due to the distortions of the bridge electric cranes and the unparallelism of the crane rails, according to the norms [4; 7.5],

$$H_{01} = 0,1F_{max} + \frac{\alpha\left(F_{max} - F_{min}\right)L}{B} = 0,1 \cdot 270 + \frac{0,01(270-60)16,5}{4,4} = 34,88 \text{ kN},$$

where F_{max} – maximum load on the crane wheel, kN;

F_{min} – minimum load on the crane wheel, kN;

α – the factor adopted equal to 0.03 for the central drive of the bridge movement mechanism and 0.01 for the separate drive;

L – crane span, m;

B – crane base, m.

Table 2.1 - Load on the crane beam

No	Load	Character-ristic load value	Safety factor for responsibility γ_n [5] at ULS/SLS (for ultimate / serviceability limit states)	Safety factor for load value at SLS γ_{fe} [4]	Design load value at SLS	Safety factor for load value at ULS γ_{fm} [4]	Design load value at ULS
1	2	3	4	5	6	7	8
Dead (kN/m) for mass:							
1	crane beam	$g_{0,b}$ = 9,14	1,1/0,975	1	8,91	1,1	11,06
2	crane rail	$g_{0,cr}$ = 1,96	1,1/0,975	1	1,91	1,1	2,37
Total dead g		g_0 = 11,10	–	–	g_e = 10,82	–	g_m = 13,43
Live (kN) for crane:							
3	in vertical plane $F_e=\gamma_n\gamma_{fe}F_0$ $F_m=\gamma_n\gamma_{fm}\psi F_0$ (ψ = 0,95 [4; 7.22])	F_0 = 270,00	1,1/0,975	1	F_e=263,25	1,1	F_m=310,36
4	in horizontal plane $H_e=\gamma_n H_{01}$ $H_m=\gamma_n\gamma_{fm}H_{01}$	H_{01} = 34,88	1,1/0,975	–	H_e =34,01	1,1	H_m =42,20

2.2 Determination of internal forces in beams cross sections

Precast concrete crane beam is calculated as a single span freely supported. The design scheme of the crane beam, loaded by two cranes, is shown in Figure 2.1.

The distance between the axis of the adjacent wheels of the two as close as possible cranes $b = A - B = 5600 - 4400 = 1200$ mm.

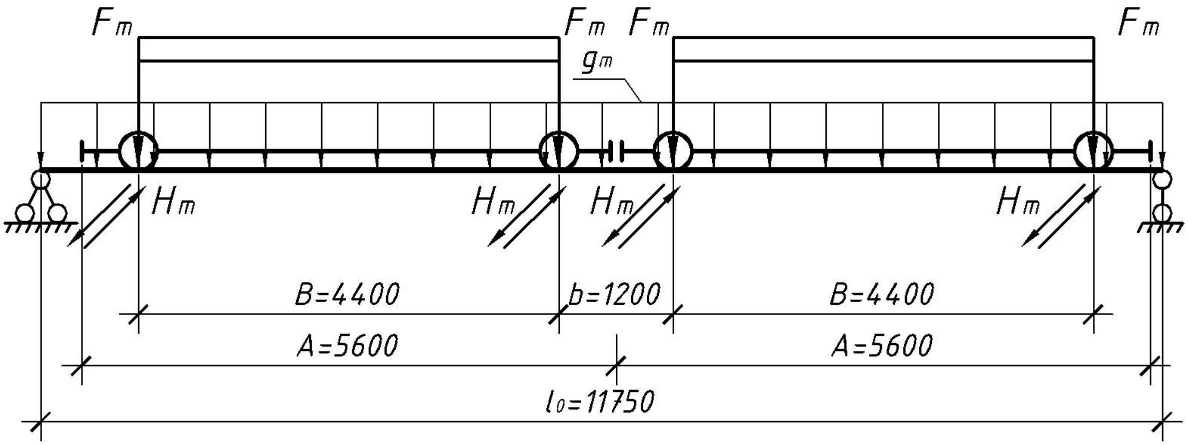


Figure 2.1 – The design scheme of the crane beam for the action of loads from two adjacent cranes

Construction of the enveloping bending moments diagram is carried out for 10 sections in length 0,1l_0 = 1175 mm (points 0 – 10).

The values of the ordinates of the diagram of bending moments in the vertical direction:

for the design value of the load at SLS

$$M_{Fe} = k_g g_e l_0^2 + k_1 k_f F_e l_0; \tag{2.1}$$

for the design value of the load at ULS

$$M_{Fm} = k_g g_m l_0^2 + k_1 k_f F_m l_0 . \tag{2.2}$$

The values of the ordinates of the diagram of bending moments in the horizontal direction:

for the design value of the load at SLS

$$M_{He} = k_1 k_f H_e l_0 ; \tag{2.3}$$

for the design value of the load at ULS

$$M_{Hm} = k_1 k_f H_m l_0 . \tag{2.4}$$

In formulas (2.1) – (2.4), the values of the coefficients k_g and k_f are taken from Table A.6 for the cross sections of the beam through $0.1l_0$, and the values of the coefficient k_1 are taken according to Table A.7, depending on the coefficients α and β.

For this case

$$\alpha = B / l_0 = 4400 / 11750 = 0{,}375,$$

$$\beta = b / l_0 = 1200 / 11750 = 0{,}102.$$

Finally, according to Table A.7, using interpolation it is got k_1 = 0,525.

The ordinates of the enveloping bending moments diagram according to formulas (2.1) – (2.4) were determined in tabular form (Tables 2.2 – 2.3).

Maximum ordinates of the enveloping shear forces diagram in the vertical direction on the supports (points 0 and 10):

for the design value of the load at SLS

$$V_{Fe} = 0,5 g_e l_0 + k_0 F_e; \tag{2.5}$$

for the design value of the load at ULS

$$V_{Fm} = 0,5 g_m l_0 + k_0 F_m. \tag{2.6}$$

Minimum ordinates of the enveloping shear forces diagram in the vertical direction at distance $0,6l_0$ = 7050 mm from the axis of each support (points 4 and 6):

for the design value of the load at SLS

$$V_{Fe} = -0,1 g_e l_0 + k_6 F_e; \tag{2.7}$$

for the design value of the load at ULS

$$V_{Fm} = -0,1 g_m l_0 + k_6 F_m. \tag{2.8}$$

In formulas (2.5) – (2.8) the values of the coefficients k_0 and k_6 are interpolated from the table A.7 at α = 0,375 and β = 0,102. It is received: k_0 = 2,45 i k_6 = 0,5.

The ordinates of the enveloping shear forces diagram according to formulas (2.5) – (2.8) were determined in tabular form (Tables 2.4 – 2.5).

Table 2.2 – Calculation of values of enveloping bending moment diagram ordinates for SLS

№ point	k_g	g_e, kN/m	l_0, m	k_f	k_1	F_e, kN	H_e, kN	M_{Fe}, kNm	M_{He}, kNm
0	0	10,82	11,75	0	0,525	263,25	34,01	0	0
1	0,045	10,82	11,75	0,4	0,525	263,25	34,01	716,8	83,9
2	0,08	10,82	11,75	0,7	0,525	263,25	34,01	1256,3	146,9
3	0,106	10,82	11,75	0,9	0,525	263,25	34,01	1619,9	188,8
4	0,12	10,82	11,75	1	0,525	263,25	34,01	1803,2	209,8
5	0,125	10,82	11,75	1	0,525	263,25	34,01	1810,7	209,8
6	0,12	10,82	11,75	1	0,525	263,25	34,01	1803,2	209,8
7	0,106	10,82	11,75	0,9	0,525	263,25	34,01	1619,9	188,8
8	0,08	10,82	11,75	0,7	0,525	263,25	34,01	1256,3	146,9
9	0,045	10,82	11,75	0,4	0,525	263,25	34,01	716,8	83,9
10	0	10,82	11,75	0	0,525	263,25	34,01	0	0

Table 2.3 – Calculation of values of enveloping bending moment diagram ordinates for ULS

№ point	k_g	g_m, kN/m	l_0, m	k_f	k_1	F_m, kN	H_m, kN	M_{Fm}, kNm	M_{Hm}, kNm
0	0	13,43	11,75	0	0,525	310,36	42,20	0	0
1	0,045	13,43	11,75	0,4	0,525	310,36	42,20	849,3	104,1
2	0,08	13,43	11,75	0,7	0,525	310,36	42,20	1488,5	182,2
3	0,106	13,43	11,75	0,9	0,525	310,36	42,20	1919,6	234,3
4	0,12	13,43	11,75	1	0,525	310,36	42,20	2137,0	260,3
5	0,125	13,43	11,75	1	0,525	310,36	42,20	2146,3	260,3
6	0,12	13,43	11,75	1	0,525	310,36	42,20	2137,0	260,3
7	0,106	13,43	11,75	0,9	0,525	310,36	42,20	1919,6	234,3
8	0,08	13,43	11,75	0,7	0,525	310,36	42,20	1488,5	182,2
9	0,045	13,43	11,75	0,4	0,525	310,36	42,20	849,3	104,1
10	0	13,43	11,75	0	0,525	310,36	42,20	0	0

The enveloping bending moment diagrams, constructed according to Tables 2.2 and 2.3, are shown in Fig. 2.2.

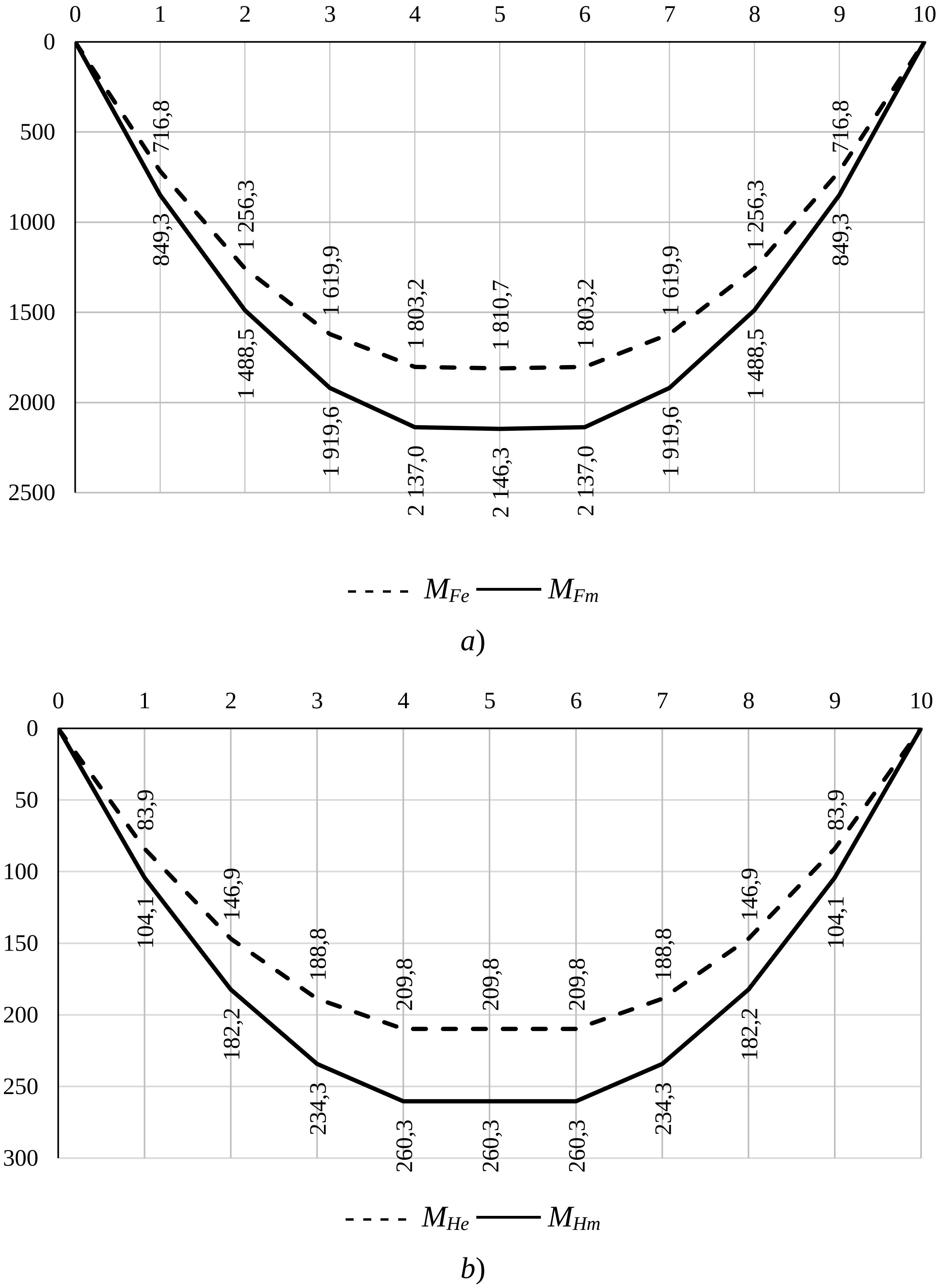


Figure 2.2 – The enveloping bending moment diagrams:

a) in vertical direction; *b*) in horizontal direction

Table 2.4 – Calculation of values of enveloping shear forces diagram ordinates for SLS

№ point	g_e, kN/m	l_0, m	k_0	k_6	F_e, kN	V_{Fe}, kN
0 (10)	10,82	11,75	2,45	–	263,3	708,7
6 (4)	10,82	11,75	–	0,5	263,3	118,9

Table 2.5 – Calculation of values of enveloping shear forces diagram ordinates for ULS

№ точки	g_m, kN/m	l_0, m	k_0	k_6	F_m, kN	V_{Fm}, kN
0 (10)	13,43	11,75	2,45	–	310,4	839,4
6 (4)	13,43	11,75	–	0,5	310,4	139,4

Between the points 0 (10) and 6 (4), the values of the shear force varies linearly, so the enveloping shear forces diagram constructed according to Tables 2.4 and 2.5 will have the form shown in Figure 2.3.

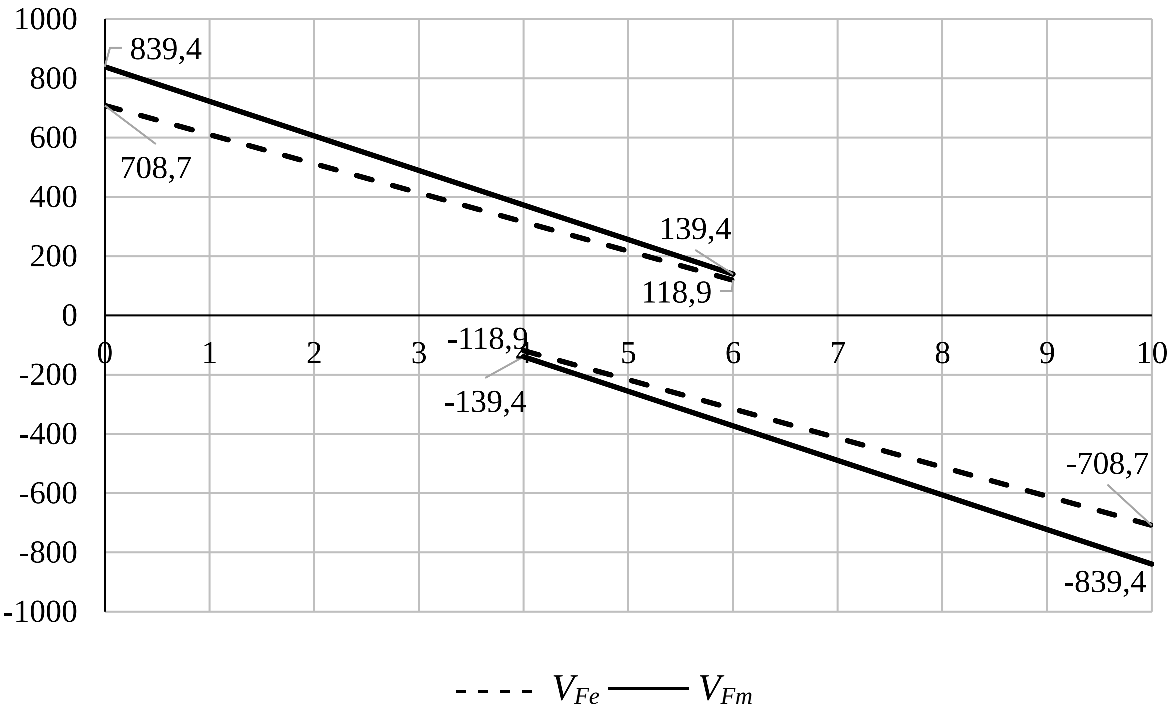


Figure 2.3 – The enveloping shear forces diagrams in vertical direction

Maximum values of internal forces in the normal cross sections of the beam by enveloping diagrams:

$M_{Fm} = 2146,3$ kNm; $M_{Hm} = 260,3$ kNm; $V_{Fm} = 839,4$ kN;

$M_{Fe} = 1810,7$ kNm; $M_{He} = 209,8$ kNm; $V_{Fe} = 708,7$ kN.

The shear force at a distance of 1.15 m from the axis of the support of the crane beam (at the place of maximum narrowing of the rib)

$V_{Fm,1,15} = 725,2$ kN.

The shear force at a distance of 2.94 m ($1 / 4l_0$) from the axis of the support of the crane beam

$V_{Fm,2,94} = 548,5$ kN.

