MINIISTRY OF EDUCATION AND SCIENCE OF UKRAINE

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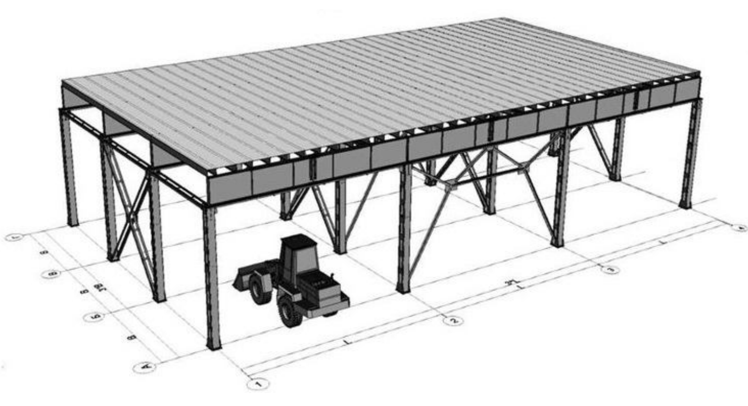
Methodological guidelines

for performing course project

on the subject

"METAL STRUCTURES. PART 1"

(for all educational forms Bachelor's Degree students
on specialty 192 – Building Industry and Civil Engineering,
of educational program "Civil and Industrial Engineering")



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CONTENT

INTRODUCTION

Methodological guidelines for performing course project "Beam cell and columns of the work platform" are intended for all educational forms Bachelor's Degree students on specialty 192 – Building Industry and Civil Engineering, of educational program "Industrial and Civil Engineering". The manual main goal is to ensure that the student has mastered his skills of composing, calculating and constructing of the beam cell elements, as well as centrally-compressed columns of the work platforms. The course project consists of an explanatory note and a graphic part on a sheet of A1 format.

1 GENERAL TERMS ABOUT BEAM CELLS

The system of load-bearing beams that form the construction of overlap or work platform is called a beam cell. Depending on the design load and dimensions in the plan, beam cells can be of three types: simplified, normal and complicated (Fig. 1). In the practice of design, the last two types are widespread. The system of load-bearing beams that form the construction of floors or work platforms is called a beam cell. Depending on the design load and dimensions in the plan, beam cells can be of three types: simplified, normal and complicated (Fig. 1). In the design practice, the last two types are widespread.

The bay of the floor beams depends on the bearing capacity of the flooring and shifts within 0,6–1,6 m for steel flooring and 1,5–3,0 m for reinforced concrete flooring. If necessary, the steel flooring might be reinforced with the stiffeners.

The bay of the auxiliary beams must be a multiple of the length of the main beam, and it can be taken within 2,0–5,0 m.

The connection of beams with each other can be superficial, level and low (Fig. 2). The distance from the lower chord of the main beam to the top of the floor is called the building height of the beam cell.

2 CALCULATION OF BEARING FLOORING

Flat steel sheets are widely used for the bearing flooring of beam cells. The distance between the edges of the shelves of adjacent beams is taken as the flooring span (Fig. 2, c). It is accepted that:

$l_{fl} / t_{fl} \leq 50$ – rigid flooring;

$l_{fl} / t_{fl} > 300$ – flexible flooring;

$50 \leq l_{fl} / t_{fl} \leq 300$ – normal flooring.

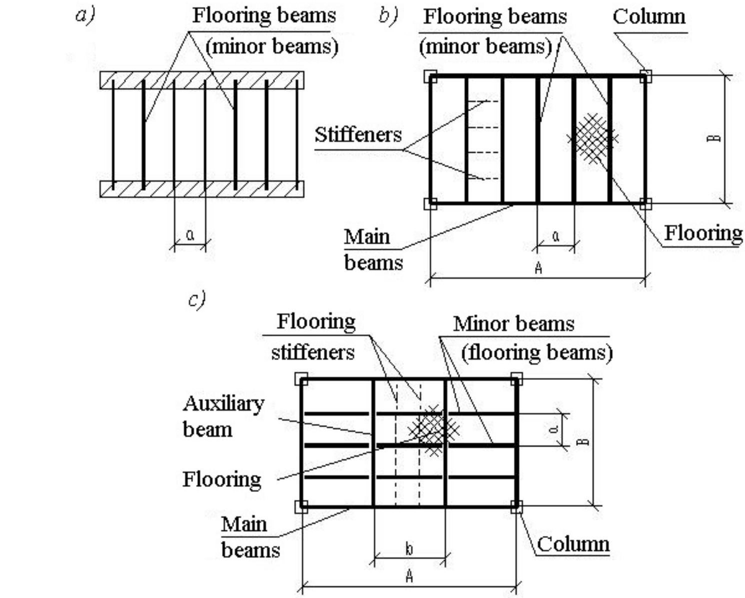

Figure 1 – Types of beam cells:
a – simplified; b – normal; c – complicated

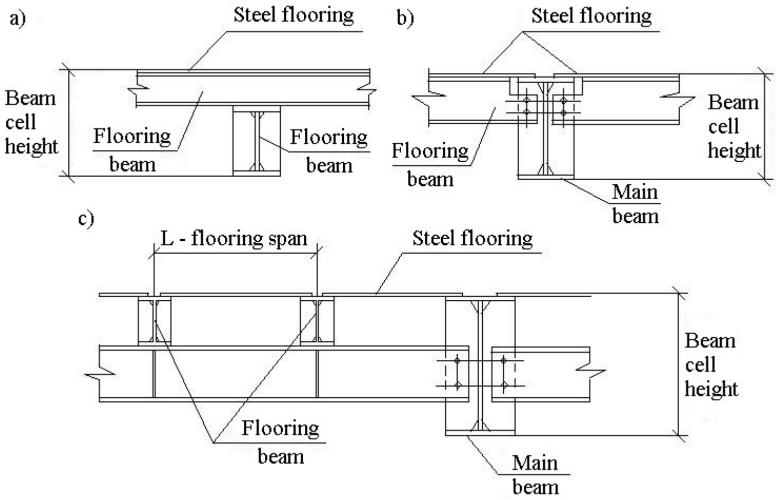

Figure 2 – Beams connection:
a – superficial; b – level; c – low

For beam floorings it is necessary to use steel sheets with thickness:

– t = 6–8 mm with loads $q \leq 10$ kN/m^2;

– t = 8–10 mm with loads $q = 20$ kN/m^2;

– t = 10–12 mm with loads q = 21-30 kN/m^2;

– t = 12–14 mm with loads $q > 30$ kN/m^2.

The flooring is joined with beams by solid or intermittent welds.

With a load of $q \leq 50$ kN/m^2 and limit relative span $l_{fl} / t_{fl} \leq 1/150$ the strength of the steel flooring hinge joint will always be ensured. Therefore, under such conditions, it should be calculated only for rigidity (deflection). In practice, from the stiffness calculation then evaluate the ratio of the span l_{fl} to its thickness t_{fl} by the formula (1):

$$\frac{l_{fl}}{t_{fl}} = \frac{4n_0}{15}\left(1+\frac{72E_1}{n_0^4 \cdot q_n}\right) = \frac{4 \cdot 150}{15} \cdot \left(1+\frac{72 \cdot 2{,}26 \cdot 10^4}{150^4 \cdot q_n}\right) = = 40 \cdot \left(1+\frac{1{,}6272}{506{,}25 \cdot q_н}\right), \tag{1}$$

where $n_0 = \left[\frac{l}{f}\right]$ – the specified ratio of the flooring span to its allowable deflection (evaluate from regulatory $\frac{f}{l} = \left[\frac{1}{150}\right]$ for floorings);

q_n – normative load on the flooring, kN/cm^2;

$$E_1 = \frac{E}{1-v^2} = \frac{2{,}06 \cdot 10^4}{1-0{,}3^2} = 2{,}26 \cdot 10^4 \text{ kN/cm}^2 \tag{2}$$

where v = 0,3 – Poisson's ratio for steel;

E_1 – modulus of elasticity when transverse deformations cannot be occurred.

3 CALCULATION OF FLOORING BEAMS

The most rational cross-section for flooring beams is rolling beams of an wide flange section or T-section. The rolling split beam is calculated in the following order:

1. Determine the bay of beams.
2. Calculate normative and estimated loads.
3. Establish the design scheme of the beam and find the maximum bending moment from the estimated loads.

4. Calculate the required moment of the cross section resistance of the beam W_r, taking into account the plastic work of the beam material:

$$W_r = C_o \times W_{pl}. \tag{3}$$

For wide flange beams and channel beams in the first approximation (oriented) C_o = 1,12 when bending is occurred in the beam wall plane, and when bending in a plane parallel to the beam chord (from the beam plane) – C_o = 1,2.

5. Using the rolling beams assortment, select profile with the moment of resistance, closest to the required moment.

6. Check the strength of the selected beam profile, taking into account the progress of plastic deformation during the bending in one of the main planes (at tangent stress $\tau \le 0{,}9R_s$, except the supporting cross-sections) by the formula (4):

$$\sigma = \frac{M}{C_1 W_{\min} R_y \gamma_c} \le 1, \tag{4}$$

where M – maximum bending moment in the designed cross-section;

W_{min} – the minimum moment of resistance net;

R_y – the design resistance of steel bending beyond the yield strength;

γ_c – coefficient of working material condition, for steel γ_c = 1,1;

R_s – design resistance of steel shear, $R_s = 0{,}58R_y$;

C_1 – coefficient, taking into account the progress of plastic deformations.

7. Check the strength of the selected profile per shear, depending on the type of connection:

– surface bearing of the beam or without shear of the beam chords;

$$\tau = \frac{QS}{I_x t_w R_y \gamma_c} \le 1; \tag{5}$$

– one-level connection with a chords shear and partially the web shear:

$$\tau = \frac{1{,}5Q}{h \cdot t_w R_y \gamma_c} \le 1. \tag{6}$$

8. Check the rigidity of the beam. For this purpose, according to the Structure Mechanics formulas, its relative bending from the regulatory loads compare with the allowable loads equal to 1/200 of the beam length.

4 CALCULATION OF COMPOSITE ELECTRIC WELDED MAIN BEAM

4.1 Selection of the cross-section of a composite beam

The cross-section of the composite electric welded beam must satisfy the requirements of strength, rigidity, general and local stability and at the same time be, if it's possible, more economical at the metal expense. One of the most important tasks, during the selection of composite beam cross-section, is evaluation of its rational height: $h = (1/8–1/12)L$.

