UKRAINE
NATIONAL UNIVERSITY OF LIFE AND ENVIRONMENTAL SCIENCES

Construction Department

METHODICAL INSTRUCTIONS
for laboratory work on the discipline "Building constructions" for students of the educational direction 192 "Construction and Civil Engineering"

Calculation of building structures for strength, rigidity and fire resistance

Kyiv-2020

УДК 624.01+699.827

Methodical instructions contain examples of calculation of building structures for strength, rigidity and fire resistance.

Recommended by the Academic Council of the Faculty of Construction and Design of the National University of Life and Environmental Sciences of Ukraine (Protocol № __3__ of October 20, 2020).

Authors: O.A. Fesenko, Ye.A. Dmitrenko

Reviewer: Associate professor, PhD, E.A. Bakulin

Educational edition

Methodical instructions for laboratory work at the discipline "Building Structures" for students of the educational direction 192 - "Construction and Civil Engineering"

Authors: FESENKO OLEH ANATOLIYOVYCH,
DMYTRENKO YEVHEN ANATOLIYOVYCH

Responsible for the issue - Associate professor O.A. Fesenko

Signed for printing.
Region - view. arch. 5.0.
Circulation ______ ex.

Format 60x84 1/16.
Cond. print. sheets
Repl №.

Editorial and publishing department of NULES of Ukraine.
Street Heroyiv Oboroni, 15, Kyiv, 03041
Tel. 527-80-49

General instructions

Calculation of building constructions by the method of limit state design is an integral part of the discipline «Building constructions», which is studied by students of the educational direction 192 «Construction and Civil Engineering».

The мethodical instructions provide for the study of safety factors, types of loads and effects on building constructions. They contain the following tasks:

- to master the requirements of current normative acts and documents of building constructions design;
- to acquire skills of analysis of calculation results by the method of ultimate limit states;
- learn how to apply safety factors for loads and materials, coefficients of responsibility and working conditions.

«Calculation of building constructions for strength, rigidity and fire resistance» contains eight laboratory works, which summarize the algorithms for collecting the load on the structures, determination of the internal forces in the structural elements, assessment of the construction bearing capacity.

Successful performance of any laboratory work is possible in compliance with the requirements of normative acts and documents with using of the reference literature.

Laboratory work № 1

Calculation of the load on the floor and coating of the building

Goal: collection and determination of load per unit area (1 m^2) on the floor and coating of the building

Work progress:

1. Establish the purpose and class of responsibility of the building, category of responsibility of the construction, area of building, types of coating and floor of the building, own weight of plates of covering and floors (tab. 1.1).
2. Set the composition (construction) of the coating (see appendix A); perform load calculation from the weight of materials and constructions that are part of the coating, taking into account their density (see appendix C); calculate the characteristic and ultimate design value of loads taking into account the coefficients of reliability for the load γ_f (see [1], tab. 5.1).
3. Set the characteristic value of the snow load depending on the construction area (see [1], appendix E); calculate the ultimate design value of snow load taking into account the coefficient of reliability for the load γ_f (see [1], tab. 8.1).
4. Calculate the total characteristic and ultimate design value for the combination of constant (dead load of materials and structures of the coating) and short-term (snow) loads; calculate the ultimate design value of the load on the coating, taking into account the coefficient of reliability for responsibility γ_n (see [2], tab. 5).
5. Set the composition (construction) of the floor (see appendix B); perform the calculation of dead load from the weight of materials and constructions that are part of the floor, taking into account their density (see appendix C); calculate the characteristic and ultimate design value of loads taking into account the coefficients of reliability for the load γ_f (see [1], tab. 5.1).
6. Establish the characteristic value of evenly distributed temporary load from people depending on the purpose of the building (see [1], table 6.2); calculate

the ultimate design value of the load from people, taking into account the safety factor for the load γ_f (see [1], p. 6.7).

7. Calculate the total characteristic and ultimate design value for the combination of constant (own weight of materials and constructions of the coating) and short-term (load from people) loads; calculate the ultimate design value of the load on the floor, taking into account the coefficient of reliability for responsibility γ_n (see [2], tab. 5).
8. The results of the calculation of the load on the floor and coating of the building write down into two tables. An example of the calculation is given in appendix D.1.

Table 1.1

Initial data for load calculation

on coating and floor of the building

<table>
<tr><td rowspan="2">The first letters of the name</td><td>Area of construction</td><td>Purpose</td><td>Class of responsibility</td><td>Dead load of the plates, kN / m^2</td><td>Type of coating</td><td>Floor type</td><td rowspan="2">Note</td></tr>
<tr><td>1</td><td>2</td><td>3</td><td>4</td><td>5</td><td>6</td></tr>
<tr><td>A, K, U</td><td>Kiyv</td><td>Hospital</td><td>CC2</td><td>3.0</td><td>8</td><td>2</td><td rowspan="10">Plates of the coating and floor - reinforced concrete round hollow</td></tr>
<tr><td>B, L, V</td><td>Odessa</td><td>Dwelling house</td><td>CC2</td><td>3.1</td><td>7</td><td>4</td></tr>
<tr><td>C, M, W</td><td>Lviv</td><td>Supermarket</td><td>CC3</td><td>3.2</td><td>1</td><td>2</td></tr>
<tr><td>D, N, X</td><td>Kharkiv</td><td>Polyclinic</td><td>CC2</td><td>3.3</td><td>2</td><td>6</td></tr>
<tr><td>E, O, Y</td><td>Chernihiv</td><td>Administrative building</td><td>CC3</td><td>3.2</td><td>5</td><td>1</td></tr>
<tr><td>F, P, Z</td><td>Rivne</td><td>Restaurant</td><td>CC2</td><td>3.1</td><td>3</td><td>7</td></tr>
<tr><td>G, Q</td><td>Poltava</td><td>School</td><td>CC2</td><td>3.0</td><td>4</td><td>5</td></tr>
<tr><td>H, R</td><td>Dnipro</td><td>Concert Hall</td><td>CC3</td><td>2.9</td><td>6</td><td>3</td></tr>
<tr><td>I, S</td><td>Donetsk</td><td>Dormitory</td><td>CC2</td><td>2.8</td><td>9</td><td>6</td></tr>
<tr><td>J, T</td><td>Uzhhorod</td><td>Bookstore</td><td>CC2</td><td>2.9</td><td>1</td><td>1</td></tr>
<tr><td colspan="8">Class of responsibility of the coating and floor constructions - Б</td></tr>
<tr><td colspan="8">Note. The initial data are taken by the first letters of the name:
Surname - from column "1"
Name - from columns "2" and "4"
Patronymic - from columns "3" and "5".</td></tr>
</table>

Control questions

1) The characteristic value of the load is…
2) The ultimate design load value is…
3) What safety factor s are used to calculate the load on the constructions of the floor and coating of the building?

Laboratory work № 2

Calculation of secondary steel floor beam

Goal: determination of bearing capacity and rigidity of steel beam of the building floor

Work progress:

1. Set the initial data for the calculation: the span of the main L (m) and secondary l (m) beams, the step of the secondary beams a (m), the strength class of steel (see tab 2.1); the characteristic and ultimate design value of the load on the floor of the building take from the results of laboratory work №1.
2. Draw a scheme of the main and secondary floor beams location.
3. Determine the design strength of rolled steel (tab. 7.1 [3]):
   - for compression, tension, bending (yield strength) according to the following formula:

$$R_y = R_{yn}/\gamma_m, \tag{2.1}$$

where R_{yn} is the characteristic strength of rolled steel, MPa;

γ_m= 1,025 – coefficient of reliability for the material, see table. 7.2, [3];

   - for shearing according to such formula:

$$R_s = 0.58 \times R_y. \tag{2.2}$$

4. Calculate the operational and ultimate design value of the load per 1 m on the secondary floor beams by multiplying the value of the load on the floor with the step of the beams:
   - operational value:

$$q_{op}^{b} = q_{char}^{pr} \times a; \tag{2.3}$$

- ultimate design value:

$$q_{ult}^{b} = q_{ult}^{pr} \times a; \tag{2.4}$$

where a - spacing of the secondary floor beams, m.

5. Determine the design internal forces in the beam:
   - bending moment (kN·m) by the formula

$$\mathrm{M}_{bend} = (q_{ult}^{b} \times l^2)/8; \tag{2.5}$$

   - shear force (kN) by the formula

$$Q_{tr} = (q_{ult}^{b} \times l)/2. \tag{2.6}$$

Draw the diagrams of internal forces M and Q.

6. Determine the moment of resistance of the section from the condition of bending strength of the beam
   - bending strength condition:

$$\frac{\mathrm{M}_{bend} \times \gamma_n}{W_{n.min} \times R_y \times \gamma_c} \leq 1, \tag{2.7}$$

   - moment of resistance of section:

$$W_{n.min} = \frac{\mathrm{M}_{bend} \times \gamma_n}{R_y \times \gamma_c}, \quad \text{см}^3. \tag{2.8}$$

where γ_c= 0.9 - the coefficient of working conditions of the beam, see tab. 5.1 [3].

7. Accept an I-beam section of according to the GOST 26020-83 assortment with the following geometric characteristics:
   - moment of resistance of section, W_x, cm^3; area of cross-section, A, cm^2;
   - moment of inertia of section, I_x, cm^4;
   - static moment of resistance of section, S_x, cm^3;
   - section wall thickness, *s*, mm; weight of 1 m, kg.
8. Clarify the load taking into account the own weight of the beam
   - operational value:

$$q_{op,1}^{b} = q_{op}^{b} + q_{o.w}; \tag{2.9}$$

   - ultimate design value:

$$q_{ult.1}^{b} = q_{ult}^{pr} + q_{o.w.} \times \gamma_f \times \gamma_n\,, \tag{2.10}$$

where $q_{o.w.}$ - own weight of the beam, kg / m.

9. Clarify the design internal efforts

- bending moment, kN·m

$$M^1_{bend} = (q^b_{ult.1} \times l^2)/8; \tag{2.11}$$

- shear force, kN

$$Q^1 = (q^b_{ult.1} \times l)/2. \tag{2.12}$$

Draw the diagrams of internal forces M and Q.

10. Check the bending strength condition of the beam:

$$\frac{M^1{}_{bend} \times \gamma_n}{W_x \times R_y \times \gamma_c \times c_x} \leq 1, \tag{2.13}$$

where c_x - coefficient for calculation taking into account the development of plastic deformations during bending, tab. M.2, [3].

11. Check the condition of the shear strength of the beam:

$$\frac{Q^1 \times S_x}{I_x \times t \times R_S \times \gamma_c} \leq 1, \tag{2.14}$$

where t - wall thickness of the section, mm.

12. Check the beam rigidity condition:

$$f = \frac{5 \times q^b_{op,1} \times l^4}{384 \times E_S \times I_x} \leq f_u = \frac{l}{200}, \tag{2.15}$$

where E_s = 2.06·10^5 MPa - modulus of elasticity of steel.

An example of the calculation of the beam is given in Appendix D.2.

Table 2.1

Initial data for the calculation of the secondary steel floor beam

Initial letters of full name	Span of the main beam, m	Span of secondary beams, m	Step of secondary beams, m	Steel grade class
	1	2	3	4
A, K, U	9,0	6,0	1,0	C245
B, L, V	9,6	4,8	1,1	C255
C, M, W	10,2	5,1	1,2	C275
D, N, X	10,8	5,4	1,3	C285
E, O, Y	11,0	5,5	1,4	C295
F, P, Z	11,8	4,2	1,5	C245
G, Q	12,0	5,0	1,6	C255
H, R	12,6	5,2	1,7	C275
I, S	13,0	5,3	1,8	C285
J, T	15,0	4,9	2,0	C295

Note. The source data takes the first letters of the full name:

Name - from the column “1”

Surname - from the columns “2” and ”4”

Patronymic - from the column “3”

Control questions:

1) What safety factor s were used in the calculation of the beam?
2) What geometrical characteristics of beam cross-section are necessary for calculation?
3) What strength conditions are performed for the beam?

Laboratory work № 3

Calculation of the node of the coating metal truss

Goal: calculation of the nodal elements cross-section of the truss; determination of the welds' length of the nodal elements.

