National Technical University
"Dnipro University of Technology"
Ministry of Science and Education of Ukraine

Faculty of Construction

Department of Construction, Geotechnics and Geomechanics

METAL STRUCTURES
Tutorial on term project

Dnipro
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Ministry of Science and Education of Ukraine
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Department of Construction, Geotechnics and Geomechanics
A S S I G M E N T
for the term project on metal structures

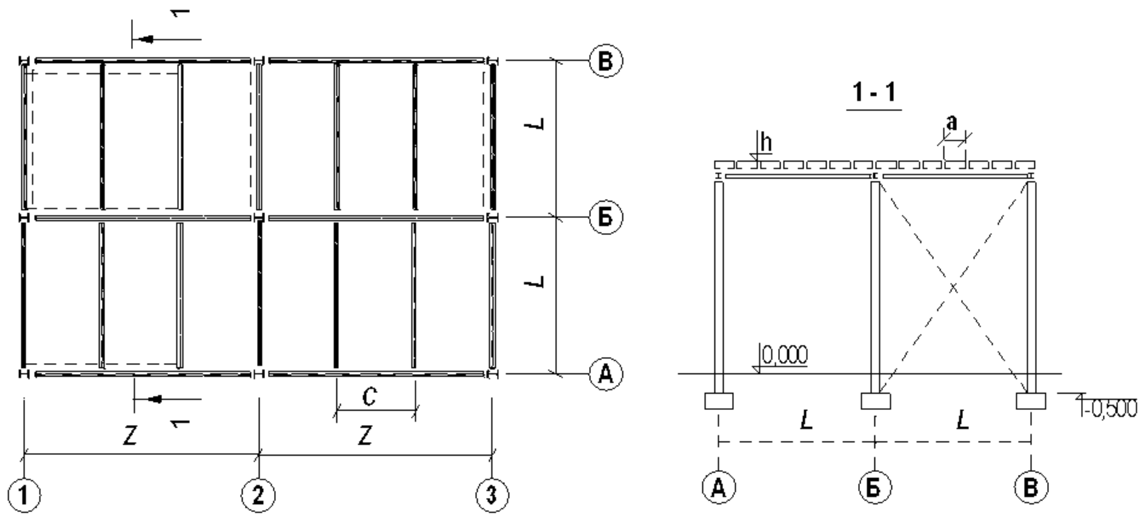

Fig.1.Scheme of building.

Initial data are taken according to the table in which the code is the first five letters of the student's name.

№ letters	Initial date	А К Ф	Б Л Х	В М Ц	Г Н Ч	Д О Ш	Є П Щ	Ж Р Е	З С Ю	І Т Я	Й У И
1	Span of secondary beam, l =	5,0	5,2	5,3	5,5	5,7	5,85	6,0	6,2	6,3	6,5
2	Span of main beam, z =	9	10	11	12	13	14	15	16	17	18
3	Floor level, h =	8,0	8,25	8,5	8,75	9,0	9,25	9,5	9,75	10,0	10,25
4	Load reliability factor	$\gamma_f = 1{,}4$			$\gamma_f = 1{,}3$				$\gamma_f = 1{,}2$		
	Characteristic live load, кN/м^2	15	16	17	18	19	20	21	22	23	24
5	Flooring thickness, $t_н$ mm	4			5			6			

Class of steel C 245, designed strength of steel R_y = 240 МПа,brand of steel Ст3 пс 6.

1. Layout and design of the panel element to the flooring

The use of metal flooring from cold-bent C-shaped panels is the most economical and technological design. The cross-sectional layout of which is determined by design and structural feasibility. Consider the sequence of calculation by example.

Initial data: characteristic load $V_1^n = 23\,\text{kN/м}^2$; load reliability factor $\gamma_f = 1{,}4$; flooring thickness $t = 5\,\text{mm}$; steel of brand - Ст3пс6, steel class C245; the designed strength of steel $R_y = 240\,\text{MPa}$ for a thickness of 2… 20 mm, the span of the secondary beam $\ell = 5{,}5\,\text{m}$; span of the main beam $Z = 13\,\text{m}$.

The coefficient of operating conditions of all elements of the building $\gamma_c = 1{,}0$ because of the absence of moving load.

The characteristic load per 1 cm of the cross section of the panel flooring at the calculated width of the strip along the panel cm (Fig. 1, a) without taking into account its own weight is:

$V^n = V_1^n \cdot b = 23 \cdot 0{,}01 = 0{,}23$ кN/см.

Designed load

$V = \gamma_f \cdot V^n = 1{,}4 \cdot 0{,}23 = 0{,}32$ кН/см.

The maximum designed length of the panel is determined by:

a) condition of strength

$$a_{1,\max} \le 3{,}63 \cdot t \cdot \sqrt{\frac{R_y}{V}} = 3{,}63 \cdot 0{,}5 \cdot \sqrt{\frac{240}{0{,}32}} = 49{,}71 \text{ cm;}$$

b) stiffness conditions

$$a_{2,\max} \le \frac{47{,}5 \cdot t}{\sqrt[3]{V^н}} = \frac{47{,}5 \cdot 0{,}5}{\sqrt[3]{0{,}23}} = 38{,}76\,\text{см.}$$

We choose the smaller value from the two ones $a_{1,\max}$ and $a_{2,\max}$

Then we calculate the width of the panel in the axes:

$$n_{кр,1} = \frac{\ell}{a_{\max}^{\min}} = \frac{5{,}5}{0{,}3876} = 14{,}19$$

Truncate the obtained value to the nearest larger integer. We accept $n_{кр,1} = 15$.

$$a_0 = \frac{\ell}{n_{кр,1}} = \frac{550}{15} = 36{,}67\text{ cm} < 38{,}76\text{cm (stiffness conditions).}$$

Accept the gap between the panels $\Delta_1 = 0{,}67$ cm (preferably within 3-5mm).

Structural width of the panel should be multiple of 0.5 cm.

$a = a_0 - \Delta_1 = 36{,}67 - 0{,}67 = 36\,\text{cm}$;

The height of the ribs inside the panel $h_0 = 20t = 20 \cdot 0{,}5 = 10\,\text{cm}$;

The width of the lower flanges of the panel is taken to be equal to $b = 8t = 8 \cdot 0,5 = 4\,\text{cm}$.

Cross section area of panel

$A = a \cdot t + 2(b \cdot t + h_0 \cdot t) = 36 \cdot 0,5 + 2(4 \cdot 0,5 + 10 \cdot 0,5) = 32\,\text{cm}^2$.

Determine the center of gravity of the panel cross-section:

Static moment relative to the X0-X0 axis (top of the deck)

$$S_{xo} = a \cdot t \cdot \frac{t}{2} + 2h_0 t\left(\frac{h_0}{2} + t\right) + 2bt\left(h_0 + \frac{t}{2} + t\right) =$$

$$= 36 \cdot 0,5 \cdot \frac{0,5}{2} + 2 \cdot 10 \cdot 0,5\left(\frac{10}{2} + 0,5\right) + 2 \cdot 4 \cdot 0,5\left(10 + \frac{0,5}{2} + 0,5\right) = 102,5\,см^3$$

Distance to neutral axis:

from the axis $X_0 - X_0$: $y_b = \frac{S_{xo}}{A} = \frac{102,5}{32} = 3,2\,\text{cm}$;

from the axis to the bottom of the rib $y_н = t + h_0 + t - y_b$

$y_н = 10 + 0,5 + 0,5 - 3,2 = 7,8\,\text{cm}$..

Under the condition of optimal selection of the cross -section of the ratio is

$$\frac{y_н}{y_в} \approx 2,5$$

The ratio is $\frac{y_н}{y_в} = \frac{7,8}{3,2} \approx 2,44$

The panel cross-section is shown in Fig.1, b.