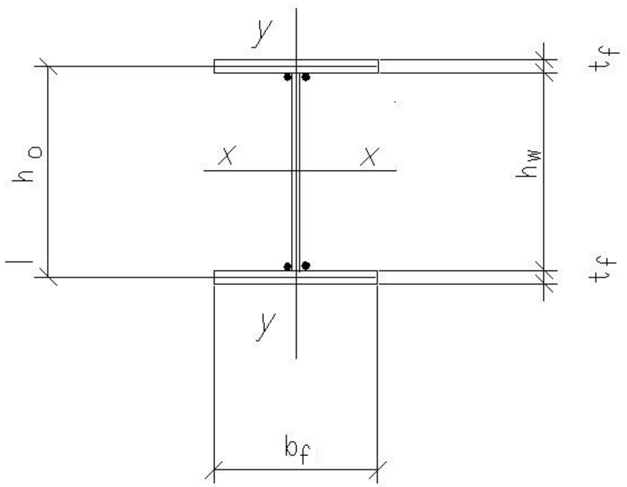

Figure 3 – Cross-section of electric welded composite beam

The composite beam calculation is carried out in the following order:

1. After accepting the beam design scheme, its loading and after determining the efforts of Q_{max} and M_{max}, as well as the required moment of resistance W_r, the beam cross-section layout begins with the determination of its height, which is found under two conditions: the stiffness h_{min} and the economy h_{opt}.

$$h_{\min} = \frac{5 \cdot R_y L^2 \sum q^n \cdot C_o}{24 \cdot E[f]\sum q^p} = \frac{5}{24} \cdot \frac{C_o R_y L^2}{24 \cdot E \dfrac{1}{400} L} \cdot \frac{\sum q^n}{\sum q^p}, \tag{7}$$

$$[f] = \frac{1}{400} L, \tag{8}$$

$$h_{opt} = (1{,}15 - 1{,}2)\sqrt{\frac{W_r}{t_w}}, \tag{9}$$

$$W_r = \frac{M_{\max}}{C_o R_y \gamma_c}. \tag{10}$$

2. The beam web thickness is pre-determined by the empirical formula (11):

$$t_w = 7 + 3h_w. \tag{11}$$

According to the shear conditions, it's necessary to determine the minimum allowable beam web thickness:

$$t_{w\min} = \frac{1{,}5Q}{hR_s}. \tag{12}$$

The final thickness of the beam web is taken according to the size of the assortment sheet steel (for design reasons it is rounding up to 6 mm).

With an unlimited construction height of the beam cell, the beam height should be slightly less than h_{opt}, but not lower than h_{min}.

When determining the beam height, it should be considered that the web height h_w must be consistent with the width of the assortment sheets, or the overall beam height must be a multiple of module 100.

3. After establishing the beam height and the web thickness, proceed to the layout of the chords. By the known value of the resistance required moment W_r of the entire cross-section and the beam height h, evaluate approximately the area A_f of each of the symmetric cross-section beam chords by the formulas (13–14):

$$\text{when } h > h_{opt}: A_f = \frac{3}{4} \cdot \frac{W_r}{h_0}, \tag{13}$$

$$\text{when } h < h_{opt}: A_f = \frac{W_r}{h_0} - \frac{t_w \cdot h_0}{6}, \tag{14}$$

where h_0 – estimated beam height.

The width of the chord's sheet is determined within $b_f = (1/5–1/3)h$, provided the overall stability of the beam. According to the technological reasons (for convenience of automatic welding) – $b_f \geq 180$ mm. After determining the chord's area and its width, it is necessary to evaluate the required thickness:

$$t_f = \frac{A_f}{b_f}. \tag{15}$$

4. The local stability of the compressed sheet chord is considered to be respected if the ratio of the calculated width of its overhang b_h to the

thickness t_f does not exceed the following values according to the requirements:

a) in the elastic stage of metal work:

$$\frac{b_{ef}}{t_f} = 0{,}5 \cdot \sqrt{\frac{E}{R_y}}\ ; \tag{16}$$

b) in the elastoplastic stage of metal work (or taking into account the progress of plastic deformations):

$$\frac{b_{ef}}{t_f} = 0{,}11\frac{h_0}{t_w}\text{, but not more than } 0{,}5 \cdot \sqrt{\frac{E}{R_y}}\ , \tag{17}$$

where h_0 – estimated beam height;

b_{ef} – beam flange overhang.

$$b_{ef} = \frac{b_f - t_w}{2}\ . \tag{18}$$

5. Next step is the determining of the geometric characteristics of the adopted beam cross-section and checking its bearing capacity (by normal and tangential stresses), as well as deformability (rigidity).

– the moment of inertia about the X axis:

$$I_x = I_w + 2 \cdot I_f = \frac{t_w \cdot h_w^2}{12} + 2 \cdot \left(t_f \cdot b_f\right) \cdot \left(\frac{h_0}{2}\right); \tag{19}$$

– the moment of resistance about the X axis:

$$W_x = \frac{I_x}{h/2} = \frac{2I_x}{h}\ ; \tag{20}$$

– normal stresses, taking into account the progress of plastic deformations, at maximum moment $M_{x,max}$:

$$\sigma_x = \frac{M_{x\max}}{C_1 W_x R_y \gamma_c} \le 1\ ; \tag{21}$$

– tangential stresses in the reference cross-section by maximum transverse force:

$$\tau = \frac{1{,}5 Q_{\max}}{t_w h_w R_s \gamma_c} \le 1\ ; \tag{22}$$

– the rigidity:

$$\frac{f}{l}=\frac{M\cdot l}{k\cdot 10E\cdot I_x}\leq\frac{1}{400}, \tag{23}$$

where $k \approx 1{,}15$ is the ratio coefficient of the calculated load to normal q_x/q_n, which translate the previously evaluated moment from the calculated loads into the moment from the normative loads.

4.2 Changing the cross-section of the composite beam in length

The cross-section of the composite beam, selected by M_{max}, can be reduced in places of moments decreasing, which are fall as the distance from the middle to its supports. Each cross-section change makes steel economy, but at the same time increases the complexity of the beam manufacturing. Therefore, it is economical only for beams with a length of 10 m or more.

The place of the cross-section chords change of the one-span electric welded beam is at a distance $x=\left(\frac{1}{6}-\frac{1}{5}\right)L$ from the support. The current moment at this place can be evaluated graphically by the moments curve (fig. 4) or by the formulas (24)–(25):

– at evenly distributed load:

$$M^*=\frac{q\cdot l}{2}-\frac{q\cdot x^2}{2}=\frac{q\cdot x\cdot(l-x)}{2}; \tag{24}$$

– when concentrated forces are loaded:

$$M^*=V\cdot x-\sum P_{ix}\cdot l_{ix}. \tag{25}$$

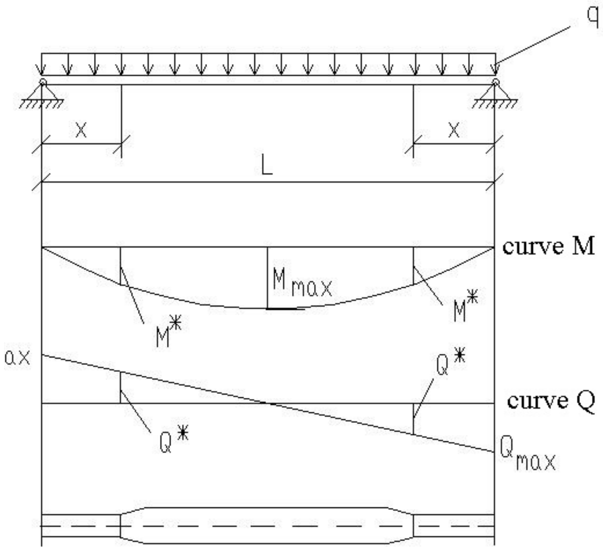

Figure 4 – Changes in the cross-section of the main beam chords

Chords sheet joints of various widths are made using a direct butt weld, which is hand welded and without physical quality control methods is unequal to the parent metal. Therefore, the resistance of the butt weld is $R_{wy} = 0{,}85R_y$.

In addition, it should be noted that in beams with variable length cross-section, the progress of plastic deformations should be considered in only one cross-section, with the most unfavorable connection (combination) of M and Q. In other cross-sections, the progress of plastic deformations is not allowed.

The calculations are performed in the following order.

Evaluate in the cross-section "X" the values of M^* and Q^* and determine the required moment of resistance at the point where the beam cross-section changes:

$$W^*_{req} = \frac{M^*}{R_{wy}\gamma_c}. \tag{26}$$

The required area of each of the modified cross-section chords is roughly evaluated by the formula:

$$A^x_{f,req} \approx \frac{W^*_{req}}{h} - \frac{t_w \cdot h_w}{6}. \tag{27}$$

Than we evaluate the chord's sheets new width, leaving the thickness t_f constant:

$$b_f^* = \frac{A_{f,req}^x}{t_f} \geq 180. \qquad (28)$$

In addition, the chord's new width shall not be less than 1/10 of the beam height (0,1h) and half of the chord width before changing the cross-section (0,5b_n), and shall also be consistent with the width of the assortment sheets.

In places where the cross-section changes at the level of the chord sheet welds, check the given stresses by the formulas:

a) at the adjoining beams (or minor beams) in the same level $\sigma_{loc} = 0$:

$$\sigma_{red} = \sqrt{\sigma_{x,1}^{*2} + 3\tau_{xy,1}^{*2}} \leq 1{,}15 R_y \gamma_c; \qquad (29)$$

b) in the presence of local load on the beam upper chord $\tau_{loc} \neq 0$ and absence of stiffeners (surface bearing of connection) of minor beams:

$$\sigma_{red} = \sqrt{\sigma_{x,1}^{*2} - \sigma_{x,1}^{*} \cdot \sigma_{loc} + \sigma_{loc}^{2} + 3\tau_{xy,1}^{*2}} \leq 1{,}15 R_y \gamma_c. \qquad (30)$$

In formulas (29)–(30):

$$\sigma_{x,1}^{*} \frac{M^* \cdot h_w}{2I_x^*} = \frac{M^*}{W_{x,1}^*}, \qquad (31)$$

$$\tau_{xy}^{*} = \frac{Q_1^* \cdot S_f^*}{I_x^* \cdot t_w}, \qquad (32)$$

$$\sigma_{loc} = \frac{F}{t_w \cdot l_{ef}}, \qquad (33)$$

where $I_x^*, W_{x,1}^*$ and S_x^* – respectively, the moment of inertia, the moment of resistance and the static moment of the modified cross-section beam chord relative to the neutral axis X;

F – support reaction of the flooring beams (minor beams);

l_{ef} – conditional length of local pressure distribution (fig. 5).