In the calculation of the beam at the stage of transportation and assembling its supports (mounting loops) are located at a distance of $0.2l_0$ from the ends. Then the maximum negative moment (on the support) of the beam mass at the coefficient of dynamism $k_d = 1,4$

$$M_{ep} = g_{ep}\left(0,2l_0\right)^2 / 2 = 12,80\left(0,2 \cdot 11,75\right)^2 / 2 = 35,3 \text{ kNm},$$

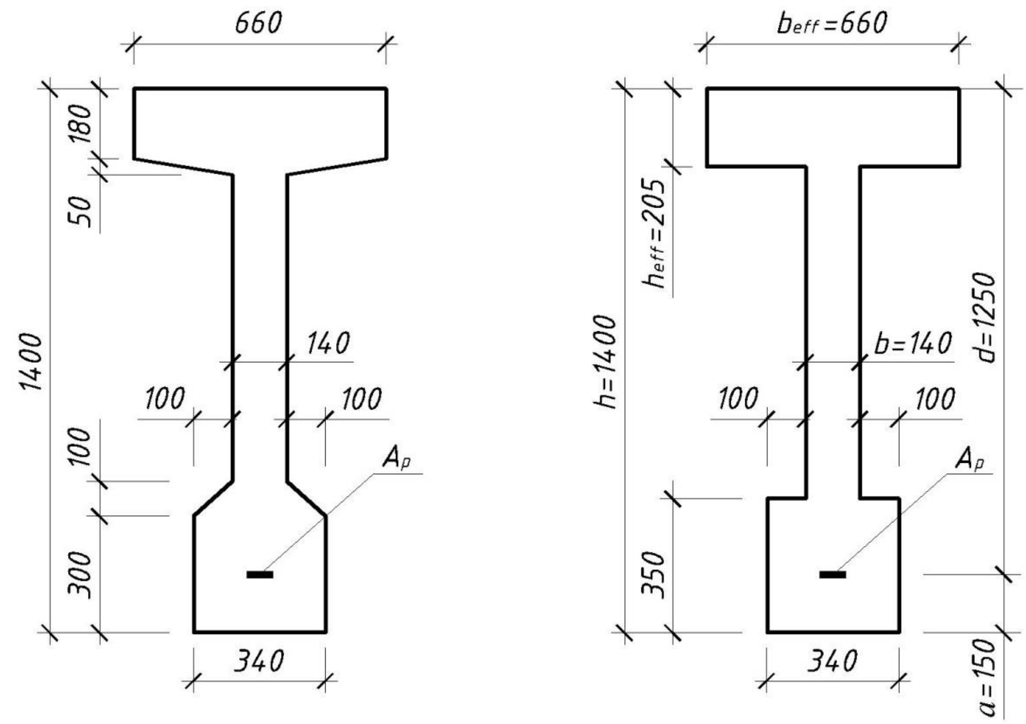
where g_{ep} – load of crane beam mass at the stage of transportation and assembling,

$$g_{ep} = k_d \cdot g_{0,b} = 1,4 \cdot 9,14 = 12,80 \text{ kN/m (Table 2.1)}.$$

2.3 Preliminary calculation of the beam strength in the normal section

The strength calculation of reinforced concrete bending elements with prestressed reinforcement A_p (in the lower area) and A'_p (in the upper area) is performed in two stages: preliminary calculation nonregistering unprestressed reinforcement A_s and A'_s and prestressed reinforcement A'_p and final calculation of all reinforcement (A_p, A'_p, A_s and A'_s).

In the strength calculations, the actual cross section of the crane beam in the middle of the span (Fig. 1.1) is reduced to design section (Fig. 2.4).



a) *b*)

Figure 2.4 – Cross-sectional shape of crane beam section in the span:
a) actual; *b*) design

Check the effective flange width b_{eff} = 660 мм (Fig. 2.4), over which uniform conditions of stress can be assumed, by condition [1; 5.3.2.1]

$$b_{eff,1} = b_{eff,2} \le 0,2 \cdot l_0 , \tag{2.9}$$

where $b_{eff,1}$, $b_{eff,2}$ – flange overhangs of T-beam on both sides;

$$b_{eff,1} = b_{eff,2} = \left(b_{eff} - b\right)/2 = (660 - 140)/2 = 260\,\text{mm};$$

b – T-beam rib width;

$$b_{eff,1} = b_{eff,2} = 260 \text{ мм} < 0,2 \cdot l_0 = 0,2 \cdot 11750 = 2350 \text{ mm}.$$

Condition (2.9) is fulfilled, therefore, in the calculations it is accepted b_{eff} = 660 mm.

Determine the design height of the flange $h_{eff} = 180 + 50/2 = 205$ mm.

Determine the effective height of the beam cross section, considering that the point of application of the resultant force in the reinforcement A_p is located at a distance $a = 150$ mm from the lower face of the cross section of the beam (Fig. 2.4),

$$d = h - a = 1400 - 150 = 1250 \text{ mm}.$$

Determine the position of the neutral axis in the design section of the beam. For this, the condition of the neutral axis location in the flange is checked

$$M_{Fm} \le M_{eff} = N_c Z_c = f_{cd} b_{eff} h_{eff} \omega (d - \chi \omega h_{eff}), \qquad (2.10)$$

where M_{eff} – the moment perceived by the beam in the assumption that the neutral axis coincides with the lower face of the flange;

N_c – resultant force in the compressed concrete area,

$$N_c = f_{cd} b_{eff} h_{eff} \omega = 33 \cdot 660 \cdot 205 \cdot 0,732 = 3268 \cdot 10^3 \text{ Н} = 3268 \text{ kN};$$

Z_c – arm of internal couple of forces (the distance between the points of application of resultant forces in compressed concrete and tensile reinforcement A_p),

$$Z_c = d - \chi\omega h_{eff} = 1250 - 0,539 \cdot 0,732 \cdot 205 = 1169 \text{ mm=1,169 m};$$

$$M_{eff} = N_c Z_c = 3268 \cdot 1,169 = 3820 \text{ kNm.}$$

Since condition (2.10) is satisfied, that is $M_{Fm} = 2146,3\,\text{kNm} < M_{eff} = 3820\,\text{kNm}$, the neutral axis is located in the top flange. Thus, a further preliminary calculation must be performed as for a rectangular beam width $b_{eff} = 660\,\text{mm}$.

First, the coefficient $\bar{\alpha}_m$ is calculated:

$$\bar{\alpha}_m = \frac{M_{Fm}}{f_{cd} \cdot b_{eff} \cdot d^2} = \frac{2146,3 \cdot 10^6}{33 \cdot 660 \cdot 1250^2} = 0,063.$$

According to Table A.8 at K = 2 value $\bar{\alpha}_m = 0,063$ corresponds to value $\bar{\zeta} = 0,965.$

Required cross-sectional area of prestressed reinforcement is

$$A_p = \frac{M_{Fm}}{f_{pd} \cdot \bar{\zeta} \cdot d} = \frac{2146,3 \cdot 10^6}{916 \cdot 0,965 \cdot 1250} = 1942\text{ mm}^2.$$

The prestressed reinforcement A'_p in the amount of 20% of the area of the reinforcement A_p is provided to ensure the crack growth resistance of the upper area of the beam, which will be tensile because of its previous compression at the stage of manufacture and assembling [6]. Taking into account the possible reduction of the normal section strength due to adding the reinforcement A'_p into the compressed area (after its prestress), as well as to advance crack growth resistance, the area of the prestressed reinforcement A_p is increased in 10%.

Then the required area of prestressed reinforcement:

$$A_p = 1942 \cdot 1{,}1 = 2136 \text{ mm}^2;$$

$$A'_p = 1942 \cdot 0{,}2 = 388 \text{ mm}^2.$$

Finally, by the assortment of reinforcement (Table A.9) it is accepted (Fig. 2.5):

in the lower area of the beam 44Ø8Bp1200 with $A_p = 2213 \text{ mm}^2$;

in the upper area of the beam 8Ø8Bp1200 with $A'_p = 402 \text{ mm}^2$.

Structural unprestressed reinforcement (Fig. 2.5):

in the lower area of the beam 4Ø10A400C with $A_s = 314 \text{ mm}^2$;

in the upper area of the beam 2Ø10A400C with $A'_s = 157 \text{ mm}^2$.

The crane beam reinforcement diagram is given in the appendix B (Fig. B.3).

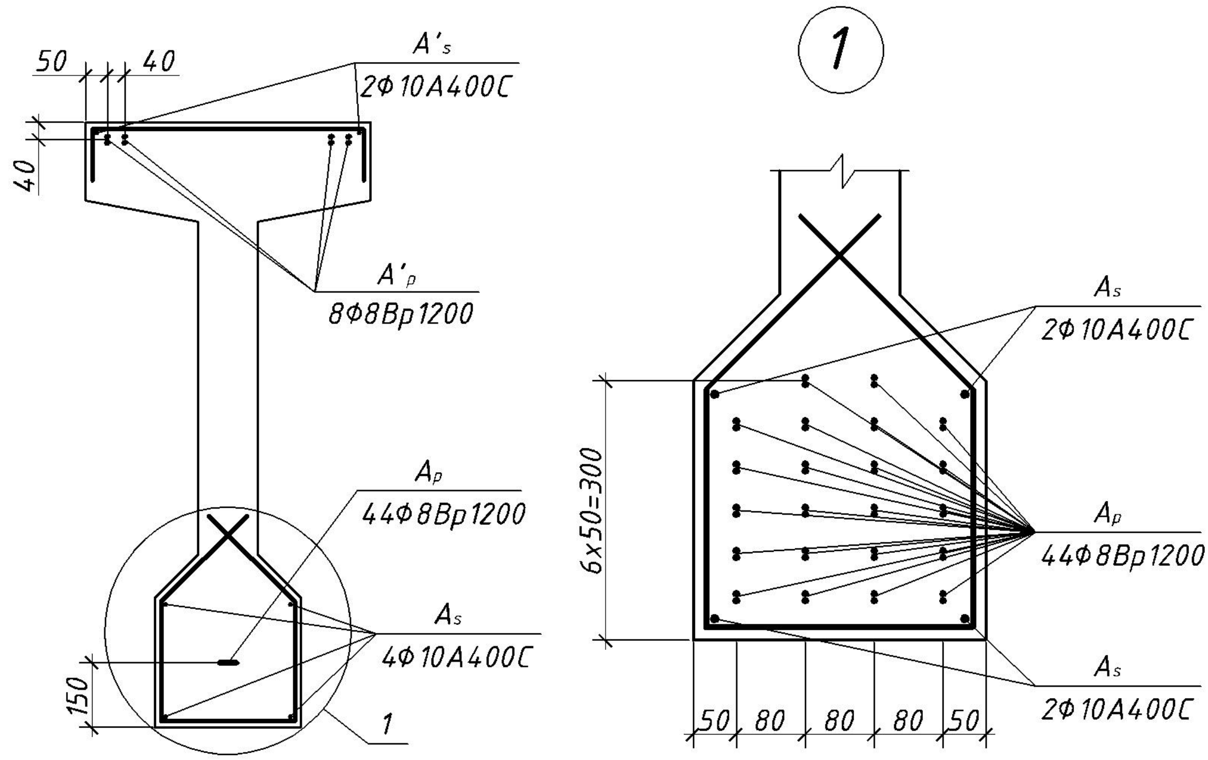


Figure 2.5 – Placement of longitudinal reinforcement in section of the crane beam

3 CHECK OF BEAM STRENGTH IN NORMAL SECTION UNDER BIAXIAL BENDING

3.1 Determination of the value of the moment causing biaxial bending

The bending moments in the design section (mid-span) of the crane beam reach the maximum values (Table 2.3) because of the external loads:

M_{Fm} = 2146,3 kNm – in vertical direction;

M_{Hm} = 260,3 kNm – in horizontal direction.

The total moment from the action of all external loads is the vector sum of moments in the vertical and horizontal planes (Fig 3.1).

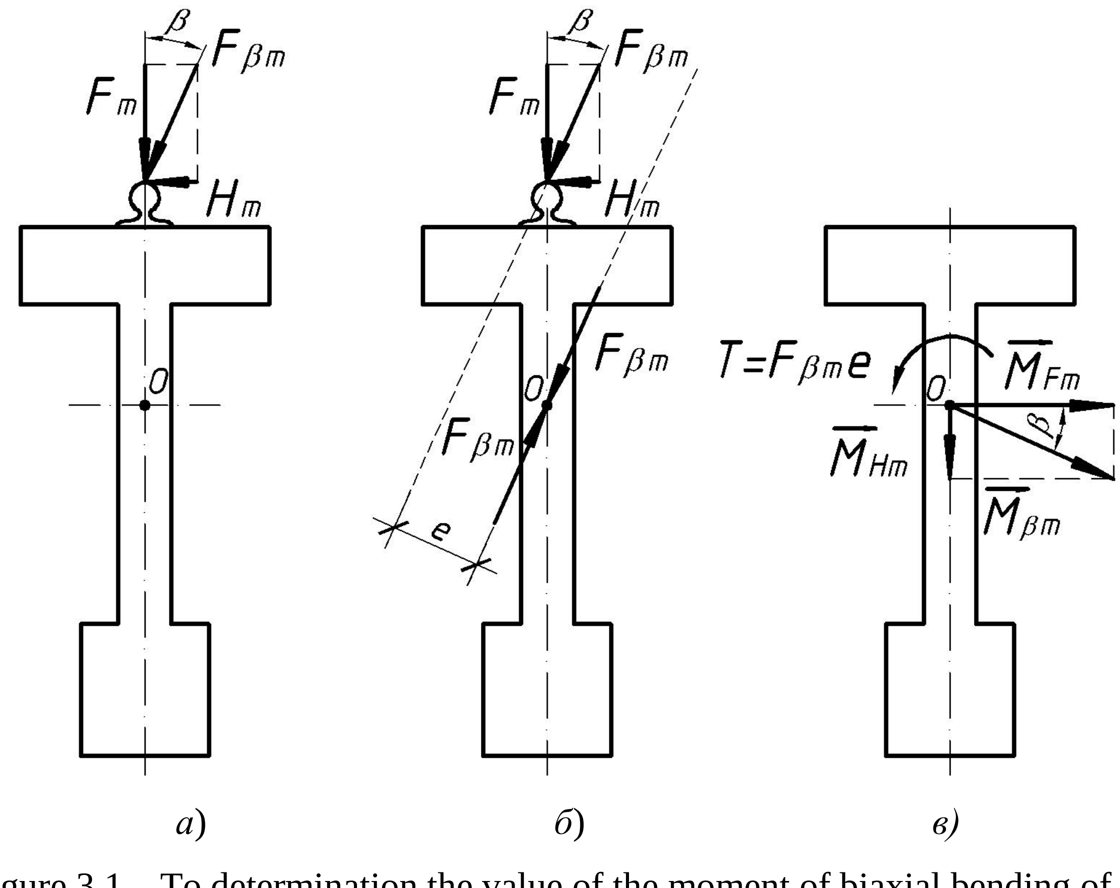


Figure 3.1 – To determination the value of the moment of biaxial bending of the beam: a) scheme of forces; b) scheme of forces in the centre of gravity of the section (O); c) scheme of moment action in the section

The bending moment causing biaxial bending

$$M_{\beta m} = \sqrt{M_{Fm}^2 + M_{Hm}^2} = \sqrt{2146,3^2 + 260,3^2} = 2162\ \text{kNm}.$$

The moment $M_{\beta m}$ acts in the plane that makes up the angle β with the vertical axis of inertia of the section

$$\beta = \text{arctg}\frac{M_{Hm}}{M_{Fm}} = \text{arctg}\frac{260,3}{2146,3} = 6,9°.$$

From Figure 3.1, it can be seen that the total force $F_{\beta m}$ does not pass through the centre of gravity of the section (O), that is, with respect to it there is a torsion component T. Further calculations only take into account the bending component of the moment, since the torsion has no significant effect.

3.2 Determining the case of calculation

3.2.1 Definition of a group of compressed area forms

Since the calculations for the action of biaxial bending by the nonlinear deformation model are rather complicated [7 – 8], and the cross-sectional area of the reinforcement is determined (p. 2.3), then, according to the norms [1; 3.1.7 (3)], in the subsequent verification calculations the rectangular stress distribution in the compressed area of concrete at an effective height λx is adopted.

Groups of forms of effective compressed areas, within which the stresses in the compressed cross-sectional area are distributed by a rectangular diagram, are differentiated depending on the value of the effective height λx for a plane bending:

$$\lambda x = \frac{f_{pd} A_p}{f_{cd} b_{eff}} = \frac{916 \cdot 2213}{33 \cdot 660} = 93\,\text{mm}.$$

Since $\lambda x = 93\,\text{mm} < 0{,}5 h_{eff} = 0{,}5 \cdot 205 = 102{,}5\,\text{mm},$ then the form of the effective compressed concrete area in the section under review belongs to the first group.

3.2.2 Determining the neutral axis position

In the first group of effective compressed area forms, there are 3 conventional neutral axis (border of rectangular stress diagram) limit positions (Table 3.1) that delineate 4 effective compressed area forms. The parameter for determining the form of the compressed area is the angle β_i (i – the number of the limit position of the neutral axis) between the vertical and the internal load plane (Fig. 3.2).

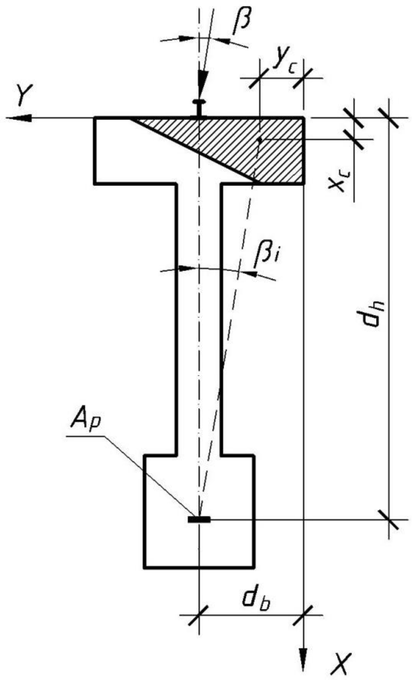


Figure 3.2 – Scheme of the beam cross section
to determination the conventional neutral axis position

Figure 3.2 shows that

$$\beta_i = \text{arctg}\left(\frac{d_b - y_c}{d_h - x_c}\right), \tag{3.1}$$

where d_b, d_h – effective depths in the direction of breadth (b) and height (h) of the beam section respectively, d_b = 330 mm, d_h = 1250 mm;

y_c, x_c – coordinates of the point of application of the resultant in the effective concrete compressed area of the considered limit form.

In order to determine the form of the effective concrete compressed area for the limit positions of the conventional neutral axis of the first group of forms of the compressed area shown in Table 3.1, calculate the values of the angle β_i of the inclination of the load plane of the internal pair of forces to the vertical and compare with the angle β between the vertical and the external load plane. The comparative analysis can be started from the first limit position of the neutral axis (Table 3.1; 1.1).

If it turns out that $\beta_i \leq \beta$, then the form of the effective compressed area is a trapezoid, which is the first in its order of formation among the forms that are delimited by the considered limit position of the conventional neutral axis.

If $\beta_i > \beta$, it is necessary to consider the following limit position of the conventional neutral axis.

Determining of the form of the effective compressed area is made in the form of table 3.2.

Area of effective compressed area for all cases is the same:

$$A_c = \frac{f_{pd} \cdot A_p}{f_{cd}} = \frac{916 \cdot 2213}{33} = 61433\,\text{mm}^2.$$

Coordinates y_c, x_c are calculated according to Table 3.1, and the angle β_i is defined by (3.1).