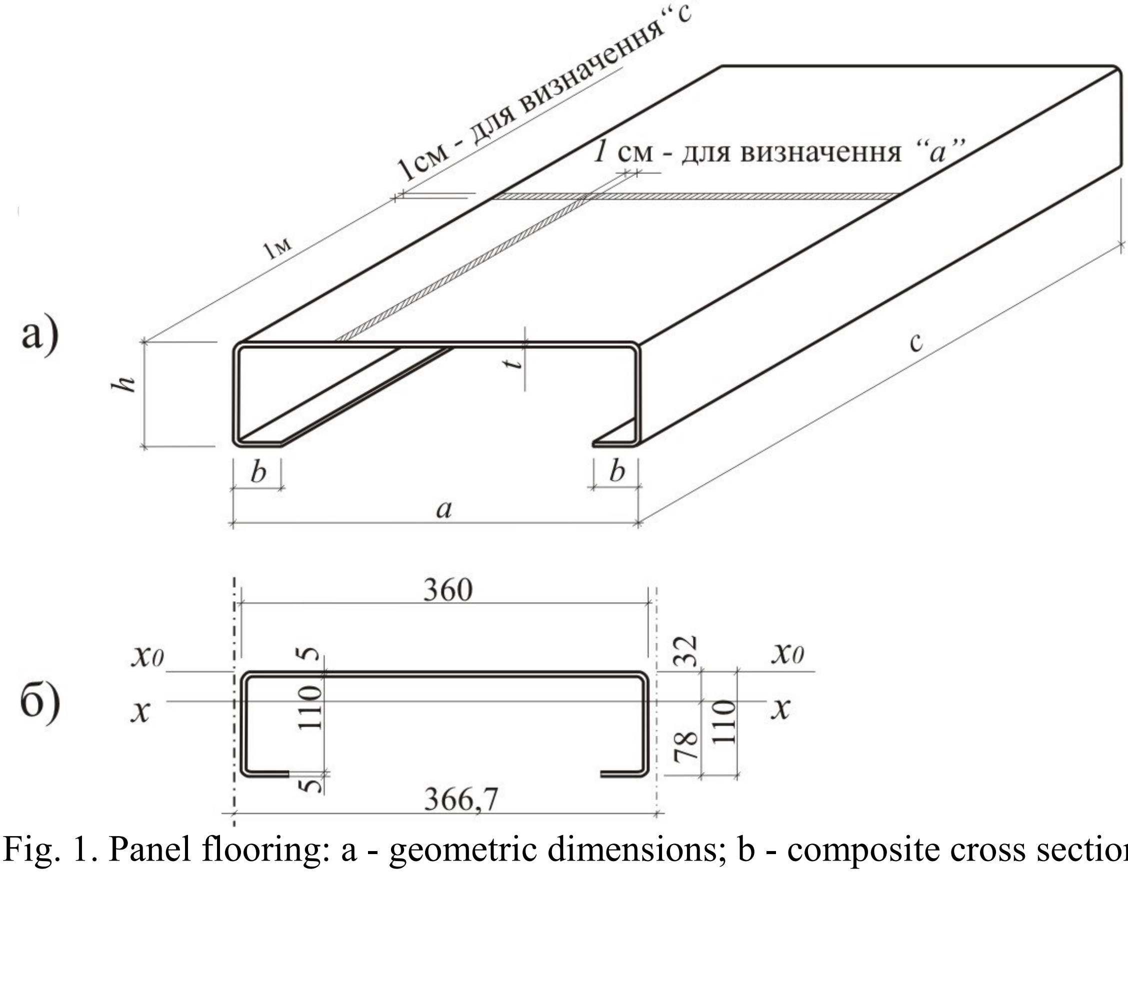

Fig. 1. Panel flooring: a - geometric dimensions; b - composite cross section

The moment of inertia of the panel section

$$I_x = \frac{at^3}{12} + at\left(y_b - \frac{t}{2}\right)^2 + 2\left[\frac{th_0^3}{12} + h_0 t\left(\frac{h_0}{2} + t - y_b\right)^2 + \frac{bt^3}{12} + bt\left(y_н - \frac{t}{2}\right)^2\right] =$$

$$= \frac{36\cdot 0,5^3}{12} + 36\cdot 0,5\left(3,2 - \frac{0,5}{2}\right)^2 +$$

$$+2\left[\frac{0,5\cdot 10^3}{12} + 10\cdot 0,5\left(\frac{10}{2} + 0,5 - 3,2\right)^2 + \frac{4\cdot 0,5^3}{12} + 4\cdot 0,5\left(7,8 - \frac{0,5}{2}\right)^2\right] = 521,35 см^4.$$

In practice own moment of deck and flanfes inertia are neglected due to smallness.

For example:

$$I_н = \frac{a\cdot t^3}{12} = \frac{36\cdot 0,5^3}{12} = 0,375\,\text{см}^4;\ I_п = 2\cdot\frac{b\cdot t^3}{12} = 2\frac{4\cdot 0,5^3}{12} = 0,083\,\text{см}^4,$$

It is less then 0,1% of $I_x = 521,35\,\text{см}^4$.

Modulus of panel cross-section:

$$W_b = \frac{I_x}{y_b} = \frac{521,35}{3,2} = 162,92\,\text{см}^3; \qquad W_н = \frac{I_x}{y_н} = \frac{521,35}{7,8} = 66,84\,\text{см}^3$$

Determining the length of the panel.

Distributed along panel load in kN / m at cm $a_0 = 36,7$ is

$$V = V_1^н \cdot a_0 \cdot \gamma_f = 0,23\cdot 36,7\cdot 1,4 = 11,82\ \text{кН/м} = 0,118\ \text{кN/cm}.$$

The maximum step of secondary beams (panel length) in terms of strength

$$c_{\max} = \sqrt{\frac{8\cdot W_н\cdot R_y}{V\cdot 10}} = \sqrt{\frac{8\cdot 66,84\cdot 240}{0,118\cdot 10}} = 329,78\,\text{см}.$$

Determine the number of panels:

$$n_{кр,2} = \frac{Z}{c_{\max}} = \frac{1300}{329,78} = 3,94$$

Truncate the resulting value to the nearest larger integer $n_{кр,2} = 4$. Then we accept the step of the secondary beams

$$c = \frac{Z}{n_{кр,2}} = \frac{1300}{4} = 325\,\text{см}$$

$c = 325$см $< c_{\max} = 329,78$ см.

Let's check up a sag of the panel at standard loading

$V^n = V_1^n \cdot a_0 = 23 \cdot 0,36 = 8,28$ кN/м = 0,0828 кN/cm.

Estimated sag of the panel between the ribs for $E = 210000$ MPa

$$f = \frac{5}{384} \cdot \frac{V^н \cdot c^4 \cdot 10}{E \cdot I_x} = \frac{5 \cdot 0,0828 \cdot 325^4 \cdot 10}{384 \cdot 210000 \cdot 521,35} = 1,099 \text{ см} < f_н = \frac{c}{250} = \frac{325}{250} = 1,3 \text{ см}.$$

Given thickness of the panel

$$t_{np} = \frac{A}{a_0} = \frac{32}{36,67} = 0,87 \text{ см}.$$

Mass of panel 1м^2

$g_{1m}^n = t_{np} \cdot \gamma_{cm} = 0,87 \cdot 78,5 = 68,3$ кг/м^2, where $\gamma_{st} = 78,5$ kg/м^2 – density of steel for sheet thicknessa $\delta = 10$ мм.

2. Calculation of the secondary beam

Initial date: the span of the secondary beam $\ell = 5,5$ m, the characteristic live load $V_1^н = 23$ kN / m2. The step of the secondary beams is taken by calculating the flooring panels $c = 3,25$ m.

The own weight of the secondary beam is taken within:

$g_b^n = 0,2...0,4$ kN / m2 = 20… 40.kg / m^2

Characteristic constant load from own weight of a flooring and secondary beam on 1 m^2 of the area

$$g_1^n = \left(g_{1m}^n + g_b^n\right) \cdot c \frac{1}{100} = \left(68,3 + 31,7\right) \cdot 325 \cdot \frac{1}{100} = 325 \text{ kN / cm} = 3.25 \text{ kN / m}$$

Estimated load from own weight

$g_1 = g_1^n \cdot \gamma_{f0} = \cdot 325 \cdot 1,05$=341.25 kN / m = 3.4125 kN / cm

Characteristic load per 1 m of secondary beam

$V^n = c \cdot V_1^n = 3,25 \cdot 23 = 74,75$ kN / m

Designed load per 1 m of secondary beam

$V = \gamma_f \cdot V^n = 1,4 \cdot 74,75 = 104,65$ kN / m

Designed total load

$q = g_1 + V = 3,41 + 104,65 = 108,06$ kN / m

.Estimated bending moment

$$M_{\max} = \frac{q\ell^2}{8} = \frac{108,06 \cdot 5,5^2}{8} = 408,60 \text{ kN·m} = 40860 \text{ kN cm}.$$

Response on support

$$F = \frac{q\ell}{2} = \frac{108,06 \cdot 5,5}{2} = 297,17 \text{ kN}.$$

The required modulus of beam cross-section

$$Wreq = \frac{M_{\max} \cdot 10 \cdot 100}{R_y} = \frac{40860 \cdot 10}{240} = 1702{,}5\ \text{cm}^3.$$

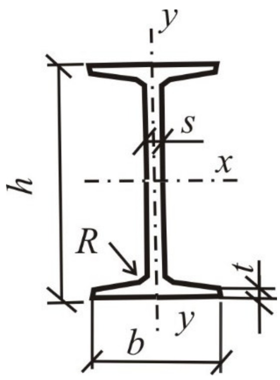

Fig. 2. To calculate the secondary beam

We accept the auxiliary beam from the closest to $Wreq$ by ГОСТ8239-72 (GOST 8239-72) (Table 4 of Annex III), I55Б2. It has у якого $W_x = 2296\,\text{cm}^3$, $I_x = 62790\,\text{cm}^4$, $s = 10\,\text{mm}$, $R = 24\,\text{mm}$, $b = 220\,\text{mm}$, $t = 15{,}5\,\text{mm}$, weight $g_1 = 97{,}9$ kg/m for 1m^2 area.

Control: $\frac{g_1}{c} = \frac{97{,}9}{3{,}25} = 30{,}12\,\text{kg/m}^2 < g_b^n = 31{,}7\,\text{kg/m}^2$.

Stresses in the beam are not checked, as $W_x > Wreq$ and $\frac{g_1}{c} < g_b^n$

Check the sag from the characteristic load

$V^n = c \cdot V_1^n = 3{,}25 \cdot 23 = 74{,}8$ kN/m = 0,748 kN/cm

$$f = \frac{5}{384}\frac{q^н l^4 10}{EI_x} = \frac{5 \cdot 78 \cdot 550^4 \cdot 10}{384 \cdot 210000 \cdot 62790 \cdot 100} = 0{,}705\,\text{cm} < f_n = \frac{l}{250} = \frac{550}{250} = 2{,}2\,\text{cm}.$$

3 Calculation of the main beam

Initial date: the span of the main beam $Z = 13$m, standard sag $f^n = \frac{Z}{400} = \frac{1300}{400} = 3{,}25\,\text{см}$ (there are no loads from rolling stock). Loads on the main beam are the responses of the secondary beams and the own weight of the main beam.

In this example, for the main beam of the middle row "B", the design loads are:

$F_1 = 2F = 2 \cdot 297{,}17 = 594{,}34\,\text{kN}$.

The characteristic uniform distributed load from the own weight of the main beam is taken depending on the span:

for $Z = 8...11$ м - $q_1^n = 2,5$ kN/m.

for $Z = 12...15$ m - $q_1^n = 3,5$ kN/m.

for $Z = 16...18$ m - $q_1^n = 5,0$ kN/m.

Designed load:

$q_1 = \gamma_{f_0} . q_1^n = 1,05 \cdot 3,5 = 3,675$ кН/м.