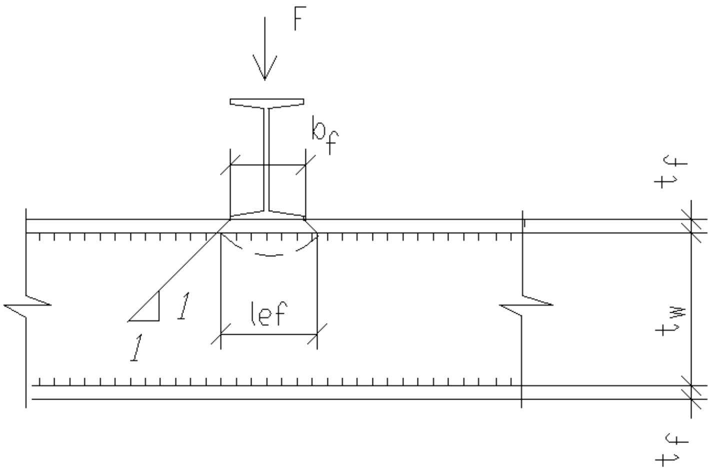

Figure 5 – Design scheme for evaluating the local distribution length load on the web of the beam

4.3 Calculation of chord welds of a beam

To calculate the chord welds, evaluate a shear force on 1 cm beam length:

$$T = \frac{Q_{\max} S_f^*}{I_x^*}, \tag{34}$$

where Q_{max} – the maximal transverse force in the reference cross-section.

We evaluate the thickness of the chord welds, which are performed by automatic welding, provided with a shear in two cross-sections.

a) on the metal weld:

$$\frac{T}{2\beta_f k_f} \le R_{wf}\gamma_{wf}\gamma_c, \tag{35}$$

$$k_f \ge \frac{T}{2\beta_f R_{wf}\gamma_{wf}\gamma_c}; \tag{36}$$

b) on metal at the boundary of the heat:

$$\frac{T}{2\beta_z k_f} \le R_{wz}\gamma_{wz}\gamma_c, \tag{37}$$

where k_f – the cathetus of the angular weld, the value of k_f is accepted according to [1];

β_f and β_z – the penetration coefficients for the calculation of the angular weld, respectively, on the metal weld and on the metal at the boundary of the heat. The value of these coefficients is taken according to [1], for steels with a yield strength of up to 58 kN/cm^2 with automatic welding they are taken β_f = 1,1 and β_z = 1.15;

γ_{wf} , γ_{wz} and γ_c – coefficients of working conditions, respectively, of the weld and construction, which are taken equal to 1,0 [1];

R_{wf} – design resistance of the angular welds to conditional shear on the metal weld according to Table 56 [1];

R_{wz} = 0,45R_u – design resistance of angular welds to the conditional shear on metal at the boundary of the heat;

R_u – tensile, butt, bending strength design resistance of the steel and temporary resistance according to Table 51 [1].

4.4 Calculation of the web local stability of the composite beam

This calculation must be carried out in accordance with the requirements of the relevant document [1].

The stability of the beam webs doesn't need to be checked if the condition for the given stresses of the formula (30) is fulfilled, as well as the conditional flexibility of the web does not exceed such values: 3,5 – in the absence of local stresses in beams with bilateral chord welds; 3,2 – in the absence of local stresses in beams with one-sided chord welds; 2,5 – in the presence of local stress in the beams with bilateral chord welds.

$$\overline{\lambda_w} = \frac{h_{ef}}{t_w}\sqrt{\frac{R_y}{E}}, \tag{38}$$

where h_{ef} – is the estimated height of the web.

The beam web should be reinforced by the transverse stiffeners if the value of the conditional flexibility of the beam web is λ_w > 3,2 in the absence of moving loads. The distance between the major transverse stiffeners should not exceed $\alpha \leq 2h_{ef}$ at $\lambda_w > 3,2$ and $\alpha \leq 2,5h_{ef}$ at $\lambda_w \leq 3,2$.

The width of the beam edge is taken at least, mm:

$$b_h \geq \frac{h_{ef}}{30} + 40. \tag{39}$$

The thickness of the beam edge:

$$t_s \geq 2b_h\sqrt{\frac{R_y}{E}}. \tag{40}$$

The calculation of the stability of the beam web of symmetrical cross-section, reinforced only by the transverse main stiffeners, in the absence of local stresses (σ_{loc} = 0) and conditional flexibility $\lambda_w \le 3{,}2$ should be performed by the formula:

$$\sqrt{\left(\sigma/\sigma_{cr}\right)^2 + \left(\tau/\tau_{cr}\right)^2} \le \gamma_c, \tag{41}$$

where γ_c – coefficient of working conditions, taken from the Table 6 [1];

$$\sigma_{cr} = \frac{C_{cr} R_y}{\overline{\lambda}_{cr}^2}, \tag{42}$$

$$\tau_{cr} = 10{,}3 \cdot \left(1 + \frac{0{,}76}{\mu^2}\right) \cdot \frac{R_s}{\overline{\lambda}_{ef}^2}. \tag{43}$$

In formula (39), coefficient C_{cr} should be taken from [1], depending on the value of the coefficient:

$$\delta = \beta \times \frac{b_f}{h_{ef}} \times \left(\frac{t_f}{t_w}\right)^3, \tag{44}$$

where b_f and t_f are respectively the width and thickness of the compressed beam chord;

β = 0,8 – the coefficient, which is taken from [1];

In the formula (43) $R_s = 0{,}58R_y$ – is the design shear resistance;

$$\overline{\lambda}_{ef} = \frac{d}{t_w}\sqrt{\frac{R_y}{E}}, \tag{45}$$

where d – smaller of the plate sides (h_{ef} or a);

μ – the ratio of the larger side of the plate to the smaller.

The calculation of the stability of the beam web of symmetrical cross-section, reinforced only by the transverse main stiffeners, in the presence of local stress ($\sigma_{loc} \ne 0$) should be performed by the formula:

$$\sqrt{\left(\sigma/\sigma_{er} + \sigma_{loc}/\sigma_{loc.cz}\right)^2 + \left(\tau/\tau_{cr}\right)^2} \le \gamma_c. \tag{46}$$

Paired transverse edges should be installed in places adjacent or abutting flooring beams. In this case it should be considered (σ_{loc} = 0) and the formula (41) should be verified.

4.5 The calculation of the supporting edge of the beam

When the hinge bearing of the main beam to the top of the column, the transference of the reaction occurs through the support edge, which is welded to the end of the beam along the perimeter of the fit (fig. 6).

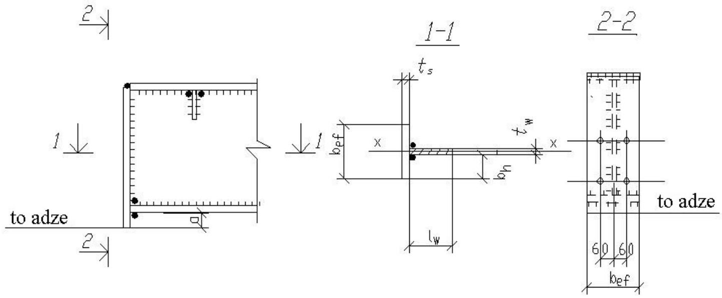

Figure 6 – Calculation scheme

The determination of the support edge size is based on the change of its end:

$$A_{req}^{h} = \frac{V_{m.b.}}{R_{p}}, \tag{47}$$

where R_p – is the calculated resistance of the end surface [1].

Structurally, the widths of the edges is taken $b_{ef} = b_s$ and only after that evaluate its thickness:

$$t_{s} \geq \frac{A_{req}^{h}}{b_{ef}}. \tag{48}$$

In addition, it is necessary to check the beam supporting segment for stability from the plane of the beam as a conditional support rod containing in the area of the cross-section A_{ef} support edge and part of the beam web:

$$A_{ef} = A_{h} + A_{w} = b_{ef} \times t_{s} + t_{w} \times 0{,}65\sqrt{\frac{E}{R_{y}}}. \tag{49}$$

This segment of the beam is calculated for compression as a rod with a calculated length equal to the height of the beam web:

$$\sigma = \frac{V_{m.b.}}{A_{ef}\varphi R_{y}\gamma_{c}} \leq 1, \tag{50}$$

where φ – the longitudinal bending coefficient, which is taken from [1] depending on the value of flexibility λ.

The calculation of the welds that attach the supporting edge to the beam web $l_w = h_w$ - 2:

a) on the metal weld:

$$k_f \geq \frac{V_{m.b.}}{2\beta_f \times l_w \times R_{wf} \times \gamma_{wf} \times \gamma_c}; \tag{51}$$

b) on metal at the boundary of the heat:

$$k_f \geq \frac{V_{m.b.}}{2\beta_z \times l_w \times R_{wz} \times \gamma_{wz} \times \gamma_c}. \tag{52}$$

The values of β_f, β_z, R_{wf}, R_{wz}, γ_{wf}, γ_{wz} and γ_c are the same as in formulas (35) and (36). It should be noted that in manual welding, according to [1] - β_f = 0,7 and β_z = 1;

After determining k_f, it is necessary to compare its value with the values of the corresponding minimum cathetuses according to [1] and accept the largest of the evaluated ones.

4.6 Calculation of the flooring beams joint with the main beams

The flooring beams joints with the main beams in the beam cells of the normal type are performed either on the surface or in the same level (fig. 7).

Of all the joints, surface support is the easiest, but, because of the possibility of main beam chord bending, it can only convey small reactions. This support can be strengthened by placing a stiffener under the flooring beam, thereby preventing the local bending of the upper chord of the main beam.

In case of minor beams surface support, the connection joint is designed constructively (fig. 7).

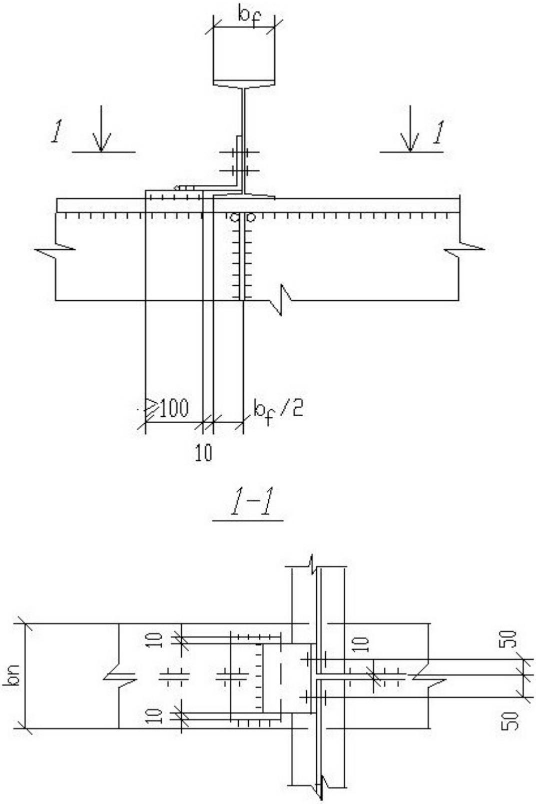

Figure 7 – Constructive scheme of beams surface support

When connection is made in same level, flooring beams are fixed to the special transverse stiffeners of the main beam (fig. 8).