Table 3.1 – Possible forms (1.1 – 1.4) of effective compressed area in T-beam for the first group of forms ($x \leq 0{,}5h_{eff}$)

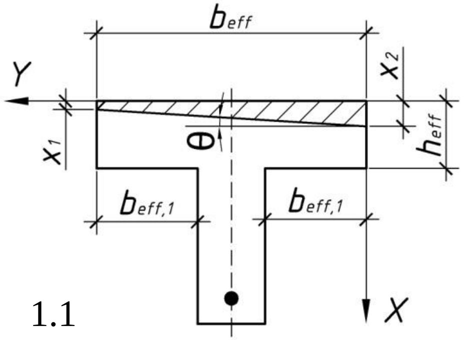
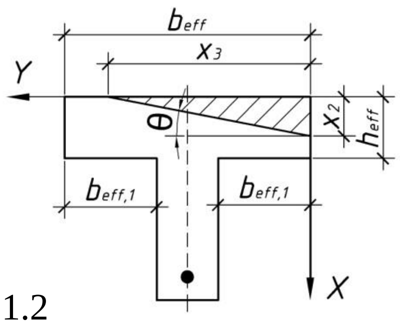
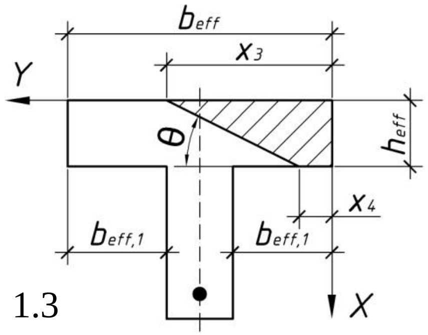
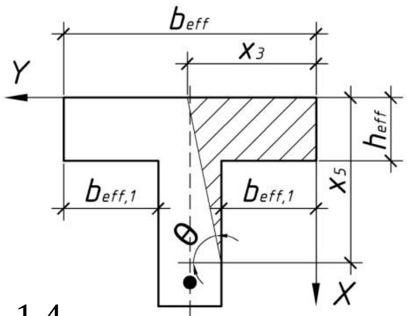
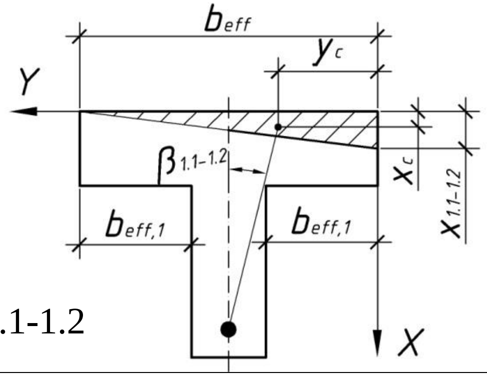
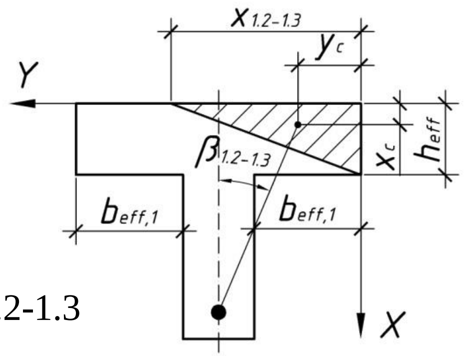
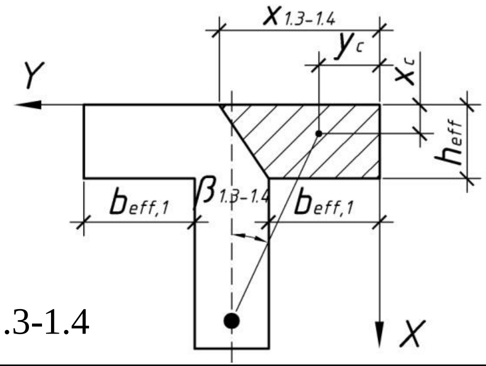
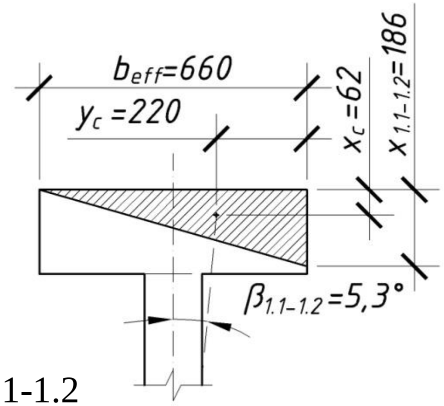
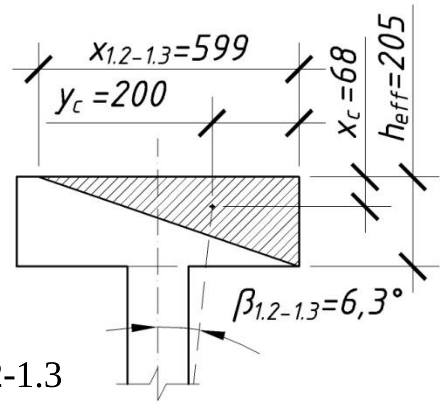
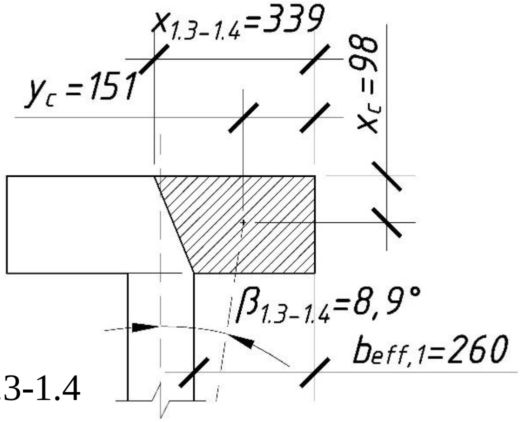
<table>
<tr>
<td>Forms of effective compressed area</td>
<td colspan="3">1.1</td>
<td colspan="3">1.2</td>
<td colspan="3">1.3</td>
<td colspan="3">1.4</td>
</tr>
<tr>
<td rowspan="2">Limit positions of conventional neutral axis</td>
<td colspan="4">1.1-1.2</td>
<td colspan="4">1.2-1.3</td>
<td colspan="4">1.3-1.4</td>
</tr>
<tr>
<td colspan="4">$x_{1.1-1.2} = \frac{2A_c}{b_{eff}}$</td>
<td colspan="4">$x_{1.2-1.3} = \frac{2A_c}{h_{eff}}$</td>
<td colspan="4">$x_{1.3-1.4} = \frac{2A_c}{h_{eff}} - b_{eff,1}$</td>
</tr>
<tr>
<td>Coordinates of resultant application point</td>
<td colspan="4">$x_c = \frac{x_{1.1}}{3}$
$y_c = \frac{b_{eff}}{3}$</td>
<td colspan="4">$x_c = \frac{h_{eff}}{3}$
$y_c = \frac{x_{1.2}}{3}$</td>
<td colspan="4">$x_c = \frac{h_{eff}\left(2b_{eff,1} + x_{1.3}\right)}{3\left(x_{1.3} + b_{eff,1}\right)}$
$y_c = \frac{x_{1.3}^2 + x_{1.3} \cdot b_{eff,1} + b_{eff,1}^2}{3\left(x_{1.3} + b_{eff,1}\right)}$</td>
</tr>
</table>

Table 3.2 – Parameters of the effective compressed area and the load plane for the limit positions of the conventional neutral axis

Limit positions of conventional neutral axis	 1.1-1.2	 1.2-1.3	 1.3-1.4
	$x_{1.1-1.2}=\frac{2\cdot 61433}{660}=186\text{ mm}$	$x_{1.2-1.3}=\frac{2\cdot 61433}{205}=599\text{ mm}$	$x_{1.3-1.4}=\frac{2\cdot 61433}{205}-260=339\text{ mm}$
Coordinates of resultant application point	$x_c=\frac{186}{3}=62\text{ mm}$ $y_c=\frac{660}{3}=220\text{ mm}$	$x_c=\frac{205}{3}=68\text{ mm}$ $y_c=\frac{599}{3}=200\text{ mm}$	$x_c=\frac{205\cdot(2\cdot 260+339)}{3(339+260)}=98\text{ mm}$ $y_c=\frac{339^2+339\cdot 260+260^2}{3(339+260)}=151\text{ mm}$
Inclination angle of the load plane	$\beta_{1.1-1.2}=\text{arctg}\frac{330-220}{1250-62}=5,3°$	$\beta_{1.2-1.3}=\text{arctg}\frac{330-200}{1250-68}=6,3°$	$\beta_{1.3-1.4}=\text{arctg}\frac{330-151}{1250-98}=8,9°$

According to the results of the calculations (Table 3.2), it can be concluded that it is case 1.3 (Table 3.1), in which the form of the effective compressed area is a trapezoid and the conventional neutral line occupies an intermediate position between the limit 1.2-1.3 and 1.3-1.4, since $\beta_{1.2-1.3} = 6,3° < \beta = 6,9° < \beta_{1.3-1.4} = 8,9°$.

Calculated parameters x_3, x_4, x_c, y_c (Table 3.1) of this trapezoidal effective compressed area can be found from the system of equations

$$\begin{cases} A_c = \dfrac{x_3 + x_4}{2} \cdot h_{eff}; \\ \text{tg}\beta = \dfrac{d_b - y_c}{d_h - x_c}; \\ y_c = \dfrac{x_3^2 + x_3 x_4 + x_4^2}{3(x_3 + x_4)}; \\ x_c = \dfrac{h_{eff}(2x_4 + x_3)}{3(x_3 + x_4)}. \end{cases} \tag{3.2}$$

The formulas for determining quantities x_3 and x_4 (Fig. 3.2) are obtained by solving the system of equations (3.2):

$$x_3 = t + \sqrt{t^2 + B\left(\text{tg}\beta(2h_{eff} - 3d) + \frac{3}{2} b_{eff} - B\right)}, \tag{3.3}$$

where $B = \dfrac{2A_c}{h_{eff}}$; $t = \dfrac{B - h_{eff}\,\text{tg}\beta}{2}$;

$$x_4 = B - x_3. \tag{3.4}$$

After performing the calculations according to formulas (3.3) and (3.4) it is found that

$$x_3 = 287 + \sqrt{287^2 + 599\left(\text{tg}6,9°(2 \cdot 205 - 3 \cdot 1250) + \frac{3}{2}660 - 599\right)} = 560\ \text{mm};$$

$$B = \frac{2A_c}{h_{eff}} = \frac{2 \cdot 61433}{205} = 599 \text{ mm};$$

$$t = \frac{B - h_{eff}\,\text{tg}\beta}{2} = \frac{599 - 205 \cdot \text{tg}6,9°}{2} = 287 \text{ mm};$$

$$x_4 = \frac{2A_c}{h_{eff}} - x_3 = B - x_3 = 599 - 560 = 39 \text{ mm}.$$

The coordinates of the point of application of the resultant in the compressed concrete area are calculated by the equations of the system (3.2):

$$x_c = \frac{h_{eff}\left(2x_4 + x_3\right)}{3\left(x_3 + x_4\right)} = \frac{205 \cdot \left(2 \cdot 39 + 560\right)}{3 \cdot (560 + 39)} = 72,8 \text{ mm};$$

$$y_c = \frac{x_3^2 + x_3 x_4 + x_4^2}{3\left(x_3 + x_4\right)} = \frac{560^2 + 560 \cdot 39 + 39^2}{3 \cdot (560 + 39)} = 187,5 \text{ mm}.$$

Check the parallelism of the external and internal load planes, ie the condition $\beta = \beta_i$ for trapezoidal compressed area,

$$\beta_{1.3} = \text{arctg}\left(\frac{d_b - y_c}{d_h - x_c}\right) = \text{arctg}\left(\frac{330 - 187,5}{1250 - 72,8}\right) = 6,9° = \beta .$$

Since equality $\beta = \beta_{1.3}$ is satisfied, then the form and parameters of the effective compressed concrete area are determined correctly.

The angle θ of inclination of the conventional neutral axis to the horizontal plane (Fig. 3.2) is determined by the formula

$$\theta = \text{arctg}\frac{h_{eff}}{x_3 - x_4} = \text{arctg}\frac{205}{560 - 39} = 21,5°.$$

The determined parameters of the effective compressed concrete area are shown in Figure 3.3.

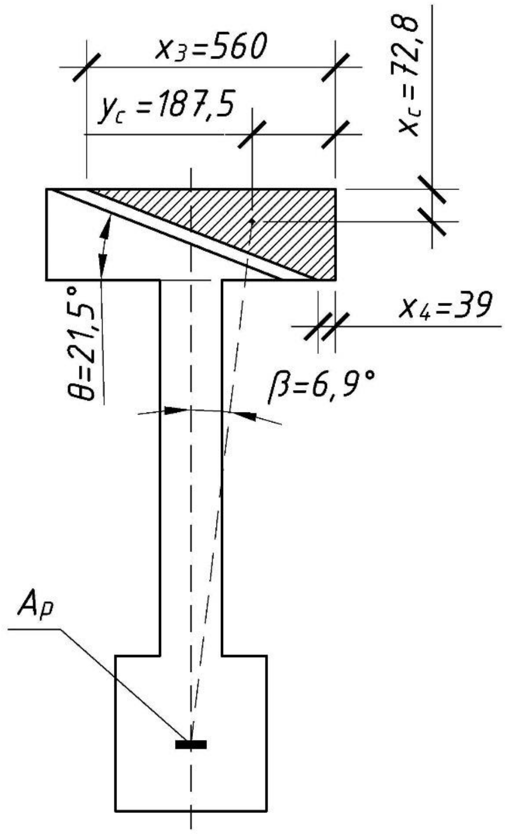


Figure 3.3 – Geometric parameters of the effective compressed concrete area

3.3 Checking the strength of the normal section

Checking the strength of the crane beam in the normal section for the effect of biaxial bending is provided by condition

$$M_{\beta m} \leq M_{\beta u}, \qquad (3.5)$$

where $M_{\beta m}$ – the bending moment from the action of the external load in the plane forming the angle β with the vertical axis of inertia of the cross section of the beam;

$M_{\beta u}$ – the bending moment that can be perceived by the beam in the same plane.

When checking the strength of the beam, it is necessary to first check the accepted hypothesis that stresses in reinforcement A_p reach the conditional yield strength at the moment of destruction under the condition

$$\bar{\xi} \le \bar{\xi}_R, \tag{3.6}$$

where $\bar{\xi} = X/D$ – the relative neutral axis depth at biaxial bending;

$\bar{\xi}_R$ – ultimate value of the relative neutral axis depth, which is calculated by the formula

$$\bar{\xi}_R = \frac{1}{1 + \dfrac{f_{pd}}{\varepsilon_{c1,cd}\eta_u E_p}} = \frac{1}{1 + \dfrac{916}{1,91 \cdot 10^{-3} \cdot 1,268 \cdot 190 \cdot 10^3}} = 0,334.$$

Determine the neutral axis depth at biaxial bending (Fig. 3.4)

$$X = \frac{x_3 \sin\theta}{\lambda} = \frac{560 \cdot \sin 21,5^\circ}{0,8} = 256\,\text{mm}.$$

Determine effective depth of the section at biaxial bending

$$D = d_b \sin\theta + d_h \cos\theta = 330 \sin 21,5^\circ + 1250 \cos 21,5^\circ = 1284\ \text{mm}.$$

Check condition (3.6)

$$\bar{\xi} = X/D = 256/1284 = 0,199 < \bar{\xi}_R = 0,334.$$

Since condition (3.6) is satisfied, the stresses in the reinforcement A_p reach the conditional yield point.

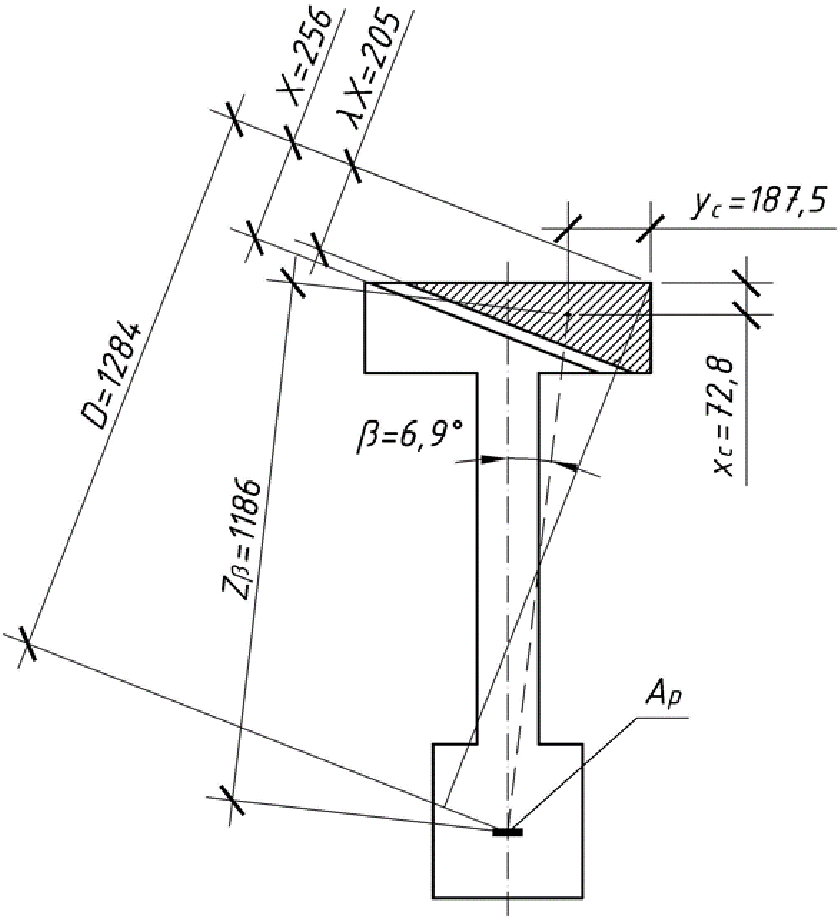


Figure 3.4 – To determination the arm of the internal pair of forces Z_β

The internal moment, which is perceived by a beam with pre-defined reinforcement A_p (2.3), is determined by the formula

$$M_{\beta u} = A_p f_{pd} Z_\beta, \tag{3.7}$$

where Z_β – arm of the internal pair of forces in the plane of action of $M_{\beta m}$ (distance between centres of application of resultant in compressed concrete and tensile reinforcement) (Fig. 3.4):

$$Z_{\beta} = \frac{d_h - x_c}{\cos\beta} = \frac{1250 - 72,8}{\cos 6,9°} = 1186 \text{ mm.}$$

The internal moment of the formula (3.7)

$$M_{\beta u} = 2213 \cdot 916 \cdot 1186 = 2404 \cdot 10^6 \text{ Нмм} = 2404 \text{ кНм} .$$

Since $M_{\beta m} = 2162$ кНм<$M_{\beta u} = 2404$ кНм, that is, condition (3.5) is satisfied, then the strength of the normal cross section for the biaxial bending is ensured.