Work progress:

1. Set the initial data for the calculation: the number of the truss node, the strength class of steel and the trusses' step - table. 3.1; ultimate design value of the load on the coating, kN / m^2 – table D.1.
2. Perform calculation of nodal loads; determine the support reactions of the truss.
3. Calculate the internal forces in the node elements of the truss by the methods of structural mechanics.
4. Define the cross section of the node elements of the truss from the paired corners, check the strength and stability of the elements.
   - strength condition under central tension and compression (p.8.1.1, [3]):

$$\frac{N \times \gamma_n}{A_n \times R_y \times \gamma_c} \leq 1, \tag{3.1}$$

where N - internal force in the element, kN, A_n – area of cross-section;

   - stability condition under central compression (p.8.1.3, [3]):

$$\frac{N \times \gamma_n}{\varphi \times A \times R_y \times \gamma_c} \leq 1, \tag{3.2}$$

where φ - coefficient of stability under central compression.

$$\varphi = \frac{0,5}{\bar{\lambda}^2} \times (\delta - \sqrt{\delta^2 - 39,48 \times \bar{\lambda}^2}) \leq 7,6/\bar{\lambda}^2, \tag{3.3}$$

$$\delta = 9,87 \times (1 - \alpha + \beta \times \bar{\lambda}) + \bar{\lambda}^2, \tag{3.4}$$

where $\bar{\lambda} = \lambda \times \sqrt{R_y/E}$ - conditional flexibility,

α and β - coefficients that characterize the initial irregularities of the form and residual stresses and are determined by tab. 8.1 [3] depending on the type of the rod cross-section and the stability curve, which are given in Appendix Ж, [3].

5. Determine the dimensions of the angular welds for the elements of the truss assembly under the action of the longitudinal force N from the condition of strength (p. 16.1.16, [3]):

$$\frac{N \times \gamma_n}{\beta_f \times k_f \times l_w \times R_{wf} \times \gamma_c} \leq 1, \qquad (3.5)$$

where β_f - coefficient, the value of which is taken from tab. 16.2, [3];

k_f – cathetus of an angular weld, mm;

l_w - design length of the angular weld, mm;

R_{wf} - the (conditional) shear design strength of the angular welds in the plane of the weld metal, MPa.

The dimensions of the angular welds and the construction of the connection must meet the following requirements (p.16.1.5, [3]):

- the cathetus of the corner weld k_f (Fig. 3.1) should not exceed *1.2t*, where *t* - the smallest of the thicknesses of the welded elements;
- the weld cathetus along the rounded edge of the shaped rolling thickness *t*, as a rule, should not exceed *0.9t*;
- the design length of the angular weld must be not less than $4k_f$ and not less than 50 mm.

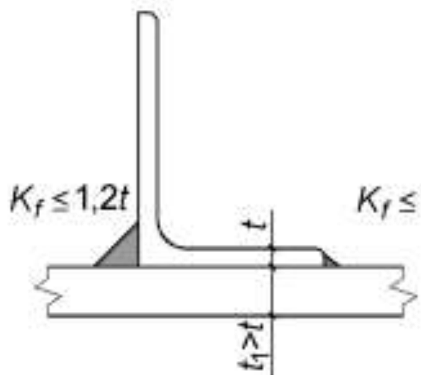


Fig. 3.1 - The sizes of angular welds

Calculation scheme of the coating truss with node numeration is given on fig.3.2

An example of the calculation of the coating metal truss is given in Appendix D.3.

Table 3.1

The initial data for the node calculation of the steel truss coating

№	Name of source data	Letters of surnames									
		A, K, U	B, L, V	C, M, W	D, N, X	E, O, Y	F, P, Z	G, Q,	H, R	I, S,	J, T
1.	Number of node	3	4	5	6	7	11	12	13	14	15
2.	Strength class of the steel	C255		C275		C285		C345		C375	
3.	Step of the truss a, m	5.0			5.5				6.0		

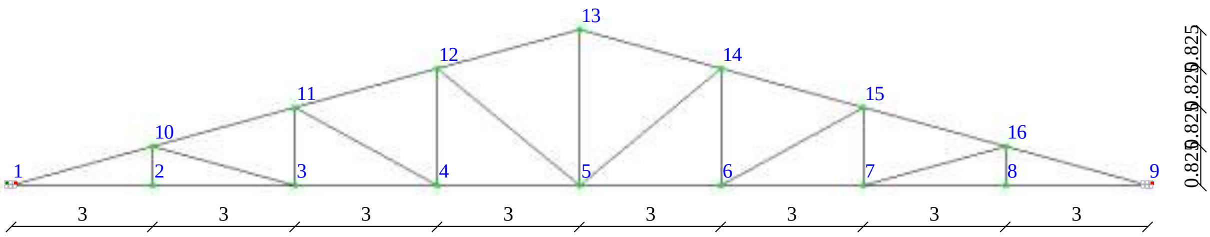


Figure 3.2 - Calculation scheme of the coating truss

Control questions

1) The strength condition of the compressed and stretched element of the truss.
2) The stability condition of the compressed element of the truss.
3) The strength condition of the angular weld of the truss element.

Laboratory work № 4

Node calculation of the wooden truss coating

Goal: perform the calculation of the nodal elements' cross-section of truss made of solid or glued wood.

Work progress:

1. Set the initial data for the calculation: type of cross-section; wood species; wood strength class; operational class of wood - 2. The number of the node and the internal forces in the elements of the node taken from laboratory work №3.
2. Determine the design value of the characteristic of wood X_d (see p.6.2.1, [4]):

$$X_d = k_{mod} \times \frac{X_k}{\gamma_M}, \quad (4.1)$$

where X_d - the characteristic value of strength, MPa (see Appendix Б, [4]);

γ_M - safety factor for the material (tab. 6.1, [4]);

k_{mod} - conversion factor that takes into account the duration of the load and temperature-humidity operating conditions (see tab. A.1, [4]).

3. Define the cross-section of the elements of the truss' node, check the strength and stability of the elements from the condition of the tensile and compressive strength of the element (p. 9.2.1, [4]):

- the condition of tensile strength (compression) along the fibers according to the formula:

$$\frac{\sigma_{t(c),0,d}}{f_{t(c),0,d}} \leq 1, \quad (4.2)$$

where $f_{t(c),0,d}$ - the design value of tensile strength (compression) along the fibers, MPa, see formula (4.1);

- design tensile stress (compression) along the fibers (p. 9.2.2, [4]) by the formula:

$$\sigma_{t(c),0,d} = \frac{N_d}{A_{net}}, \qquad (4.3)$$

- the required cross-sectional area of the element, mm^2, according to the formula:

$$A_{net} = \frac{N_d}{f_{t,0,d}}, \qquad (4.4)$$

where N_d - is the calculated tensile (compression) force along the fibers of the element, kN. Should be taken from the laboratory work №3.

The sizes of element's section made from solid or glued wood accept by the recommended assortment of lumber, see Appendix E, [4].

4. Check the stability of the elements of the truss' node (see paragraph 9.3.3, [4])
   - the condition of resistance to compression along the fibers according to the formula:

in the plane of the truss $$\frac{\sigma_{t(c),0,d}}{k_{c,y} \times f_{t(c),0,d}} \leq 1, \qquad (4.5)$$

out of the truss' plane $$\frac{\sigma_{t(c),0,d}}{k_{c,z} \times f_{t(c),0,d}} \leq 1, \qquad (4.6)$$

where $k_{c,y}$ and $k_{c,z}$ are the coefficients of longitudinal bending

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}} \qquad (4.7)$$

$$k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}} \qquad (4.8)$$

$$k_y = 0,5\left(1 + \beta_c\,(\lambda_{rel,y} - 0,3) + \lambda_{rel,y}^2\right), \qquad (4.9)$$

$$k_z = 0,5\left(1 + \beta_c\,(\lambda_{rel,z} - 0,3) + \lambda_{rel,z}^2\right) \qquad (4.10)$$

- the relative flexibility of the element

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \qquad (4.11)$$

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}}. \qquad (4.12)$$

- the flexibility of the central-tensioned element:

$$\lambda = \frac{l_{ef}}{i}, \quad (4.13)$$

where l_{ef} – design length of the element according to the tab. 9.1, [4].

i - the radius of inertia of the element's cross section relative to the corresponding axis.

The location scheme of the cross-section's axes of the wooden element is shown in Fig. 4.1.

An example of the calculation of the node of the coating metal truss is given in Appendix D.4.

Table 4.1

The initial data for the calculation of the node of the wooden truss coating

<table>
<tr><th rowspan="2">№</th><th rowspan="2">Name of source data</th><th colspan="10">Letters of surnames</th></tr>
<tr><th>A, K, U</th><th>B, L, V</th><th>C, M, W</th><th>D, N, X</th><th>E, O, Y</th><th>F, P, Z</th><th>G, Q,</th><th>H, R</th><th>I, S,</th><th>J, T</th></tr>
<tr><td>1.</td><td>Wood species</td><td colspan="2">Pine</td><td colspan="2">Spruce</td><td colspan="2">Oak</td><td colspan="2">Maple</td><td colspan="2">Beech</td></tr>
<tr><td rowspan="3">2.</td><td colspan="11">Wood strength class (Appendix Б, [4])</td></tr>
<tr><td>coniferous, table Б.1</td><td colspan="2">C30</td><td colspan="2">C35</td><td colspan="2">C40</td><td colspan="2">C45</td><td colspan="2">C50</td></tr>
<tr><td>deciduous, table Б.2</td><td colspan="3">D34</td><td colspan="2">D40</td><td colspan="2">D50</td><td colspan="3">D60</td></tr>
<tr><td>3.</td><td colspan="11">Operational class of wood - 2</td></tr>
<tr><td>4.</td><td colspan="11">Type of cross-section - square</td></tr>
</table>

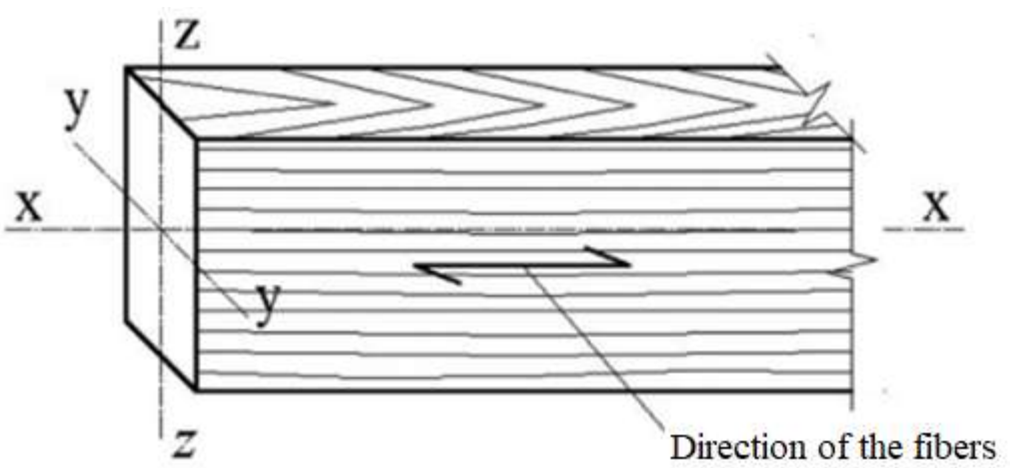


Рисунок 4.1 - The location scheme of the cross-section’s axes of the wooden element

Control questions

1) Types of wood used for building structures.
2) What characteristics of wood are needed to calculate for the calculation on tension and compression.
3) Condition of tensile and compressive strength.

Laboratory work № 5

Calculation of the wooden glued beam on bending

Goal: performing the selection of the cross-section of the beam; checking the bending and chipping strength of the beam.

Work progress:

1. Set the initial data for the calculation: beam length *l*, m; step of the beams *a*, m; types of wood; wood strength class; operational class of wood - 2 (see tab. 5.1). The maximum design value of the load on the floor of the building should be taken from the results of laboratory work №1.
2. Determine the design value X_d of the characteristics of wood (see p.6.2.1, [4]):

   - design value of bending strength relatively to the main axis *y*

$$f_{m,y,d} = k_{mod} \times \frac{f_{m,g,k}}{\gamma_M}, \quad (5.1)$$

where $f_{m,g,k}$, MPa – the characteristic value of the bending strength of homogeneous glued wood;

γ_M = 1,25 – safety factor for the material for glued wood (tab. 6.1, [4]);

- design value of chipping strength:

$$f_{v,d} = k_{mod} \times \frac{f_{v,g,k}}{\gamma_M}, \quad (5.2)$$

where $f_{v,g,k}$, MPa - characteristic value of chipping strength of homogeneous glued wood.

3. Calculate design values of internal forces acting in cross-section of the beam – bending moment $M_{y,d}$ and shear force V_d.
4. Determine the cross-section of the beam from the condition of bending strength (paragraph 9.4.1, [4]):

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1, \quad (5.3)$$

where $f_{m,y,d}$, MPa - the design value of the bending strength relative to the main axis y, see formula (5.1).