Structural scheme of main beam is shown on Fig. 3.

Response of support (without response of secondary beam supported by column)

$F_b = \frac{F_1 \cdot n}{2} + \frac{q_1 \cdot Z}{2} = \frac{594,34 \cdot 3}{2} + \frac{3,675 \cdot 13}{2} = 915,40$ kN, where

n – number of concentrated forces (responses of secondary beams supported by main beam)

$n = n_{кр,2} - 1 = 4 - 1 = 3$;

Z – main beam span;

g_1 – designed weight of 1m of the main beam.

Estimated bending moments:

$$M_1 = F_b \cdot c - \frac{q_1 \cdot c^2}{2} = 915,4 \cdot 3,25 - \frac{3,675 \cdot 3,25^2}{2} = 2956 \text{ kN·m};$$

$$M_2 = F_b \cdot 2c - \frac{q_1 \cdot (2c)^2}{2} - F_1 \cdot c =$$

$$= 915,40 \cdot 6,5 - \frac{3,675 \cdot 6,5^2}{2} - 594,34 \cdot 3,25 = 3941 \text{ kN·m};$$

$M_2 = M_{\max}$

Estimated shear forces:

$Q_1 = F_б = 915,40$ kN;

$Q_2^r = F_b - q_1 \cdot c = 915,4 - 3,675 \cdot 3,25 = 903,46$ kN;

$Q_2^l = Q_2^r - F_1 = 903,46 - 594,34 = 309,12$ kN;

$Q_3^r = F_b - F_1 - q_1 \cdot \frac{z}{2} = 915,40 - 594,34 - 3,675 \cdot 6,5 = 297,17$ kN;

$Q_3^l = Q_3^r - F_1 = 297,17 - 594,34 = -297,17$ kN.

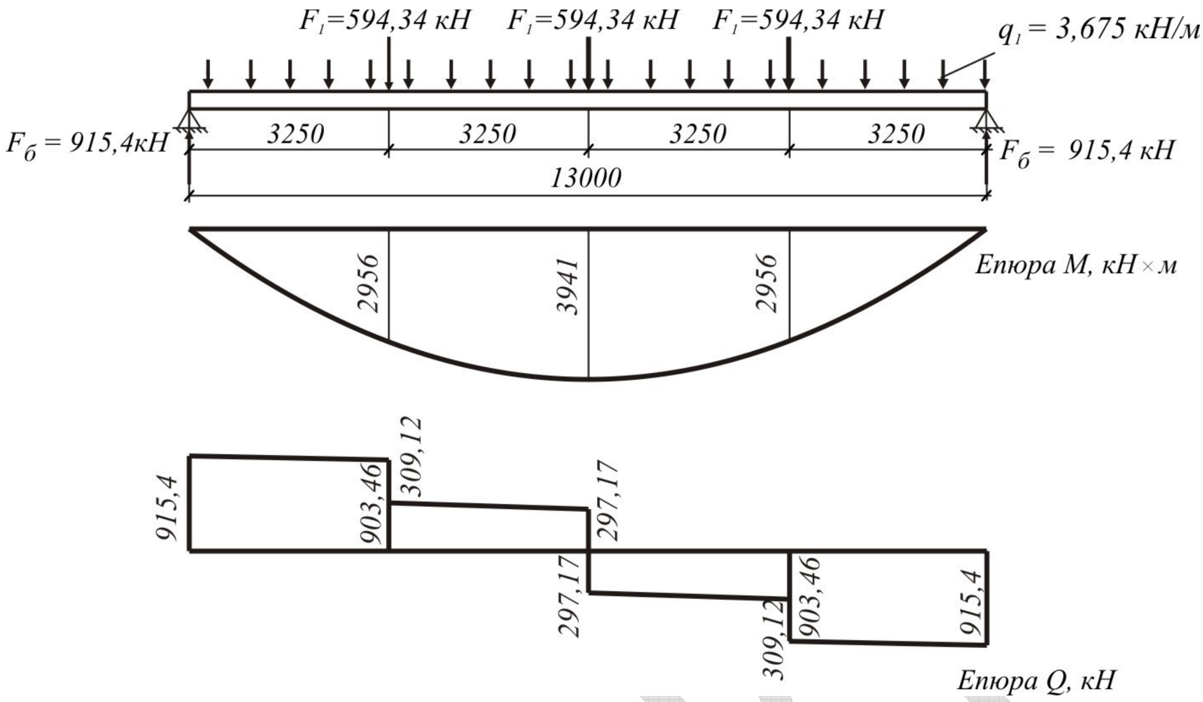

Fig. 3. Structural scheme and plots of forces acting in the cross sections of the main beam

Sections of the main beam are selected for two options: from steel of one brand (monometallic) and from steel of two brands (bimetallic).

The first variant - the main beam from brand of steel Ст3пс6, for a sheet of $t = 4...20$ mm, $R_y = 240\,\text{MPa}$

The required modulus of cross-section of the main beam:

$$W_{req} = \frac{M_{\max} \cdot 10}{R_y} = \frac{394100 \cdot 10}{240} = 16420\,\text{см}^3.$$

We assume κ_w - the wall ratio.

$$\kappa_w = \frac{z}{10} + (10...20) = \frac{1300}{10} + 10 = 140$$

Determine the optimal and minimum height of the main beam.

Optimal height from the condition of minimum weight:

$$h_{opt} = \sqrt[3]{\frac{3}{2}\kappa_w \cdot W_{req}} = \sqrt[3]{\frac{3}{2}140 \cdot 16420} = 151,1\,\text{cm}.$$

The minimum height from the condition of deflection is determined by one of the formulas:

$$h_{\min} = \frac{5}{24} \cdot \frac{R_y \cdot z}{E} \cdot \frac{z}{f_n} \cdot \left(\frac{g_1^n + V_1^n \cdot l}{\gamma_{f_0} \cdot g_1^n + \gamma_f \cdot V_1^n \cdot l} \right) =$$

$$= \frac{5}{24} \cdot \frac{240 \cdot 1300}{210000} \cdot \frac{1300}{3{,}25} \cdot \left(\frac{3{,}5 + 23 \cdot 5{,}5}{1{,}05 \cdot 3{,}5 + 1{,}4 \cdot 23 \cdot 5{,}5} \right) = 89{,}02 \text{ cm, where } \gamma_{f0} = 1{,}05 \text{ -}$$

where is the coefficient of reliability for the own weight of metal structures; γ_f - reliability factor for live load is given in the task form for the term project; the standard sag for beams

We accept the height of the wall within the optimal and greater than taking into account the standard width of the rolled steel according to table. 1 annex III - 1500 mm. With a shelf thickness of 20 mm, the approximate total height of the beam is $h = 150 + 2 \cdot 2 = 154$ cm (see Fig. 4, a).

a) b)

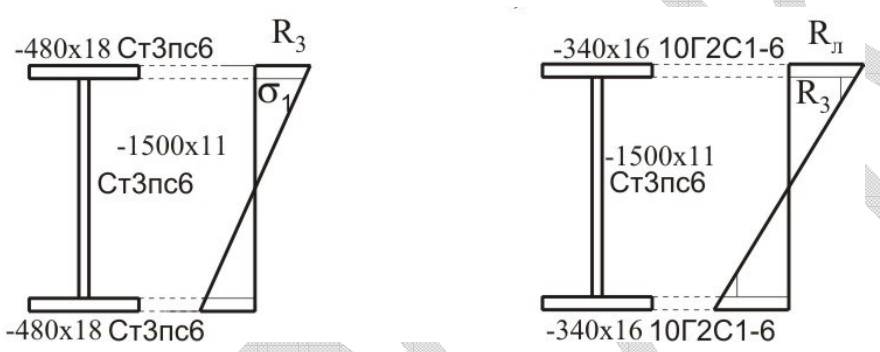

Fig. 4. To calculate the main beam: a) monometallic, b) bimetallic

We accept the height of the beam $h_w = 1500$ mm (according to Table 1 of Annex III).

Wall thickness:

a) from the condition of shear strength at Rs = 0,58.Ry = 0,58.240 = 139,2 MPa

$$t = \frac{1{,}2 \cdot Q_{\max} \cdot 10}{h_w \cdot R_s} = \frac{1{,}2 \cdot 915{,}4 \cdot 10}{150 \cdot 139{,}2} = 0{,}52 \text{ cm};$$

b) on the condition of local stability

$$t = \frac{h_w}{k_w} = \frac{150}{140} = 1{,}07 \text{ cm}.$$

We accept a wall with a standard thickness of $t_w = 11$ mm (according to tab. 1 of Annex III).

We accept the wall $(h_w \times t_w)$ -1500×11

The required moment of inertia of the section:

cm4.

Required moment of inertia of the beam:

$$I_{req} = W_{req} \cdot \frac{h_w + 4}{2} = 16420 \cdot \left(\frac{150+4}{2}\right) = 1264340\ \text{cm}^4.$$

Required moment of inertia of shelves

$$I_{sh} = I_{req} - I_w = 1264340 - \frac{1{,}1 \cdot 150^3}{12} = 954965\ \text{cm}^4,$$

where $I_w = -\frac{t_w \cdot h_w^3}{12}$

Required cross-section area of shelves

$$A_f = \frac{2I_{sh}}{(h_w+2)^2} = \frac{2 \cdot 954965}{(150+2)^2} = 82{,}66\ \text{cm}^2.$$

$$A_f = b_f \cdot t_f;\ b_f = 25 \cdot t_f^2;\ t_f = \sqrt{\frac{A_f}{25}} = \sqrt{\frac{82{,}66}{25}} = 1{,}82\ \text{см}.$$

We accept shelves made of universal steel of standard sizes (according to Table 1 of Annex III) - 480x18 mm, ie $A_f = 86{,}4\ \text{cm}^2$.