4 DETERMINATION OF PARAMETERS OF REINFORCEMENT PRE-STRESSING

4.1 Geometric characteristics of the reduced section

Geometric characteristics are determined for the reduced normal section, located in the middle of the span of the beam. The section is conditionally divided into 7 parts (Fig. 4.1).

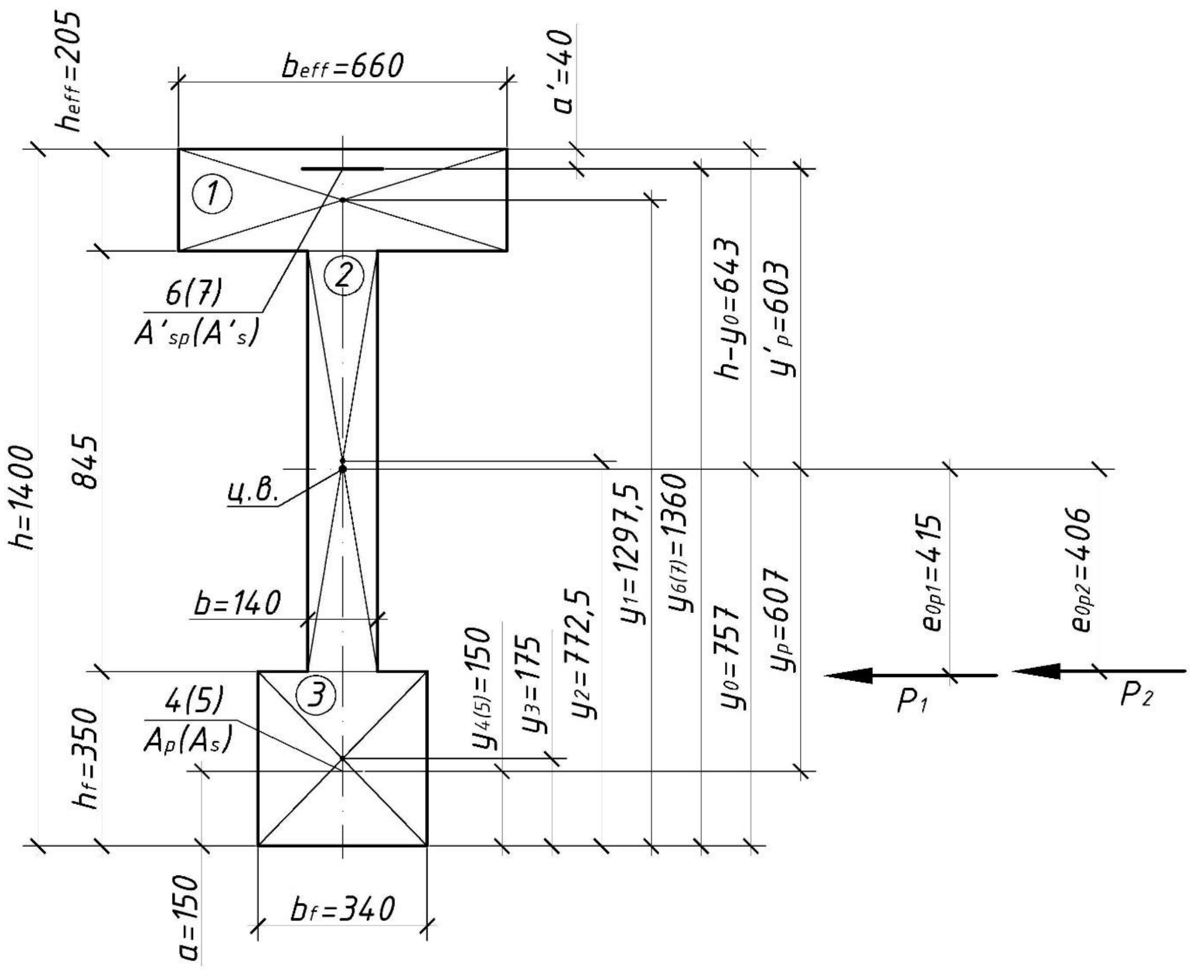


Figure 4.1 – Schematic diagram of the reduced section for the calculation of geometric characteristics and prestressing forces:
$A_p = 2213\,\text{mm}^2$ (44Ø8Bp1200); $A'_p = 402\,\text{mm}^2$ (8Ø8Bp1200);
$A_s = 314\,\text{mm}^2$ (4Ø10A400C); $A'_s = 157\,\text{mm}^2$ (2Ø10A400C)

The reduced section area

$$A_{red} = \sum_{i=1}^{7} A_i = 135300 + 117600 + 119000 + 10512 + 1649 + 1910 + 824 =$$
$$= 388 \cdot 10^3 \text{mm}^2,$$

where A_i – area of the i-th parth of the section, thus:

$A_1 = b_{eff} \cdot h_{eff} = 660 \cdot 205 = 135300$ mm^2;

$A_2 = b \cdot \left(h - h_{eff} - h_f\right) = 140 \cdot (1400 - 205 - 350) = 117600$ mm^2;

$A_3 = b_f \cdot h_f = 350 \cdot 340 = 119000$ mm^2;

$A_4 = \alpha_p \cdot A_p = 4,75 \cdot 2213 = 10512$ mm^2;

$\alpha_p = \dfrac{E_p}{E_{cm}} = \dfrac{190000}{40000} = 4,75;$

$A_5 = \alpha_s \cdot A_s = 5,25 \cdot 314 = 1649$ mm^2;

$\alpha_s = \dfrac{E_s}{E_{cm}} = \dfrac{210000}{40000} = 5,25;$

$A_6 = \alpha_p \cdot A'_p = 4,75 \cdot 402 = 1910$ mm^2;

$A_7 = \alpha_s \cdot A'_s = 5,25 \cdot 157 = 824$ mm^2.

Static moment of reduced section about axis 1-1, which coincide with the lower edge of the section,

$$S_{red} = \sum_{i=1}^{7} S_i = \sum_{i=1}^{7} A_i y_i = 135300 \cdot 1297{,}5 + 117600 \cdot 772{,}5 + 119000 \cdot 175 +$$
$$+ 10512 \cdot 150 + 1649 \cdot 150 + 1910 \cdot 1360 + 824 \cdot 1360 = 294 \cdot 10^6 \text{ mm}^3,$$

where y_i – distance from axis 1-1 to the centre of gravity of the i-th part of the section, thus:

$$y_1 = h - \frac{h_{eff}}{2} = 1400 - \frac{205}{2} = 1297{,}5 \text{ mm};$$

$$y_2 = h_f + \frac{h - h_{eff} - h_f}{2} = 350 + \frac{1400 - 205 - 350}{2} = 772{,}5 \text{ mm};$$

$$y_3 = \frac{h_f}{2} = \frac{350}{2} = 175 \text{ mm};$$

$$y_4 = a = 150 \text{ mm};$$

$$y_5 = a = 150 \text{ mm};$$

$$y_6 = h - a' = 1400 - 40 = 1360 \text{ mm};$$

$$y_7 = h - a' = 1400 - 40 = 1360 \text{ mm}.$$

The distance from the extreme tensile fiber (axis 1-1) to the center of gravity of the reduced section

$$y_0 = \frac{S_{red}}{A_{red}} = \frac{294 \cdot 10^6}{388 \cdot 10^3} = 757 \text{ мм}.$$

Moment of inertia I_{red} of the reduced section about horizontal axis passing through the centre of its gravity

$$I_{red} = \sum_{i=1}^{7} I_i = 400 \cdot 10^8 + 69 \cdot 10^8 + 415 \cdot 10^8 + 39 \cdot 10^8 + 6 \cdot 10^8 + 7 \cdot 10^8 +$$
$$+ 3 \cdot 10^8 = 939 \cdot 10^8 \text{ мм}^4,$$

where I_i – moment of inertia of i-th part of the reduced section about horizontal axis passing through the centre of its gravity, thus:

$$I_1 = \frac{b_{eff} h_{eff}^3}{12} + A_1 \cdot (y_1 - y_0)^2 = \frac{660 \cdot 205^3}{12} + 135300 \cdot (1297{,}5 - 757)^2 =$$
$$= 400 \cdot 10^8 \text{ mm}^4;$$

$$I_2 = \frac{b \cdot (h - h_{eff} - h_f)^3}{12} + A_2 \cdot (y_2 - y_0)^2 = \frac{140 \cdot (1400 - 205 - 350)^3}{12} +$$
$$+117600 \cdot (772{,}5 - 757)^2 = 69 \cdot 10^8 \text{ mm}^4;$$

$$I_3 = \frac{b_f h_f^3}{12} + A_3 \cdot (y_3 - y_0)^2 = \frac{340 \cdot 350^3}{12} + 119000 \cdot (175 - 757)^2 =$$
$$= 415 \cdot 10^8 \text{ mm}^4;$$

$$I_4 = A_4 \cdot (y_4 - y_0)^2 = 10512 \cdot (150 - 757)^2 = 39 \cdot 10^8 \text{ mm}^4;$$

$$I_5 = A_5 \cdot (y_5 - y_0)^2 = 1649 \cdot (150 - 757)^2 = 6 \cdot 10^8 \text{ mm}^4;$$

$$I_6 = A_6 \cdot (y_6 - y_0)^2 = 1855 \cdot (1360 - 757)^2 = 7 \cdot 10^8 \text{ mm}^4;$$

$$I_7 = A_7 \cdot (y_7 - y_0)^2 = 824 \cdot (1360 - 757)^2 = 3 \cdot 10^8 \text{ mm}^4.$$

Modulus of the reduced section W_{red} about lower edge (axis 1-1)

$$W_{red} = \frac{I_{red}}{y_o} = \frac{939 \cdot 10^8}{757} = 124 \cdot 10^6 \text{ мм}^3.$$

Modulus of the reduced section W'_{red} about upper edge

$$W'_{red} = \frac{I_{red}}{h - y_o} = \frac{939 \cdot 10^8}{1400 - 757} = 146 \cdot 10^6 \text{ мм}^3.$$

4.2 Determination of pre-stress value and its losses

The A_p and A'_p reinforcement are mechanically pre-tensioned. In accordance with the requirements of the norms [1; 5.10.2.1] the pre-stress value of the reinforcement shall not exceed the following values

$$\sigma_{p,\max} \le \begin{cases} 0,9 f_{p0,1k}; \\ 0,8 f_{pk}. \end{cases} \quad (4.1)$$

For Bp1200 condition (4.1) takes the following form:

$$\sigma_{p,\max} \le \begin{cases} 0,9 \cdot 1145 = 1030 \text{ MPa}; \\ 0,8 \cdot 1260 = 1008 \text{ MPa}. \end{cases}$$

Accept $\sigma_{p,\max} = 1000$ MPa.

The pre-stress force in A_p,

$$P_{\max} = \sigma_{p,\max} \cdot A_p = 1000 \cdot 2213 = 2213000 \text{ N=2213 kN.}$$

The pre-stress force in A'_p,

$$P'_{\max} = \sigma_{p,\max} \cdot A'_p = 1000 \cdot 402 = 402000 \text{ N=402 kN.}$$

The transfer strength of concrete is assigned not less than C12 / 15 and not less than 50% of the accepted class of concrete according to the norms [3; 3.1.1.7]. It is accepted

$$f_{cp} = 0,7C = 0,7 \cdot 60 \approx 45 \text{ МПа},$$

which corresponds to concrete class C35/45.

4.2.1 Immediate losses of prestress for pre-tensioning

Losses due to short term relaxation of stresses in reinforcement at mechanical method of tension [3; 3.3.5.2]:

$$\Delta P_r = A_p\left(0,22\frac{\sigma_{p,\max}}{f_{p0,1k}}-0,1\right)\sigma_{p,\max} = 2213\cdot\left(0,22\cdot\frac{1000}{1145}-0,1\right)\cdot 1000 =$$
$$= 204\cdot 10^3 \text{ Н=204 кН;}$$

$$\Delta P_r' = A_p'\left(0,22\frac{\sigma_{p,\max}}{f_{p0,1k}}-0,1\right)\sigma_{p,\max} = 402\cdot\left(0,22\cdot\frac{1000}{1145}-0,1\right)\cdot 1000 =$$
$$= 37\cdot 10^3 \text{ Н=37 кН.}$$

Losses due to temperature difference during steam curing [3; 3.3.5.3]:

$$\Delta P_\theta = 0,5A_pE_p\alpha_c(T_{\max}-T_0) = 0,5\cdot 2213\cdot 1,9\cdot 10^5\cdot 10^{-5}\cdot 65 = 137\cdot 10^3 \text{ Н=}$$
$$\text{=137 кН;}$$

$$\Delta P_\theta' = 0,5A_p'E_p\alpha_c(T_{\max}-T_0) = 0,5\cdot 402\cdot 1,9\cdot 10^5\cdot 10^{-5}\cdot 65 = 25\cdot 10^3 \text{ Н=}$$
$$\text{=25 кН,}$$

where α_c – coefficient of linear thermal expansion of concrete, by the norms [2; 3.1.2.4] $\alpha_c = 1\cdot 10^{-5}$ C^{-1};

$T_{\max} - T_0$ – difference between the maximum and initial temperatures of concrete near the pre-stressed reinforcement. In the absence of accurate data on the temperature difference it is acceptable $T_{\max} - T_0 = 65^\circ$C.

Losses due to deformation of steel stops at nonsimultaneous tensioning of reinforcement in the absence of data on the form design and manufacturing technology in accordance with the norms [3; 3.3.5.4] it is acceptable:

$$\Delta P_3 = 30A_p = 30\cdot 2213 = 66\cdot 10^3 \text{ H=66 kN};$$

$$\Delta P_3' = 30A_p' = 30\cdot 402 = 12\cdot 10^3 \text{ H=12 kN}.$$

Losses in anchors that occur when jamming in the channels of anchoring devices during anchoring, after tensioning and as a result of deformation of the anchors themselves [3; 3.3.5.7]:

$$\Delta P_4 = \frac{\Delta l}{l}E_pA_p = \frac{2}{13000}\cdot 1{,}9\cdot 10^5\cdot 2213 = 65\cdot 10^3 \text{ H=65 kN};$$

$$\Delta P_4' = \frac{\Delta l}{l}E_pA_p' = \frac{2}{13000}\cdot 1{,}9\cdot 10^5\cdot 402 = 12\cdot 10^3 \text{ H=12 kN},$$

where Δl – reduction of the anchors or displacement of the tendon in the anchor clips. In the absence of more accurate data, it is acceptable $\Delta l = 2$ mm;

l – distance between the outer faces of the stops.

Losses due to the instantaneous deformation of concrete, taking account the order in which the tendons are stressed [3; 3.3.5.5]:

$$\Delta P_{el} = A_pE_p\sum\left(\frac{j\Delta\sigma_c(t)}{E_{cm}(t)}\right) = 2213\cdot 1{,}9\cdot 10^5\cdot\frac{1/2\cdot 10{,}90}{37{,}5\cdot 10^3} = 61\cdot 10^3 \text{ H=61 kN};$$

$$\Delta P_{el}' = A_p'E_p\sum\left(\frac{j\Delta\sigma_c'(t)}{E_{cm}(t)}\right) = 402\cdot 1{,}9\cdot 10^5\cdot\frac{1/2\cdot 0}{37{,}5\cdot 10^3} = 0 \text{ kN},$$

where $j=\frac{(n-1)}{2n}$, n – the number of identical tendons successively prestressed. As an approximation this may be taken as 1/2;

$E_{cm}(t)$ – the average value of the concrete modulus of elasticity at time t when the prestress is applied to it, $E_{cm}(t)$ = 37,5 GPa (for transfer strength C35/45);

$\Delta\sigma_c(t),\ \Delta\sigma'_c(t)$ – the variation of stress in concrete at the centre of gravity of the tendons A_p та A'_p applied at time t:

$$\Delta\sigma_c(t)=\frac{P}{A_{red}}+\frac{Pe_{0p}y_p}{I_{red}}=\frac{2057\cdot10^3}{388\cdot10^3}+\frac{2057\cdot10^3\cdot607\cdot421}{939\cdot10^8}=10,90\text{ MPa},$$

$$\Delta\sigma'_c(t)=\frac{P}{A_{red}}-\frac{Pe_{0p}y'_p}{I_{red}}=\frac{2057\cdot10^3}{388\cdot10^3}-\frac{2057\cdot10^3\cdot603\cdot421}{939\cdot10^8}=-0,26\text{ MPa},$$

where P – force in tendons after pre-determined immediate losses:

$$P=P_p+P'_p=1741+316=2057\text{ kN},$$

$$P_p=P_{\max}-\Delta P_r-\Delta P_\theta-P_3-P_4=2213-204-137-66-65=1741\text{ kN},$$

$$P'_p=P'_{\max}-\Delta P_r-\Delta P'_\theta-P'_3-P'_4=402-37-25-12-12=316\text{ kN};$$

e_{0p} – eccentricity of force P applying,

$$e_{0p}=\frac{P_p\cdot y_p-P'_p\cdot y'_p}{P}=\frac{1741\cdot607-316\cdot603}{2057}=421\text{ mm};$$

$y_p,\ y'_p$ – distances from the centre of gravity of the reduced cross section to the centre of gravity of the reinforcement A_p and A'_p respectively:

$$y_p = y_0 - a = 757 - 150 = 607 \text{ mm},$$

$$y'_p = h - y_0 - a' = 1400 - 757 - 40 = 603 \text{ mm}.$$

The compressive stresses in concrete of the structure arising from the action of the prestressing force applied during tensioning or the transmitting of the prestressing force shall be limited [1; 5.10.2.2 (4)]

$$\Delta\sigma_c(t) \le 0{,}6 f_{ck}(t), \tag{4.2}$$

where $f_{ck}(t)$ – the characteristic value of the compressive strength of concrete at time t when the prestressing force is applied to it, $f_{ck}(t) = 32$ MPa (for transfer strength C35/45).

Check the condition (4.2)

$$\Delta\sigma_c(t) = 10{,}90 \text{ MPa} < 0{,}6 f_{ck}(t) = 0{,}6 \cdot 32 = 19{,}2 \text{ MPa}.$$

Since $\Delta\sigma'_c(t) < 0$, in further calculations it is accepted $\Delta\sigma'_c(t) = 0$.