5. Estimate the value of bending stress (paragraph 9.4.1, [4]):

$$\sigma_{m,y,d} = \frac{M_{y,d}}{W_{y,d}} \tag{5.3}$$

where $M_{y,d}$, кN*m – design bending moment;

$W_{y,d}$, mm^3 – required resistance moment of cross-section relative to *y* axis.

6. Estimate the required resistance moment of beam cross-section:

$$W_{y,d} = \frac{M_{y,d}}{f_{m,y,d}}, \tag{5.5}$$

The sizes of element's cross-section are accepted on assortment.

7. Estimate required value of chipping stress (paragraph 9.4.2, [4]):

$$\tau_d = \frac{V_d \cdot S_{br}}{I_{br} \cdot b_{ef}} \tag{5.6}$$

where V_d, кН – design shear force;

S_{br}, мм^3 – static moment (gross) part of sheared cross-section relative to the neutral axis;

I_{gr}, мм^4 – the moment of inertia of the section (gross) relative to the neutral axis;

b_{ef}, мм - the estimated width of the cross-section of the beam.

8. Check the strength condition of the beam:

- on bending

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1 \tag{5.7}$$

- on chipping

$$\frac{\tau_d}{f_{v,d}} \leq 1 \tag{5.8}$$

An example of the calculation of the wooden floor beam is given in Annex D.5.

Table 5.1

Initial data for the calculation of the wooden beam per bend

№	Name of source data	Letters of surnames									
		A, K, U	B, L, V	C, M, W	D, N, X	E, O, Y	F, P, Z	G, Q	H, R	I, S	J, T
1.	Beam length, m	6	4.8	5.1	5.4	5.5	4.2	5.6	4.7	6.3	4.5
2.	Step of the beams, m	1.0		2.0		1.5		1.8		1.2	
3.	Type of wood	Pine		Spruce		Oak		Maple		Beech	
4.	Strength class of glued wood, table Б.3	GL 24h		GL 28h			GL 32h			GL 36h	
5.	Operational class of wood - 2										
6.	The type of cross-section - rectangular										

Control questions

1) What are the peculiarities of glued beams’ calculation?
2) What characteristics of wood needed to calculate the beam on the bending and chipping?
3) Bending and chipping strength conditions.

Laboratory work № 6

Calculation of a stone masonry pier from ceramic brick

Goal: choose a mark of ceramic brick and cement-sand mortar to perform calculation of the stone masonry walls of building.

Work progress:

1. Set the initial data for the calculation: the number of floors of the house n_{fl}; floor height of the house (m); window opening width (m) (tab. 6.1). The maximum design value of the load on the floor and coating of the building should be taken from the results of laboratory work №1 (see tab. D.1, D.2).
2. Draw a scheme of the window openings location in the outer wall of the building.
3. Collect the load transmitted from the floor and coating to the outer wall of the building.
4. Calculate the design internal forces in the pier of the 1st floor: the longitudinal force N_{Ed} (kN) and the bending moment M_{Ed} (kN·m).
5. Condition for ensuring the load-bearing capacity of masonry:

$$N_{\mathrm{Ed}} < N_{\mathrm{Rd}}, \quad (6.1)$$

where N_{Ed}, кН – design value of the vertical load on the wall;

N_{Rd}, кН – design value of the load-bearing capacity of the wall.

6. Determine the required value of compressive strength of masonry made of ceramic bricks f_d (p. 11.1.2.1.2, [5]).

$$f_d = \frac{N_{Ed}}{\Phi_i \cdot b_{\text{п}} \cdot t}, \quad (6.2)$$

where Φ_i – the coefficient of reduction of the load-bearing capacity of the wall above or below the wall, which depends on the flexibility of the wall and the eccentricity of the load,

$$\Phi_i = 1 - 2\frac{e_i}{t}, \quad (6.3)$$

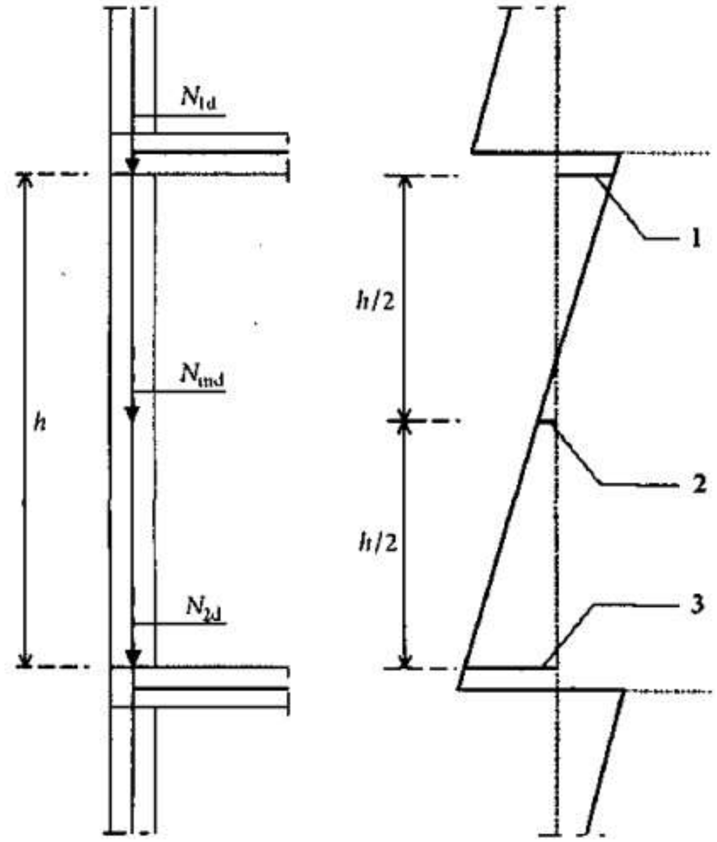
e_i - eccentricity of application of loading from above or from below a wall;

$$e_i = \frac{M_{id}}{N_{id}} e_{he} + e_{init} \geq 0{,}05t, \quad (6.4)$$

M_{id} – the calculated bending moment at the top or bottom of the wall caused by the eccentricity of the load in the area of support of the floor slabs (floor);

N_{id} – design vertical load at the top or bottom of the wall;

t, mm – thickness of the wall; $b_п$, mm– pier width.



1 –moment at the top of the wall M_{1d}; 2 –moment in the middle of the wall M_{2d}; 3 – moment in the bottom of the wall M_{3d}

Figure 6.1 – Design vertical load scheme and bending moments in the masonry walls

7. Accept stone masonry of ceramic brick with strength f_d on heavy cement-sand mortar f_m; determine the compressive strength of masonry. An example of the calculation of the wooden floor beam is given in Appendix D.6.

Table 6.1

Initial data for the calculation of the pier of the outer wall

1st floor of the building

№	Name of initial data	Letters of surnames									
		A, K, U	B, L, V	C, M, W	D, N, X	E, O, Y	F, P, Z	G, Q	H, R	I, S	J, T
1.	Number of floors	4				5			6		
2.	Floor height, m	2.8		3.0		3.2		3.3		3.5	
3.	Dimensions in the axes ($L_1 x L_2$),m	6,5x6,0		6,6x6,4		6,3x6,5		6,8x6,3		6,4x6,2	
4.	Dimensions of a window opening (bxh), m	1,5x1,2		1,5x1,6				1,8x1,5		2,0x1,8	
5.	The thickness of the outer wall, mm	510					380				
6.	Loads on the floor and coating - from laboratory work №1, tables D.1 and D.2										

Control questions

1) Types and groups of masonry elements.
2) Compressive strength of masonry elements.
3) Design value of the vertical resistance of a wall or column of masonry.

Laboratory work № 7

Calculation of fire resistance for reinforced concrete beam

Goal: determine the fire resistance of reinforced concrete unstressed freely-supported beam using tabular data and zonal method.

Work progress:

1. Set the degree of fire resistance of the building, the class of concrete, the dimensions of the cross-section and the distance to the axis of the beam, reinforcement, longitudinal reinforcement of the beam - reinforcement class, number and diameter of rods (see table 7.1);
2. Set the normalized fire resistance class of the beam depending on the degree of fire resistance of the building according to [6] (see table D.1);
3. Determine the fire resistance of the beam according to the tabular data according to [10]:
   - calculate the design values of the load on the beam during a fire $q_{Ed,fi}$ and under normal conditions $q_{\mathrm{E}d}$;
   - calculate the design values of bending moments in the beam during a fire $M_{Ed,fi}$ and under normal conditions M_{Ed};
   - calculate the reduction factor η_{fi} that determines the load level of the beam during a fire by the formula:

$$\eta_{fi} = \frac{M_{d,fi}}{M_{Ed}} \leq 0{,}7. \tag{7.1}$$

   - compare the geometric characteristics of the cross section of the beam with the minimum required values in table D.6;
4. Determine the fire resistance of the beam by zonal method, check the condition of fire resistance:
   - divide half of the cross-section of the beam into n≥3 parallel zones of equal thickness;
   - calculate the average temperature for each cross-sectional area;

- determine graphically the coefficients of reduction of concrete strength on compression $k_c(\theta_i)$;
- determine the average coefficient of concrete strength reduction, which takes into account the temperature change of each cross-sectional zone, according to the formula:

$$k_{c,\,m} = \frac{(1-0{,}2/n)}{n} \cdot \sum_{i=1}^{n} k_{c,(\theta_i)} \tag{7.2}$$

- calculate the width of damaged cross-sectional area of the beam by the formula:

$$a_z = w\left[1 - \frac{k_{c,m}}{k_{c,(\theta_M)}}\right], \tag{7.3}$$

- reduce the cross-sectional dimensions of the beam by the value of a_z, mm;
- calculate the temperature in the reinforcing bars of the beam;
- calculate the reduced strength of the beam reinforcement by the formula:

$$f_{sd,\,fi}(\theta_m) = k_{v(\theta)} \times f_{sd}, \tag{7.4}$$

where $k_v(\theta) = \frac{\sum k(\theta_i)}{n_v}$ - the average coefficient of the strength reduction of the *v*-th reinforcement row,

θ - temperature of the *i*-th reinforcing rod;

$k(\theta i)$ - the coefficient of reduction of the strength of the *i*-th rod;

n_v - the number of reinforcing rods in the *v*-th reinforcing row.

- calculation of the residual bearing capacity of the beam perform for a reduced cross-section with reduced reinforcement strength:

determine the height of the compressed zone of concrete from the equation of equilibrium of the reduced cross section of the beam:

$$\lambda x = A_s \times f_{sd},\, f_i\,(\theta_m) / f_{cd,\,fi}(20) \times b_{fi}, \tag{7.5}$$

A_s - cross-sectional area of tensioned reinforcement;

$f_{sd,fi}(\theta_m)$ - design tensile strength of the reinforcement at higher temperature θ_m;

$f_{cd,\,fi\,(20)} = f_{ck} / \gamma_{c,\,fi}$ - design compressive strength of concrete at normal temperature;

b_{fi} - the width of the reduced cross section;

determine the shoulder of the inner pair of forces - compression in concrete and tensioned reinforcement:

$$z = (d_{fi} - 0,5 \times \lambda x), \tag{7.6}$$

d_{fi} - working height of the reduced cross section;

z - the distance between the stretched reinforcement and the compressed zone of concrete;

λx - estimated height of the compressed zone of concrete;

- determine the bearing capacity of the reduced cross-section of the beam by the formula:

$$M_u = A_s \times f_{sd,fi}(\theta_m) \times z \tag{7.7}$$

- check the condition of the strength of the beam in case of fire by the formula:

$$M_u > M_{Ed,fi}. \tag{7.8}$$

An example of the reinforced concrete floor beam calculation on the fire resistance is given in Appendix D.7.