The rationality of the selection and layout of the cross section of the main beam is characterized by a parameter for low-carbon steels $\alpha = 0{,}5 \pm 0{,}05$.

For low-alloy steel beam $\alpha = 0{,}6 \pm 0{,}05$.

In this example

$$\alpha = \frac{A_w}{A_w + 2A_f} = \frac{h_w \cdot t_w}{h_w \cdot t_w + 2 \cdot b_f \cdot t_f} = \frac{150 \cdot 1{,}1}{150 \cdot 1{,}1 + 2 \cdot 48 \cdot 1{,}8} = 0{,}49$$

that is, within the optimal.

The moment of inertia of the composite section

$$I_x = \frac{t_w h_w^3}{12} + 2b_f t_f \left(\frac{h_w + t_f}{2}\right)^2 = \frac{1{,}1 \cdot 150^3}{12} + 2 \cdot 48 \cdot 1{,}8 \left(\frac{150+1{,}8}{2}\right)^2 = 1304843\ \text{cm}^4$$

Note: The eighn moment of inertia of the shelves is neglected due to the small value, for example, in this example: $I_n = \frac{48 \cdot 1{,}8^3}{12} = 23{,}33\ \text{см}^4$ and is $\frac{23{,}33}{1304843} \cdot 100\% = 0{,}0018\%$

Checking normal stress:

$$\sigma = \frac{M_{\max} \cdot 10y}{I_x} = \frac{394100 \cdot 10 \left(\frac{150 + 2 \cdot 1{,}8}{2}\right)}{1304843} = 231{,}96\ \text{МПа} < R_y = 240\ \text{МПа},$$

де $y = \left(h_w + 2t_f\right)/2$

Checking of shear stresses: static moment of half of section of a welded I-beam.

$$S_T = b_f t_f\left(\frac{h_w + t_f}{2}\right) + \frac{h_w t_w}{2}\cdot\frac{h_w}{4} = 48\cdot 1{,}8\left(\frac{150+1{,}8}{2}\right) + \frac{150\cdot 1{,}1}{2}\cdot\frac{150}{4} = 9651{,}51\,\text{см}^4\text{;MPa.}$$

The sag check is performed for the characteristic load forming the bending moment:

$$M^n = M_{\max}\frac{g_1^n + V_1^n\cdot l}{\gamma_{f_0}\cdot g_1^n + \gamma_f\cdot V_1^n\cdot l} = 3941\frac{3{,}5 + 23\cdot 5{,}5}{1{,}05\cdot 3{,}5 + 1{,}4\cdot 23\cdot 5{,}5} = 2834\text{kNm} = \quad =283400$$

kNcm.

The calculated deflection for the case of a complex loading can be determined by an approximate formula

$$f = \frac{M^n\cdot Z^2}{E\cdot I_x} = \frac{283400\cdot 1300^2}{210000\cdot 1304843} = 1{,}75\,\text{см} < \frac{1300}{400} = 3{,}25\,\text{см}.$$

We change the cross section of the shelves at a distance $\frac{Z}{6}$ from the support, ie $\frac{13}{6} = 2{,}17$ m:

$$M_{z/6} = 915{,}39\cdot 2{,}17 - \frac{3{,}675\cdot 2{,}17^2}{2} = 1977{,}74\,\text{kNm} = 197774\text{ kNcm;}$$

$$Q_{z/6} = 915{,}39 - 3{,}675\cdot 2{,}17 = 907{,}4\,\text{KN;}$$

$$W_{req}^{new} = \frac{M_{z/6}\cdot 10}{R_y} = \frac{197774\cdot 10}{240} = 8240{,}58\,\text{см}^3;$$

$$I_{req}^{new} = W_{req}^{new}\cdot\frac{hw + 2\cdot t_f}{2} = 8240{,}58\cdot\frac{150 + 2\cdot 1{,}8}{2} = 632876{,}54\,\text{см}^4;$$

$$I_f^{new} = I_{req}^{new} - I_w = 632876{,}5 - \frac{1{,}1\cdot 150^3}{12} = 323502\,\text{см}^4;$$

$$A_f^{new} = \frac{2I_f^{new}}{\left(h_w + t_f\right)^2} = \frac{2\cdot 323502}{(150+1{,}8)^2} = 28{,}07\,\text{см}^2.$$

The smallest width of a shelve of a beam from a condition of support of a flooring makes 180 mm, ie cm. $b_f^n = 18\,\text{cm}$

Keeping the height of the belt, we accept the changed cross section of the shelves - 180x18, ie $A_f^n = b_f^n\cdot t_f = 18\cdot 1{,}8$ 32.4 cm^2

The moment of inertia of the changed section

$$I_x^n = \frac{t_w h_w}{12} + 2A_f^n\left(\frac{h_w + t_f}{2}\right)^2 = \frac{1{,}1\cdot 150^3}{12} + 2\cdot 18\cdot 1{,}8\left(\frac{150+1{,}8}{2}\right)^2 = 682675\,\text{см}^4.$$

The greatest wall stresses in the place of change of sections of shelves:

$$\sigma_1^n = \frac{M_{z/6} 10\, y}{I_x^n} = \frac{197774 \cdot 10\left(\frac{150}{2}\right)}{682675} = 217{,}3\,\text{MPa} < R_y = 240 \text{ MPa; where } y = \frac{h_w}{2}.$$

$$\tau_1 = \frac{Q_{z/6} 10}{h_w t_w} = \frac{907{,}4 \cdot 10}{150 \cdot 1{,}1} = 54{,}99 \text{ MPa} < R_s = 139{,}2 \text{ MPa}.$$

Checking the reduced stresses

$$\sigma_{red} = \sqrt{\sigma_1^2 + 3\tau_1^2} = \sqrt{217{,}3^2 + 3 \cdot 54{,}99^2} = 237{,}25 <$$
$$< 1{,}15 R_y = 1{,}15 \cdot 240 = 276 MPa$$

Calculation of weld joint at section $\frac{Z}{6}$ for electrode Э42 with $R_{wf} = 180\,\text{MPa}$.

Minimal weld legs k_f are given in Table 1, where

$\Delta A_f = t_f (b_f - b_f^n) = 1{,}8 \cdot (48 - 18) = 54$ см².

$$k_f = \frac{1}{2} \cdot \frac{Q_{z/6} 10 A_f \left(\frac{h_w + t_f}{2}\right)}{I_x^n \beta_f R_{wf}} = \frac{1}{2} \cdot \frac{907{,}4 \cdot 10 \cdot 32{,}4 \left(\frac{150 + 1{,}8}{2}\right)}{682675 \cdot 1 \cdot 180} = 0{,}18\,\text{см},$$

where β_f – factor taken to equal $\beta_f = 1.$ for single pass automatic welding.

The leg of the weld is taken in accordance with table 1.

So for shelve sheets with a thickness of $t_f = 18$ mm of steel brand Ст3пс6, the recommended leg of the weld is $k_f = 7$, which is more than calculated ($k_f = 0{,}18$ мм).

The place of the actual change of the cross section of the shelves is attributed to the support at a distance from the theoretical place of change (2.16 m) by the value of:

$$a_1 = \frac{0{,}5 \cdot \Delta A_f \cdot R_y}{2\beta_f \cdot k_f \cdot R_{wf}} = \frac{0{,}5 \cdot 54 \cdot 240}{2 \cdot 1 \cdot 0{,}7 \cdot 180} = 25{,}71\,\text{см};$$

Where $\Delta A_f = \left(b_f - b_f^n\right) \cdot t_f = (48 - 18) \cdot 1{,}8 = 54$ cm²

We accept $a_1 = 26$ см.

Check of the general stability of a monometallic main beam.

Determined according to the requirements for a symmetrical I-beam with the parameters:

$$\frac{h}{b_f} = \frac{h_w + 2t_f}{b_f} = \frac{150 + 2 \cdot 1{,}8}{48} = 3{,}2, \text{ ie within } 1 \le \frac{h}{b} < 6;$$

$$\frac{b_f}{t_f} = \frac{48}{1{,}8} = 26{,}7, \text{ ie within } \frac{b_f}{t_f} < 35,$$

where h is the height of the beam in cm; b_f – width of shelve in cm; t_f – thickness of shelve.

We accept $l_{ef} = c$ the distance between the fastenings of the beam, which prevents the rotation of the cross section, equal to the step of the secondary beams $l_{ef} = c$= 3.25 m, and the maximum allowable distance at which the general stability is provided and its verification is not required, is:

$$\left|\frac{l_{ef}}{b_f}\right| = \left[0,35 + 0,0032\frac{b_f}{t_f} + \left(0,76 - 0,02\frac{b_f}{t_f}\right) \cdot \frac{b_f}{h_w + 2t_f}\right] \cdot \sqrt{\frac{E}{R_y}} =$$

$$= \left[0,35 + 0,0032 \cdot \frac{48}{1,8} + \left(0,76 - 0,02 \cdot \frac{48}{1,8}\right)\frac{48}{150 + 2 \cdot 1,8}\right] \cdot \sqrt{\frac{210000}{240}} = 14,97.$$

Actually at $l_{ef} = 325$ см $b_f = 48$ см;

$\frac{l_{ef}}{b} = \frac{325}{48} = 6,77 < \left|\frac{l_{ef}}{b_f}\right| = 14,97$, that is, the general stability is ensured and there is no need for verification.

Table 1

Restriction of the minimum leg of fillet weld on DBN B 2.6 - 163: 2010

Thickness of beam shelve, t_f мм	Manual weld		Automatic weld	
	Minimal weld leg k_f ,мм at the yield strength of steel σ_y (MPa)			
	till 430	431-580	Till 430	431-580
4-5	4	5	3	4
6-10	5	6	4	5
11-16	6	7	5	6
17-22	7	8	6	7
23-32	8	9	7	8
33-40	9	10	8	9
41-80	10	12	9	10

Check of local stability of a wall of a beam of the complicated section.