Total immediate losses:

$$\sum \Delta P_i = \Delta P_r + \Delta P_\theta + \Delta P_3 + \Delta P_4 + \Delta P_{el} = 204 + 137 + 66 + 65 + 61 = 533 \text{ кН},$$

$$\sum \Delta P'_i = \Delta P'_r + \Delta P'_\theta + \Delta P'_3 + \Delta P'_4 + \Delta P'_{el} = 37 + 25 + 12 + 12 + 0 = 86 \text{ кН}.$$

The initial force in the reinforcement (at time t = t_0), applied to the concrete immediately after tension and the pre-stress transfer, is determined by reducing the pre-stress force by the amount of immediate losses [3; 3.3.4.3]:

$$P_{m0}(x) = P_{\max} - \sum \Delta P_i = 2213 - 533 = 1680 \text{ kN},$$

$$P'_{m0}(x) = P'_{\max} - \sum \Delta P'_i = 402 - 86 = 316 \text{ kN}.$$

The prestressing force at the time $t = t_0$ applied to the concrete immediately after tensioning or after transfer of prestressing shall not exceed the following value [1, 5.10.3 (1)]

$$\frac{P_{m0}(x)}{A_p} \le \begin{cases} 0,75 f_{pk}; \\ 0,85 f_{p0,1k}. \end{cases} \tag{4.3}$$

Check condition (4.3) for tendons A_p and A'_p respectively:

$$\frac{P_{m0}(x)}{A_p} = \frac{1680 \cdot 10^3}{2213} = 760 \text{ MPa} < \begin{cases} 0,75 f_{pk} = 0,75 \cdot 1260 = 945 \text{ MPa}; \\ 0,85 f_{p0,1k} = 0,85 \cdot 1145 = 973 \text{ MPa}, \end{cases}$$

$$\frac{P'_{m0}(x)}{A'_p} = \frac{316 \cdot 10^3}{402} = 786 \text{ MPa} < \begin{cases} 0,75 f_{pk} = 0,75 \cdot 1260 = 945 \text{ MPa}; \\ 0,85 f_{p0,1k} = 0,85 \cdot 1145 = 973 \text{ MPa}. \end{cases}$$

Condition (4.3) is fulfilled.

4.2.2 Long-term losses of prestress

Losses due to the reduction of strain, caused by the deformation of concrete due to creep and shrinkage, under the permanent loads and due to the relaxation of stress in the steel under tension.

A simplified method to evaluate long term losses at location x under the permanent loads is given by following expression [3; 3.3.6.2]:

$$\Delta P_{c+s+r} = A_p \Delta\sigma_{p,c+s+r} = A_p \frac{\varepsilon_{cs} E_p + 0{,}8\Delta\sigma_{pr} + \alpha_p \varphi(t,t_o)\sigma_{c,Q,P}}{1+\alpha_p \dfrac{A_p}{A_c}\left(1+\dfrac{A_c}{I_c} z_{cp}^2\right)\left(1+0{,}8\varphi(t,t_o)\right)} =$$

$$= 2213 \cdot \frac{38{,}9 \cdot 10^{-5} \cdot 1{,}9 \cdot 10^5 + 0{,}8 \cdot 92 + 4{,}75 \cdot 1{,}5 \cdot 9{,}0}{1+4{,}75 \cdot \dfrac{2213}{373 \cdot 10^3}\left(1+\dfrac{373 \cdot 10^3}{884 \cdot 10^8} \cdot 622^2\right)\left(1+0{,}8 \cdot 1{,}5\right)} =$$

$$= 398 \cdot 10^3 \text{ N} = 398 \text{ kN},$$

$$\Delta P'_{c+s+r} = A'_p \Delta\sigma'_{p,c+s+r} = A'_p \frac{\varepsilon_{cs} E_p + 0{,}8\Delta\sigma'_{pr} + \alpha_p \varphi(t,t_o)\sigma'_{c,Q,P}}{1+\alpha_p \dfrac{A'_p}{A_c}\left(1+\dfrac{A_c}{I_c} z'^2_{cp}\right)\left(1+0{,}8\varphi(t,t_o)\right)} =$$

$$= 402 \cdot \frac{38{,}9 \cdot 10^{-5} \cdot 1{,}9 \cdot 10^5 + 0{,}8 \cdot 92 + 4{,}75 \cdot 1{,}5 \cdot 1{,}3}{1+4{,}75 \cdot \dfrac{402}{373 \cdot 10^3}\left(1+\dfrac{373 \cdot 10^3}{884 \cdot 10^8} \cdot 588^2\right)\left(1+0{,}8 \cdot 1{,}5\right)} =$$

$$= 60 \cdot 10^3 \text{ N} = 60 \text{ kN},$$

where $\Delta\sigma_{p,c+s+r}$ – variation of stress in the tendons due to creep, shrinkage and relaxation at location *x*, at time *t* ;

ε_{cs} – estimated shrinkage strain for final shrinkage[3; 3.1.3.8],

$$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca} = (30{,}7+8{,}2) \cdot 10^{-5} = 38{,}9 \cdot 10^{-5};$$

ε_{cd} – drying shrinkage strain,

$$\varepsilon_{cd} = k_n \cdot \varepsilon_{cd,0} = 0{,}903 \cdot 0{,}34 = 0{,}307‰ = 30{,}7 \cdot 10^{-5};$$

$\varepsilon_{cd,0}$ – final unrestrained drying shrinkage, $\varepsilon_{cd,0} = 0{,}34‰$ – for concrete class C50/60 at relative humidity 60% by norms [3; Table 3.2];

k_n – coefficient, depending on notional size h_0 of the cross-section, $k_n = 0,903$ by interpolation at h_0 = 165 mm [3; Table 3.3];

h_0 – notional size of the cross-section,

$$h_0 = \frac{2A_c}{u} = \frac{2 \cdot 372 \cdot 10^3}{4520} = 165 \text{ mm};$$

A_c – concrete cross section area,

$$A_c = A_1 + A_2 + A_3 = 135300 + 117600 + 119000 = 372 \cdot 10^3 \text{ mm}^2;$$

u – perimeter of the drying surface,

$$u = 2 \cdot 1400 + 2 \cdot 660 + 2 \cdot 340 - 2 \cdot 140 = 4520 \text{ mm};$$

ε_{ca} – autogenous shrinkage strain,

$$\varepsilon_{ca} = 2,5(f_{ck} - 10) \cdot 10^{-6} = 2,5 \cdot (43 - 10) \cdot 10^{-6} = 8,2 \cdot 10^{-5};$$

$\Delta\sigma_{pr}$, $\Delta\sigma'_{pr}$ – absolute magnitudes of stresses change in tendons A_p and A'_p respectively at location x at time t, caused by the relaxation of the prestressed reinforcement:

$$\Delta\sigma_{pr} = \frac{\Delta P_r}{A_p} = \frac{204 \cdot 10^3}{2213} = 92 \text{ MPa},$$

$$\Delta\sigma'_{pr} = \frac{\Delta P'_r}{A'_p} = \frac{37 \cdot 10^3}{402} = 92 \text{ MPa};$$

$\varphi(t,t_o)$ – creep coefficient at a time t and load application at time t_0, $\varphi(t,t_o)=1{,}5$ – for concrete class C50/60 and relative humidity 40 – 75 % by norms [3; Table3.1];

I_c – moment of inertia of the concrete section,

$$I_c = I_1 + I_2 + I_3 = 400\cdot10^8 + 69\cdot10^8 + 415\cdot10^8 = 884\cdot10^8 \text{ мм}^4;$$

z_{cp}, z'_{cp} – distances from the center of gravity of the concrete section to tendons A_p and A'_p respectively:

$$z_{cp} = \frac{S_c}{A_c} - a = \frac{287\cdot10^6}{372\cdot10^3} - 150 = 622 \text{ mm},$$

$$z'_{cp} = h - \frac{S_c}{A_c} - a' = 1400 - \frac{287\cdot10^6}{372\cdot10^3} - 40 = 588 \text{ mm};$$

S_c – static moment of the concrete cross-sectional area about axis 1-1 coinciding with the lower face of the cross-section

$$S_c = A_1y_1 + A_2y_2 + A_3y_3 = 135300\cdot1297{,}5 + 117600\cdot772{,}5 + 119000\cdot175 = \\ = 287\cdot10^6 \text{ mm}^3;$$

$\sigma_{c,QP}$, $\sigma'_{c,QP}$ – initial stress in the concrete adjacent to the tendons A_p and A'_p due to self-weight, prestress and any other quasi-permanent actions:

$$\sigma_{c,QP} = \frac{P_1}{A_{red}} + \frac{P_1e_{0p1}y_p}{I_{red}} - \frac{M_by_p}{I_{red}} = \frac{1996\cdot10^3}{388\cdot10^3} + \frac{1996\cdot10^3\cdot415\cdot607}{939\cdot10^8} - \\ - \frac{232\cdot10^6\cdot607}{939\cdot10^8} = 9{,}0 \text{ MPa},$$

$$\sigma'_{c,QP} = \frac{P_1}{A_{red}} - \frac{P_1 e_{0p1} y'_p}{I_{red}} + \frac{M_b y'_p}{I_{red}} = \frac{1996 \cdot 10^3}{388 \cdot 10^3} - \frac{1996 \cdot 10^3 \cdot 415 \cdot 603}{939 \cdot 10^8} +$$

$$+ \frac{232 \cdot 10^6 \cdot 603}{939 \cdot 10^8} = 1,3 \text{ MPa}.$$

P_1 – force in prestressed tendons after all immediate losses,

$$P_1 = P_{m0}(x) + P'_{m0}(x) = 1680 + 316 = 1996 \text{ kN};$$

e_{0p1} – eccentricity of applying force P_1 about centre of gravity of the reduced cross section,

$$e_{0p1} = \frac{P_{m0}(x) \cdot y_p - P'_{m0}(x) \cdot y'_p}{P_1} = \frac{1680 \cdot 607 - 316 \cdot 603}{1996} = 415 \text{ mm};$$

M_b – bending moment due the beam's self-weight,

$$M_b = \frac{g_m \cdot l_0^2}{8} = \frac{13,43 \cdot 11,75^2}{8} = 232 \text{ kNm}.$$

Prestress force taking into account all losses

$$P_2 = P_{m,t}(x) + P'_{m,t}(x) = 1282 + 256 = 1538 \text{ kN},$$

where $P_{m,t}(x)$, $P'_{m,t}(x)$ – the average value of the prestress at time $t > t_0$ in tendons A_p and A'_p respectively after all losses:

$$P_{m,t}(x) = P_{m0}(x) - \Delta P_{c+s+r} = 1680 - 398 = 1282 \text{ kN};$$

$$P'_{m,t}(x) = P'_{m0}(x) - \Delta P'_{c+s+r} = 316 - 60 = 256 \text{ kN}.$$

Eccentricity of applying force P_2 about centre of gravity of the reduced cross section

$$e_{0p2} = \frac{P_{m,t}(x) y_p - P'_{m,t}(x) y'_p}{P_2} = \frac{1282 \cdot 607 - 256 \cdot 603}{1538} = 406 \text{ mm}.$$

5 SHEAR RESISTANCE OF THE CRANE BEAM

5.1 Calculation of the beam strength in cross section on the support

5.1.1 The design value for the shear resistance without shear reinforcement

Calculation of beam strength in an inclined section is performed according to the norms [1; 6.2.2]. A cross section located at the maximum shear force at the axes of the supports (points 0 and 10) is considered. The design value of the applied shear force on the support according to 2.2 is

$V_{Fm,0} = V_{Fm,\max} = 839,4$ кН.

Shear reinforcement is not required by calculation if the condition is met

$$V_{Fm,\max} \le V_{Rd,c} = \left(C_{Rd,c} k \left(100 \rho_l f_{ck}\right)^{1/3} + k_1 \sigma_{cp} \right) bd, \tag{5.1}$$

where $V_{Rd,c}$ – design value for the shear resistance,

$$V_{Rd,c} \ge V_{Rd,c\min} = \left(V_{\min} + k_1 \sigma_{cp} \right) bd;$$

$$C_{Rd,\,c} = \frac{0,18}{\gamma_c} = \frac{0,18}{1,3} = 0,14,$$

γ_c – partial factors for concrete [2; Table 2.1];

$$k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{1250}} = 1,4 < 2;$$

$\rho_l \le 0,02$ – longitudinal reinforcement ratio,

$$\rho_l = \frac{A_p}{b \cdot d} = \frac{2213}{340 \cdot 1250} = 0,0052 < 0,02;$$

$k_1 = 0,15;$

$$\sigma_{cp} = \frac{N_{Ed}}{A_c} = \frac{1538 \cdot 10^3}{528 \cdot 10^3} = 2,91 \text{ MPa} < 0,2 f_{cd} = 0,2 \cdot 33 = 6,6 \text{ MPa},$$

N_{Ed} – axial force in the cross-section due to loading or prestressing, $N_{Ed} = P_2 = 1538$ kN;

A_c – area of concrete cross section on the support (Fig. 1.1),

$$A_c = 660 \cdot 180 + 340 \cdot (1400 - 180 - 30) + (660 - 340) \cdot 15 = 528 \cdot 10^3 \text{ mm}^2;$$

$$V_{\min} = 0,035 k^{3/2} f_{ck}^{1/2} = 0,035 \cdot 1,4^{3/2} \cdot 43^{1/2} = 0,380 \text{ MPa}.$$

Check the condition (5.1):

$$V_{Fm,\max} = 839,4 \text{ kN} > V_{Rd,c} = \left(0,14 \cdot 1,4 \cdot \left(100 \cdot 0,0052 \cdot 43\right)^{1/3} + 0,15 \cdot 2,91\right) \times$$
$$\times 1250 \cdot 340 = 420 \cdot 10^3 \text{ N=420 kN};$$

$$V_{Rd,c} = 420 \text{ kN} > V_{Rd,c\min} = \left(0,380 + 0,15 \cdot 2,91\right) \cdot 1250 \cdot 340 = 347 \cdot 10^3 \text{ H=347 kN}.$$

Since condition (5.1) is not fulfilled, the beam requires shear reinforcement by the calculation.

5.1.2 Calculation of shear reinforcement

The calculation is performed as for elements that require shear reinforcement in accordance with the requirements of norms [1; 6.2.3].

According to design requirements [3; 8.2.6] and taking into account the design experience [9], shear reinforcement 2∅10A400C ($A_{sw} = 157\,\text{mm}^2$) step s = 200 мм is accepted. In this approach, the solution of the shear reinforcement calculation problem is reduced to checking the accepted reinforcement for the perception of shear force.

To ensure the strength of the beam in an inclined section, the following conditions are necessary:

$$V_{Fm,\max} \le V_{Rd,\max} = \frac{a_{sw} \cdot b \cdot z \cdot \nu \cdot f_{cd}}{\cot\theta + \tan\theta}; \tag{5.2}$$

$$V_{Fm,\max} \le V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} \cot\theta, \tag{5.3}$$

where a_{cw} – coefficient that takes into account the stress level in the concrete of the compressed chord [3; 4.6.3.3]. Since the average compression stress σ_{cp} in concrete caused by pre-stress in the reinforcement does not need to be calculated at a distance less than 0,5$d$ctgθ from the face of the support, then a_{cw} = 1 is taken when calculating the shear strength on the support;

b – the minimum section width, b = 340 mm;

z – the inner lever arm corresponding to the maximum bending moment in the element under consideration,

$$z = 0,9d = 0,9 \cdot 1250 = 1125\,\text{mm};$$

ν – coefficient of reduction of the strength of concrete with cracking in shear [3; 4.6.2.7],

$$\nu = 0,6\left(1 - \frac{f_{ck}}{250}\right) = 0,6\left(1 - \frac{43}{250}\right) = 0,497;$$

A_{sw} – shear reinforcement area, $A_{sw} = 157\,\text{mm}^2$;

s – shear reinforcement step, s = 200 mm;

f_{ywd} – design yield strength of stirrups, f_{ywd} = 285 МПа (Table 3.1);

θ – angle between concrete compression struts and the main tension chord;

$$1 \le \cot\theta \le 2,5.$$

Take $\cot\theta = 2,5$ and check conditions (5.2) and (5.3):

$$V_{Fm,\max} = 839,4\text{ kN} < V_{Rd,\max} = \frac{340 \cdot 1125 \cdot 0,497 \cdot 33}{2,5 + 0,4} = 2163,2 \cdot 10^3\text{ N}=$$
$$= 2163,2\text{ kN};$$

$$V_{Fm,\max} = 839,4\text{ kN} > V_{Rd,s} = \frac{157}{200} \cdot 1125 \cdot 285 \cdot 2,5 = 629,2 \cdot 10^3\text{ N}=$$
$$= 629,2\text{ kN}.$$

Since condition (5.3) is not fulfilled, the accepted amount of shear reinforcement it is not sufficient to ensure the strength of the beam in the inclined section. Therefore, it is necessary to increase its diameter or reduce the step.

Determine the required step of the shear reinforcement from condition (5.4) by accepting $V_{Rd,s} = V_{Fm,\max} = 839,4\,\text{kN}$,

$$s = \frac{A_{sw}}{V_{Fm,\max}} z f_{ywd} \text{ctg}\theta = \frac{157}{839,4 \cdot 10^3} \cdot 1125 \cdot 285 \cdot 2,5 = 149,9\,\text{mm}.$$

Take step s = 140 mm.

5.2 Calculation of the beam strength in cross section at a distance 1.15 m from the support axis

5.2.1 The design value for the shear resistance without shear reinforcement

Calculation of beam strength in an inclined section is performed in accordance with the requirements of norms [1; 6.2.2]. The cross section located at the place of maximum narrowing of the rib of the beam (at a distance of 1.15 m from the support axis) under the action of the maximum shear force arising on the section of the beam with the width of the ribs *b* = 140 mm is considered. The design value of the applied shear force in the section under 2.2 is

$V_{Fm,1.15} = 725,2$ kN.

Check condition (5.1)

$$V_{Fm,1.15} = 725,2\ \text{kN} > V_{Rd,c} = \left(0,14 \cdot 1,4 \cdot \left(100 \cdot 0,013 \cdot 43\right)^{1/3} + 0,15 \cdot 4,13\right) \times$$
$$\times 1250 \cdot 140 = 240 \cdot 10^3\ \text{N} = 240\ \text{kN},$$

where

$$V_{Rd,c} = 240\ \text{kN} > V_{Rd,c\min} = \left(0,380 + 0,15 \cdot 4,13\right) \cdot 1250 \cdot 140 = 175 \cdot 10^3\ \text{N} = 175\ \text{kN},$$

$$\rho_l = \frac{A_p}{b \cdot d} = \frac{2213}{140 \cdot 1250} = 0,013 < 0,02;$$

$$\sigma_{cp} = \frac{N_{Ed}}{A_c} = \frac{1538 \cdot 10^3}{372 \cdot 10^3} = 4,13\ \text{MPa} < 0,2 f_{cd} = 0,2 \cdot 33 = 6,6\ \text{MPa},$$

A_c – concrete cross-section area in the span (4.2.2).