Table 7.1

Initial data for the calculation of the reinforced concrete beam for fire resistance

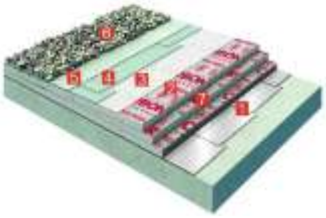
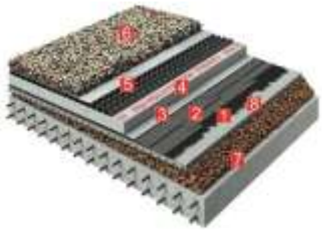
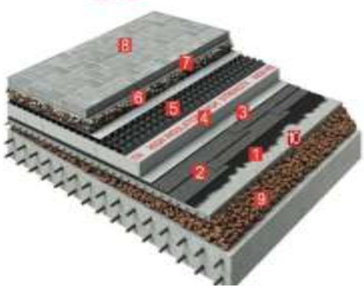
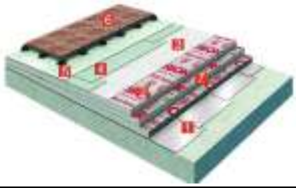
№	Initial data	The first letters of the name									
		A, K, U	B, L, V	C, M, W	D, N, X	E, O, Y	F, P, Z	G, Q	H, R	I, S	J, T
1.	The degree of fire-resistance of the building	I				II			III		
2.	Concrete class	C16 / 20			C20 / 25		C25 / 30			C30 / 35	
3.	Section dimensions, *hxb*, mm	500x250			600x250		600x300			400x200	
4.	Distance to the axis of the reinforcement, mm	15		20		25		30		40	
5.	Longitudinal reinforcement	4Ø18 A500 / As = 1018 mm^2		3Ø22 A400 / 1140 mm^2		3Ø20 A500 / As = 942 mm^2		2Ø25 A400 / As = 982 mm^2		2Ø28 A400 / 1232 mm^2	

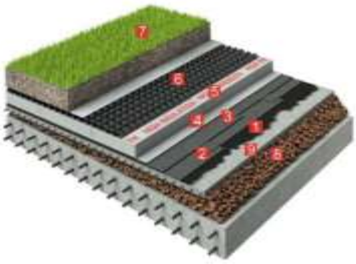
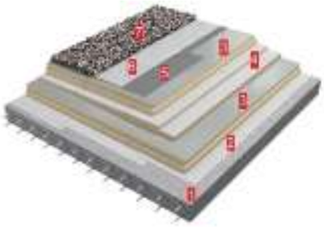
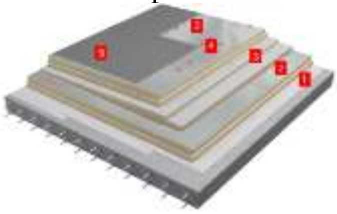
Control questions

1) What determines the degree of fire resistance of a building?
2) Limit states of building structures on fire resistance.
3) Methods of calculation of building structures for fire resistance.

Appendix A

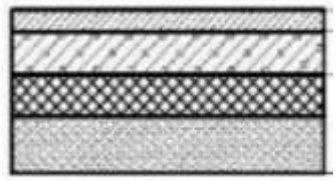
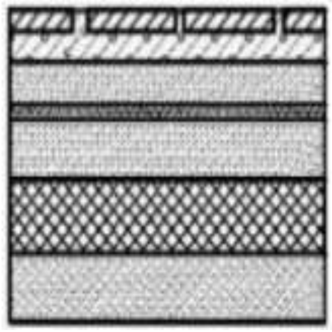
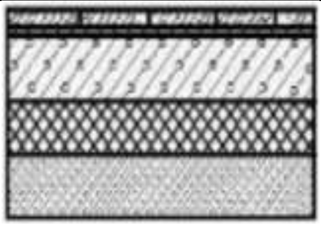
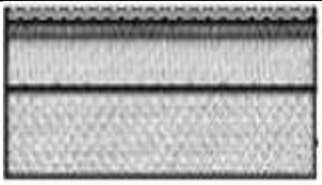
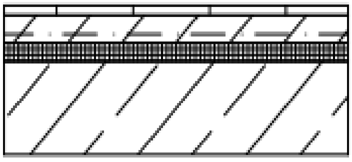
Examples of coating compositions

№	Name of the SWEETONDALE coating system, sketch	The composition of the coating			
		The name of the layer	Density, kg / m3	Thickness, mm	
				I temperature zone	II temperature zone
1	2	3	4	5	6
1	SD-ROOF Ballast	Reinforced concrete slab	2500	220	220
		1 - Bipol EPP	1200	2	2
		2, 7 - Extrusion expanded polystyrene CARBON PROF / CARBON PROF RF	26-42	220	200
		4 - Polymer membrane LOGICROOF V-RP	1200	2	2
		6 - Ballast	1600	70	70
2	SD-ROOF Inverse	Reinforced concrete slab	2500	220	220
		7 - Expanded clay gravel	600	70	70
		8 - C / s screed	1800	50	50
		2 - Bipole EPP	1200	2	2
		4 - Extrusion expanded polystyrene CARBON PROF / CARBON PROF RF	26-42	200	180
		5 - Drainage membrane	1200	2	2
		6 - Ballast	1600	70	70
3	SD-ROOF Sidewalk	Reinforced concrete slab	2500	220	220
		9 - Expanded clay gravel	600	70	70
		10 - C / s screed	1800	50	50
		2 - Bipole EPP	1200	2	2
		4 - Extrusion expanded polystyrene XPS CARBON PROF RF	26-42	200	180
		5 - Drainage membrane	1200	2	2
		6 - Ballast	1600	70	70
		8 - Paving slabs	2000	40	40
4	SD-ROOF Terrace	Reinforced concrete slab	2500	220	220
		1 - Bipol EPP	1200	2	2
		2, 7 - Extrusion expanded polystyrene XPS CARBON PROF RF	26-42	220	200
		4 - Polymer membrane LOGICROOF V-RP	1200	2	2
5	SD-ROOF Auto	Reinforced concrete slab	2500	220	220
		7 - Expanded clay concrete	1200	70	70
		8 - C / s screed	1800	50	50
		2 - Technoelast EPP	1200	2	2

№	Name of the SWEETONDALE coating system, sketch	The composition of the coating			
		The name of the layer	Density, kg / m3	Thickness, mm	
				I temperature zone	II temperature zone
1	2	3	4	5	6
		3 - Extrusion expanded polystyrene XPS CARBON SOLID 500	26-42	210	190
		5 - Reinforced concrete slab	2500	100	100
		6 - Asphalt concrete	2100	70	70
6	SD-ROOF Green 	Reinforced concrete slab	2500	220	220
		8 - Expanded clay gravel	600	70	70
		9 - C / s screed	1800	50	50
		2 - Technoelast EPP	1200	2	2
		3 - Technoelast GREEN	1200	2	2
		5 - Extrusion expanded polystyrene XPS CARBON PROF RF	26-42	180	160
		6 - Drainage membrane	1200	2	2
		7 - Soil	800	150	150
7	SD-ROOF PIR ballast 	1 - Reinforced concrete slab	2500	220	220
		2 - Bipole EPP	1200	2	2
		3, 4 - Plates heat-insulating PIR	36	130	120
		5 - Polymer membrane LOGICROOF V-RP	1200	2	2
		7 - Ballast	1600	70	70
8	SD-ROOF Optima 	Reinforced concrete slab	2500	220	220
		1 - Bipol EPP	1200	2	2
		2, 3 - Plates heat-insulating PIR	36	130	120
		5 - Polymer membrane LOGICROOF V-RP	1200	2	2
9	SD-ROOF Prof. 	Reinforced concrete slab	2500	220	220
		1 - Bipol EPP	1200	2	2
		2, 3 - Extrusion expanded polystyrene CARBON PROF / CARBON PROF RF	26-42	220	200
		6 - Polymer membrane LOGICROOF V-RP	1200	2	2

Appendix B

Examples of floor structures

№ p / p	Sketch	Floor construction
1	2	3
1		Mosaic concrete - C 12/15-20mm Screed of cement-sand mortar M150 – 44mm Thermal insulation expanded polystyrene – 50-60mm Reinforced concrete floor slab
2		Ceramic tile – 13mm Layer and filling of seams with cement-sand mortar M150 - 15mm Screed with cement-sand mortar M150 – 20mm 1 layer of waterproofing – 20mm Screed with cement-sand mortar M150 – 40mm Thermal insulation expanded polystyrene – 50-60mm Reinforced concrete floor slab
3		Artificial parquet – 19mm Layer of fast-curing waterproof mastic – 1mm Expanded clay concrete Y=1300-1400kg/m^3/M75 – 50mm Thermal insulation expanded polystyrene – 50-60mm Reinforced concrete floor slab
4		Multilayer linoleum – 8mm Layer of adhesive mastic – 5mm Screed with cement-sand mortar M150 – 40mm Reinforced concrete floor slab
5		-Floor covering - 10-20 -Concrete screed C8 reinforced with a grid according to DSTU B.V.2.6-173: 2011 d5 Bp-1 with a cell of 100x100-60mm - Sound insulation of a plate of extruded expanded polystyrene of 28 kg / m3 λ= 0,041 W / (m * C) (flammability group not lower than G2) DSTU B.V.2.7-8-94 - 40 mm -Reinforced concrete slab - 200mm
6	Tile from porcelain stoneware for a floor on grout of seams with mix - 8-10 mm Adhesive mixture - 10 mm Deep penetration primer Concrete screed class. C25 / 30 reinforced with polymer fiber 1.2 kg / m^2 - 50 mm Polyethylene membrane - 1 layer of 200 microns Reinforced concrete floor slab – 250мм	
7	The remote floor from a terrace board on an inclining coupler - 200 mm Extruded expanded polystyrene - 70 mm Reinforced concrete floor slab	

Appendix C

Density of materials

Material	Density kg/m^3	Material	Density kg/m^3
Concrete		**Wood, products from it and other natural organic materials**	
Agloporite concrete	1000…1800	Oak	700
Concrete heavy	2400	Building cardboard, multilayer	650
Concrete on blast furnace slag	1200…1800	Facing cardboard	1000
Vermiculite concrete	400…800	Tow	150
Reinforced concrete	2500	Fibreboard and chipboard	400…800
Expanded clay concrete	1000…1800	Reed plates	200…300
Pumice concrete	800…1600	Peat slabs	200…300
Perlite concrete	600…1200	Fibrolite and arbolite slabs on portland cement	300…800
Tuff concrete	1200…1800	Pine and spruce	500
Shungizitobeton	1000…1400	Glued plywood	600
Solutions		**Heat-insulating materials**	
Gypsum perlite	600	The mats and strips of fiberglass are pierced	150
Covering qypsum sheets	600		
Gypsum slabs	1000…1200		
Complex (sand, lime, cement)	1700		
Cement-perlite	800…1000		
Cement-sand	1800		
Cement-slag	1200…1400		
Masonry, natural stone		**Roofing, waterproofing, cladding and roll materials**	
Limestone	1400…2200	Asphalt concrete	2100
Granite, gneiss, basalt, marble	2800	Oil bitumen	1000…1400
Masonry of ceramic hollow bricks	1200…1600	Products from foamed perlite on bituminous binder	300…400
Masonry of solid silicate brick on cement-sand mortar	1800	Polyvinylchloride linoleum, multi-layered	1600…1800
Masonry of silicate hollow bricks	1400…1500	Polyvinylchloride linoleum on a fabric basis	1400…1800
Tuff	1400…2000	Polyfoam PVC-1, PV-1	100…125
Masonry of ceramic bricks on cement-sand mortar	1800	Expanded polystyrene	100…150
The same, on a cement-slag solution	1700	Polyurethane foam	40…80
The same, on cement-perlite mortar	1600	Perlite plastic concrete	100…200
		Perlite phosphogenic products	200…300
		Mineral wool boards of the increased rigidity on organophosphogenic and starch binder	200
		Mineral wool boards on synthetic binder	50…125

		Mineral wool boards soft, semi-rigid and rigid on synthetic and bituminous binder Plates from resolo-phenol-foldehyde polyfoam Plates from glass staple fiber on synthetic binder	100…300 40..100 50
Backfill Foam glass, gas glass Sand Crushed stone from blast furnace slag, slag pumice, agloporite Crushed stone and sand from the made foam perlite Asbestos-cement sheets, flat Glassine, roofing material, roofing felt, gidroizol	200…400 1600 400…80 200…600 1600…1800 600	**Metals, glass** Aluminum Copper Glass sheets Steel Cast iron	2600 8500 2500 7850 7200

Appendix D

Examples of calculation

D.1 Example of calculating the load on the floor and coating of the building

Initial calculation data: construction area - Chernihiv; purpose of the building - residential building, responsibility class - CC2; own weight of the plate - 3.30 kN / m^2; type of covering - 8, type of overlapping - 2. Values of reliability coefficient for responsibility γ_n are shown in tab. E.1