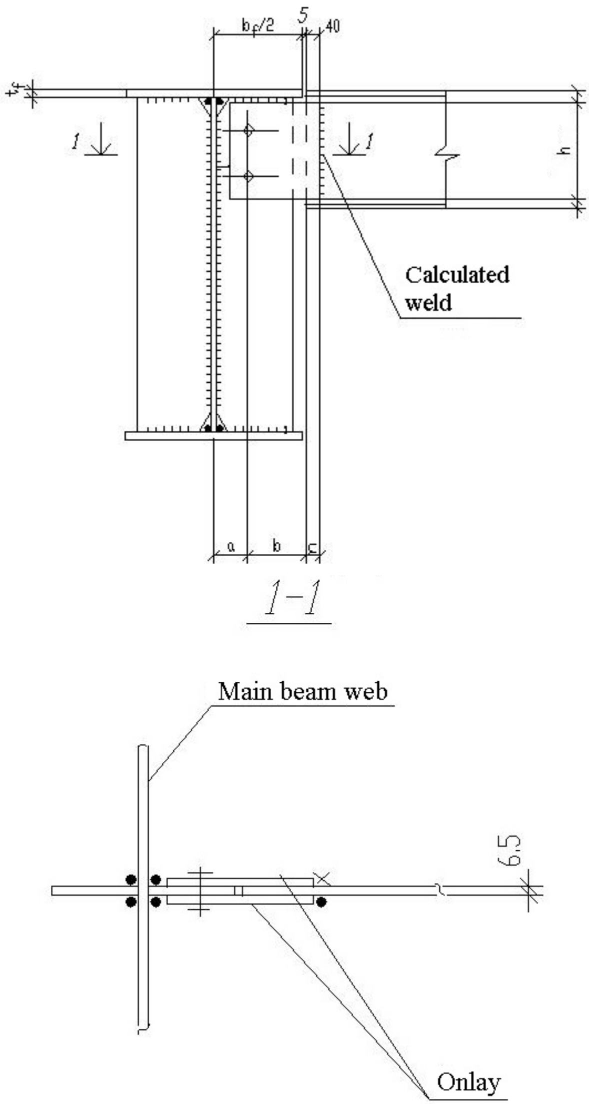

Figure 8 – Constructive scheme of beams connection in same level

The purpose of beam connections calculation is to determine the size of onlays, edges, welds or the number of shear bolts that attach the beams to each other.

The design force – is the supporting reaction of the flooring beam, which is increased by 20 %. In this case, as a rule, the thickness of the edges and onlays is taken not less than the web thickness of the minor beam.

The calculations are performed in the following sequence:

1. Evaluation of the strength of structurally accepted onlays is performed according to the formula (53):

$$\sigma_{red} = \sqrt{\sigma^2 + 3\tau^2} \le 1{,}15 R_y \gamma_c, \tag{53}$$

where γ_c – coefficient of working conditions, taken from the Table 6 from [1];

$$\sigma = \frac{M_{onlay}^{\max}}{W_{onlay} R_y \gamma_c} \le 1, \tag{54}$$

$$\tau = \frac{1{,}2 V_{f.b.}}{\sum A_{onlay} R_y \gamma_c} \le 1, \tag{55}$$

$$M_{onlay}^{\max} = 1{,}2 V_{f.b.} \times (b + c), \tag{56}$$

$$W_{onlay} = \frac{\sum t_{onlay} \times h_{onlay}^2}{6}, \tag{57}$$

$$\sum A_{onlay} = h_{onlay} \times \sum t_{onlay}. \tag{58}$$

If the condition of the formula (53) is not fulfilled, then it is necessary to change the onlays thickness.

2. The calculation of the welds that attach the onlays to the web of the flooring beam. Such welds are usually performed by manual welding and the weld cathetus k_f is assigned within the range from the minimum allowable [1] to the maximum possible $k_f = 1{,}2 t_{min}$ (where t_{min} is the smallest of the welding sheets thickness).

Welds stresses from calculated force V_{fb}:

a) on the metal weld:

$$\tau_f = \frac{1{,}2 V_{f.b.}}{2\beta_f \times k_f \times l_w}; \tag{59}$$

b) on metal at the boundary of the heat:

$$\tau_z = \frac{1{,}2 V_{f.b.}}{2\beta_z \times k_w \times l_w}. \tag{60}$$

The support reaction to the weld center of gravity creates a bending moment:

$$M = 1{,}2V_{f.b.} \times (b + c). \tag{61}$$

Welds stresses from bending moment action:

a) on the metal weld:

$$\sigma_f = \frac{M}{W_f} = \frac{6M}{2\beta_f \times k_f \times l_w^2}; \tag{62}$$

b) on metal at the boundary of the heat:

$$\sigma_z = \frac{M}{W_z} = \frac{6M}{2\beta_z \times k_f \times l_w^2}. \tag{63}$$

Evaluating the welds strength is performed by the formulas:

a) on the metal weld:

$$\sigma_{f,level} = \sqrt{\sigma_f^2 + \tau_f^2} \le R_{wf}\gamma_{wf}\gamma_c; \tag{64}$$

b) on metal at the boundary of the heat:

$$\sigma_{z,level} = \sqrt{\sigma_z^2 + \tau_z^2} \le R_{wz}\gamma_{wz}\gamma_c. \tag{65}$$

3. The calculation of the bolts that attach the onlays to the edges of the main beam. Before the calculation we need to take the class and the diameter of the bolts.

Estimated effort N_b, which can be sensed by one bolt in the connection:

a) on shear condition:

$$N_b = R_{bs} \times \gamma_b \times A \times n_s, \tag{66}$$

where R_{bs} – design shear resistance of bolts [1];

$\gamma_b = 0{,}9$ – coefficient of joint working conditions;

A – the estimated area of the bolt cross-section;

$n_s = 2$ is the number of shear areas;

b) on crushing condition:

$$N_b = R_{bp} \times \gamma_b \times d \times \sum t_{\min}, \tag{67}$$

where R_{bp} – design crushing resistance of the elements connected by bolts, is taken according to [1];

d – bolt rod diameter;

$\sum t_{min}$ – is the smallest total thickness of the elements compressed in one direction.

The number of bolts in the connection is evaluated by the formula:

$$n \geq \frac{V_{f.b.}}{N_{b\min} \times \gamma_c}. \tag{68}$$

It should be noted that the bolts must be installed in accordance with the requirements of [1].

5 EXAMPLE OF THE BEAM CELL ELEMENTS COMPOSING AND CALCULATION

To master the skills of composing, calculating and constructing beam cell elements, let us consider the following example. So, it is necessary to arrange and perform calculations of beam cell elements of a normal type (fig. 9).

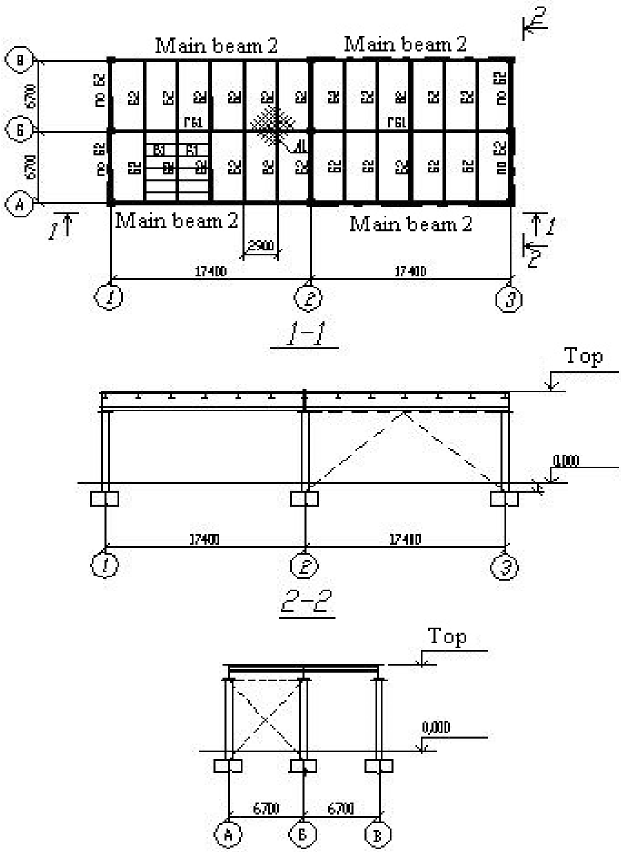

Figure 9 – Constructive scheme of beam cell elements

Basic data: L = 17 400 mm; l = 6 700 mm; a = 2 900 mm; q_{char} = 4,8 kN/m^2; a = 2 900 mm q_{const} = 0,994 kN/m^2;

Only a static load is applied to the beam cell, so its structural elements can be made of C235 and C245 steel, whose design resistance is R_y = 23 kN/cm^2 and R_y = 24 kN/cm^2, respectively.

5.1 Calculation of flooring beams

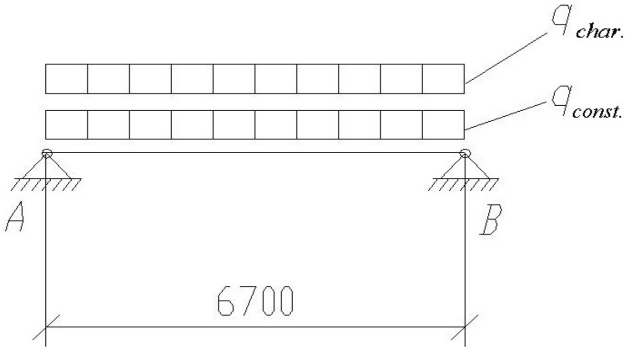

Figure 10 – Calculation scheme of beam B2

1. First, we need to evaluate the longitudinal design load on the beam with an approximate consideration of its own weight:

$$q_{l.d.} = \left(q_{const} \times \gamma_{fn0} + q_{char} \times \gamma_{fm} + q_{flooring} \times \gamma_{fn}\right) \times k \times a_1 =$$

$$= \left(0{,}994 \times 1{,}1 + 4{,}8 \times 1{,}2 + 0{,}785 \times 1{,}05\right) \times 1{,}02 \times 2{,}9 = 22{,}566 \frac{kN}{m};$$

2. Evaluating of the moment in span:

$$M_{\max} = \frac{22{,}566 \times 6{,}7^2}{8} = 126{,}623 kN \times m = 12662{,}3 kN \times cm;$$

3. Determining of the required moment of resistance, taking into account the progress of plastic deformations of the beam material:

$$W_{nomp} = \frac{M_{\max}}{C_{appr} R_y \gamma_c} = \frac{12662{,}3}{1{,}12 \times 24 \times 0{,}95} = 446{,}86 cm^3.$$