Since condition (5.1) is not fulfilled, the beam requires shear reinforcement by the calculation.

5.2.2 Shear reinforcement calculation

According to 5.1.2 shear reinforcement is accepted 2∅10A400C ($A_{sw} = 157$ mm^2), step s = 140 mm.

Check are conditions (5.2) and (5.3) satisfied by accepting $\text{ctg}\theta = 2,5$:

$$V_{Fm,\max} = 725,2 \text{ kN} < V_{Rd,\max} = \frac{1,13 \cdot 140 \cdot 1125 \cdot 0,497 \cdot 33}{2,5 + 0,4} = 1006,5 \cdot 10^3 \text{ N} = 1006,5 \text{ kN};$$

$$V_{Fm,\max} = 725,2 \text{ kN} < V_{Rd,s} = \frac{157}{140} \cdot 1125 \cdot 285 \cdot 2,5 = 899,0 \cdot 10^3 \text{ N} = 899,0 \text{ kN},$$

where b – minimal breadth of the section, b = 140 mm;

a_{cw} – coefficient taking into account the level of stresses in the compressed chord [3; 4.6.3.3],

$$a_{cw} = 1 + \frac{\sigma_{cp}}{f_{cd}} = 1 + \frac{4,13}{33} = 1,13$$

at $\sigma_{cp} = 4,13 \text{ MPa} < 0,25 f_{cd} = 0,25 \cdot 33 = 8,25 \text{ MPa}$;

σ_{cp} – average compression stress in concrete caused by pre-stress in the reinforcement,

$$\sigma_{cp} = \frac{P_2}{A_c} = \frac{1538 \cdot 10^3}{372 \cdot 10^3} = 4,13 \text{ MPa};$$

A_c – concrete cross-section area in the span.

Since conditions (5.2) and (5.3) are fulfilled, the strength of the beam for the action of the shear force in the considered section with the accepted reinforcement is ensured.

5.3 Calculation of the beam strength in cross section at a distance 1/4l_0 from the support axis

Since the values of the shear force in the middle third of the span are significantly smaller than on the supports, it is possible to reduce the intensity of the shear reinforcement of the beam at this section by checking such a measure to save the reinforcement by calculation.

The design value of the applied shear force in the section at a distance 1/4l_0 (2,94 m) from support axis (2.2)

$V_{Fm,2.94} = 548,5$ kN.

Condition (5.1) is not executed for the section under consideration (5.2.1):

$$V_{Fm,2.94} = 548,5\text{ kN} > V_{Rd,c} = 240\text{ kN},$$

therefore, the beam requires shear reinforcement by the calculation.

Condition (5.2) for the section under consideration is satisfied (5.2.2).

Define the necessary step of the shear reinforcement 2∅10 A400C from condition (5.3), taking $V_{Rd,s} = V_{Fm,2.94} = 548,5$ kN:

$$s = \frac{A_{sw} \cdot z \cdot f_{ywd} \cdot ctg\theta}{V_{Fm,2.94}} = \frac{157 \cdot 1125 \cdot 285 \cdot 2,5}{548,5 \cdot 10^3} = 229,4\text{ mm.}$$

Take finally in the middle third of the span of the beam the step of the shear reinforcement s = 220 mm (Annex B, CP2).

According to the results of the calculation for the action of the shear force, a plane cage CP2 was constructed for reinforcing the crane beam (Annex B).

6 CALCULATION OF THE CRANE BEAM AT SERVICEABILITY LIMIT STATE

The calculation of cracks in a reinforced concrete member must be carried out by condition [2; 7.3.3]

$$S \le S_{w,ult}, \tag{6.1}$$

where S – efforts from external loads in the section under consideration;

$S_{w,ult}$ – ultimate effort that can be taken by a reinforced concrete element before cracks.

When considering the calculation for the crane beam the condition (6.1) takes the form

$$M_r \le M_{crc}, \tag{6.2}$$

where M_r – the moment of the external forces located on one side of the section under consideration about an axis parallel to the neutral axis passing through the core point furthest from the tensile area whose crack formation is checked;

M_{crc} – the moment perceived by the beam in the formation of a crack.

6.1 Calculation of the formation of normal cracks at the stage of manufacture

The calculation of the crack appearing in the crane beam at the stage of manufacture and assembling is carried out at the level of the upper face by condition (6.2), in which:

M_r – the moment of external forces about the axis passing through the core point equal to the bending moment from the mass of the beam, taking into account the dynamic coefficient k_d = 1,4, that coincides with sign of the moment from the pre-stress of the section (2.2), $M_r = M_{ep}$ = 35,3 kNm;

M_{crc} – the moment perceived by the beam in the formation of a crack,

$$M_{crc} = f_{ctm}(t) \cdot W'_{pl} - M_{rp} = 3{,}2 \cdot 219 \cdot 10^6 - 77{,}8 \cdot 10^6 = 623{,}0 \cdot 10^6 \text{ Nmm} = \\ = 623{,}0 \text{ kNm},$$

where $f_{ctm}(t)$ – mean tensile strength of concrete at time t when pre-stress is applied to it, $f_{ctm}(t)$ = 3,2 МПа – for transfer strength C35/45 (Table A.2);

W'_{pl} – elastic-plastic modulus of the reduced section about its upper face, which is determined taking into account non-elastic strains of tensioned concrete,

$$W'_{pl} = \gamma \cdot W'_{red} = 1{,}5 \cdot 146 \cdot 10^6 = 219 \cdot 10^6 \text{ mm}^3;$$

M_{rp} – moment of force P_1 about an axis, that passing through the lower core point,

$$M_{rp} = P_1(e_{0p1} - r) = 1996 \cdot 10^3 \cdot (415 - 376) = 77{,}8 \cdot 10^6 \text{ Nmm} = 77{,}8 \text{ kNm};$$

r – the distance from the centre of gravity of the reduced section to the lower core point,

$$r = \frac{\varphi \cdot W'_{red}}{A_{red}} = \frac{1 \cdot 146 \cdot 10^6}{388 \cdot 10^3} = 376 \text{ mm};$$

φ – coefficient accepted within

$$0,7 \le \varphi = 1,6 - \frac{\sigma_{cp}}{f_{ck}(t)} \le 1; \qquad (6.3)$$

σ_{cp} – stresses in concrete at the level of extreme compressed fibre due to force P_1,

$$\sigma_{cp} = \frac{P_1}{A_{red}} + \frac{P_1 e_{0p1} y_0}{I_{red}} = \frac{1996 \cdot 10^3}{388 \cdot 10^3} + \frac{1996 \cdot 10^3 \cdot 415 \cdot 757}{939 \cdot 10^8} = 11,82 \text{ MPa};$$

$f_{ck}(t)$ – the characteristic value of concrete compressive strength at time t when pre-stress is applied to it, $f_{ck}(t)$ = 32 MPa – for transfer strength C35/45 (Table A.2).

Because by the formula (6.3) $\varphi = 1,6 - 11,82/32 = 1,23 > 1$, then accept $\varphi = 1$.

Since $M_r = 35,3 \text{ kNm} < M_{crc} = 623,0 \text{ kNm}$, then normal cracks do not appear at the stage of manufacture of crane beam.

6.2 Calculation for the formation of normal cracks at the stage of exploitation

Calculations for the crack appearing at the stage of exploitation of the reinforced concrete crane beam is carried out at the level of the lower face of the reinforced concrete cross section. The fracture strength is verified by the condition (6.2) in which:

M_r – the moment of external forces about the axis passing through the core point equal to the maximum bending moment from the design value of the external load at SLS (2.2), $M_r = M_{Fe}$ = 1810,7 kNm;

M_{crc} – the moment perceived by the beam in the formation of a crack,

$$M_{crc} = f_{ctm} W_{pl} + M_{rp} = 4{,}1 \cdot 186 \cdot 10^6 + 1116{,}6 \cdot 10^6 = 1879{,}2 \cdot 10^6 \text{ Nmm} =
= 1879{,}2 \text{ kNm},$$

where W_{pl} – elastic-plastic modulus of the reduced section about its lower face, which is determined taking into account non-elastic strains of concrete,

$$W_{pl} = \gamma \cdot W_{red} = 1{,}5 \cdot 124 \cdot 10^6 = 186 \cdot 10^6 \text{ mm}^3;$$

M_{rp} – moment of force P_2 about the axis passing through the upper core point,

$$M_{rp} = P_2 (e_{0p2} + r') = 1538 \cdot 10^3 \cdot (406 + 320) = 1116{,}6 \cdot 10^6 \text{ Nmm} = 1116{,}6 \text{ kNm},$$

where r' – the distance from the centre of gravity of the reduced section to the upper core point,

$$r' = \frac{\varphi \cdot W_{red}}{A_{red}} = \frac{1 \cdot 124 \cdot 10^6}{388 \cdot 10^3} = 320 \text{ мм};$$

φ – coefficient determined by inequality (6.3), where:

σ_{cp} – stresses in concrete at the level of extreme compressed fibre from the force P_2 and the external bending moment,

$$\sigma_{cp} = \frac{P_2}{A_{red}} - \frac{P_2 e_{0p2}(h - y_0)}{I_{red}} + \frac{M_{Fe}(h - y_0)}{I_{red}} = \frac{1538 \cdot 10^3}{388 \cdot 10^3} -
- \frac{1538 \cdot 10^3 \cdot 406 \cdot (1400 - 757)}{939 \cdot 10^8} + \frac{1810{,}7 \cdot 10^6 \cdot (1400 - 757)}{939 \cdot 10^8} = 12{,}08 \text{ MPa};$$

$f_{ck}(t) = f_{ck}$ – the characteristic value of concrete compressive strength at time $t > t_0$, $f_{ck} = 43$ MPa (Table 1.1).

Because by the formula (6.3) $\varphi = 1{,}6 - 12{,}08/43 = 1{,}32 > 1$, then accept $\varphi = 1$.

Since $M_r = 1810{,}7\ \text{kNm} < M_{crc} = 1879{,}2\ \text{kNm}$, then the crack resistance of the beam at the stage of exploitation of the crane beam is ensured

According to the results of the calculations of the beam at ULS and SLS and in accordance with the structural requirements of the current normative documents for the design of reinforced concrete structures [1 – 3], the crane beam of 12 m span for a bridge crane with a load capacity Q=32/5 t is designed. Working drawings of the beam and the main reinforcement cages are given in Annex B.

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ANNEX A

TABLES OF VALUES OF PARAMETERS AND CHARACTERISTICS USED IN THE CALCULATIONS

Table A.1 – Characteristics of the bridge cranes

Load capacity, Q, t	Crane span, L, m	Crane base, B, mm	Crane breadth, A, mm	Middle crane operation mode (4K – 5K)*			Heavy crane operation mode (6K – 7K)		
				Crane mass, m_c, t	Trolley mass, m_t, t	F_{max}, kN	Crane mass, m_c, t	Trolley mass, m_t, t	F_{max}, kN
5	16,5	4000	5260	11	2	55	12	3	57
	22,5	4000	5260	13		60	14		62
	28,5	4200	5460	19		75	19		76
10	16,5	4350	5600	13	3	84	14	4	89
	22,5	4350	5600	16		93	18		99
	28,5	4600	5850	21		105	22		110
16	16,5	4350	5450	19	4	133	19	5	139
	22,5	4350	5450	21		142	21		149
	28,5	4600	5700	27		161	27		166
16/3,2	16,5	4350	5450	20	5	138	20	6	144
	22,5	4350	5450	22		147	22		154
	28,5	4600	5700	28		166	28		171
20/5	16,5	4400	5600	22	7	188	26	7	195
	22,5	4400	5600	28		195	32		210
	28,5	5000	6200	38		235	42		240
32/5	16,5	4400	5600	30	10	260	34	10	270
	22,5	4400	5600	36		275	40		290
	28,5	5000	6200	45		305	47		305

*Note: for cranes of the 1K – 3K mode group, the indicators can be taken the same as for the group 4K – 5K.

Table A.2 – Strength and deformability characteristics of concrete

Strength classes for concrete											Analytical relation / Explanation
	C8/10	C12/15	C16/20	C20/25	C25/30	C30/35	C32/40	C35/45	C40/50	C45/55	C50/60	
$f_{ck,cube}$ (МПа)	10	15	20	25	30	35	40	45	50	55	60	
$f_{cm,cube}$ (МПа)	13	19	25	32	38	45	51	58	64	71	77	$f_{cm,cube} = f_{cm,cube}/(1\text{-}1{,}64V_c)$
$f_{ck,prism}$ (МПа)	7,5	11	15	18,5	22	25,5	29	32	36	39,5	43	
f_{cd} (МПа)	6	8,5	11,5	14,5	17	19,5	22	25	27,5	30	33	$f_{cd} = f_{ck}/\gamma_c$
f_{ctm} (МПа)	1,2	1,6	1,9	2,2	2,6	2,8	3	3,2	3,5	3,8	4,1	
$f_{ctk,\,0{,}05}$ (МПа)	0,8	1,1	1,3	1,5	1,8	2	2,1	2,2	2,5	2,7	3	$f_{ctk,\,0{,}05}=0{,}7f_{ctm}$ 5% fractile
$f_{ctk,\,0{,}95}$ (МПа)	1,6	2	2,5	2,9	3,4	3,6	3,9	4,2	4,6	4,9	5,3	$f_{ctk,\,0{,}95}=1{,}3f_{ctm}$ 95% fractile
E_{cm} (ГПа)	18	23	27	30	32,5	34,5	36	37,5	39	39,5	40	
E_{ck} (ГПа)	15	20	23	26	29	31	32	34	35	36	37	
E_{cd} (ГПа)	12,6	16,3	20	23	25	27	28,5	30,5	32	33	34	
$\varepsilon_{c\,1,ck}$ (‰)	1,57	1,61	1,66	1,71	1,76	1,81	1,86	1,90	1,94	1,98	2,02	
$\varepsilon_{c\,1,cd}$ (‰)	1,56	1,58	1,62	1,65	1,69	1,72	1,76	1,80	1,84	1,87	1,91	
$\varepsilon_{cu\,1,ck}$ (‰)	4,5	4,4	4,15	3,85	3,55	3,25	3	2,83	2,63	2,5	2,4	
$\varepsilon_{cu\,1,cd}$ (‰)	3,75	3,7	3,59	3,44	3,28	3,1	2,93	2,72	2,57	2,43	2,29	
$\varepsilon_{c\,3,ck}$ (‰)	0,5	0,55	0,65	0,71	0,76	0,82	0,91	0,94	1,03	1,1	1,16	$\varepsilon_{c\,3,ck} = f_{ck,prism}/E_{ck}$
$\varepsilon_{c\,3,cd}$ (‰)	0,48	0,52	0,58	0,63	0,68	0,72	0,77	0,83	0,86	0,91	0,97	$\varepsilon_{c\,3,cd} = f_{cd}/E_{cd}$
$\varepsilon_{cu\,3,ck}$ (‰)	4,05	3,96	3,73	3,46	3,2	2,93	2,7	2,55	2,37	2,25	2,16	$\varepsilon_{cu\,3,ck} = 0{,}9\,\varepsilon_{cu\,1,ck}$
$\varepsilon_{cu\,3,cd}$ (‰)	3,38	3,33	3,23	3,1	3	2,8	2,64	2,45	2,31	2,19	2,06	$\varepsilon_{cu\,3,cd} = 0{,}9\,\varepsilon_{cu\,1,cd}$
*) - values $f_{ck,cube}$ in the table given for the values of the coefficient of variation $V_c =$ 13,5 %												

Table A.3 – Strength and deformability characteristics of prestressing steel

Charac-teristic	A600 A600C A600K	A800 A800K A800CK	A1000	Bp 1200	Bp 1300	Bp 1400	Bp 1500	K1400 (K-7)	K1500 (K-7)	K1500 (K-19)
f_{pk} (MPa)	630	840	1050	1260	1365	1470	1575	1470	1575	
$f_{p0,1k}$ (MPa)	575	765	955	1145	1240	1335	1430	1335	1430	
γ_s	1,2			1,25				1,2		
f_{pd} (MPa)	479	638	796	916	992	1068	1144	1113	1192	
E_p (MPa)	$1{,}9 \times 10^5$							$1{,}8 \times 10^5$		
ε_{uk}	0,020	0,0180		0,0160				0,0140		
ε_{ud}	0,018	0,0162		0,0144				0,0126		

Table A.4 – Strength and deformability characteristics of reinforcing steel

Characteristic	A240C	A400C	A500C		B500
			Ø8 – 22	Ø25 – 40	
f_{yk} (MPa)	240	400	500		500
γ_s	1,05	1,10	1,15	1,20	1,20
f_{yd} (MPa)	229	364	435	417	417
f_{ywd} (MPa)	170	285	300		300
E_s (MPa)	$2{,}1 \times 10^5$				$1{,}9 \times 10^5$
ε_{ud}	0,025		0,020		0,012

Table A.5 – Values of the parameters ω, φ, χ depend on K and η_u

Parameter	K								
	1,18	1,5	2	2,5	3	3,5	4	4,5	5
η_u	1,075	1,2	1,268	1,309	1,339	1,363	1,382	1,398	1,412
ω	0,587	0,673	0,732	0,767	0,792	0,811	0,826	0,838	0,848
φ	0,3835	0,4214	0,443	0,455	0,462	0,467	0,471	0,474	0,476
χ	0,591	0,555	0,539	0,53	0,526	0,523	0,52	0,518	0,517

Table A.6 – Coefficients k_g and k_f for constructing the enveloping bending moment diagram for a single span crane beam loaded by two cranes [11]

Point No	k_g	k_f
0	0	0
1	0,045	0,4
2	0,080	0,7
3	0,106	0,9
4	0,120	1,0
5	0,125	1,0
6	0,120	1,0
7	0,106	0,9
8	0,080	0,7
9	0,045	0,4
10	0	0

Table A.7 – Coefficients k_1, k_0 and k_6 for constructing the enveloping bending moment and shear force diagrams for a single span crane beam loaded by two cranes [11]

α	Coefficients	β		
		0,10	0,15	0,20
0,3	k_1	0,60	0,55	
	k_0	2,60	2,50	
	k_6	0,50	0,50	
0,4	k_1	0,50	0,48	0,45
	k_0	2,40	2,30	2,20
	k_6	0,50	0,45	0,40
0,5	k_1		0,43	0,40
	k_0		2,20	2,10
	k_6		0,55	0,50