To ensure the local stability of the wall and the structural design of the joints of the secondary beams with the main provide structural transverse ribs in the step of the auxiliary beams c = 3.25 m (since it exceeds $2h_w = 300$ cm, it is necessary to set intermediate ribs). Check of local stability is carried out in the support (I) and middle (II) sections of the wall at points located at distances $\frac{h_w}{2} = \frac{150}{2} = 0,75$ m from the nearest to the support of the rib, shown in Fig. 5

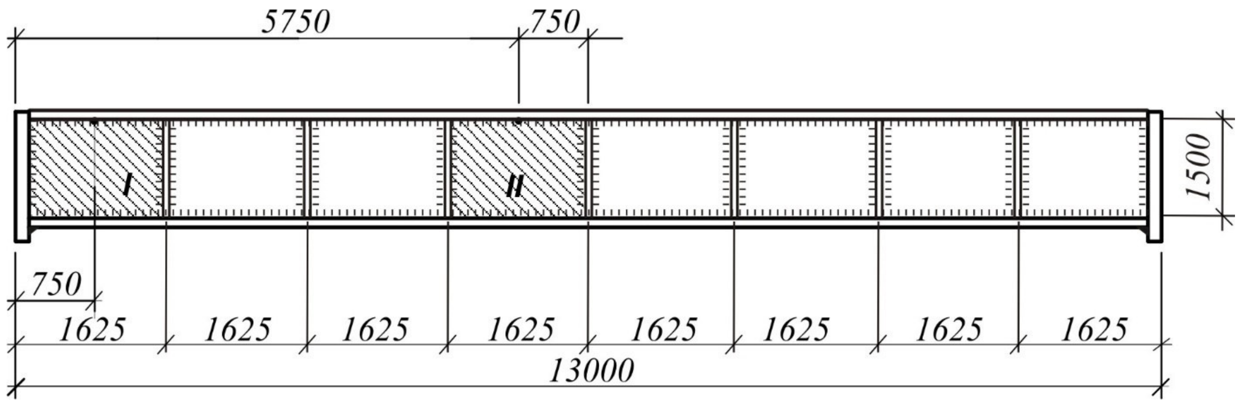

Fig. 5 Sectors I andта II

In the case of step ribs $a < h_w$, the positions of the control points are taken in the middle of the compartment between the ribs, and if $a > h_w$- at a distance from the left rib (to the support). In our case $a = c = 3{,}25\,\text{м} > h_w = 1{,}5\,\text{м}$. Determine the calculated forces at the points tested for local stability points.

Sector 1

$$M_{0,75} = 915{,}4 \cdot 0{,}75 - \frac{3{,}675 \cdot 0{,}75^2}{2} = 685{,}52\,\text{кН·м} = 68552\text{kN·см};$$

$$Q_{0,75} = 915{,}4 - 3{,}675 \cdot 0{,}75 = 912{,}64\ \text{kN}$$

$I_x^n = 682675\,\text{см}^4$ (див. стор. 12)

Estimated stresses:

$$\sigma_1 = \frac{M_{0,75} \cdot 10 \cdot y_1}{I_x^n} = \frac{68552 \cdot 10 \cdot 75}{682675} = 75{,}31\text{MPa, де } y_1 = \frac{h_w}{2} = \frac{150}{2} = 75\,\text{см}.$$

$$\tau_1 = \frac{Q_{0,75} \cdot 10}{h_w \cdot t_w} = \frac{912{,}64 \cdot 10}{150 \cdot 1{,}1} = 55{,}31\,\text{MPa}$$

To calculate the critical normal stresses, determine the parameter δ.

$$\delta = \beta \frac{b_f}{h_{ef}} \left(\frac{t_f}{t_w}\right)^3 = 0{,}8 \cdot \frac{48}{150} \left(\frac{1{,}8}{1{,}1}\right)^3 = 1{,}12;$$

where the coefficient $\beta = 2$ is for undercrane beam with unwelded rail; $\beta = \infty$ is for beams with flooring attached to the upper shelve; $\beta = 0{,}8$- for other beams.

From table 2 on linear interpolation we define parameter $c_{cr} = 31{,}704$ according to value δ.

Conditional flexibility of the wall:

$$\bar{\lambda}_w = \frac{h_w}{t_w}\sqrt{\frac{R_y}{E}} = \frac{150}{1{,}1}\sqrt{\frac{240}{210000}} = 4{,}6.$$

Critical normal stress:

$$\sigma_{cr} = \frac{c_{cr} \cdot R_y}{\bar{\lambda}_w^2} = \frac{31{,}704 \cdot 240}{4{,}6^2} = 359{,}6\,\text{МПа}.$$

To calculate the critical shear stresses, determine the ratio of the larger side of the sector d to the smaller h_w (step of ribs 1.625 m, wall height 150 cm),

$\mu = \frac{162,5}{150} = 1,08$ i $\bar{\lambda}_{ef} = \frac{h_w}{t_w}\sqrt{\frac{R_y}{E}} = \frac{150}{1,1}\sqrt{\frac{240}{210000}} = 4,6$, ie $\bar{\lambda}_{ef} = \bar{\lambda}_w$, since the smaller side of the compartment. $d \le h_w$

Table 2

Coefficient c_{cr} for welded beams

δ	< 0,8	1,0	2,0	4,0	6,0	10,0	>30
c_{cr}	30	31,5	33,2	34,6	34,8	35,1	35,5

Estimated shear resistance of steel $R_s = 0,58 \cdot R_y = 0,58 \cdot 240 = 139,2$ MPa.

Critical shear stress:

$$\tau_{cr} = 10,3\left(1 + \frac{0,76}{\mu^2}\right)\cdot\frac{R_s}{\bar{\lambda}_{ef}} = 10,3\left(1 + \frac{0,76}{1,08^2}\right)\cdot\frac{139,2}{4,6^2} = 111,9 \text{ MPa.}$$

Check of local stability of a sector I at support of secondary beams on stiffening ribs, ie at absence in a beam of the concentrated forces on the top shelve and local stress $\sigma_{loc} = 0$, we carry out by the formula:

$$\sqrt{\left(\frac{\sigma_1}{\sigma_{cr}}\right)^2 + \left(\frac{\tau_1}{\tau_{cr}}\right)^2} = \sqrt{\left(\frac{75,31}{359,6}\right)^2 + \left(\frac{55,31}{111,9}\right)^2} = 0,537 < 1.0.$$

Thus, the local stability of sector I is provided.

Sector 2

When supporting the secondary beams on the stiffeners, i.e. $\sigma_{loc} = 0$, the calculated forces in cross section:

$$M_{5,625} = 915,40 \cdot 5,625 - 594,34(5,625 - 3,25) - \frac{3,675 \cdot 5,625^2}{2} = 3679,42 \text{ кN·м} =$$

=367942кN·см;

$Q_{5,625} = 915,40 - 594,34 - 3,675 \cdot 5,625 = 300,39$ кN;

$I_x = 1304843$ см^4.

Estimated stresses at the test point:

$$\sigma_2 = \frac{M_{5,625} \cdot 10 \cdot y_1}{I_x} = \frac{367942 \cdot 10 \cdot 75}{1304843} = 211,5 \text{ МПа;}$$

$$\tau_2 = \frac{Q_{5,625} \cdot 10}{h_w \cdot t_w} = \frac{300,39 \cdot 10}{150 \cdot 1,1} = 18,20 \text{ МПа.}$$

Critical stresses of loss of stability σ_{cr} and τ_{cr} we accept according to check of a sector 1 as on all parameters sectors are identical.

$$\sqrt{\left(\frac{\sigma_2}{\sigma_{cr}}\right)^2+\left(\frac{\tau_2}{\tau_{cr}}\right)^2}=\sqrt{\left(\frac{211,5}{359,6}\right)^2+\left(\frac{18,20}{111,9}\right)^2}=0,61<1,0.$$

Thus, the structurally accepted arrangement of the stiffeners provides local stability of the beam wall and the 2nd sector.

At the height of the main beams having $\overline{\lambda}_w \geq 5,5$, in addition to the transverse ribs, also provide a longitudinal stiffening rib. Check of local stability in this case should be carried out according to the corresponding formulas of item 7.7 DBN B 2.6 - 163: 2010. When supporting the secondary beams on the upper shelve of the main beams in these places set the transverse stiffeners, local stresses do not occur and check the local stability is carried out as described above.

The second option is a bimetallic main beam. Wall from steel Ст3пс6 with $R_y = 240$ MPa (thickness is 4-20 mm); shelves from steel 10Г2С1-6 of 10-20 mm thick from $R_{y1} = 345$ MPa.

Determining the height of the main beam and the cross section of the wall is similar to the first option, ie take $h_w = 150$ см, $t_w = 1,1$ см.

Part of the bending moment, which is perceived by the wall of the beam:

$$M_w = R_y \cdot \frac{t_w \cdot h_w^2}{4 \cdot 10}\left[1-\frac{1}{3}\left(\frac{R_y}{R_{y1}}\right)^2\right],$$

Where $R_y = 240$ MPa (for Ст3пс6) the calculated resistance of carbon steel, and $R_{y1} = 345$ MPa (for 10Г2С1-6) - the calculated resistance of alloy steel.

$$M_w = 240\frac{1,1\cdot 150^2}{4\cdot 10}\left[1-\frac{1}{3}\left(\frac{240}{345}\right)^2\right]=124545 \text{ кN·см.}=1245,25\text{kNm}$$

The proportion of bending moment, which is perceived by the belts of the beam:

$M_f = M_{max} - M_w = 394100 - 124545 = 269555$ кN·см.