According to the assortment of SSTU 8768:2018 [2] for beam B2 we accept the wide-flange section I30 with the following geometric characteristics:

$W_x = 472$ cm^3; $I_x = 7080$ cm^4; $S = 268$ cm^3;
$q = 36{,}5$ kg/m $= 0{,}365$ kN/m; $A_I = 46{,}5$ cm^2;

4. Determining of the design load on the beam, taking into account its own weight:

$$q'_{l.d.} = \left(q_{const} \times \gamma_{fn} + q_{char} \times \gamma_{fm} + q_{flooring} \times \gamma_{fn}\right) \times a_1 + q \times \gamma_{fn} =$$

$$= \left(0{,}994 \times 1{,}1 + 4{,}8 \times 1{,}2 + 0{,}785 \times 1{,}05\right) \times 2{,}9 + 0{,}365 \times 1{,}05 = 22{,}507 \text{ kN/m};$$

5. Determining of the real maximum moment and the supporting reaction of the beam:

$$M^1_{\max} = \frac{22{,}507 \cdot 6{,}7^2}{8} = 126{,}292 \text{ kN·m} = 12629{,}2 \text{ кN·cm};$$

$$V^1_A = V^1_B = q^1_p \cdot l/2 = 22{,}507 \cdot 6{,}7/2 = 75{,}398 \text{ kN} = 75{,}4 \text{ kN};$$

6. Verification of the strength of the selected beam profile:

$$\tau = \frac{Q \cdot S}{I_x \cdot t_w \cdot \gamma_c} = \frac{75{,}4 \cdot 268}{70807 \cdot 0{,}65 \cdot 1{,}1} = 3{,}992 \text{ кN/cm}^2 < 0{,}58 \cdot 23 = 13{,}34 \text{ kN/cm}^2,$$

$$\sigma = \frac{M^1_{\max}}{C_1 \cdot W_x \cdot \gamma_c} = \frac{12629{,}2}{1{,}12 \cdot 472 \cdot 0.95} = 25{,}1 \text{ kN/cm}^2 \geq 24 \text{ kN/cm}^2,$$

Overtension of I30 is:

$$\delta = \frac{25{,}1 - 24}{24} \cdot 100\% = 5\% \geq [5\%];$$

7. Verification of the rigidity of the beam:

$$\frac{f}{l} = \frac{M^p \cdot l}{k \cdot 10E \cdot I_x} = \frac{12629{,}2 \cdot 670}{1{,}15 \cdot 10 \cdot 2{,}06 \cdot 10^4 \cdot 7080} =$$

$$= \frac{8{,}462}{1677{,}25} = \frac{1}{198{,}2} < \left[\frac{1}{200}\right].$$

The condition is not fulfilled, so for the beam, according to the assortment of SSTU 8768:2018 [2], we accept larger wide-flange section I30a with the following geometric characteristics:

W_x = 518 cm^3; I_x = 7780 cm^4; S = 292 cm^3;
q = 39,5 kg/m =0,395 kN/m;

$$\sigma = \frac{M^1_{\max}}{C_1 \cdot W_x \cdot \gamma_c} = \frac{12629{,}2}{1{,}12 \cdot 518 \cdot 0{,}95} = 22{,}9 \text{ kN/cm}^2 \leq 24 \text{ кN/cm}^2,$$

Overtension of I30a is:

$$\delta = \frac{24 - 22.9}{24} \cdot 100\% = 4{,}53\% \leq [5\%],$$

$$\frac{f}{l} = \frac{12629{,}2 \cdot 670}{1{,}15 \cdot 10 \cdot 2{,}06 \cdot 10^4 \cdot 7780} = \frac{1}{217} < \left[\frac{1}{200}\right].$$

5.2 Calculation of the composite main beam MB1

Construction material:

– at sheet thickness up to 20 mm – steel of class C245 with design resistance R_y = 24 kN/cm^2;

– for sheet thicknesses larger than 20-30 mm – steel of class C255 with design resistance R_y = 23 kN/cm^2 [1].

Bending of the main beam 1/400*L*.

The MB1 beam is loaded with concentrated forces – the reactions of minor beams.

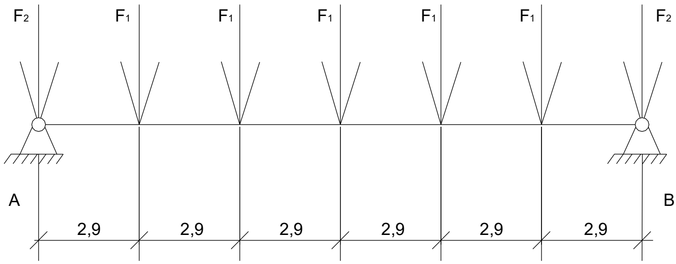

Figure 11 – Calculation scheme of beam MB1

$$F_1 = 2V_{MB1} = 2\text{x}75{,}4 = 150{,}8 \text{ kN};$$
$$F_2 = V_{MB1} = 75{,}4 \text{ kN}.$$

5.3 Selection of the beam MB1 cross-section

1. Determining of the reactions, maximum transverse force and moment in span, taking into account the estimated weight of the beam:

$$V_A = V_B = \frac{\sum F_1 + \sum F_2}{2} \cdot k =$$
$$= \frac{5 \cdot 150{,}8 + 2 \cdot 75{,}4}{2} \cdot 1{,}05 = 475{,}02 \text{ kN},$$

where k = 1,03-1,05 – coefficient of the approximate beam self-weight considering.

$$Q_{\max} = V_A - F_2 \cdot k = 475{,}02 - 75{,}4 \cdot 1{,}05 = 396{,}35 \text{ kN};$$

$$M_{\max} = \left[\left(\frac{V_A}{k} - F_2\right) \cdot 8{,}7 - F_1(5{,}8 + 2{,}9)\right] \times k =$$

$$= [396{,}35 \cdot 8{,}7 - 150{,}8 \cdot 8{,}7] \cdot 1{,}05 = 2276{,}4 \text{ kN·m} = 227640 \text{ kN·cm}.$$

2. Determination of the necessary moment of resistance, taking into account the progress of plastic deformations of the beam material:

$$W_{req}=\frac{M_{\max}}{C_{appr}R_y\gamma_c}=\frac{227640}{1{,}12\cdot 24\cdot 0{,}95}=8915cm^3;$$

3. Determination of the optimal beam height by the formula (9), approximately taking its height:

$$h\approx\left(\frac{1}{12}\div\frac{1}{8}\right)\cdot l\approx\frac{1}{11}\cdot 17{,}4\approx 1{,}582\,m.$$

According to the empirical formula (10), we evaluate the web thickness:

$$t_w=7+3h_{(m)}=7+3\cdot 1{,}58=11{,}74mm.$$

Accepting web thickness t_w = 11 mm.

$$h_{opt}=(1{,}15-1{,}2)\sqrt{\frac{W_{req}}{t_w}}=1{,}2\sqrt{\frac{8915}{1{,}1}}=108{,}03mm;$$

4. On condition of rigidity at the formula (8) we find the minimum height of the beam:

$$h_{\min}=\frac{5}{24}\cdot\frac{C_{appr}R_yL^2}{E[f]}\cdot\frac{\sum F^n}{\sum F^w}=72{,}5\cdot\frac{C_{appr}R_yL}{E}=$$

$$=72{,}5\cdot\frac{1{,}12\cdot 24\cdot 1740}{2{,}06\cdot 10^4}=164{,}6\,cm,$$

where $\dfrac{\sum F^n}{\sum F^w}\approx 0{,}87.$

Accepting $h_w=1250$ mm;

5. On the assumption of the web working conditions for tangent stresses on the support, we determine the admissible web thickness by the formula (11):

$$t_{w,\min}=\frac{1{,}5Q^{\max}}{h\cdot R_s}=\frac{1{,}5\cdot 396.35}{125\cdot 0{,}58\cdot 24}=0{,}342\ \text{cm}.$$

6. Determination of the minimum permissible beam web thickness, based on the condition, that no longitudinal stiffeners are used to ensure the local stability:

$$t_{w,\min}=\frac{h\sqrt{R_y/E}}{5{,}5}=\frac{125\sqrt{24/2{,}06\cdot 10^4}}{5{,}5}=\frac{5{,}46112}{5{,}5}=0{,}776\ \text{cm};$$

Comparing the calculated web thickness with the accepted t_w = 11 mm, we conclude that it can be reduced and accepted t_w = 10 mm, which satisfies the strength requirements on the tangent stresses and does not need its support by longitudinal stiffeners to provide local stability.

7. Composing of the beam cross-section (fig. 12).

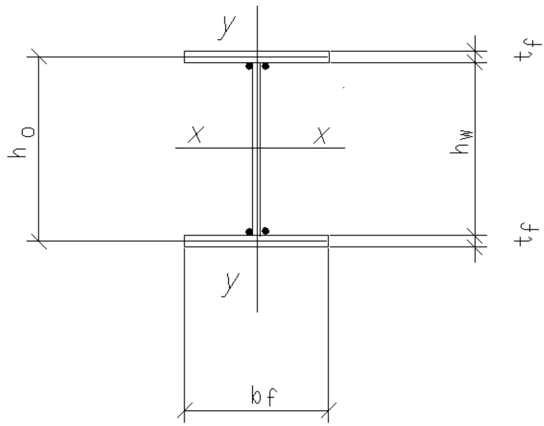

Figure 12 – Transverse cross-section of the beam

According to the assortment of sheet steel SSTU 8540:2015 [3], the beam web is accepted from a sheet of 1250×10 mm. Approximately, we define the thickness of the chords sheets t_f = 20 mm:

$$\left(t_w \le t_f \le (2-3)t_w\right).$$

Then, the height of the beam will be h = 1250 + 2 × 20 = 1290 mm.

Because the accepted height of the beam h = 1290 mm > h_{opt} = = 900 mm, the approximate area of each chord of the beam is evaluated by the formula (12):

$$A_f = \frac{3}{4} \cdot \frac{W_{req}}{h_0} = \frac{3}{4} \cdot \frac{8915}{129-2} = 52{,}64 cm^2.$$

Beam chord width is determined within the recommended ratio:

$$b_f = \left(\frac{1}{5} \div \frac{1}{3}\right) \cdot h = \frac{1}{4} \cdot 129 \approx 32{,}25 mm.$$

According to the assortment of universal sheet steel SSTU 8541:2015 [4], we accept beam chord width of 340 mm, then $t_f = A_f / b_f$ = = 52,64 / 34,0 = 1,55 cm, so the thickness of the beam chord t_f = 20 mm. The beam chord is taken from the steel strip 340 × 20.