Table A.8 – Coefficients $\bar{\xi}, \bar{\zeta}, \bar{\alpha}_m$

K=2			K=2,5			K=3			K=3,5		
$\bar{\xi}$	$\bar{\zeta}$	$\bar{\alpha}_m$	$\bar{\xi}$	$\bar{\zeta}$	$\bar{\alpha}_m$	$\bar{\xi}$	$\bar{\zeta}$	$\bar{\alpha}_m$	$\bar{\xi}$	$\bar{\zeta}$	$\bar{\alpha}_m$
0,014	0,995	0,010	0,013	0,995	0,010	0,013	0,995	0,010	0,012	0,995	0,010
0,027	0,989	0,020	0,026	0,989	0,020	0,025	0,989	0,020	0,025	0,990	0,020
0,041	0,984	0,030	0,039	0,984	0,030	0,038	0,984	0,030	0,037	0,984	0,030
0,055	0,978	0,039	0,052	0,979	0,039	0,050	0,979	0,039	0,049	0,979	0,039
0,068	0,973	0,049	0,065	0,973	0,049	0,063	0,974	0,049	0,062	0,974	0,049
0,082	0,968	0,058	0,078	0,968	0,058	0,076	0,968	0,058	0,074	0,969	0,058
0,096	0,962	0,067	0,091	0,963	0,067	0,088	0,963	0,067	0,086	0,963	0,067
0,109	0,957	0,077	0,104	0,958	0,077	0,101	0,958	0,077	0,099	0,958	0,077
0,123	0,952	0,086	0,117	0,952	0,086	0,114	0,953	0,086	0,111	0,953	0,086
0,137	0,946	0,095	0,130	0,947	0,095	0,126	0,947	0,095	0,123	0,948	0,095
0,150	0,941	0,103	0,143	0,942	0,104	0,139	0,942	0,104	0,136	0,943	0,104
0,164	0,935	0,112	0,156	0,936	0,112	0,151	0,937	0,112	0,148	0,937	0,112
0,178	0,930	0,121	0,169	0,931	0,121	0,164	0,932	0,121	0,160	0,932	0,121
0,191	0,925	0,129	0,182	0,926	0,130	0,177	0,926	0,130	0,173	0,927	0,130
0,205	0,919	0,138	0,195	0,920	0,138	0,189	0,921	0,138	0,185	0,922	0,138
0,219	0,914	0,146	0,208	0,915	0,146	0,202	0,916	0,147	0,197	0,916	0,147
0,232	0,908	0,154	0,222	0,910	0,155	0,215	0,911	0,155	0,210	0,911	0,155
0,246	0,903	0,163	0,235	0,904	0,163	0,227	0,905	0,163	0,222	0,906	0,163
0,260	0,898	0,171	0,248	0,899	0,171	0,240	0,900	0,171	0,234	0,901	0,171
0,273	0,892	0,178	0,261	0,894	0,179	0,252	0,895	0,179	0,247	0,895	0,179
0,287	0,887	0,186	0,274	0,889	0,187	0,265	0,890	0,187	0,259	0,890	0,187
0,301	0,881	0,194	0,287	0,883	0,194	0,278	0,884	0,195	0,271	0,885	0,195
0,314	0,876	0,202	0,300	0,878	0,202	0,290	0,879	0,202	0,284	0,880	0,202
0,328	0,871	0,209	0,313	0,873	0,209	0,303	0,874	0,210	0,296	0,875	0,210
0,342	0,865	0,216	0,326	0,867	0,217	0,316	0,869	0,217	0,308	0,869	0,217
0,355	0,860	0,224	0,339	0,862	0,224	0,328	0,863	0,224	0,321	0,864	0,225
0,369	0,855	0,231	0,352	0,857	0,231	0,341	0,858	0,232	0,333	0,859	0,232

Continuation of Table A.8

К=2			К=2,5			К=3			К=3,5		
$\bar{\xi}$	$\bar{\zeta}$	$\bar{\alpha}_m$	$\bar{\xi}$	$\bar{\zeta}$	$\bar{\alpha}_m$	$\bar{\xi}$	$\bar{\zeta}$	$\bar{\alpha}_m$	$\bar{\xi}$	$\bar{\zeta}$	$\bar{\alpha}_m$
0,382	0,849	0,238	0,365	0,851	0,238	0,353	0,853	0,239	0,345	0,854	0,239
0,396	0,844	0,245	0,378	0,846	0,245	0,366	0,847	0,246	0,358	0,848	0,246
0,410	0,838	0,252	0,391	0,841	0,252	0,379	0,842	0,253	0,370	0,843	0,253
0,423	0,833	0,258	0,404	0,835	0,259	0,391	0,837	0,259	0,382	0,838	0,260
0,437	0,828	0,265	0,417	0,830	0,266	0,404	0,832	0,266	0,395	0,833	0,266
0,451	0,822	0,271	0,430	0,825	0,272	0,417	0,826	0,273	0,407	0,828	0,273
0,464	0,817	0,278	0,443	0,820	0,279	0,429	0,821	0,279	0,419	0,822	0,280
0,478	0,811	0,284	0,456	0,814	0,285	0,442	0,816	0,286	0,432	0,817	0,286
0,492	0,806	0,290	0,469	0,809	0,291	0,454	0,811	0,292	0,444	0,812	0,292
0,505	0,801	0,296	0,482	0,804	0,297	0,467	0,805	0,298	0,456	0,807	0,298
0,519	0,795	0,302	0,495	0,798	0,303	0,480	0,800	0,304	0,469	0,801	0,305
0,533	0,790	0,308	0,508	0,793	0,309	0,492	0,795	0,310	0,481	0,796	0,311
0,546	0,785	0,314	0,521	0,788	0,315	0,505	0,790	0,316	0,493	0,791	0,316
0,560	0,779	0,319	0,534	0,782	0,321	0,518	0,784	0,322	0,506	0,786	0,322
0,574	0,774	0,325	0,547	0,777	0,326	0,530	0,779	0,327	0,518	0,780	0,328
0,587	0,768	0,330	0,560	0,772	0,332	0,543	0,774	0,333	0,530	0,775	0,333
0,601	0,763	0,336	0,573	0,766	0,337	0,555	0,769	0,338	0,543	0,770	0,339
0,615	0,758	0,341	0,586	0,761	0,343	0,568	0,763	0,343	0,555	0,765	0,344
0,628	0,752	0,346	0,599	0,756	0,348	0,581	0,758	0,349	0,567	0,760	0,349
0,642	0,747	0,351	0,612	0,751	0,353	0,593	0,753	0,354	0,580	0,754	0,355
0,656	0,741	0,356	0,625	0,745	0,358	0,606	0,748	0,359	0,592	0,749	0,360
0,669	0,736	0,361	0,639	0,740	0,363	0,619	0,742	0,364	0,604	0,744	0,365
0,683	0,731	0,365	0,652	0,735	0,367	0,631	0,737	0,369	0,617	0,739	0,369
0,697	0,725	0,370	0,665	0,729	0,372	0,644	0,732	0,373	0,629	0,733	0,374
0,710	0,720	0,374	0,678	0,724	0,376	0,656	0,726	0,378	0,641	0,728	0,379
0,724	0,715	0,379	0,691	0,719	0,381	0,669	0,721	0,382	0,654	0,723	0,383
0,738	0,709	0,383	0,704	0,713	0,385	0,682	0,716	0,387	0,666	0,718	0,388
0,751	0,704	0,387	0,717	0,708	0,389	0,694	0,711	0,391	0,678	0,713	0,392

Continuation of Table A.8

К=2			К=2,5			К=3			К=3,5		
$\bar{\xi}$	$\bar{\zeta}$	$\bar{\alpha}_m$	$\bar{\xi}$	$\bar{\zeta}$	$\bar{\alpha}_m$	$\bar{\xi}$	$\bar{\zeta}$	$\bar{\alpha}_m$	$\bar{\xi}$	$\bar{\zeta}$	$\bar{\alpha}_m$
0,765	0,698	0,391	0,730	0,703	0,394	0,707	0,705	0,395	0,691	0,707	0,396
0,779	0,693	0,395	0,743	0,697	0,398	0,720	0,700	0,399	0,703	0,702	0,400
0,792	0,688	0,399	0,756	0,692	0,401	0,732	0,695	0,403	0,715	0,697	0,404
0,806	0,682	0,402	0,769	0,687	0,405	0,745	0,690	0,407	0,728	0,692	0,408
0,820	0,677	0,406	0,782	0,682	0,409	0,757	0,684	0,411	0,740	0,686	0,412
0,833	0,671	0,410	0,795	0,676	0,412	0,770	0,679	0,414	0,752	0,681	0,416
0,847	0,666	0,413	0,808	0,671	0,416	0,783	0,674	0,418	0,765	0,676	0,419
0,861	0,661	0,416	0,821	0,666	0,419	0,795	0,669	0,421	0,777	0,671	0,423
0,874	0,655	0,419	0,834	0,660	0,423	0,808	0,663	0,425	0,789	0,665	0,426
0,888	0,650	0,422	0,847	0,655	0,426	0,821	0,658	0,428	0,802	0,660	0,429
0,902	0,644	0,425	0,860	0,650	0,429	0,833	0,653	0,431	0,814	0,655	0,432
0,915	0,639	0,428	0,873	0,644	0,432	0,846	0,648	0,434	0,826	0,650	0,435
0,929	0,634	0,431	0,886	0,639	0,435	0,858	0,642	0,437	0,839	0,645	0,438
0,943	0,628	0,434	0,899	0,634	0,437	0,871	0,637	0,440	0,851	0,639	0,441
0,956	0,623	0,436	0,912	0,628	0,440	0,884	0,632	0,442	0,863	0,634	0,444
0,970	0,618	0,438	0,925	0,623	0,442	0,896	0,627	0,445	0,876	0,629	0,447
0,984	0,612	0,441	0,938	0,618	0,445	0,909	0,621	0,447	0,888	0,624	0,449
0,997	0,607	0,443	0,951	0,613	0,447	0,922	0,616	0,450	0,900	0,618	0,451
			0,964	0,607	0,449	0,934	0,611	0,452	0,913	0,613	0,454
			0,977	0,602	0,451	0,947	0,606	0,454	0,925	0,608	0,456
			0,990	0,597	0,453	0,959	0,600	0,456	0,937	0,603	0,458
						0,972	0,595	0,458	0,950	0,598	0,460
						0,985	0,590	0,460	0,962	0,592	0,462
						0,997	0,584	0,462	0,974	0,587	0,464
									0,987	0,582	0,465
									0,999	0,577	0,467

Table A.9 – Range of reinforcement

Nominal diameter of a bar[1], mm	Design area of a bar cross-section, mm^2, at their number									Theoretical mass of 1 m[3], kg	Diameter of reiofocement classes[2]			
	1	2	3	4	5	6	7	8	9		A240C	A400C	A500C	B500
3	7,1	14,1	21,2	28,3	35,3	42,4	49,5	56,5	63,6	0,056	-	-	-	+
4	12,6	25,1	37,7	50,3	62,8	75,4	88	100,5	113,1	0,099	-	-	-	+
5	19,6	39,3	58,9	78,5	98,2	117,8	137,4	157,1	176,7	0,154	-	-	-	+
6	28,3	57	85	113	141	170	198	226	254	0,222	+	+	+	+
8	50,3	101	151	201	251	302	352	402	452	0,395	+	+	+	+
10	78,5	157	236	314	393	471	550	628	707	0,616	+	+	+	+
12	113,1	226	339	452	565	679	792	905	1018	0,888	+	+	+	+
14	153,9	308	462	616	770	924	1078	1232	1385	1,208	+	+	+	-
16	201,1	402	603	804	1005	1206	1407	1608	1810	1,579	+	+	+	-
18	254,5	509	763	1018	1272	1527	1781	2036	2290	1,998	+	+	+	-
20	314,2	628	942	1257	1571	1885	2199	2513	2827	2,466	+	+	+	-
22	380,1	760	1140	1521	1901	2281	2661	3041	3421	2,984	+	+	+	-
25	490,9	982	1473	1963	2454	2945	3436	3927	4418	3,854	+	+	+	-
28	615,8	1232	1847	2463	3079	3695	4310	4926	5542	4,834	+	+	+	-
32	804,2	1608	2413	3217	4021	4825	5630	6434	7238	6,313	+	+	+	-
36	1017,9	2036	3054	4072	5089	6107	7125	8143	9161	7,991	+	+	+	-
40	1256,6	2513	3770	5027	6283	7540	8796	10053	11310	9,864	+	+	+	-

1 The nominal diameter of reinforcing bars of the periodic profile corresponds to the nominal diameter of equivalent in cross-sectional area smooth bars.

2 The + sign indicates the availability of diameter in the range for the reinforcement of this class.

3 Theoretical mass of 1 m or wire class Bp-1 is taken equal: at Ø3 – 0,052 kg; Ø4 – 0,092 kg; Ø5 – 0,144 kg.

Table A.10 – Range of wire-rope reinforcement*

Class	Nominal diameter, mm	Theoretical mass of 1 m, kg	The design cross-sectional area of reinforcing ropes, mm^2, at their number									
			1	2	3	4	5	6	7	8	9	10
К1400 К1500 (К-7)	6	0,173	22,7	45,4	68,1	90,8	113,5	136,2	158,9	181,6	204,3	227
	9	0,402	51,0	102,0	153,0	204,0	255,0	306,0	357,0	408,0	459,0	510
	12	0,714	90,6	181,2	271,8	362,4	453,0	543,6	634,2	724,8	815,4	906
	15	1,116	141,6	283,2	424,8	566,4	708,0	849,6	991,2	1132,8	1274,4	1416
К1500 (К-19)	14,2	1,014	128,7	257,4	386,1	514,8	643,5	772,2	900,9	1029,6	1158,3	1287

* Note: The nominal diameter of the reinforcing rope corresponds to the diameter of the circle circumscribed around its cross section.

ANNEX B

DRAWINGS OF THE CRANE BEAM

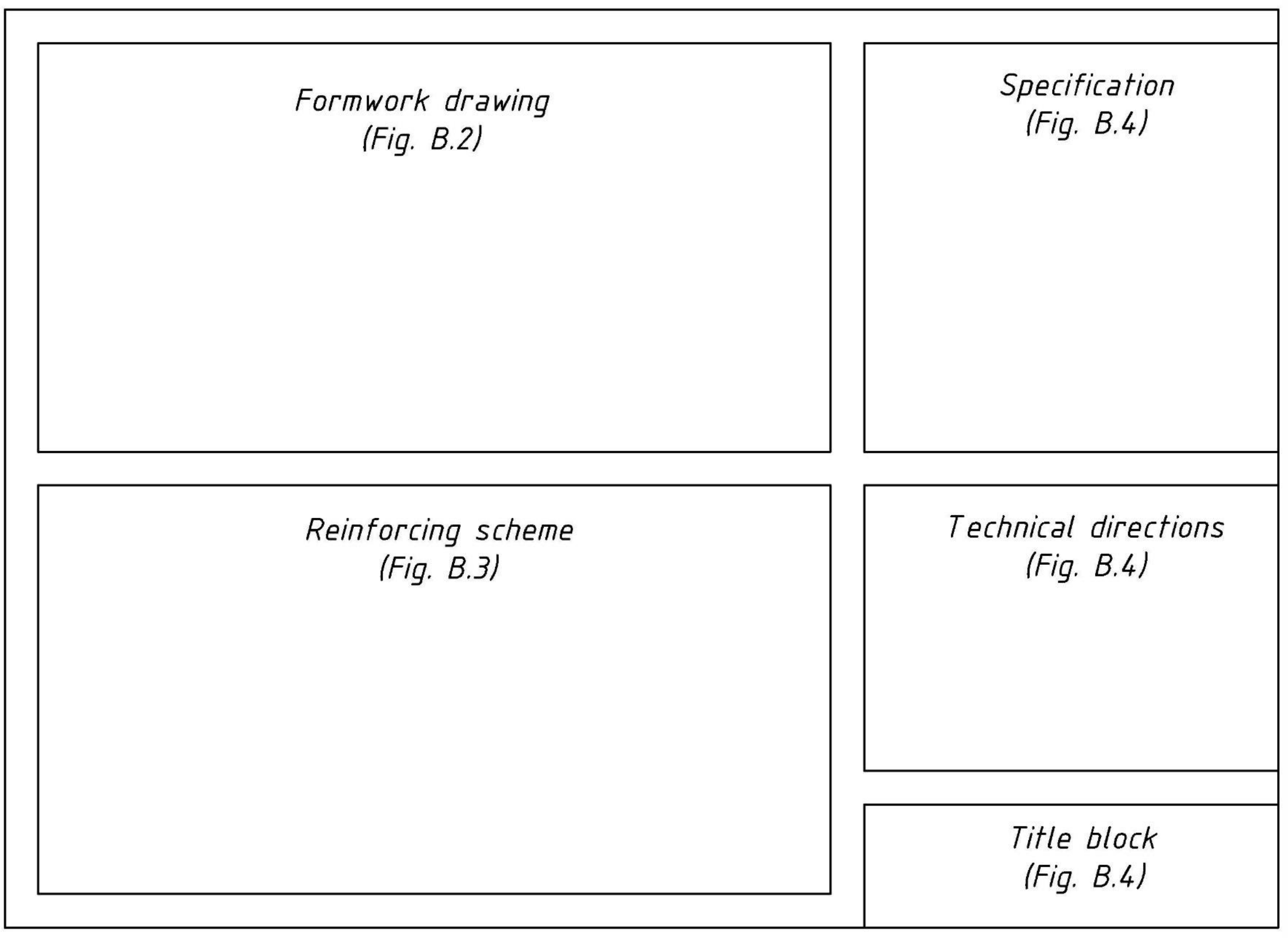


Figure B.1 – Variant of arrangement of drawings of the crane beam on the A2 sheet