The required cross-sectional area of steel shelves 10Г2С1-6

$$A_f = \frac{M_n \cdot 10}{R_{y1}\left(h_w + t_f\right)} = \frac{269555\cdot 10}{345(150+1,8)} = 51,47 \text{ см}^2.$$

$$t_f = \sqrt{\frac{A_f}{25}} = \sqrt{\frac{51,47}{25}} = 1,43 \text{ см}.$$

$b_f = \dfrac{A_f}{t_f} = \dfrac{51,47}{1,6} = 32,16$ см. Af new= 32*1,4=44.8 cm^2

We accept $t_f = 16$ мм (according to table. 1 of appendix III).

We accept standard strips - 340x16 mm, (according to table. 1 of appendix III); $A_f = 54,4$ см^2, the coefficient of rationality of the selected cross section:

$$\alpha = \frac{A_w}{Aw + 2A_f} = \frac{1{,}1 \cdot 150}{1{,}1 \cdot 150 + 2 \cdot 34 \cdot 1{,}6} = 0{,}602 = 0{,}6 \pm 0{,}05$$, ie accepted for alloy steel is optimal. Aw=hw*tw alpha is in 0,55….0,65

The moment of inertia of the selected section of the bimetallic beam:

$$I_x = \frac{t_w h_w^3}{12} + 2b_f t_f \left(\frac{h_w + t_f}{2}\right)^2 = \frac{1{,}1 \cdot 150^3}{12} + 2 \cdot 34 \cdot 1{,}6 \left(\frac{150 + 1{,}6}{2}\right)^2 = 934500{,}6 \text{ см}^4$$

Bearing capacity of the composite cross section by bending moment:

$$M_{bc} = R_{y1} \cdot A_f \left(h_w + t_f\right)\frac{1}{10} + R_y \cdot \frac{t_w \cdot h_w^2}{4 \cdot 10}\left[1 - \frac{1}{3}\left(\frac{R_y}{R_{y1}}\right)^2\right] =$$

$$= 345 \cdot 54{,}4 \cdot \left(150 + 1{,}6\right)\frac{1}{10} + 240\frac{1{,}1 \cdot 150^2}{4 \cdot 10}\left[1 - \frac{1}{3}\left(\frac{240}{345}\right)^2\right] = 409068{,}3 \text{ кН·см} >$$

$> M_{\max} =$ 394100 кН·см.

Mbc=344755.38kNcm=3447,55kNm>Mmamx=2458kNm

The obligatory variant (after calculation of a welded I-beam) for the main beam should be considered selection of section from I-beams with parallel faces of shelves (wide-shelf) according to ТУ 14-2-24-72.

W_{req} =16400 см³ (in our case it is impossible to accept a rolling beam from steel of a strength class С245).

Therefore, we choose a rolled beam made of alloy steel class С345.

$$W_{req} = \frac{M_{\max} \cdot 10}{R_{y1}} = \frac{394100 \cdot 10}{335} = 11764 \text{ см}^3.$$

According to the range, we accept I 100Б4 ГОСТ 8239-72 (Table 3, Annex III), which has $W_x = 12940$ см³, $I_x = 655400$ см⁴, маса $g^t = 314{,}5$ kg/м.

Weight of a rolling beam

$g_2 = g^m \cdot Z \cdot k_k = 314{,}5 \cdot 13 \cdot 1{,}05 = 4293$ kg;

where k_k - design factor, in the welded beam k_k = 1.25 (for stiffeners, welding); in the rolling beam k_k = 1.05 (only for support ribs, as intermediate are absent).

Check the sag of the rolling beam by the formula:

$$f = \frac{M^n \cdot Z^2}{E \cdot I_x} = \frac{283400 \cdot 1300^2}{200000 \cdot 655400} = 3{,}65 > f^n = 3{,}25 \text{ см};$$

where $E = 2 \cdot 10^5$ MPa for alloy steels of class C345, ie the stiffness condition is not met. The calculations take into account the bulk density of steel $\rho = 7{,}85 \text{ g} / \text{cm3}$ == 0.00785 kg / cm3. Weight of monometallic beam

$$g_1 = \left(2b_f t_f + h_w t_w\right) z \rho k_k =$$
$$= \left(2 \cdot 48 \cdot 1{,}8 + 150 \cdot 1{,}1\right) \cdot 1300 \cdot 0{,}00785 \cdot 1{,}25 = 4309 \text{ kg.}$$

$$g_2 = \left(2b_f t_f + h_w t_w\right) z \rho k_k = \\ = \left(2 \cdot 32 \cdot 1,4 + 125 \cdot 0,9\right) \cdot 1300 \cdot 0,00785 \cdot 1,25 = 4309 \text{ kg}$$

3. Calculation of the connection of the secondary beam to the main beam

The connection of the secondary beam to the main one is carried out by means of a bolted joint on bolts of normal precision. Preliminaty we accept pre-bolts M22 (($d_b = 22$ mm) class of strength 4.6 in the holes $d_h = 24$ mm.

2. Estimated resistance to shear $R_{bs} = \gamma_b \cdot R_{bs}^T = 0,9 \cdot 150 = 135$ MPa,

3. Where $\gamma_b = 0,9$ is the coefficient of operating conditions of the bolted connection, equal to the number of bolts in the package less than 10

4. R_{bs}^T - bolt resistance according to table 3 of Annex II

$R_{bp} = \gamma_b \cdot R_{bp}^T = 0,9 \cdot 360 = 324$ MPa**,**

5. where R_{bs}^T - the crush resistance according to table. 4 of Annex II.

6. Determine the bearing capacity of the 1st bolt:

1. a) on the shear

$$N_1 = \frac{\pi d_b^{\,2}}{4} \cdot n_s \cdot R_{bs} \cdot \frac{1}{10} = \frac{3,14 \cdot 2,2^2}{4} \cdot 1 \cdot 135 \cdot \frac{1}{10} = 51,3 \text{ kN;}$$

b) on the crush

$$N_1 = s \cdot d_b \cdot R_{bp} = 1 \cdot 2,2 \cdot \frac{324}{10} = 71,28 \text{ kN.}$$

The required number of bolts

$$n_b = \frac{F}{N_{1\min}} = \frac{297,17}{51,3} = 5,79$$, , we accept 6 bolts.

In this variant it is necessary to carry out additional check of a wall of a beam weakened on a support by holes of $d_h = 24$ mm for bolts of $d_b = 22$ mm and cutouts of shelves with sections of a wall.

Estimated height for the accepted I-beam I 55B2:

$h_1 = h_b - 2(t + R + \Delta) = 54,7 - 2(1,55 + 2,4 + 0,5) = 50,25$ см

=44,3-2*(1,1+2,1+0,5)=36,9cm

where t_f is the thickness of the shelf, R is the radius of curvature, Δ is the thickness of the shelf.

The weight of 1 m of beam is 97.9 kg, then the distributed load from its own weight is equal to the mass of $\frac{97,9}{3,25} = 30,12$ kg / m2 = 0.301 kN / m2.

We accept rounded $h_1 = 50$ cm and we check up the basic section of a beam on a wall in section where fixing bolts are placed.

$$\tau = \frac{F10}{(h_1 - n_b\, d_h)s} = \frac{297{,}17 \cdot 10}{(50{,}25 - 6 \cdot 2{,}4) \cdot 1} =$$ s=7,5mm=0,75cm

$$= 83{,}47\text{MPa} < R_s = 0{,}58\, R_y = 0{,}58 \cdot 240 = 139{,}2\text{MPa}$$

$$a = \frac{b_f}{2} + 1{,}5 = \frac{48}{2} + 1{,}5 = 25{,}5\,\text{cm};$$

where 1.5 cm - the gap between the belts of the main and secondary beams.

Bending moment in section II

$$M_{II} = F \cdot a = 297{,}17 \cdot 25{,}5 = 7577{,}8\,\text{kN см};$$

$$W_{II} = \frac{s \cdot h_1^2}{6} = \frac{1 \cdot 50{,}25^2}{6} = 420{,}84\,\text{см}^3;$$

Wii=0,78*(36,9)^2/6=177cm^3

$$\sigma_{II} = \frac{M_{II} \cdot 10}{W_{II}} = \frac{7577{,}8 \cdot 10}{420{,}84} = 180{,}06\text{MPa} < R_y = 240\text{MPa};$$

The strength is not provided so we accept I45Б1

$$\tau_{II} = \frac{F10}{h_1\, s} = \frac{297{,}17 \cdot 10}{50{,}25 \cdot 1} = 59{,}14\text{MPa};$$

$$\sigma_{red} = \sqrt{\sigma_{II}^2 + 3\tau_{II}^2} = \sqrt{180{,}06^2 + 3 \cdot 59{,}14^2} = 207{,}15\,\text{MPa} <$$
$$< 1{,}15 R_y = 1{,}15 \cdot 240 = 276\,\text{MPa}$$

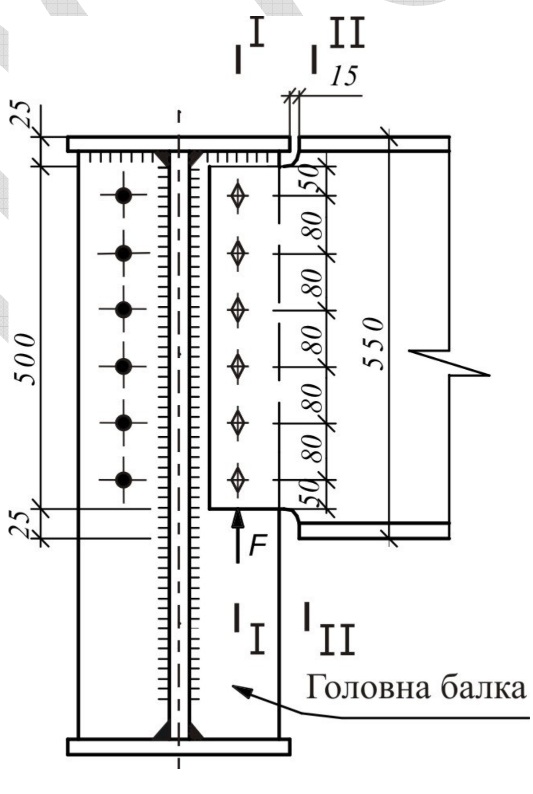

Fig. 6 Connection the secondary beam to the main beam

Calculation of the coupling of the main beam with the column

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Option 1. Surface conjugation (see Fig. 7). The support reaction of the main beam is transmitted to the support plate of the column through the crumpling of the end face of the support rib. In the support rib, the design will be the section 1-1, working on the wrinkling, as the calculated resistance to the wrinkling of the end is greater than the compression.