8. Verification of the local stability of the compressed beam chord in the elastoplastic stage of work by the formula (15):

$$\frac{b_{ef}}{t_f} = 0{,}11 \cdot \frac{h_0}{t_w} = 0{,}11 \cdot \frac{129-2}{1{,}0} = 13{,}97 < 0{,}5\sqrt{\frac{E}{R_y}} = 0{,}5\sqrt{\frac{2{,}06 \cdot 104}{24}} = 14{,}65 .$$

Determination of the beam chord overhang by the formula (16):

$$b_{ef} = \frac{b_f - t_w}{2} = \frac{34-1{,}0}{2} = 16{,}5 \text{ cm}.$$

And evaluation the actual ratio:

$$\frac{b_{ef}}{t_f} = \frac{16{,}5}{2} = 8{,}25 < 14{,}65;$$

9. Evaluation of the geometric characteristics of the beam cross-section:

$$I_x = \frac{t_w \cdot h_w^3}{12} + 2A_f\left(\frac{h_0}{2}\right)^2 + 2\frac{b_f \cdot t_f^3}{12} =$$

$$= \frac{1{,}0 \cdot 125^3}{12} + 2 \cdot (34 \cdot 2) \cdot 63{,}5^2 + 2 \cdot \frac{34 \cdot 2^3}{12} =$$

$$= 162760 + 548386 = 711146 \text{ cm}^4,$$

$$W_x = \frac{I_x}{h/2} = \frac{711146}{63{,}5} = 11199 \ge 8915 \text{ cm}^3,$$

$$A_I = 2A_f + A_w = 2 \cdot (34 \cdot 2) + 125 \cdot 1{,}0 = 261 \text{ cm}^2;$$

10. Verification of the selected beam cross-section strength:

$$\sigma = \frac{M_{\max}}{C_1 \cdot W_x \cdot \gamma_c} = \frac{227640}{1{,}113 \cdot 11199 \cdot 0.95} = 19{,}22 < 24 \text{ kN/cm}^2,$$

where $C_1 = C$ – the value of C is evaluated according to [1] depending on the value of the ratio $\frac{A_f}{A_w} = \frac{2 \cdot 34}{1{,}0 \cdot 125} = 0{,}544$; $C = 1{,}113$.

Evaluation of the safety margin:

$$\delta = \frac{24-19{,}22}{24} \cdot 100\% = 19{,}9\% \ge [5\%],$$

$$\tau = \frac{1{,}5Q_{\max}}{t_w \cdot h_w \cdot \gamma_c} = \frac{1{,}5 \cdot 396{,}35}{1{,}0 \cdot 125 \cdot 0{,}95} = 5{,}01 < 0{,}58 \cdot 24 = 13{,}92 kN/cm^2;$$

11. Verification of the beam elasticity by the formula (20):

$$\frac{f}{l} = \frac{M_{\max}^{p} \cdot l}{k \cdot 10E \cdot I_x} = \frac{227640 \cdot 1740}{1{,}15 \cdot 10 \cdot 2{,}06 \cdot 10^4 \cdot 711146} = \frac{1}{425} < \left[\frac{1}{400}\right].$$

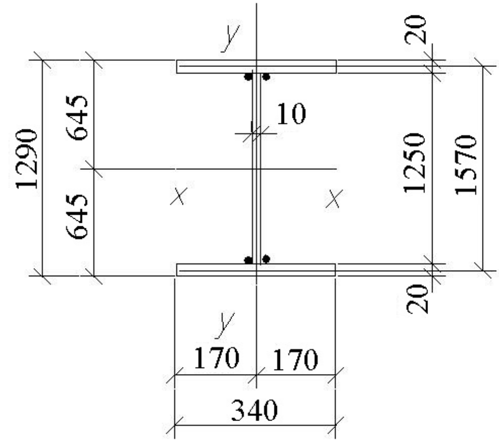

Figure 13 – Transverse cross-section of the beam

5.4 Changing the beam MB1 cross-section

1. Beam chords cross-section changing position is accepted at following distance from its support:

$$x = \left(\frac{1}{5} \div \frac{1}{6}\right)L = \frac{1}{5{,}8} \cdot 17{,}4 = 3 \text{ m};$$

2. Determination in the cross-section "*X*" the value of M^* and Q^*:

$$M^*_{x=3} = \left(V_A - F_1\right) \cdot 3 - F_2 \cdot 0{,}1 =$$

$$= \left(475{,}02 - 75{,}4\right) \cdot 3 - 150{,}8 \cdot 0{,}1 = 1183{,}8 \text{ kN·m},$$

$$Q^*_{x=3} = V_A - F_2 - F_1 = 475{,}02 - 75{,}4 - 150{,}8 = 248{,}82 \text{ kN};$$

3. Evaluation of the required moment of resistance at the changing position of the beam cross-section:

$$W^*_{req} = \frac{M^*}{R_{wy} \cdot \gamma_c} = \frac{118380}{0{,}85 \cdot 24 \cdot 0.95} = 6108 cm^3,$$

where $R_{wy} = 0{,}85R_y$ – the calculated resistance of the butt weld by manual welding, without the use of physical methods of quality control;

4. Evaluation of the required area of each of the chords of the changed cross-section:

$$A^*_{f_{nomp}} \approx \frac{W^*_{req}}{h} - \frac{t_w \cdot h_w}{6} = \frac{6108}{129} - \frac{1{,}0 \cdot 125}{6} = 47{,}35 - 20{,}83 = 26{,}52 cm^3;$$

5. Leaving the thickness of the chord's sheets as constant, we evaluate their new width:

$$b_f^* = \frac{A_f^*}{t_f} = \frac{26{,}52}{2{,}0} = 13{,}26 \text{ cm.}$$

According to the design requirements and in accordance with the assortment (according to SSTU 8541:2015) we accept a new chord width of 180 mm (from a sheet of 180 × 20 mm).

The new chord width corresponds with the requirements:

$$b_f^* = 180 > 0{,}1h = 0{,}1 \cdot 1250 = 125 \text{ mm},$$
$$b_f^* = 180 > 0{,}5b_f = 0{,}5 \cdot 340 = 170 \text{ mm},$$
$$b_{f,min}^* = 180 > 180 \text{ mm;}$$

6. Determination of the geometric characteristics of the changed cross-section of the beam:

$$I_x^* = \frac{t_w \cdot h_w^3}{12} + 2A_f^* \left(\frac{h_0}{2}\right)^2 + 2\frac{b_f \cdot t_f^3}{12} =$$

$$= \frac{1{,}0 \cdot 125^3}{12} + 2 \cdot (18 \cdot 2) \cdot 63{,}5^2 + 2 \cdot \frac{18 \cdot 2^3}{12} = 162670 + 29322 = 452992 cm^4,$$

$$W_x^* = \frac{I_x^*}{h/2} = \frac{2I_x^*}{h} = \frac{2 \cdot 452992}{129} = 7023 \text{ cm}^3;$$

7. Verification of the strength of the selected beam cross-section (at a distance of 3 m from its support):

$$\sigma_x^* = \frac{M_{x=3}^*}{W_x^* \cdot \gamma_c} = \frac{118380}{7023 \cdot 0{,}95} = 17{,}74 \text{ kN/cm}^2 =$$
$$= R_{wy} = 0{,}85R_y = 0{,}85 \cdot 24 = 20{,}4 \text{ kN/cm}^2.$$

8. Verification of the given stresses at the level of chord welds in the cross-section changing position:

$$\sigma_{red}^* = \sqrt{\sigma_{1x}^{*2} + 3\tau_x^{*2}} = \sqrt{16{,}33^2 + 3 \cdot 1{,}26^2} \approx$$
$$\approx 22{,}58 \text{ kN/cm}^2 \le 1{,}15 \cdot 24 \cdot 0{,}95 = 30{,}36 \text{ kN/cm}^2;$$

where τ_1^* and σ_1^* – are normal and tangential stresses at position 1 of the beam cross-section (at the level of the chord welds):

$$\sigma_x^* = \frac{M_{x=3}^*}{I_x^* \Big/ \frac{h_w}{2}} = \frac{M_{x=3}^*}{2I_x^*} \cdot h_w = \frac{118380}{2 \cdot 452992} \cdot 125 = 16{,}33 \text{ kN/cm}^2;$$

$$\tau_x^* = \frac{Q_{x=3}^* \cdot S_f^*}{I_x^* \cdot t_w} = \frac{248{,}82 \cdot 2286}{452992 \cdot 1{,}0} = 1{,}26 \text{ kN/cm}^2,$$

where S_f^* – statical moment of the chord, relative to the neutral axis "X" of the beam cross-section:

$$S_f^* = b_f^* \cdot t_f \cdot \left(\frac{h_0}{2}\right) = 18 \cdot 2 \cdot \frac{127}{2} = 2286 \text{ cm}^3.$$

The conditions are fulfilled, so the beam strength at the chords cross-section changing position is provided.

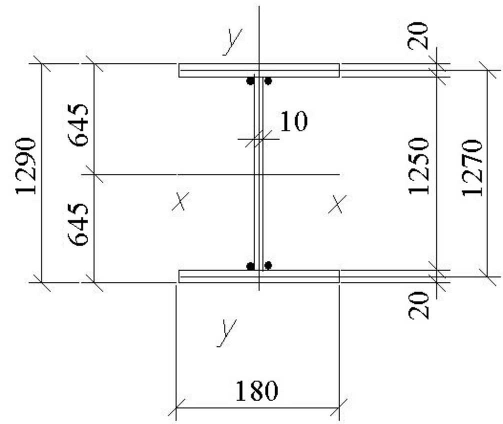

Figure 14 – Changed cross-section of the beam

5.5 Verification of the beam MB1 general stability

First, we need to determine the necessity of the beam MB1 calculation on the general stability.

1. According to the requirements of [1], the general stability of the beam doesn't need to be checked in a case of transferring load through a solid flooring, which continuously resists on the compressed chord of the beam and is securely connected with it. In our case, this condition is

fulfilled, because the compressed chord of the beam is securely fastened with minor beams (whose bay is 2,9 m) and steel flooring, welded to the upper chords of the beams.

2. For example, consider the option when the steel flooring, for reasons beyond our control, will not be welded to the compressed chord of the beam MB1, or secondary beams will be superficially supported by the beam MB1. Then, according to the requirements of [1], the general stability of the beam also does not need to be verificated if the condition of formula (40) in [1] is fulfilled, namely if:

$$\frac{l_p}{b_f} < \left[0{,}41 + 0{,}0032\frac{b_f}{t_f} + \left(0{,}73 - 0{,}016\frac{b_f}{t_f}\right)\cdot\frac{b_f}{h}\right]\cdot\sqrt{\frac{E}{R_y}},$$

where l_p – is the distance between the joints of the compressed chord of the beam from the plane of the beam, the distance between the secondary beams; if the relationship $\frac{b_f}{t_f}$ < 15, then in the formula it is necessary to accept this size of the relation equal to $\frac{b_f}{t_f}$ = 15:

$$\left[0{,}41 + 0{,}032\cdot 15 + \left(0{,}73 - 0 < 016\cdot 15\right)\cdot\frac{18}{129}\right]\times$$

$$\times\sqrt{\frac{2{,}06\cdot 10^4}{24}} = \left[0{,}41 + 0{,}048 + 0{,}49\cdot 0{,}17\right] =$$

$$= 0{,}5416\cdot 29{,}3 = 15{,}87\ ,$$

$\frac{l_p}{b_f}$ = 290/18 = 15,82 < 15,87, that is, requirement [1] is fulfilled and the general stability of the beam does not need to be verificated.