Table D.1

Calculation of the load on the floor of the building

Name of the load	Load calculation			Characteristic value, kN / m^2	Coefficient of reliability	Design calculated value, kN / m^2
	Thickness, m	Density, t / m^3	9.81			
Constant loads						
Coating type SD-ROOF Optima						
Polymer membrane LOGICROOF V-RP - 2 mm	0.002×1.2×9.81 =			0.024	1.20	0.028
Heat-insulating plates - 130 mm	0.13×0.036×9.81 =			0.046	1.20	0.055
Heat-insulating plates - 130 mm	0.13×0.036×9.81 =			0.046	1.20	0.055
Bipol EPP - 2 mm	0.002×1.2×9.81 =			0.024	1.20	0.028
Reinforced concrete slab coating - 220 mm				3.30	1.10	3.63
Total (permanent)				3.44		3.8
Short-term load						
Snow, Chernihiv	DBN B.1.1-2: 2006, Appendix E			1.72	1.14	1.96
Total (permanent + short-term)				5.16		5.76
Total, taking into account the responsibility of the house, γ_n= 1.05	CC2, category B				1.05	6.05

Table D.2

Calculation of the load on the floor of the building

Name of the load	Load calculation			Characteristic value, kN / m^2	Coefficient of reliability	Design calculated value, kN / m^2
	Thickness, m	Density, t / m^3	9.81			
1	2	3	4	5	6	7
Constant loads						
Ceramic tile - 13 mm	0.013×1.4×9.81 =			0.18	1.2	0.21
Layer of filling of seams with cement-sandy solut. M150 - 15 mm	0.015×1.8×9.81 =			0.26	1.3	0.34
Screed from cement-sandy solut. M150 - 20 mm	0.02×1.8×9.81 =			0.35	1.3	0.46
1 layer of hydrolysis - 20 mm	0.02×0.6×9.81 =			0.12	1.2	0.14
Screed from cement-sandy solut. M150 - 40 mm	0.04×1.8×9.81 =			0.71	1.3	0.92
Thermal insulation from expanded polystyrene - 50 mm	0.05×0.15×9.81 =			0.07	1.2	0.09
Reinforced concrete slab - 220 mm				3.30	1.1	3.63
Total (permanent)				4.99		5.79
Short-term load						
Evenly distributed temporary load from people (residential building)	DBN B.1.1-2: 2006, table 6.2			1.50	1.3	1.95
Total (constant + short-term)				6.49		7.74
Total, taking into account the responsibility of the building, γ_n = 1.05	CC2, category B				1.05	8.13

Conclusion: According to the results of the calculation, the maximum calculated values of the loads on the floor and coating of the building were calculated, which are 6.05 kN / m^2 and 8.13 kN / m^2, respectively.

D.2 Example of calculation of a secondary steel floor beam

Initial calculation data:

The span of the main beam, m - L = 10.8

The span of the secondary beam, m - l = 5.4

Step of secondary beams, m - a = 1.5

Steel strength class - C285

Characteristic value of loading per 1 m^2 of floor, kN/m^2 - q^{fl}_{char} = 6.49 (see Table D.2);

The design calculated value of the load per 1 m^2 of floor, kN/m^2 - $q^{fl}_{d,calc}$ = 8.13 (see table D.2).

Calculation progress:

1. Draw the plan of the building floor constructions location (fig. D.1).

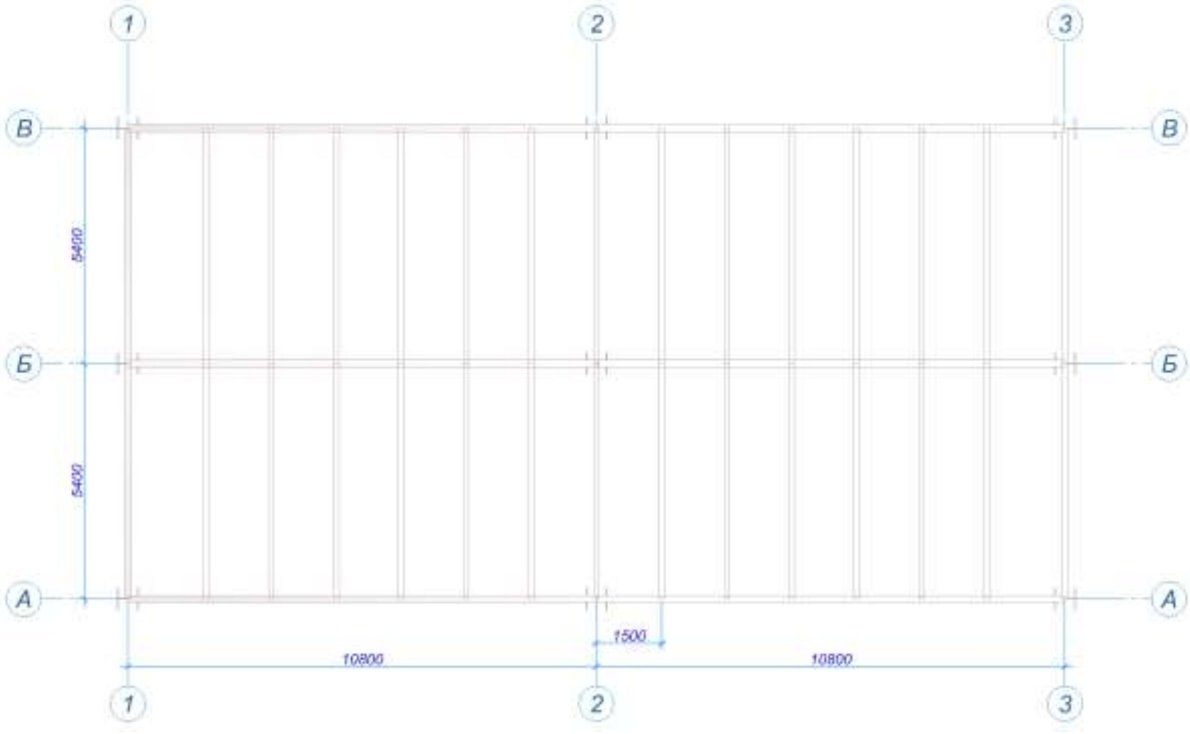


Figure D.1 -The plan of the floor constructions of the building

2. Determine the design strength of rolled steel

   - for compression, tension, bending (yield strength):

$$R_y = \frac{R_{yn}}{\gamma_m} = \frac{285}{1,025} = 278,05 \text{ МПа}$$

   - cutting, shear:

$$R_s = 0.58 \times R_y = 0,58 \times 278,05 = 161,27 \text{ МПа} .$$

3. Calculate the operational and design calculated value of the load per 1 m.p. of the secondary floor beams

   - operational value:

$$q_{op}^{b} = q_{char}^{fl} \times a = 6.49 \times 1.5 = 9.735 \text{ kN / m},$$

   - design calculated value:

$$q_{d,calc}^{b} = q_{d,calc}^{fl} \times a = 8.13 \times 1.5 = 12.2 \text{ kN / m}.$$

4. Determine the design values of internal efforts:

   - bending moment

$$\text{M}_{bend} = \frac{q_{d,calc}^{b} \times l^2}{8} = \frac{12,2 \times 29,16}{8} = 44.45 \text{ kN·m};$$

   - transverse force

$$Q = \frac{q_{d,calc}^{b} \times l}{2} = \frac{12,2 \times 5,4}{2} = 32,94 \text{ кН}.$$

Construct plots of internal efforts of a beam: bending moments M (Fig. D.2) and transverse forces Q (Fig. D.3).

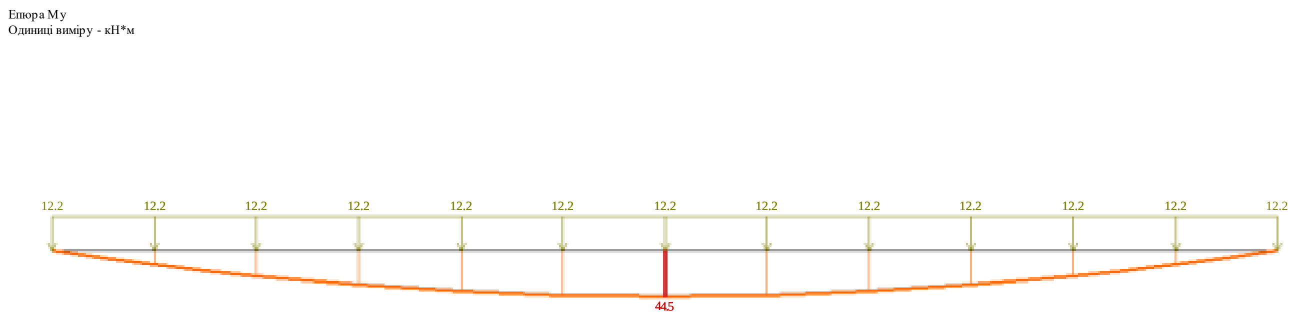


Figure D.2 - Diagram of bending moments M_{bend}, kN·m, in the beam

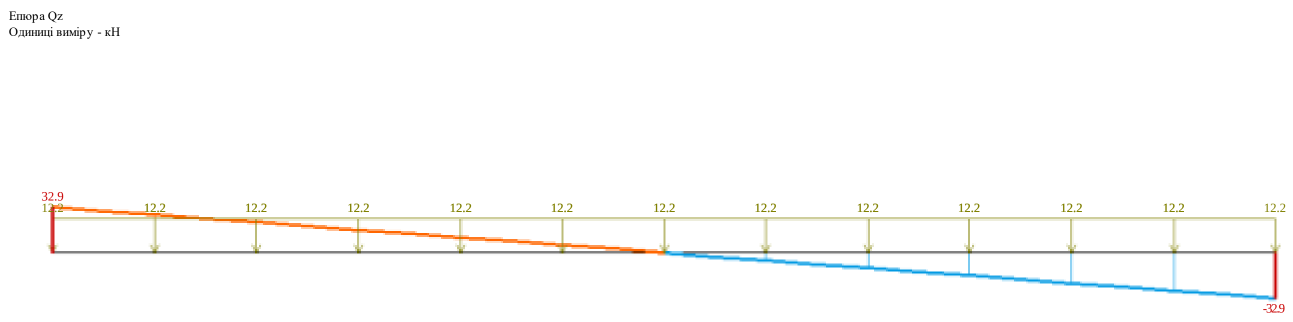


Figure D.3 - Plot of transverse forces Q, kN, in the beam

5. Determine the moment of resistance of the section from the condition of bending strength of the beam:

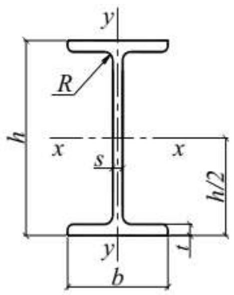
$$W_{n.min} = \frac{M_{bend} \times \gamma_n}{R_y \times \gamma_c} = \frac{44{,}45 \times 10^6}{278{,}05 \times 0{,}9} = 186137{,}58\ mm^3 = 186{,}14\ sm^3;$$

We accept the I-beam section of the beam №20Б1 (Fig. D.4) with the following characteristics:

$W_x = 194{,}3\ sm^3$; A = 28.49 sm^2;

$I_x = 1943\ sm^4$; $s = 5{,}6\ mm$;

$S_x = 110{,}3\ sm^3$; Weight 1m, kg - 22.4.



Сортамент сталевих двутаврів

Двотаври сталеві гарячекатані з паралельними гранями поличок за ГОСТ 26020-83

Приклад позначення:
I30Ш1 / ГОСТ 26020-83

№ профілю	Розміри					Маса 1 м, кг	Площа перерізу, см^2	Довідкові величини для осей						
	h	b	s	t	R			x - x				y – y		
	мм							I_x, см^4	W_x, см^3	i_x, см	S_x, см^3	I_y, см^4	W_y, см^3	i_y, см
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
20Б1	200	100	5,6	8,5	12	22,4	28,49	1943	194,3	8,26	110,3	142,3	28,5	2,23
23Б1	230	110	5,6	9	12	25,8	32,91	2996	260,5	9,54	147,2	200,3	36,4	2,47
26Б1	258	120	5,8	8,5	12	28,0	35,62	4024	312,0	10,63	176,6	245,6	40,9	2,63
30Б1	296	140	5,8	8,5	15	32,9	41,92	6328	427,0	12,29	240,0	390,0	55,7	3,05
35Б1	346	155	6,2	8,5	18	38,9	49,63	10060	581,7	14,25	328,6	529,6	68,3	3,27
35Б2	349	155	6,5	10	18	43,3	55,17	11550	662,2	14,47	373,0	622,9	80,4	3,36

Figure D.4 - Assortment of I-beams with parallel faces of shelves

6. Specify loading taking into account weight of a beam
   - operational value:

$$q_{op,1}^{b} = q_{op}^{b} + q_{d.l.} = 9{,}735 + 0{,}0224 \times 9{,}81 = 9{,}95\ kN/m;$$

   - design calculated value:

$$q_{d.calc,1}^{b} = q_{d,calc}^{b} + q_{d.l} \times \gamma_f \times \gamma_n = 12{,}2 + 0{,}0224 \cdot 9{,}81 \cdot 1{,}05 \cdot 1{,}05 = 12{,}44\, kN/m.$$

7. Specify the calculated internal efforts:
   - bending moment

$$\mathrm{M}_{bend}^{1} = \frac{q_{d,calc,1}^{b} \times l^2}{8} = \frac{12{,}44 \times 5{,}4^2}{8} = 45{,}34\ kN\cdot\mathrm{m};$$

   - transverse force

$$Q^1 = \frac{q_{d,calc,1}^{b} \times l}{2} = \frac{12{,}44 \times 5.4}{2} = 33{,}59\ kN.$$

8. Check the bending strength of the beam:

$$\frac{\mathrm{M}^1{}_{bend} \times 10^3}{W_x \times R_y \times \gamma_c \times \mathrm{c}_x} = \frac{45{,}34 \times 10^3}{194{,}3 \times 278{,}05 \times 0{,}9 \times 1{,}087} = 0{,}86 < 1.$$

   The bending strength of the beam is provided.