Required cross-sectional area of the support rib:

$$A_{r,req} = \frac{(F_b + 0{,}5F_1)\cdot 10}{R_p} = \frac{(915{,}40 + 0{,}5\cdot 594{,}34)\cdot 10}{324} = 37{,}42\,\text{см}^2,$$

where F_b is responce of the main beam;

We accept support rib - 200×20 (by width of the shelve) so $b_r = 20\,\text{см}; t_r = 2\,\text{см}$.

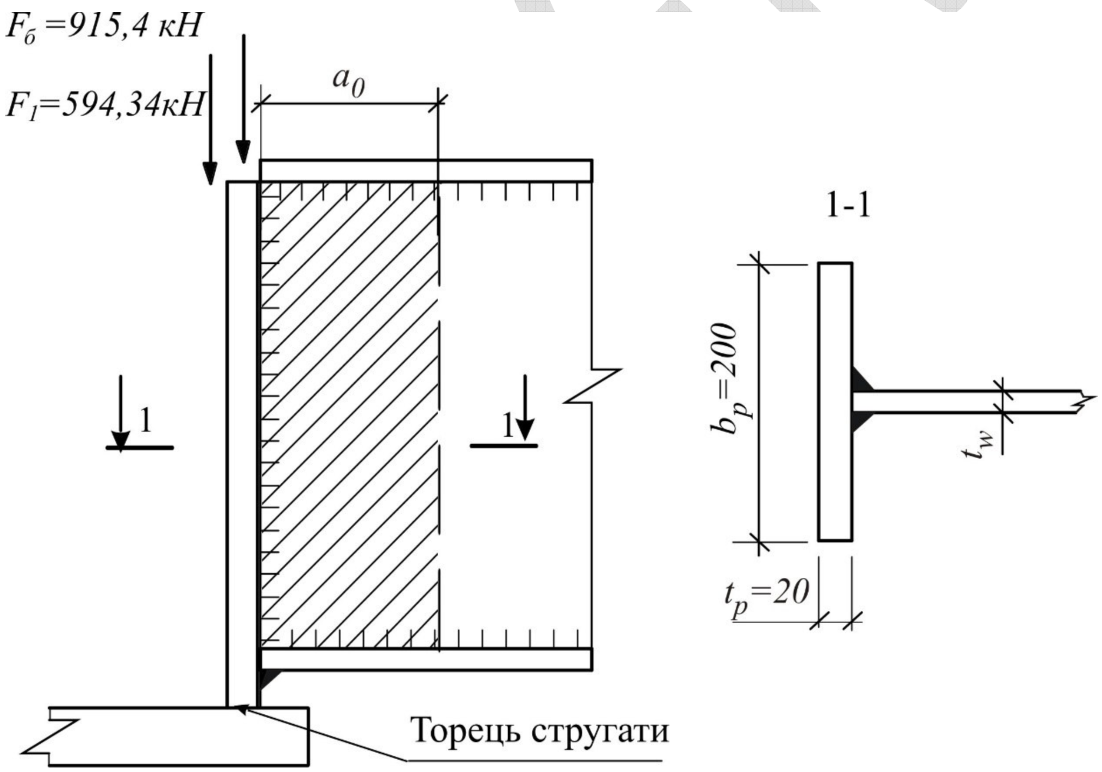

Fig. 7 Joint of the surface connection of the main beam with the column

Check the conditional reference section 1-1 for stability from the plane of the beam (relative to the axis X-X):

$$A_r = b_r\cdot t_r + 0{,}65\cdot t_w^2\sqrt{\frac{E}{R_y}} = 20\cdot 2 + 0{,}65\cdot 1{,}1^2\cdot\sqrt{\frac{210000}{240}} = 63{,}26\,\text{см}^2;$$

$$I_{x1} = \frac{b_r^3 \cdot t_r}{12} = \frac{2 \cdot 20^3}{12} = 1333\text{ см}^4;\ i_x = \sqrt{\frac{I_{x1}}{A_r}} = \sqrt{\frac{1333}{63,26}} = 4,59\text{ см};$$

$$\lambda_x = \frac{h_w}{i_x} = \frac{150}{4,59} = 32,67;$$

$\varphi_x = 0,921$ (from Table 3 by interpolation);

$$\sigma = \frac{\left(F_b + 0,5 \cdot F_1\right) \cdot 10}{\varphi_x \cdot A_r} = \frac{\left(915,4 + 0,5 \cdot 594,34\right) \cdot 10}{0,912 \cdot 63,26} = 210,18\text{ MPa} < R_y = 240\text{ Mpa}$$

$$a_o = 0,65 t_w \sqrt{\frac{E}{R_y}}$$

Table 3

Coefficent φ for stell element (ДБН В 2.6 - 163:2010) with R_y,Mpa

λ	200	240	280	320	360	400
10	0,988	0,987	0,985	0,984	0,983	0,981
20	967	962	959	955	952	949
30	939	931	924	917	911	905
40	906	894	883	873	863	854
50	869	852	836	822	809	796
60	827	805	785	766	749	721
70	782	754	724	687	654	623
80	734	686	641	602	566	532
90	665	612	565	522	483	447
100	599	542	493	448	408	369
110	537	478	427	381	338	306
120	479	419	366	321	287	260
130	425	364	313	276	247	223
140	376	315	272	240	215	195
150	328	276	239	211	189	171

Note. Intermediate values φ for λ and R_y, and, which are not in the table, should be determined by linear interpolation.

Annex I

Mechanical characteristics of steel

Table 1

Steel Brand accordance to steel class to according GOST 27772-88 (Table 51 ДБН В2.6 - 163:2010 II-23-81*)

Class of steel ГОСТ27772	Brand of steel	
	Steel brands and thickness of rolling steel, мм	ГОСТ або ТУ
C235	ВСт3кп2 ВСт3кп2-1	ГОСТ 380-94 ТУ I4-I-3023
C245	ВСт3пс6 (sheet – to 20, shaped – to 30) ВСт3пс6-1	ГОСТ 380 ТУ I4-I-3023
C255	ВСт3сп5, ВСт3Гпсб, ВСт3пс6 (sheet – up 20 to 40, shaped – up to 30) ВСт3сп5-1, ВСт3Гпс5-1	ГОСТ 380 ТУ I4-I-3023
C275	ВСт3пс6-2	ТУ I4-I-3023
C285	ВСт3сп5-2, ВСт3Гпс5-2	ТУ I4-I-3023
C345, C345T	09Г2	ГОСТ 19281 ГОСТ 19282
	09Г2С 15ХСНД (sheet to 10, shaped to 20) 14Г2 (sheet, shaped to 20)	ГОСТ 19282
	12Г2С гр.1	ТУ I4-I-4323
	09Г2 гр.1, 09Г2 гр.2, 09Г2С гр.1, 14Г2 гр.1 (shaped – to 20)	ТУ I4-I-3023
C375	12Г2С гр.2	ТУ I4-I-4323
	09Г2С гр.2, 14Г2 гр.1 (shaped – Up20), 14Г2 гр.2 (shaped – to 20)	ТУ I4-I-3023
	14Г2 (sheet, shaped – Up20) 10Г2С1, 15ХСНД (sheet – Up10, shaped – Up20), 10ХСНД (sheet – to 10, shaped – unlimeted)	ГОСТ I928I, ГОСТ 19282

Table 2

Regulatory and design trength for tension, compression and bending of sheet and shaped rolling in accordance with GOST 27772-88 for steel structures of buildings and structures

Table 51* ДБН В 2.6 - 163:2016

Steel class	thickness мм	Design strength of rolled steel, MPa			
		Steel sheet		Shaped steel	
		R_y	R_u	R_y	R_u
C235	From 2 to 20	230	350	230	350
	Up 20 to 40	220	350	220	350
	Up 40 to 100	210	350	-	-
	Up 100	190	350	-	-
C245	From 2 to 20	240	360	240	360
	Up 20 to 30	-	-	230	360
C255	From 2 to 3,9	250	370	-	-
	From 4 to 10	240	370	250	370
	Up 10 to 20	240	360	240	360
	Up 20 to 40	230	360	230	360
C275	From 2 to 10	270	370	270	380
	Up 10 to 20	260	360	270	370
C285	From 2 to 3,9	280	380	-	-
	From 4 to 10	270	380	280	390
	Up 10 to 20	260	370	270	380
C345	From 2 to 10	335	480	335	480
	Up 10 to 20	315	460	315	460
	Up 20 to 40	300	450	300	450
	Up 40 to 60	280	440	-	-
	Up 60 to 80	270	430	-	-
	Up 80 to 160	260	420	-	-
C345T	From 4 to 10	335	460	335	460
C375	From 2 to 10	365	500	365	500
	Up 10 to 20	345	480	345	480
	Up 20 to 40	325	470	325	470

Annex II

Таблиця 1

Welding materials corresponding to the strength of the steel elements, standard support of the weld metal, the calculated support of the metal of the corner joints