5.6 Calculation of chord welds of the beam MB1

By means of such welds chords are attached to a beam wall:

1. Evaluating of the shear force per 1 cm of beam length:

$$T = \frac{Q_{\max}\cdot S_f^*}{I_x^*} = \frac{396{,}35\cdot 286}{452992} = 4{,}74\ \text{kN};$$

2. Determination of the thickness of the belt welds, which are performed by automatic welding.

The calculation of welds is carried out on the metal of the fusion boundary, because:

$$\beta_f \cdot R_{wf} = 1{,}1 \cdot 1{,}8 = 19{,}8 \text{ kN/cm}^2 > \beta_z \cdot R_{wz} =$$

$$= \beta_z \cdot 0{,}45 R_u = 1{,}15 \cdot 0{,}45 \cdot 36 = 18{,}63 \text{ kN/cm}^2,$$

$$k_f \geq \frac{T}{2\beta_z \cdot R_{wz} \cdot \gamma_{wz} \cdot \gamma_c} = \frac{4{,}74}{2 \cdot 1{,}15 \cdot 0{,}45 \cdot 36 \cdot 1{,}0 \cdot 1{,}0} =$$

$$= 0{,}127 \text{ cm} \approx 1{,}3 \text{ mm},$$

where $\beta_f = 1{,}1$; $\beta_z = 1{,}15 -$ melting coefficients, which are accepted according to [1];

$\gamma_{wf} = 1$; $\gamma_{wz} = 1$; $\gamma_c = 1 -$ coefficients of working conditions, respectively, of the weld and of the structure, are accepted according to [1];

R_{wf} = 18 kN/cm^2 – the calculated resistance of the corner welds on the weld metal;

R_{wz} = 0,45R_u – the calculated resistance of the corner welds on the metal of the fusion boundary;

R_u = 36 kN/cm^2 – calculated resistance of steel to tensile, compression and bending by temporary resistance [1].

However, at a thickness t = 20 mm (thicker of the welded elements) the cathetuses of the welds should be taken at least 6 mm, so constructively we accept k_f = 6 mm > 1,3 mm.

5.7 Calculation of supporting edge of the beam MB1

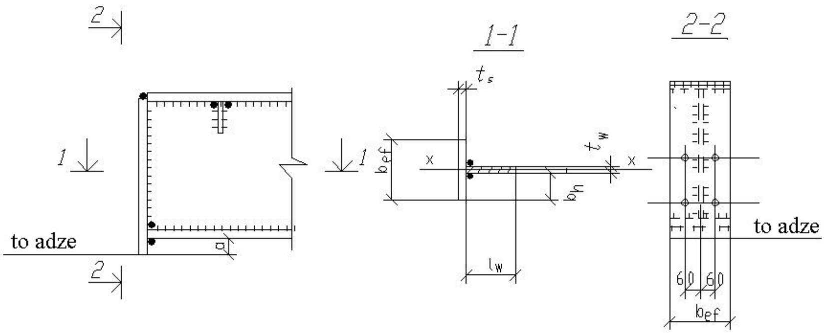

Figure 15 – Calculated scheme

1. Determination of the dimensions of the supporting rib.

Evaluation of the required cross-sectional area:

$$A_{req}^{h}=\frac{V_{MB}}{R_{p}}=\frac{475{,}02}{33{,}6}=14{,}14\ \text{cm}^2.$$

Structurally, we accept the width of the support rib:

$$b_h=\frac{b_{ef}-t_w}{2}=\frac{18-1{,}0}{2}=8{,}5<0{,}5\sqrt{\frac{E}{R_y}}=$$

$$=0{,}5\sqrt{\frac{20600}{24}}=14{,}65$$

So, the condition is fulfilled.

Evaluating the required rib thickness:

$$t_s\geq\frac{A_{req}^{h}}{b_{ef}}=\frac{14{,}14}{18}=0{,}79cm.$$

Structurally, we accept $t_s=14$ mm.

2. Verification of the stress in the rib from the condition of its crushing:

$$\sigma=\frac{V_{MB}}{A_h\cdot\gamma_c}=\frac{475{,}02}{(18\cdot1{,}4)\cdot1{,}0}=26{,}39kN/cm^2<33{,}6kN/cm^2$$

3. Verification of the stability of the support rib from the plane of the beam.

The support rib is considered as a conditional hinged-supported compressed rod of complex cross-section, the height of which is equal to the height of the beam wall.

The total area of the rod consists of the area of the support rib and the area of the wall part of the beam with length $0{,}65t_w\sqrt{\frac{E}{R_y}}$.

$$A_{ef}=A_h+A_w=b_{ef}\cdot t_s+0{,}65\sqrt{\frac{E}{R_y}}\cdot t_w=$$

$$=18\cdot1{,}4+0{,}65\sqrt{\frac{2{,}06\cdot10^4}{24}}\cdot1{,}0=39{,}2+20{,}95=60{,}15\ \text{cm}^2.$$

Next, we evaluate the necessary geometric characteristics of the rod:

$$I_x=\frac{t_s\cdot b_h^3}{12}=\frac{1{,}4\cdot18^3}{12}=2561\ \text{cm}^4,$$

$$i_x=\sqrt{\frac{I_x}{A_{ef}}}=\sqrt{\frac{2561}{60{,}15}}=6{,}52\ \text{cm},$$

Then:

$$\lambda_x = \frac{h_w}{i_x} = \frac{125}{6{,}52} = 20{,}54 \rightarrow \varphi = 0{,}949,$$

$$\sigma = \frac{V_{MB}}{\varphi \cdot A_{ef} \cdot \gamma_c} = \frac{475{,}02}{0{,}949 \cdot 60{,}15 \cdot 0{,}95} = 16{,}67 kN/cm^2 < 24 kN/cm^2.$$

4. Determination of the cathetuses of the welds k_f, which attach the support rib to the wall of the beam:

$$l_w = h_w - 2 = 125 - 2 = 123 \text{ cm}.$$

Since, the welds will be made by hand welding, then $\beta_f = 0{,}7$ and $\beta_z = 1$.

The weld cathetus is determined from the condition of the shear on the weld metal, because of:

$$\beta_f \cdot R_{wf} = 0{,}7 \cdot 1{,}8 = 12{,}6 \text{ kN/cm}^2 > \beta_z \cdot R_{wz} =$$

$$= \beta_z \cdot 0{,}45 R_u = 1 \cdot 0{,}45 \cdot 37 = 16{,}6 \text{ kN/cm}^2,$$

$$k_f \geq \frac{V_{ГБ}}{2\beta_f \cdot R_{wf} \cdot \gamma_{wf} \cdot \gamma_c \cdot l_w} = \frac{475{,}02}{2 \cdot 0{,}7 \cdot 18 \cdot 1 \cdot 1 \cdot 123} = 0{,}31 cm.$$

According to the requirements of [1] in these conditions, the weld cathetus must be at least 6 mm. Structurally, we accept it as k_f = 8 mm.

5.8 Verification of the wall local stability of the composite beam MB1

1. Verification of the necessity to check the local stability of the beam wall $\overline{\lambda_w}$.

According to [1], the stability of the beam walls does not need to be verificated if the condition is fulfilled for the given stresses, and the conditional flexibility of the wall $\overline{\lambda_w}$ does not exceed (in the absence of local stresses in beams with bilateral welds) 3,5, i.e. $\overline{\lambda_w}$ < 3,5.

$$\sigma^*_{red} = \sqrt{\sigma^{*2}_{1x} + 3\tau^{*2}_x} = \sqrt{16{,}33^2 + 3 \cdot 1{,}26^2} \approx$$

$$\approx 22{,}58 \text{ kN/cm}^2 \leq 1{,}15 \cdot 24 \cdot 0{,}95 = 30{,}36 \text{ kN/cm}^2,$$

where σ^*_{red}, σ^*_{x1}, $\tau^*_{xy,1}$ – accordingly, the stresses at the level of the chord welds at the place of change of the chords cross-section, the normative and tangential stresses at point 1 at the level of the chord welds.

In our case, the first condition is fulfilled, so we need to verificate the second requirement.

Evaluation of the conditional flexibility of the beam wall:

$$\overline{\lambda_w} = \frac{h_w}{t_w}\sqrt{\frac{R_y}{E}} = \frac{125}{1{,}0}\sqrt{\frac{24}{2{,}06\cdot 10^4}} = 4{,}26 > 3{,}5,$$

that is, in our case, one of the regulatory conditions [1] is not fulfilled, so the stability of the beam wall must be verificated.

We install the main stiffeners in the places of attachment (support) of secondary beams through one step.

Other secondary beams will be attached to short ribs (cross-section 2–2).

2. Verification of the local stability of the wall in the first compartment length α_1 = 2900 mm *at a distance* $x = a_1 - h_w / 2 = 2900 - 1250 / 2 =$ $= 2275$ mm.

We evaluate the moment and the transverse force at a distance X_1 from the support of the beam, i.e. in the calculated cross-section:

$$M_{x=2,275} = \left(V_{MB} - F_1\right)\cdot X_1 = (475{,}02 - 75{,}4)\cdot 2{,}275 = 909{,}14 kN \times cm;$$

$$Q_{\max} = V_{MB} - F_1 = 399{,}62 kN.$$

Verification of the local stability of the wall is performed according to the formula:

$$\sqrt{\left(\sigma/\sigma_{cr}\right)^2 + \left(\tau/\tau_{cr}\right)^2} \le \gamma_c,$$

where γ_c = 1,1.

$$\sigma = \frac{M_{x=2,275}}{I_x^*}\cdot\frac{h_w}{2} = \frac{90914}{452992}\cdot\frac{125}{2} \approx 12{,}55$$ kN/cm² – stresses at the level of chord welds at a distance x = 2,275 m from the beam support.

$$\tau_{x=2,275} = \frac{Q_{x=2,275}}{h_w\cdot t_w} = \frac{399{,}62}{125\cdot 1{,}0} = 3{,}2$$ – average tangential stresses at a distance x = 2,275 m from the support of the beam.