9. Check the shear strength of the beam:

$$\frac{Q^1 \times S_x}{I_x \times t \times R_s \times \gamma_c} = \frac{33{,}59 \times 110{,}3 \times 10}{1943 \times 0{,}56 \times 161{,}27 \times 0{,}9} = 0{,}23 < 1.$$

   The shear strength of the beam is provided.

10. Check the rigidity of the beam:

$$f = \frac{5 \times q_{op,1}^{b} \times l^4 \times 10^8}{384 \times E_s \times I_x} = \frac{5 \cdot 9{,}95 \cdot 5{,}4^4 \cdot 10^8}{384 \cdot 2{,}06 \cdot 10^5 \cdot 1943} = 27{,}52\ \text{мм} \approx \frac{l}{200} = \frac{5400}{200} = 27\ mm.$$

   The rigidity of the beam is provided.

Conclusion: According to the results of the calculation, the definition of the cross section of the secondary steel floor beam was performed; bending and shear strength as well as beam rigidity are provided.

D.3 Example of calculation of the metal truss of the coating

Initial calculation data:

Truss node number - 6

Steel strength class - C345

Step of trusses a, m - 5.0

The characteristic value of the load per 1 m2 of coating, kN/m^2 - $q^{coat}_{d,\,calc}$ = 6.05 (see Table D.1).

Calculation progress:

1. Perform the calculation of the nodal loads of the truss (Fig. D.5)

- on the support nodes of the truss:

$$F_{sup} = 0{,}5 \times q^{coat}_{d,calc} \times l_m \times a/\cos\alpha = 0{,}5 \times 6{,}05 \times 3{,}0 \times 5{,}0/0{,}9642 = 47{,}06\ kN,$$

- on the middle nodes of the truss:

$$F_{mid} = q^{coat}_{d,calc} \times l_m \times a/\cos\alpha = 6{,}05 \times 3{,}0 \times 5{,}0/0{,}9642 = 94{,}12\ kN,$$

where cos α = 0,9642 - cosine of the angle of inclination of the upper belt of the coating truss.

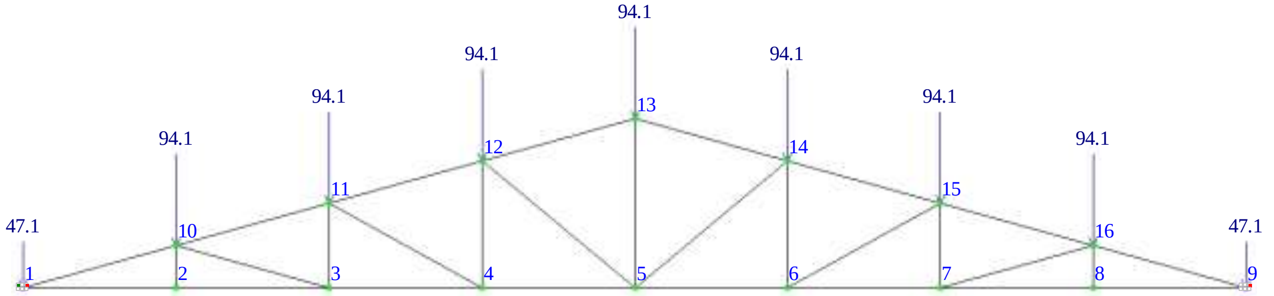


Figure D.5 - The scheme of loading of a truss of the coating

Determine the support reactions of the truss:

$$R_1 = R_9 = \sum (F_{mid} + F_{sup})/2 = \frac{94{,}12 \times 7 + 47{,}06 \times 2}{2} = 376{,}48\ kN.$$

2. Calculate the internal forces in the elements of the node №6 of the coating truss

The internal forces in the elements of the node №6 of the coating truss are determined by the perforating sections method or moment points (Ritter's) [12]. Locations of cross-sections of the truss are shown in Fig. D.6.

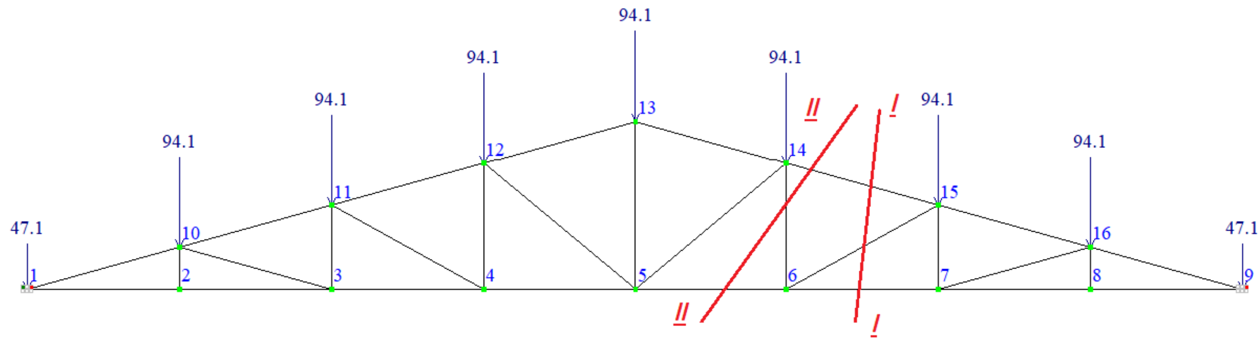


Figure D.6 - Scheme of the truss sections location

To determine the internal forces in the elements №№ 6-7, 6-15, 6-14 and 6-5 consider the equilibrium of the right side of the truss. Compose the equation of equilibrium:

$$\sum M_{15} = 0;\ N_{6-7} \times 1{,}65 + F_{mid} \times 3{,}0 + F_{sup} \times 6{,}0 - R_9 \times 6 = 0,,$$

$$\sum M_9 = 0; \quad N_{6-15} \times 1{,}65 \times cos\beta + N_{6-15} \times 6{,}0 \times sin\beta - F_{mid} \times 9 = 0,,$$

$$\sum M_9 = 0; \quad N_{6-14} \times 9{,}0 - F_{mid} \times 6{,}0 - F_{mid} \times 3{,}0 = 0,,$$

$$\sum M_{14} = 0;\ N_{6-5} \times 2{,}475 + F_{mid} \times 3{,}0 + F_{mid} \times 6{,}0 + F_{sup} \times 9{,}0 -$$
$$R_9 \times 9{,}0 = 0.$$

From the equilibrium equations determine the internal forces in the elements of the node №6:

$$N_{6-7} = \left(R_9 \times 6 - F_{mid} \times 3{,}0 - F_{sup} \times 6{,}0\right)/1{,}65 =$$

$$= (376{,}48 \times 6 - 94{,}12 \times 3{,}0 - 47{,}06 \times 6{,}0)/1{,}65 = 1026{,}76\ kN,$$

$$N_{6-15} = (F_{mid} \times 6{,}0 + F_{mid} \times 3{,}0)/(1{,}65 \times cos\beta + 6{,}0 \times sin\beta) =$$

$$= (94{,}12 \times 6{,}0 + +94{,}12 \times 3{,}0)/(1{,}65 \times 0{,}876 + 6{,}0 \times 0{,}482) = 195{,}3\ kN,$$

$$N_{6-14} = (F_{mid} \times 6{,}0 + F_{mid} \times 3{,}0)/9{,}0 = (94{,}12 \times 6{,}0 + 94{,}12 \times 3{,}0)/9{,}0$$
$$= \quad = 94{,}12 \text{ кН}$$

$$N_{6-5} = (R_9 \times 9{,}0 - F_{mid} \times 3{,}0 - F_{mid} \times 6{,}0 - F_{sup} \times 9{,}0)/2{,}475 =$$

$$= (376{,}48 \times 9{,}0 - 94{,}12 \times 3{,}0 - 94{,}12 \times 6{,}0 - 47{,}06 \times 9{,}0)/2{,}475$$

$$= 855{,}64\ kN.$$

Internal forces in the elements of the truss are determined by the results of static calculation in the software package LIRA-SAPR 2017 (Fig. D.7).

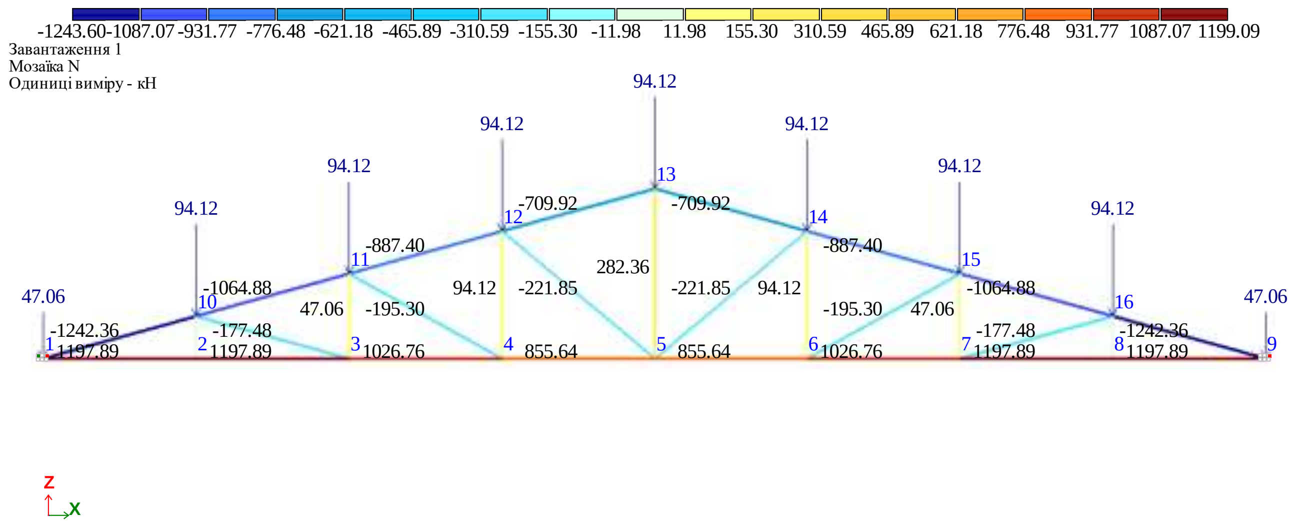


Figure D.7 - Internal forces *N* (kN) in the elements of the truss

3. Select the sections of the elements of the node №6 of the truss from the paired corners on the condition of tensile and compressive strength

- for the element of the lower belt №6-7 the cross-sectional area is:

$$A_{cal}^{6-7} = \frac{N_{6-7}}{R_y \times \gamma_c} = \frac{1026{,}76 \cdot 10}{336{,}59 \cdot 1{,}00} = 30{,}5\ sm^2,$$

where $R_y = R_{yn} / \gamma_m = 345 / 1{,}025 = 336.59$ MPa - the design stress of steel for tension and compression.

According to the assortment of DSTU 2251-93 accept a section from paired corners 100×100×8 mm with the following geometric characteristics: cross-sectional area $A^{6\text{-}7} = 2\times19{,}24 = 38{,}48\ sm^2$, radius of inertia $i_x = 3{,}05$ sm, $i_y = 4{,}47$ sm (with a thickness of 10 mm).