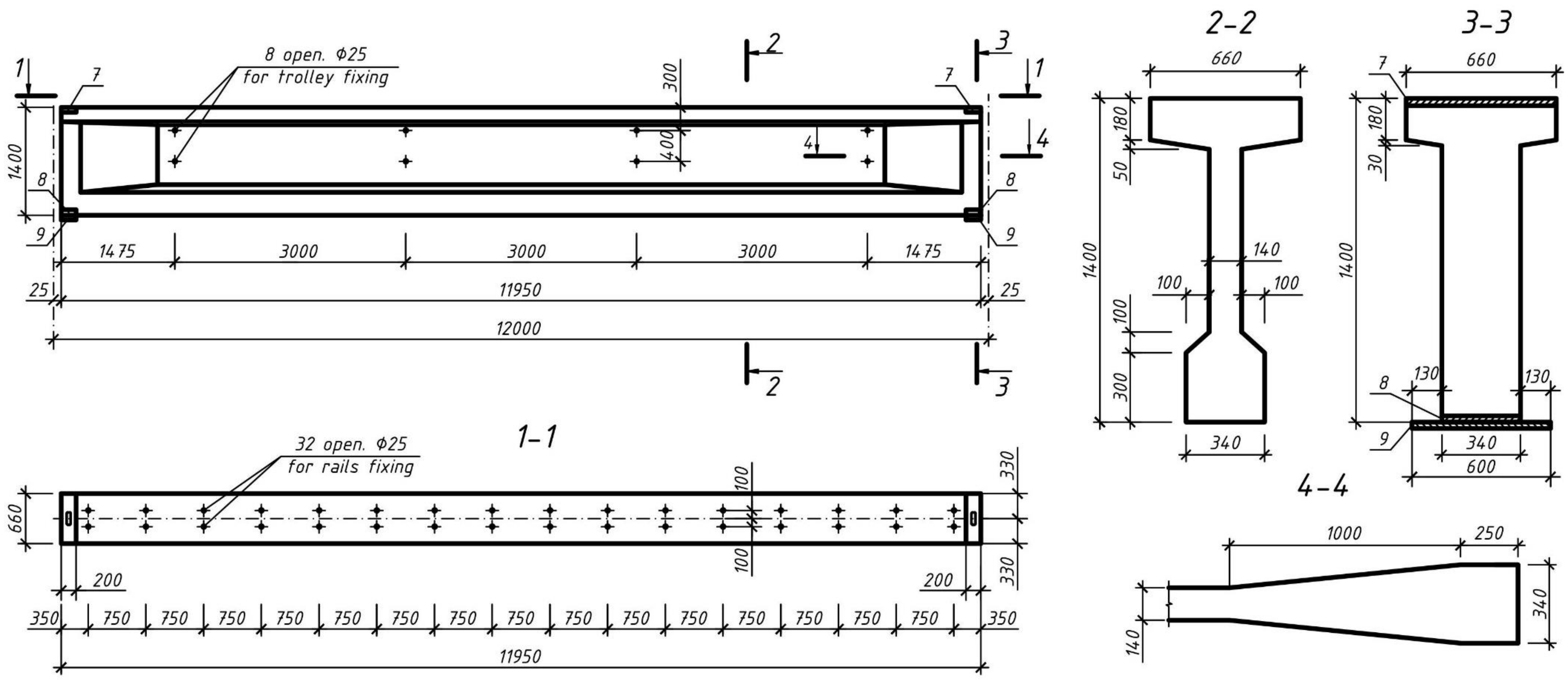


Figure B.2 – Formwork drawing

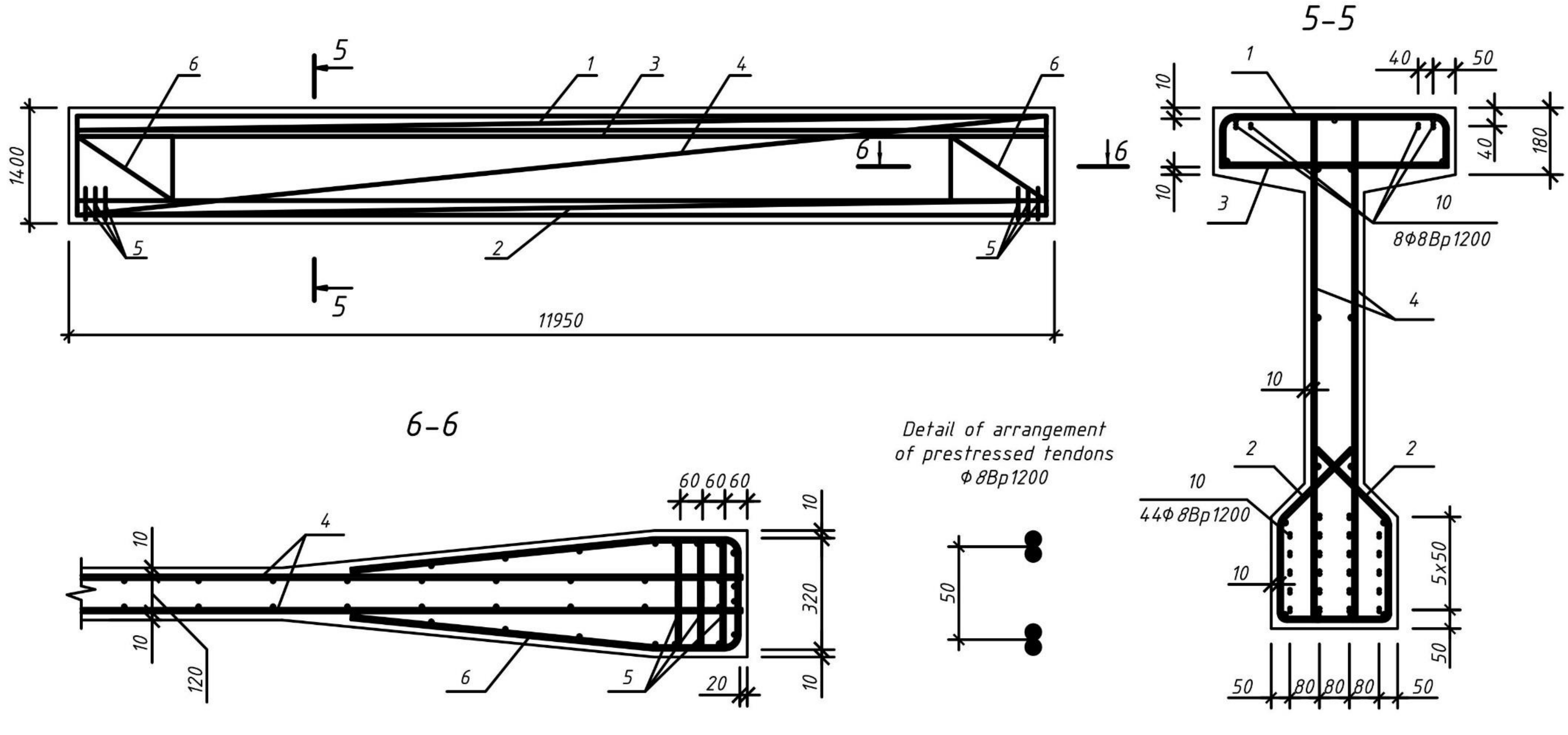


Figure B.3 – Reinforcing scheme

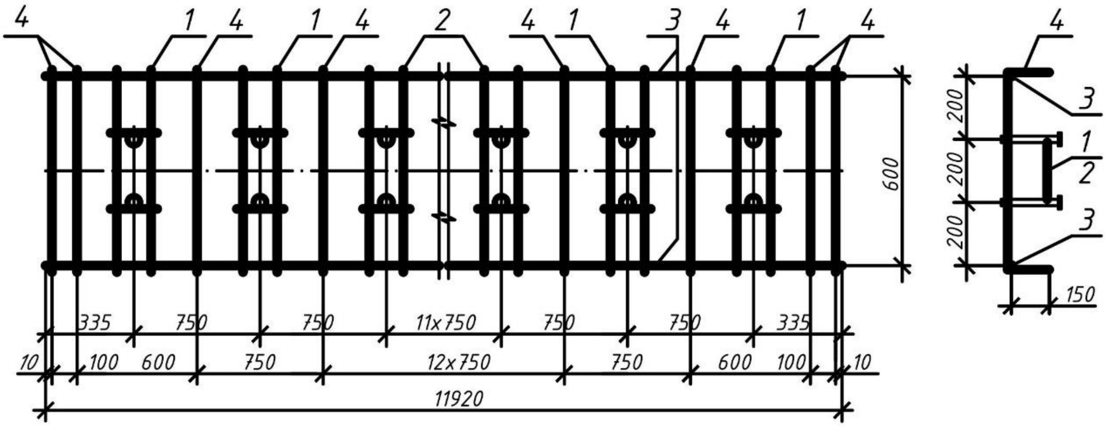
Specification

Pos.	Designation	Denomination		Qua.	Notice
		Documentation			
	SRC-BC-12-Bp1200-CD	Composite drawing			
		Composite items			
1	SRC-BC-12-Bp1200-10	Cage spatial	CS1	1	73,7 kg
2	SRC-BC-12-Bp1200-20	Cage spatial	CS2	1	34,4 kg
3	SRC-BC-12-Bp1200-30	Cage plane	CP1	1	34,0 kg
4	SRC-BC-12-Bp1200-40	Cage plane	CP2	2	89,7 kg
5	Series КЭ-01-50	Cage plane	K-8	12	
6	Series КЭ-01-50	Cage plane	K-9	2	
7	Series КЭ-01-50	Item embedded	M-1	2	
8	Series КЭ-01-50	Item embedded	M-2	2	
9	Series КЭ-01-50	Item embedded	M-3	2	
		Details			
		Prestressed tendons			
10		Ф8 Bp1200 GOST 7348-81 l=13000		52	267,0 kg
		Materials			
		Concrete class C50/60			4,64 m^3

1. Mode of prestressing - mechanical on anchors.
2. The stress that is controlled σ_{con} = 970 MPa.
3. Transfer concrete strength not less 45 MPa.
4. Corrosion protection of embedded items with zinc metallization thickness 150 мк.
5. Reinforcing cages are welded by DSTU B V.2.6-169:2011.

Ch.	Quant.	Sheet	Doc.	Signature	Date	501-БПм.10173.CP			
Developed		Alkelani A.O.				Crane beam SRC-BC-12-Bp1200	Stage	Sheet	Sheets
Checked		Harkava O.V.					CP	1	5
N.control		Harkava O.V.				Formwork drawing, reinforcing scheme	NU "Poltava Polytechnic" Department RCMSSM		
Approved		Pavlikov A.M.							

Figure Б.4 – Specification, technical instructions and title block



Pos.	Designation	Demomination	Qua.	Notice
		Documentation		
	SRC-BC-12-Bp1200-10-CD	Composite drawing		
		Composite items		
1	SRC-BC-12-Bp1200-10-01	Cage plane CP3	4	13,37 kg
2	SRC-BC-12-Bp1200-10-02	Cade plane CP4	12	35,41 kg
		Details		
3	SRC-BC-12-Bp1200-10-03	Ø10 A400C ДСТУ 3760:2006 l=11920	2	14,41 kg
4	SRC-BC-12-Bp1200-10-04	Ø10 A400C ДСТУ 3760:2006 l=900	19	10,53 kg
		Materials		
		Steel mass		73,7 kg
		Ø10 A400C ДСТУ 3760:2006		52,8 kg

Ch.	Quan.	Sheet	Doc.	Signature	Date
Developed		Alkelani A.O.			
Checked		Harkava O.V.			
N. control		Harkava O.V.			
Approved		Pavlikov A.M.			

SRC-BC-12-Bp1200-10

Cage spatial
CS1

Stage	Mass	Scale
CP	look spec.	1:20 1:100
Sheet 2	Sheets 5	

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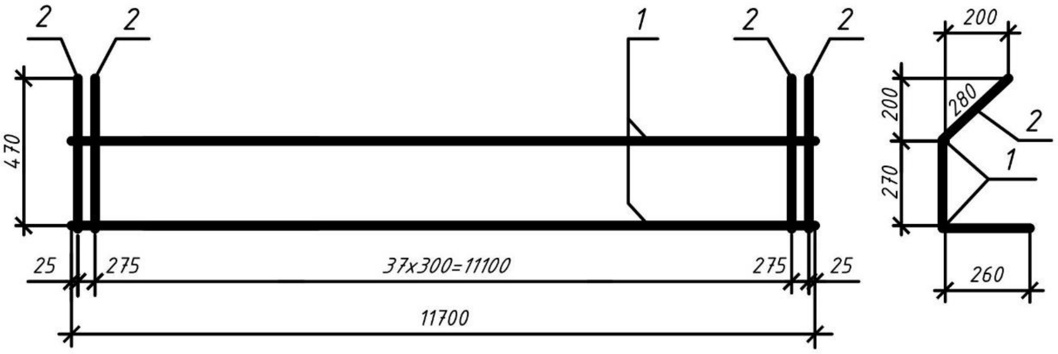

Pos.	Designation	Demomination	Qua.	Notice
		Documentation		
	SRC-BC-12-Bp1200-20-CD	Composite drawing		
		Details		
1	SRC-BC-12-Bp1200-20-01	Ø10 A400C ДСТУ 3760:2006 l=11700	2	14,41 kg
2	SRC-BC-12-Bp1200-20-02	Ø10 A400C ДСТУ 3760:2006 l=810	40	19,96 kg
		Materials		
		Steel mass		34,4 kg
		Ø10 A400C ДСТУ 3760:2006		34,4 kg

Ch.	Quan.	Sheet	Doc.	Signature	Date
Developed		Alkelani A.O.			
Checked		Harkava O.V.			
N. control		Harkava O.V.			
Approved		Pavlikov A.M.			

SRC-BC-12-Bp1200-20

Cage spatial
CS2

Stage	Mass	Scale
CP	look spec.	1:20 1:100
Sheet 2	Sheets 5	

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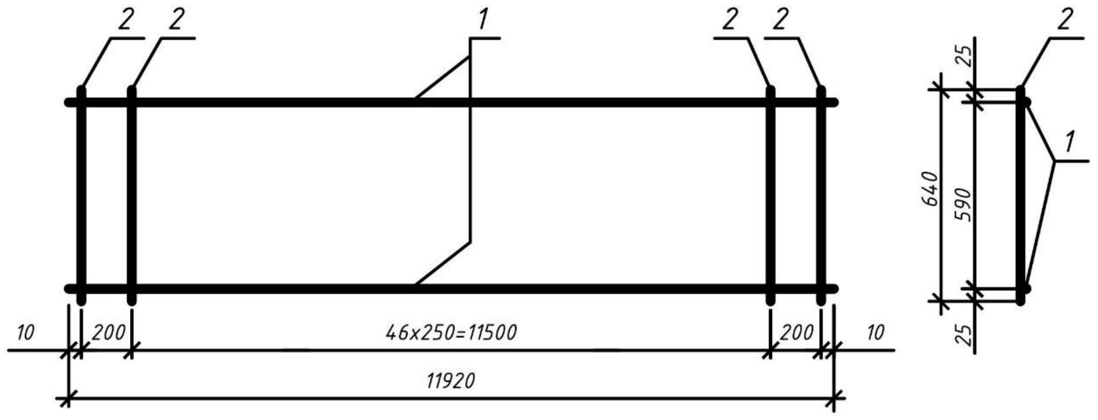


Pos.	Designation	Demomination	Qua.	Notice
		Documentation		
	SRC-BC-12-Bp1200-30-CD	Composite drawing		
		Details		
1	SRC-BC-12-Bp1200-30-01	Ø10 A400C ДСТУ 3760:2006 l=11920	2	14,69 kg
2	SRC-BC-12-Bp1200-30-02	Ø10 A400C ДСТУ 3760:2006 l=640	49	19,32 kg
		Materials		
		Steel mass		34,0 kg
		Ø10 A400C ДСТУ 3760:2006		34,0 kg

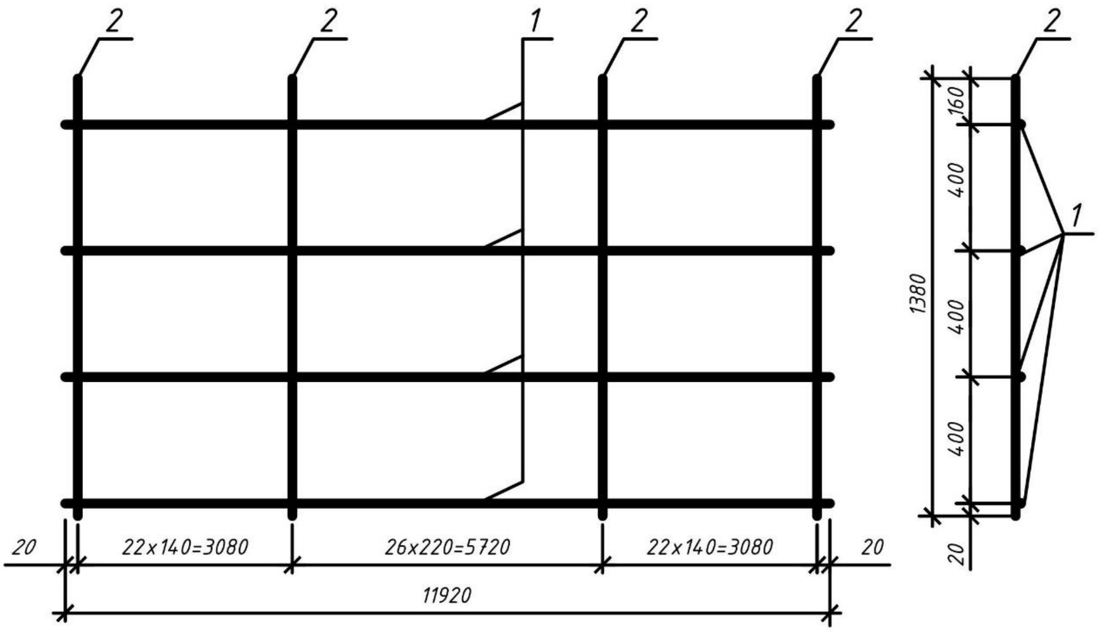
SRC-BC-12-Bp1200-30

Ch.	Quan.	Sheet	Doc.	Signature	Date
Developed		Alkelani A.O.			
Checked		Harkava O.V.			
N. control		Harkava O.V.			
Approved		Pavlikov A.M.			

Cage plane
CP1

Stage	Mass	Scale
CP	look spec.	1:20 1:100
Sheet 2	Sheets 5	

NU "Poltava Polytechnic"
Department RCMSSM



Pos.	Designation	Demomination	Qua.	Notice
		Documentation		
	SRC-BC-12-Bp1200-40-CD	Composite drawing		
		Details		
1	SRC-BC-12-Bp1200-40-01	Ø10 A400C ДСТУ 3760:2006 l=11920	4	29,37 kg
2	SRC-BC-12-Bp1200-40-02	Ø10 A400C ДСТУ 3760:2006 l=1380	71	60,36 kg
		Materials		
		Steel mass		89,7 kg
		Ø10 A400C ДСТУ 3760:2006		89,7 kg

Ch.	Quan.	Sheet	Doc.	Signature	Date
Developed		Alkelani A.O.			
Checked		Harkava O.V.			
N. control		Harkava O.V.			
Approved		Pavlikov A.M.			

SRC-BC-12-Bp1200-40

Cage plane
CP2

Stage	Mass	Scale
CP	look spec.	1:20 1:100
Sheet 2		Sheets 5

NU "Poltava Polytechnic"
Department RCMSSM

Educational edition

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Harkava Olha Viktorivna

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