Critical normal and tangential stresses:

$$\sigma_{cr} = \frac{C_{cr}\cdot R_y}{\overline{\lambda_w^2}} = \frac{30{,}3\cdot 24}{4{,}26^2} = 40{,}07 \text{ kN/cm}^2,$$

$$\overline{\lambda_w} = \frac{d}{t_w}\sqrt{\frac{R_y}{E}} = \frac{125}{1{,}0}\sqrt{\frac{24}{2{,}06\cdot 104}} = 4{,}26,$$

where $d = h_w$ = 125 cm;

C_{cr} – coefficient for composite beams, according to Table 21 [1], depending on the value of the coefficient:

$$\delta = \beta \cdot \frac{b_f}{h_w} \cdot \left(\frac{t_f}{t_w}\right)^3 = 0{,}8 \cdot \frac{18}{125} \cdot \left(\frac{2{,}0}{1{,}0}\right)^3 = 0{,}92 .$$

According to [1] for beam cells beams coefficient $\beta = 0{,}8$;

According to [1] at $\delta = 0{,}92$ – $C_{cr} = 30{,}3$.

$$\tau_{cr} = 10{,}3 \cdot \left(1 + \frac{0{,}76}{\mu^2}\right) \cdot \frac{R_s}{\lambda_w^2} = 10{,}3 \cdot \left(1 + \frac{0{,}76}{1{,}82^2}\right) \times \frac{0{,}58 \cdot 24}{4{,}26^2} \approx 8{,}4 \text{ kN/cm}^2,$$

$\mu = \dfrac{227{,}5}{125} = 1{,}82$ – the ratio of the larger side to the smaller, i.e. $\mu = \dfrac{a}{h_w}$.

After determination of all components, we perform a verification:

$$\sqrt{\left(\frac{12{,}55}{40{,}07}\right)^2 + \left(\frac{3{,}2}{8{,}4}\right)^2} = \sqrt{0{,}594} = 0{,}77 < 1{,}1,$$

The condition is fulfilled.

If the condition is not fulfilled, it is necessary to reduce the bay of the main ribs of the stiffeners or increase the wall thickness of the beam.

3. Determination of the width and thickness of the main stiffeners.

According to the relevant requirements [1], the width of the rib:

$$b_h \geq \frac{h_w}{30} + 40mm = \frac{1250}{30} + 40 = 81{,}66mm.$$

We accept $b_n = 85$ mm.

Rib thickness from C235 steel is $R_y = 23$ kN/cm^2.

$$t_s \geq 2b_h\sqrt{\frac{R_y}{E}} = 2 \cdot 0{,}85 \cdot \sqrt{\frac{23}{2{,}06 \cdot 10^4}} = 0{,}668 \text{ cm}.$$

We accept $t_s = 8$ mm

The ribs will be welded to the wall of the beam by manual welding, according to [1] with bilateral welds $h_f \geq 6$ mm.

5.9 Calculation of secondary beams connection with the main beam MB1

In our example, the beams are connected in one level.

Consider one of the possible options for connecting the beams and perform the calculation of its elements. Preliminarily, for structure reasons, we set the overall dimensions of the onlays.

For connection of secondary beams I30a with the main beam MB1 we accept onlays with the sizes 250 × 250 × 8 mm from steel of the C235 class. Onlays are attached during installation to the edges of the main beam with coarse precision bolts of class 4.6 with a diameter of d = 20 mm.

The calculated force is the reference reaction of the secondary beam, which is increased by 20 %:

$$P = 1{,}2 \cdot V_{S.B} = 1{,}2 \cdot 75{,}4 = 90{,}48 kN.$$

1. Verification of the strength of structurally accepted onlays.

To make this, we pre-evaluate:

a) the value of the calculated moment:

$$M_{slope} = 90{,}48 \cdot 20{,}0 = 1809{,}60 kN \times cm;$$

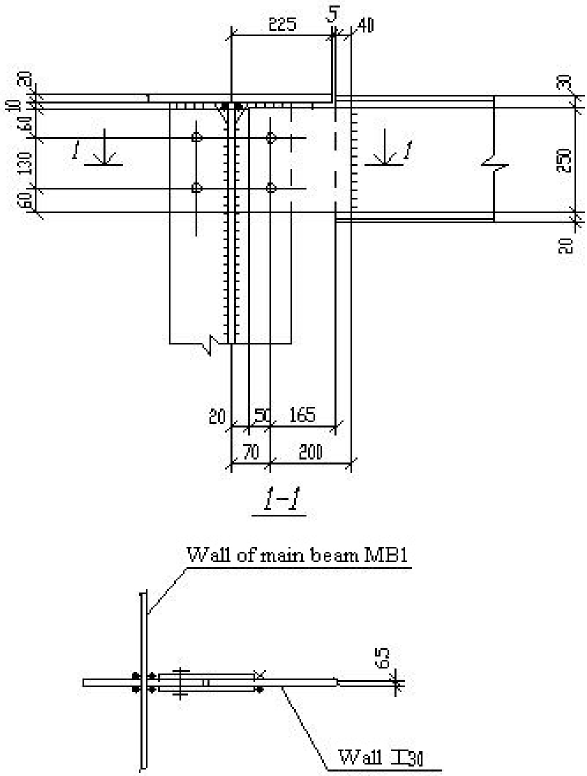

Figure 16 – Estimated connection of the beams

b) the moment of onlays resistance:

$$M_{slope} = \frac{2 \cdot 0,8 \cdot 25^2}{6} = 166,66cm^3;$$

c) the total cross-sectional area of the onlays:

$$\sum A_{slope} = 25 \cdot 0,8 \cdot 2 = 40,0cm;$$

d) normal and tangential stresses in the onlays:

$$\sigma_{slope} = \frac{M_{slope}}{W_{slope}} = \frac{1809,60}{166,66} = 10,86kN/cm^2,$$

$$\tau_{slope} = \frac{P}{\sum A} = \frac{90,48}{40,0} = 2,26kN/cm^2,$$

$$\sigma_{red} = \sqrt{\sigma_{slope}^2 + 3\tau_{slope}^2} = \sqrt{10,86^2 + 3 \cdot 2,26^2} =$$

$$= 11,54 \text{ kN/cm}^2 < R_y \cdot \gamma_c = 1,15 \cdot 23 \cdot 1,1 = 29,1 \text{ kN/cm}^2.$$

The condition is fulfilled, i.e. the bearing capacity of the onlays is provided.

2. Calculation of the welds attaching onlays to the wall of a secondary beam.

These welds are performed by manual welding, the weld cathetus must be taken:

– the minimum allowable: $k_{f,min}$ = 5 mm;

– the maximum allowable: $k_{f,max} = 1,2t_{min} = 1,2t_w = 1,2 \cdot 6,5 = 7,8$ mm.

So, we accept k_f = 8 mm.

3. Determination of the tangential stresses in the welds from the calculated force P:

a) on the metal weld:

$$\tau_f = \frac{P}{2\beta_f \cdot k_f \cdot l_w} = \frac{90,48}{2 \cdot 0,7 \cdot 0,8 \cdot (25-1)} = 3,37 \text{ kN/cm}^2;$$

b) on the metal at the fusion boundary:

$$\tau_z = \frac{P}{2\beta_z \cdot k_f \cdot l_w} = \frac{90,48}{2 \cdot 1 \cdot 0,8 \cdot (25-1)} = 2,36 \text{ kN/cm}^2.$$

4. Evaluation of the bending moment, which is created by the support reaction of the beam relative to the center of gravity of the weld:

$$M = 90,48 \cdot 20,0 = 1809,60 \text{ kN·cm}.$$

5. Determination of the normal stresses in the welds from the action of the bending moment:

a) on the metal weld:

$$\sigma_f = \frac{M}{W_f} = \frac{6M}{2\beta_f \cdot k_f \cdot l_w^2} = \frac{6 \cdot 1809{,}6}{2 \cdot 0{,}7 \cdot 0{,}8 \cdot (25-1)^2} = 16{,}83 \text{ kN/cm}^2;$$

b) on the metal at the fusion boundary:

$$\sigma_z = \frac{M}{W_z} = \frac{6M}{2\beta_z \cdot k_f \cdot l_w^2} = \frac{6 \cdot 1809{,}6}{2 \cdot 1 \cdot 0{,}8 \cdot (25-1)^2} = 11{,}78 \text{ kN/cm}^2.$$

6. Verification of the welds strength:

a) equivalent stresses on the metal weld:

$$\sigma_f = \sqrt{\sigma_f^2 + \tau_f^2} = \sqrt{16{,}83^2 + 3{,}37^2} = 17{,}164 \text{ kN/cm}^2 <$$

$$< R_{wf} \cdot \gamma_c = 18 \cdot 1 = 18{,}0 \text{ kN/cm}^2;$$

7. Calculation of the bolts attaching onlays to an edge of the main beam.

For connection we accept M20 class 4.6 bolts.

Determination of the design forces that can be perceived by one bolt in the connection:

a) from the conditions of the shear:

$$N_b = R_{bs} \cdot \gamma_b \cdot A_b \cdot n_s = 15 \cdot 0{,}9 \cdot \left(3{,}14 \cdot 1^2\right) \cdot 2 = 87{,}78kN$$

where R_{bs} = 15 kN/cm^2 – the calculated resistance of the bolt shear [1];

$\gamma_b = 0{,}9$ – coefficient of working conditions of the connection [1];

A_b – the cross-sectional area of the bolt; $n_s = 2$ – the number of shear areas.

b) from the conditions of the crushing:

$$N_b = R_{bp} \cdot \gamma_b \cdot d \cdot \sum t_{\min} = 36{,}5 \cdot 0{,}9 \cdot 2 \cdot 0{,}8 \approx 52{,}6kN$$

where R_{bp} = 36,5 – the design crushing resistance of the bolted elements (steel C235) [1]; d = 2 cm – the diameter of the bolt; Σt_{min} = 0,8 the smallest total thickness of the elements compressed in one direction.

8. Determination of the required number of bolts in the connection:

$$n \geq \frac{P}{[N_{\min}] \cdot \gamma_c} = \frac{90{,}48}{52{,}6 \cdot 1} = 1{,}72.$$

We accept two bolts, which location is shown in Figure 16. The bolts must be arranged in accordance with the requirements of [1].

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GLOSSARY

Beam cell – балочна клітка
Overlap – перекриття
Flooring – настил
Flooring beam – балка настилу
Stiffener – ребро жорсткості
Wide flange section – двотавровий переріз
T-section – тавровий переріз
Beam web – стінка балки
Beam flange – полка балки
Overhang – звис полки
Curve – епюра, крива
Beam chord – пояс балки
Butt weld – стиковий зварний шов
Cross-section, cross section – переріз
Edge – ребро
Shear – зріз
Surface support – поверхове обпирання
Same level support – однорівневе обпирання
Onlay – накладка
Span – проліт
Bay – крок

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з курсу

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*(для студентів усіх форм навчання
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