Check the flexibility of the lower belt element №6-7

- in the plane of the truss:

$$\lambda_x = l_{ef,x} / i_x = 300 / 3.05 = 97{,}72 < \lambda_u = 400,$$

- out of the truss plane:

$$\lambda_y = l_{ef,\,y} / i_y = 600 / 4.47 = 134.23 < \lambda_u = 400.$$

The value of the element flexibility №6-7 does not exceed the limit value, see table 13.10, [3].

Similarly, perform the calculation of other elements of the node №6 of the truss. The results of the calculation are given in table. D.3.

Table D.3

The results of the calculation of the elements of the node №6 of the coating truss

№ element	Design effort, kN	Section of the element	Cross-sectional area, cm^2	Estimated length, cm		Radius of inertia, sm		Flexibility			Condition of strength and stability	Cathetus of an angular weld, mm		Design length of the angular weld, sm	
				$l_{ef,x}$	$l_{ef,y}$	i_x	i_y	λ_x	λ_y	λ_u		k_f^o	k_f^n	l_w^o	l_w^n
Lower belt	1197,89	2⎣100x10	38,48	300	600	3.05	4.52	98,36	132,74	400	0.925	6	5	8.92	5.07
Brace 6-15	-195.3	2⎣56x5	10.82	273 .9	342.4	1.72	2.69	159.24	127.29	161.83	0.80	6	5	10.04	5.65
Rack 6-14	94.12	2⎣50x4	7.78	198	247.5	1.54	2.43	128.57	101.85	400	0.36	6	5	5.36	3.24

4. Determine the dimensions of the angular welds for the elements of the lower belt of the truss node №6. Design force N = $N_{6\text{-}7}$–$N_{6\text{-}7}$ = 1026.76 - 855.64 = 171.12 kN. Manual welding perform by the electrode brand Э42А, the design strength of the angular weld R_{wf} = 180 MPa = 18kN/sm^2, β_f = 0.7. The cross section of the lower belt is taken from paired corners 100×100×8 mm. Cathetus of the edge weld accept 6 mm, pen - 5 mm. Calculate the lengths of the angular welds:

- on the edge

$$l_{wf}^0 = \frac{0{,}7 \times N}{2 \times \beta_f \times k_f \times R_{wf} \times \gamma_c} + 1 = \frac{0{,}7 \times 171{,}12}{2 \times 0{,}7 \times 0{,}6 \times 18 \times 1{,}0} + 1 = 8{,}92\ sm,$$

- by pen

$$l_{wf}^n = \frac{0{,}3 \times N}{2 \times \beta_f \times k_f \times R_{wf} \times \gamma_c} + 1 = \frac{0{,}3 \times 171{,}12}{2 \times 0{,}7 \times 0{,}5 \times 18 \times 1{,}0} + 1 = 5{,}07\ sm.$$

The results of the calculation of angular welds for other elements of the node №6 of the coating truss are shown in table D.3.

Conclusion: Based on the results of the calculation, the cross-section was selected and the dimensions of the angular welds of the coating truss elements were determined.

D.4 Example of calculation of a wooden truss coating node

<u>Initial calculation data:</u>

Type of cross section - rectangular;

Wood species - spruce;

Wood strength class – C40;

Operational class of wood - 2.

<u>Calculation progress:</u>

1. Internal forces in the elements of the truss are determined by the results of static calculation in the software package LIRA-CAD 2017 (Fig. D.7).

2. Determine the design strength strength of wood:

- for tension along the fibers:

$$f_{t,0,d} = k_{mod}\frac{f_{t,0,k}}{\gamma_M} = 0{,}6\times\frac{24{,}0}{1{,}3} = 11{,}08 \text{ МПа},$$

- for compression along the fibers:

$$f_{c,0,d} = k_{mod}\frac{f_{\text{c},0,k}}{\gamma_M} = 0{,}6\times\frac{26{,}0}{1{,}3} = 12 \text{ МПа},$$

where $k_{mod} = 0{,}6$ - transition factor, which takes into account the influence of the load duration (constant and medium-term load by snow) and humidity, which corresponds to the 2nd operating class;

$f_{t,0,k} = 11{,}08$ МПа - characteristic value of resistance of hardwoods of class D30;

$f_{c,0,k} = 12$ МПа – characteristic value of compressive strength along fibers of hardwood class C40;

$\gamma_m = 1{,}3$ - safety factor for material characteristics (solid wood).

Characteristic values of strength, rigidity and density, as well as reliability coefficients γ_M for wood are given in Appendix E, table E.3-E.6.

3. Determine the required cross-sectional area of the elements of the truss' node №6:

- for the upper belt element

$$A^{1-10} = \frac{N_{1-10}}{f_{c,0,d}} = \frac{1242{,}36 \cdot 10^3}{12} = 103530\ mm^2$$

Accept the cross-section of the lower belt of the truss in size 380x380 mm; cross-sectional area A^{1-10}=380·380=144400 mm^2.

4. Check the strength of the upper belt element of the truss for compression along the fibers:

$$\sigma_{c,0,d} = \frac{N^{1-10}}{A_{net}} = \frac{1242{,}36 \cdot 10^3}{144400} = 8{,}6\ MPa \le f_{c,0,d} = 12\ MPa.$$

The compressive strength of the section is provided.

5. Check the stability of the upper belt element of the truss' node (p. 9.3.3, [4])

- condition of stability to compression along the fibers by the formula:

in the plane of the truss $\frac{\sigma_{c,0,d}}{k_{c,y} \times f_{c,0,d}} = \frac{8{,}6}{0{,}957 \times 12} = 0{,}749 \le 1,$

out of the truss' plane $\frac{\sigma_{c,0,d}}{k_{c,z} \times f_{c,0,d}} = \frac{8{,}6}{0{,}724 \times 12} = 0{,}990 \le 1,$

where $k_{c,y}$ i $k_{c,z}$ – longitudinal bending coefficients:

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}} = \frac{1}{0{,}631 + \sqrt{0{,}631^2 - 0{,}477^2}} = 0{,}957,$$

$$k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}} = \frac{1}{1{,}019 + \sqrt{1{,}019^2 - 0{,}953^2}} = 0{,}724,$$

$k_y = 0{,}5(1+\beta_c(\lambda_{rel,y} - 0{,}3) + \lambda^2_{rel,y}) = 0{,}5(1+0{,}2 \cdot (0{,}529 - 0{,}3) + 0{,}529^2) = 0{,}631,$

$k_z = 0{,}5(1+\beta_c(\lambda_{rel,z} - 0{,}3) + \lambda^2_{rel,z}) = 0{,}5(1+0{,}2 \cdot (1{,}058 - 0{,}3) + 1{,}058^2) = 1{,}019;$

β_c = 0,2 – straightness coefficient for solid wood elements;

relative flexibility of element

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} = \frac{28{,}36}{3{,}14}\sqrt{\frac{26}{9333{,}33}} = 0{,}477,$$

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} = \frac{56{,}73}{3{,}14}\sqrt{\frac{26}{9333{,}33}} = 0{,}953,$$

- flexibility of the element in the plane of the truss:

$$\lambda_y = l_{ef,y} / i_y = 3111{,}37 / 109{,}7 = 28{,}36,$$

- flexibility of the element out of the truss' plane:

$$\lambda_z = l_{ef,z} / i_z = 6222{,}74 / 109{,}7 = 56{,}73,$$

- moment of inertia of the cross-section of the upper belt:

$$i_{y(z)} = \sqrt{\frac{I_{y(z)}}{A}} = \sqrt{\frac{380^4/12}{380^2}} = 109{,}7\ mm.$$

- the estimated value of 5% of the quantile of the elasticity modulus of solid wood along the fibers:

$$E_{0,05} = 2/3 \cdot E_{0,mean} = 2/3 \cdot 14000 = 9333{,}33 \text{ MPa},$$

- the average value of the elasticity modulus of solid wood along the fibers (tab. Б.1, [4]):

$$E_{0,mean} = 14000 \text{ MPa}.$$

Similarly, calculate other elements of a wooden truss. The results of the calculation of the wooden truss' elements are given in table. D.4.

Conclusion: According to the results of the calculation, the cross-sections of the wooden truss' elements were calculated.

The results of the calculation of the wooden truss' coating elements

№ element	Design force, kN	The cross-sectional dimensions of the element, mm	Area of cross-section, mm^2	Estimated length, mm		Radius of inertia, mm		Flexibility		Relative flexibility		Longitudinal bending coefficient		Conditions of strength and stability		
															in the plane	out of the plane
				$l_{ef,y}$	$l_{ef,z}$	i_y	i_z	λ_y	λ_z	$\lambda_{rel,y}$	$\lambda_{rel,z}$	$k_{c,y}$	$k_{c,z}$	$\frac{\sigma_{c,0,d}}{f_{c,0,d}} \le 1$	$\frac{\sigma_{c,0,d}}{k_{c,y} \times f_{c,0,d}} \le 1$	$\frac{\sigma_{c,0,d}}{k_{z,y} \times f_{c,0,d}} \le 1$
Upper belt	-1242,36	380x380	144400	3111,37	6222,74	109,7	109,7	28,36	56,73	0,477	0,953	0,631	1,019	0,717	0,749	0,990
Brace 6-15	-195,3	50x50	2500	3423,8	3423,8	14,43	14,43	273,2	273,2	3,985	3,985	14,337	14,337	0,003	0,183	0,183
Stand 6-14	94,12	50x50	2500	-	-	-	-	-	-	-	-	-	-	0,007	-	-
Lower belt	1197,89	330x330	108900	-	-	-	-	-	-	-		-	-	0,993	-	-

D.5 Example of calculation of a wooden beam for bending

Initial calculation data:

Beam length l, m – 5,4;

Step of the beams, m – a = 1.5;

Wood species - pine;

Wood strength class - GL 32h;

Operational class of wood - 2;

Type of cross section - rectangular;

The maximum design value of the load on the floor of the building, kN / m^2 – 8,13.

Calculation progress:

1. Determine the design characteristics of wood:
   - the design value of bending strength relative to the main axis *y*:

$$f_{m,y,d} = k_{mod} \times \frac{f_{m,k}}{\gamma_M} = 0{,}6 \times \frac{32}{1{,}25} = 15{,}36\ MPa,$$

   - calculated value of chipping strength:

$$f_{v,d} = k_{mod} \times \frac{f_{v,k}}{\gamma_M} = 0{,}6 \times \frac{3{,}8}{1{,}25} = 1{,}82\ MPa$$

2. Calculate the maximum calculated value of the load per 1 m.p. floor beams

$$q^{b}_{d.calc} = q^{fl}_{d.calc} \times a = 8{,}13 \times 1{,}5 = 12{,}2 \text{ kN/m}.$$

3. Determine the calculated bending moment by the formula:
   - Bending moment

$$\mathrm{M}_{y,d} = \frac{q^{b}_{d,calc} \times l^2}{8} = \frac{12{,}2 \times 29{,}16}{8} = 44{,}45 \text{ kN*m};$$

   - Shear force

$$V_d = \frac{q^{b}_{d,calc} \times l}{2} = \frac{12{,}2 \times 5{,}4}{2} = 32{,}94\ kN$$

4. Determine the required moment of resistance of the beam by the formula:

$$W_{y,d} = \frac{M_{y,d}}{f_{m,y,d}} = \frac{44{,}45 \cdot 10^6}{15{,}36} = 2{,}89 \cdot 10^6\ mm^3 = 2894{,}5\ sm^3,$$

Accept a section of a glued beam 150 mm wide from boards of 150x40 mm; find the required section height:

$$h = \sqrt{\frac{6 \cdot W_{y,d}}{b}} = \sqrt{\frac{6 \cdot 2{,}89 \cdot 10^6}{150}} = 340{,}27\ mm.$$

Pre-accept the beam with a cross section $b \times h = 150 \times 360$ mm (рис. Г.5).