<table>
<tr><td rowspan="4">Class of steel</td><td colspan="4">Welding materials</td></tr>
<tr><td>under the flux</td><td colspan="2">in carbon dioxide (according to GOST 8050-85) or in its mixture with argon (GOST 10157-79 *)</td><td rowspan="3">coated electrodes GOST 9467-75 *</td></tr>
<tr><td colspan="3">Brand</td></tr>
<tr><td>flux (according to GOST 9087-81 *)</td><td colspan="2">сварного дроту (ГОСТ 2246-70*)</td></tr>
<tr><td>C235, C245, C255,C275, C285, 20,ВСт3кп,ВСт3пс, ВСт3сп</td><td>АН-348-А, АН-60</td><td>Св-08А, Св-08ГА</td><td rowspan="2">Св-08Г2С</td><td>Э42,Э46</td></tr>
<tr><td>C345, C345Т, C375, C375Т, C390, C390Т, C390К, C440, 16Г2АФ, 09Г2С</td><td>АН-47, АН-43, АН-17-М, АН-348-А[1]</td><td>Св-10НМА, Св-10Г2, Св-08ГА, Св-10ГА</td><td>Э50</td></tr>
<tr><td>C345К</td><td>АН-348-А</td><td>Св-08Х1ДЮ</td><td>Св-08 ХГ2СДЮ</td><td>Э50А</td></tr>
<tr><td>C235, C245, C255, C275, C285, ВСт3кп, ВСт3пс, ВСт3сп</td><td>АН-348-А</td><td>Св-08А , Св-08ГА</td><td rowspan="2">Св-08Г2С</td><td>Э42А, Э46А</td></tr>
<tr><td>C345, C345Т, C375, C375Т, 09Г2С</td><td>АН-47, АН-43, АН-348-А</td><td>Св-10НМА, Св-10Г2, Св-08ГА, Св-10ГА</td><td>Э50А</td></tr>
</table>

Таблиця 2

Нормативні та розрахункові опори металу швів зварних з`єднань з кутовими швами

Welding materials		R_{wun}, MPa	R_{wf}, MPa
Electrode type (GOST 9467-75)	Wire brand		
Э42, Э42А	Св-08, Св-08А	410	180
Э46, Э46А	Св-08ГА	450	200
Э50, Э50А	Св-10ГА, Св-08Г2С,Св-08Г2СЦ, ПП-АН8, ПП-АН3	490	215

Table 3

Design strength of bolts to stretch and shear, MPa

stress strain state	Symbol	Design strength MPa of bolt class of strength						
		4.6	4.8	5.6	5.8	6.6	8.8	10.9
Shear	R_{bs}	150	160	190	200	230	320	400
Tension	R_{bt}	170	160	210	200	250	400	500

Table 4

Design bearing strength of bolted elements Rbp, MPa

Temporary resistance of steel of connected elements, R_u, MPa	Design bearing strength of bolted elements Rbp	
	Class of accuracy A	Class of accuracy B and C, high-strength bolts without adjustable tension
360	475	430
365	485	440
370	495	450
380	515	465
390	535	485
400	560	505
430	625	565
440	650	585
450	675	605
460	695	625
470	720	645
480	745	670
490	770	690
500	795	710
510	825	735
520	850	760
530	875	780
540	905	805
570	990	880
590	1045	930

Annex III

Table 1

Rolled sheet assortment

Type of rolling steel	Hot-rolled strips steel ГОСТ 103-76	Low-alloyed plate strip universal steel ГОСТ 82-70	Rolled steel plates ГОСТ 10903 -74
Width of detail, мм	b≥200 мм	200мм≥b ≥1050мм	b≤1050мм
Standard recommended width	30 38 40 45 50 56 60 65 70 75 80 85 90 95 100 105 110 120 125 130 140 150 160 170 180 190 200	220 250 280 300 320 340 360 380 400 420 450 480 500 530 560 600 630 650 670 710 750 800 850 900 950 1000 1050	1250 1400 1500 1600 1700 1800 1900 2000 2100 2200 2300 2400 2500
Recommended thickness, mm	4 5 6 7 8 9 10 11 12 14 16 18 20 22 25 28 30 32 36 40 45 50 55 6		

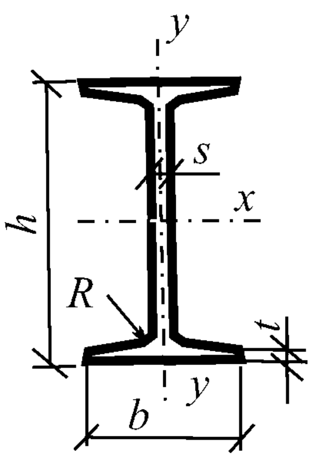

Table 2

GOST 26020-83 Hot-rolled steel I-beam with parallel flange edges. Assortment.

Profile Number	h	b	s	t	r	Cross - section area, $см^2$	Mass 1м, кг	Reference values for axes						
								x - x				y - y		
	мм							I_x, $см^4$	W_x, $см^3$	S_x, $см^3$	i_x, см	I_y, $см^4$	W_y, $см^3$	i_y, см
Normal I-beam														
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
10Б1	100	55	4,1	5,7	7	10,32	8,1	171	34,2	19,7	4,07	15,9	5,8	1,24
12Б1	117,6	64	3,8	5,1	7	11,03	8,7	257	43,8	24,9	4,83	22,4	7,0	1,42
12Б2	120	64	4,4	6,3		13,21	10,4	318	53,0	30,4	4,90	27,7	8,6	1,45
14Б1	137,4	73	3,8	5,6	7	13,39	10,5	435	63,3	35,8	5,70	36,4	10,0	1,65
14Б2	140	73	4,7	6,9		16,43	12,9	541	77,3	44,2	5,74	44,9	12,3	1,65
16Б1	157	82	4,0	5,9	9	16,18	12,7	689	87,8	49,5	6,53	54,4	13,3	1,83
16Б2	160	82	5,0	7,4		20,09	15,8	968	108,7	61,9	6,58	68,3	16,6	1,84
18Б1	177	91	4,3	6,5	9	19,58	15,4	1063	120,1	67,7	7,37	81,9	18,0	2,04
18Б2	180	91	5,3	8,0		23,95	18,8	1317	146,3	83,2	7,41	100,8	22,2	2,05
20Б1	200	100	5,6	8,5	12	28,49	22,4	1943	194,3	110,3	8,26	142,3	28,5	2,23
23Б1	230	110	5,6	9,0		32,91	25,8	2996	260,5	147,2	9,54	200,3	36,4	2,47
26Б1	258	120	5,8	8,5		35,62	28,0	4024	312,0	176,6	10,63	245,6	40,9	2,63

Continuation Table 4

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
26Б2	261	120	6,0	10,0	12	39,70	31,2	4654	356,6	201,5	10,83	288,8	48,1	2,70
30Б1	296	140	5,8	8,5	15	41,92	32,9	6328	427,0	240,0	12,29	390,0	55,7	3,05
30Б2	299	140	6,0	10,0		46,67	36,6	7293	487,8	273,8	12,50	458,6	65,5	3,13
35Б1	246	155	6,2	8,5	18	49,53	38,9	10060	581,7	328,6	14,25	529,6	68,3	3,27
35Б2	349	155	6,5	10,0		55,17	43,3	11550	662,2	373,0	14,47	622,9	80,4	3,36
40Б1	392	165	7,0	9,5	21	61,25	48,1	15750	803,6	456,0	16,03	714,9	86,7	3,42
40Б2	396	165	7,5	11,5		69,72	54,7	18530	935,7	529,7	16,30	865,0	104,8	3,52
45Б1	443	180	7,8	11,0	21	76,23	59,8	24940	1125,8	639,5	18,09	1073,7	119,3	3,75
45Б2	447	180	8,4	13,0		85,96	67,5	28870	1291,9	732,9	18,32	12,7	141,0	3,84
50Б1	492	200	8,8	12,0	21	92,98	73,0	37160	1511,0	860,4	19,99	1606,0	160,6	4,16
50Б2	496	200	9,2	14,0		102,80	80,7	42390	1709,0	970,2	20,30	1873,0	187,3	4,27
55Б1	543	220	9,5	13,5	24	113,37	89,0	55680	2051,0	1165,0	22,16	2404,0	218,6	4,61
55Б2	547	220	10,0	15,5		124,75	97,9	62790	2296,0	1302,0	22,43	2760,0	250,9	4,70
60Б1	593	230	10,5	15,5	24	135,26	106,2	78760	2656,0	1512,0	24,13	3154,0	274,3	4,83
60Б2	597	230	11,0	17,5		147,30	115,6	87640	2936,0	1669,0	24,39	3561,0	309,6	4,92
70Б1	691	260	12,0	15,5	24	164,70	129,3	125930	3645,0	2095,0	27,65	4556,0	350,5	5,26
70Б2	697	260	12,5	18,5		183,60	144,2	145912	4187,0	2393,0	28,19	5437,0	418,2	5,44
80Б1	791	280	13,5	17,0	26	203,30	159,5	199500	5044,0	2917,0	31,33	6244,0	446,0	5,54
80Б2	798	280	14,0	20,5		226,60	177,9	232200	5820,0	3343,0	32,01	7527,0	537,6	5,76
90Б1	893	300	15,0	18,5	30	247,10	194,0	304400	6817,0	3964,0	35,09	8365,0	557,6	5,82
90Б2	900	300	15,5	22,0		272,40	213,8	349200	7760,0	4480,0	35,80	9943,0	662,8	6,04
100Б1	990	320	16,0	21,0	30	293,82	230,6	446000	9011,0	5234,0	38,96	11520,0	719,9	6,26
100Б2	998	320	17,0	25,0		328,90	258,2	516400	10350,0	5980,0	39,62	13710,0	856,9	6,46
100Б3	1006	320	18,0	29,0		364,00	285,7	587700	11680,0	6736,0	40,18	15900,0	993,9	6,61
100Б4	1013	320	19,5	32,5		400,60	314,5	655400	12940,0	7470,0	40,45	17830,0	1114,3	6,67

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