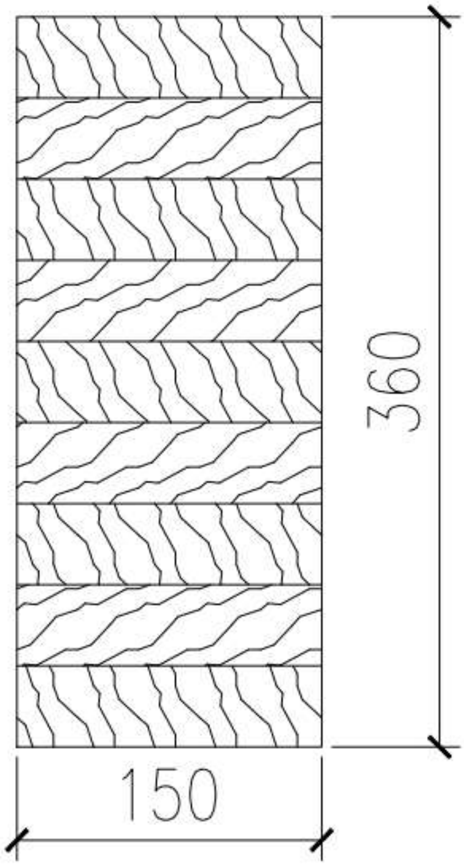


Figure D.8 – Cross-section of glued beam

Define geometrical characteristics of the accepted cross-section of a glued beam:

$$W_{y,d} = \frac{bh^2}{6} = \frac{150 \cdot 360^2}{6} = 3240000\ mm^3;$$

$$I_{br} = \frac{bh^3}{12} = \frac{150 \cdot 360^3}{12} = 583200000\ mm^4;$$

$$S_{br} = \frac{A}{2} \cdot z = \frac{A}{2} \cdot \frac{h}{4} = \frac{150 \cdot 360 \cdot 360}{2 \cdot 4} = 2430000\ mm^3;$$

$$b_{ef} = 150\ mm.$$

Check up the durability of the accepted section of a glued beam:

- on bending at normal stresses:

$$\sigma_{m,y,d} = \frac{M_{y,d}}{W_{y,d}} = \frac{44{,}45 \cdot 10^6}{3240000} = 13{,}72\ MPa \leq\ f_{m,y,d} = 15{,}36\ MPa,$$

- on chipping at shear stresses:

$$\tau_d = \frac{V_d \cdot S_{br}}{I_{br} \cdot b_{ef}} = \frac{32{,}94 \cdot 10^3 \cdot 2430000}{583200000 \cdot 150} = 0{,}915\ MPa \leq f_{\mathrm{v},d} = 1{,}82\ MPa.$$

<u>Conclusion:</u> The bending and chipping strength of the accepted cross section of the glued beam is provided.

D.6 Example of masonry pier calculation of an external wall

Initial calculation data:

Number of floors of the house n_{fl} - 4;

Height of a floor of the house, m - 3,0;

Dimensions in axes ($L_1 x L_2$), m – 6,8x6,3;

Dimensions of window opening (bxh), m – 1,5x1,6;

The thickness of external wall, mm - 510;

The maximum design value of the load on the floor and coating of the building, kN / m^2 - $q^{ceil}_{d,\ calc}$ = 6,05, $q^{fl}_{d,\ calc}$ = 8,13.

Calculation progress:

1. Draw a scheme of the window openings' location in the external wall of the building.

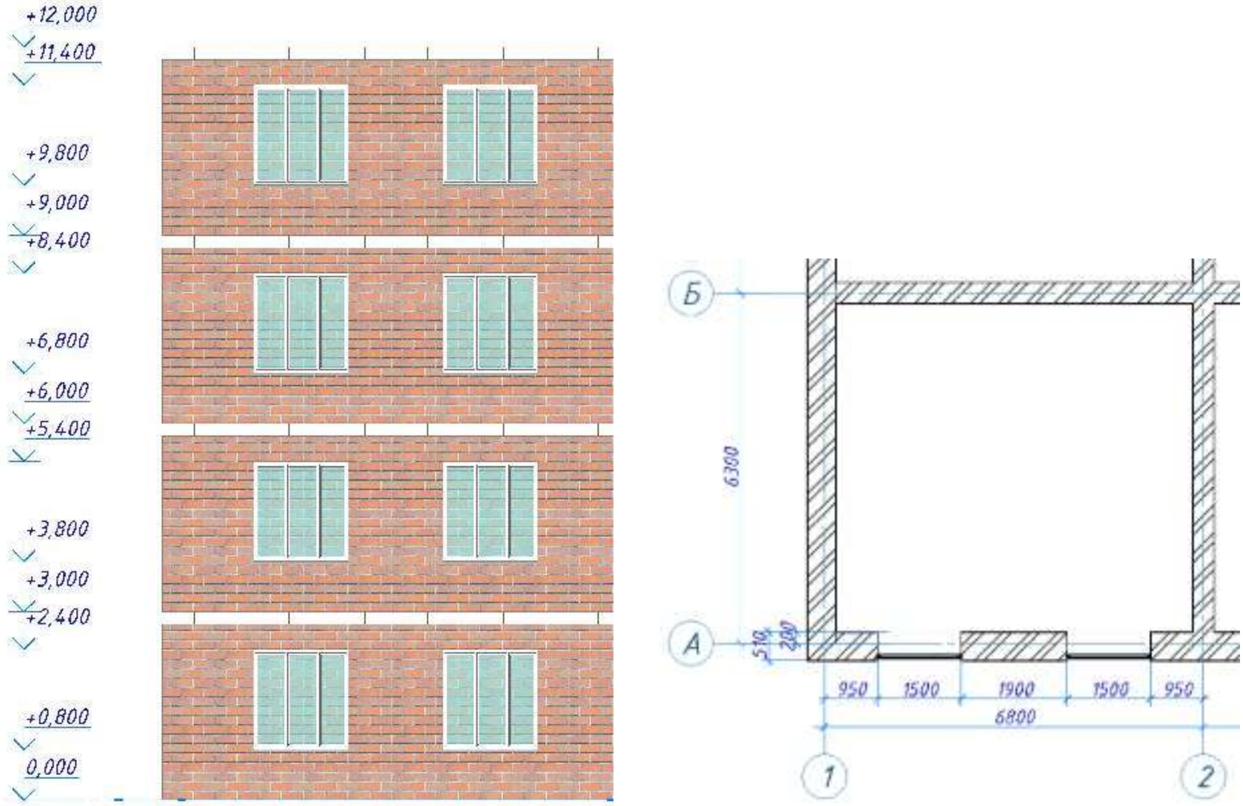


Figure D.9 – The dimensions of the partition and the loading area of the wall

2. Determine the width of the pier is:

$$b_{pier} = \frac{l_1 - 2b_{op}}{2} = \frac{6{,}8 - 2 \cdot 1{,}5}{2} = 1{,}9\ m$$

The loading area of the wall is:

$$A_w = \text{B}_{pier} \cdot \text{H}_{pier} = 3{,}4 \cdot 10{,}6 - 3 \cdot 1{,}5 \cdot 1{,}6 = 28{,}84\ m^2$$

3. The weight of the wall, which is transferred to the partition (masonry and plaster) with a wall thickness of 2 bricks ($t = 0{,}51\ m$):

$$G = A_w \cdot (t \cdot D_k \cdot \gamma_f + \delta \cdot D_{\text{ш}} \cdot \gamma_f)$$

$$= 28{,}84 \cdot (0{,}51 \cdot 18 \cdot 1{,}1 + 0{,}02 \cdot 2 \cdot 20 \cdot 1{,}3) = 321{,}22\ kN$$

4. Loads from weight of floor and coating with the loading area:

$$A_{load} = \frac{L_1 - 0{,}2}{2} \cdot L_2 = \frac{6{,}8 - 0{,}35}{2} \cdot 6{,}3 = 20{,}32\ m^2$$

Coating load:

$$F_{coat} = q_d^{coat} \cdot A_2 = 6{,}05 \cdot 20{,}32 = 122{,}92\ kN$$

Load on the floor:

$$F_{fl} = q_d^{fl} \cdot A_2 = 8{,}43 \cdot 20{,}32 = 171{,}3\ kN$$

The total design load on the wall is:

$$N_{Ed} = G_w + F_{coat} + \left(n_{fl} - 1\right) \cdot F_{fl} = 321{,}22 + 122{,}92 + 3 \cdot 171{,}3 = 958{,}03\ kN$$

5. Eccentricity of application of loading from above floor:

$$e = \frac{t}{2} - \frac{C}{3} = \frac{510}{2} - \frac{200}{3} = 188{,}33\ mm = 18{,}83\ sm$$

Moment on the floor support:

$$M = F_{fl} \cdot e = 171{,}3 \cdot 0{,}188 = 32{,}26\ kN \cdot m$$

The free height of the partition is equal to the height of the floor in the light:

$$h = h_{fl} - h_\delta = 3000 - 300 = 2700\ mm = 2{,}7\ m$$

The flexibility of the pier is determined by the formula:

$$\lambda = \frac{h}{t} = \frac{2700}{510} = 5{,}29 \leq 27$$

Moment at the level of the top of the window opening:

$$M_{i,d} = \frac{32{,}26}{2{,}7} \cdot (0{,}8 + 1{,}6) = 28{,}68\ kN \cdot m$$

The value of the random eccentricity:

$$e_{i,net} = \frac{h}{450} = \frac{2700}{450} = 6{,}0\ mm = 0{,}6\ sm$$

The reduced eccentricity makes:

$$e_i = \frac{M_{i,d}}{N_{Ed}} + e_{i,net} = \frac{28,68}{958,03} + 6,0 = 35,93\ mm \geq 0,05t = 25,5\ sm$$

Coefficient taking into account flexibility and eccentricity:

$$\Phi_i = 1 - 2 \cdot \frac{e_i}{t} = 1 - 2 \cdot \frac{35,93}{510} = 0,859$$

The required value of the compressive strength of the masonry:

$$N_{Rd} = \frac{N_{Ed}}{\Phi_i \cdot b_п \cdot t} = \frac{958,03 \cdot 10^{3}}{0,859 \cdot 1900 \cdot 510} = 1,15\ MPa$$

Based on the correspondent design parameters, accept the brick masonry strength $f_b = 12,5$ МПа (M125) on a heavy solution of strength $f_m = 1,0$ МПа (M10)

The compressive strength of the masonry is $f_d = 1,2\ MPa$.

Conclusion: According to the results of the calculation of the compressive strength of the masonry, the strength mark of ceramic brick and the mark of heavy cement-sand mortar for compression was adopted.

Table D.4

Estimated compressive strengths of brick masonry of all types on heavy mortars according to DBN V. 2.6-162

The strength of brick or stone f_b, MPa	Design strength f_d, MPa (kgs / sm^2), on compression of masonry from all types of bricks and ceramic stones with dense vertical hollows up to 12 mm wide with height of a masonry row 50 ... 150 mm on heavy solutions at solution durability f_m								According the strength of solution	
	20,0	15,0	10,0	7,5	5,0	2,5	1,0	0,4	0,2	zero
30,0	3,9(39)	3,6(36)	3,3(33)	3,0(30)	2,8(28)	2,5(25)	2,2(22)	1,8(18)	1,7(17)	1,5(15)
25,0	3,6(36)	3,3(33)	3,0(30)	2,8(28)	2,5(25)	2,2(22)	1,9(19)	1,6(16)	1,5(15)	1,3(13)
20,0	3,2(32)	3,0(30)	2,7(27)	2,5(25)	2,2(22)	1,8(18)	1,6(16)	1,4(14)	1,3(13)	1,0(10)
15,0	2,6(26)	2,4(24)	2,2(22)	2,0(20)	1,8(18)	1,5(15)	1,3(13)	1,2(12)	1,0(10)	0,8(8)
12,5	-	2,2(22)	2,0(20)	1,9(19)	1,7(17)	1,4(14)	1,2(12)	1,1(11)	0,9(9)	0,7(7)
10,0	-	2,0(20)	1,8(18)	1,7(17)	1,5(15)	1,3(13)	1,0(10)	0,9(9)	0,8(8)	0,6(6)
7,5	-	-	1,5(15)	1,4(14)	1,3(13)	1,1(11)	0,9(9)	0,7(7)	0,6(6)	0,5(5)
5,0	-	-	-	1,0(10)	-	0,9(9)	0,7(7)	0,6(6)	0,5(5)	0,35(3,5)
3,5	-	-	-	0,9(9)	0,8(8)	0,7(7)	0,6(6)	0,45(4,5)	0,4(4)	0,25(2,5)

Note.

The design strength of the masonry on mortars of strength class from 4 to 50 should be reduced by applying reducing coefficients: 0,85 - for masonry on stiff cement mortars (without the addition of lime or clay), light and lime mortars up to 3 months; 0.9 - for masonry on cement mortars (without lime or clay) with organic plasticizers.

It is not required to reduce the compressive design strength for masonry of high quality - the seam of the mortar is indicated under the frame with leveling and compaction of the mortar with a rail. The project indicates the mark of mortar for ordinary masonry and for high quality masonry.

D.7 Example of fire resistance calculation for reinforced concrete beam

Initial calculation data:

According to DBN [6] for the first degree of fire resistance of the building the standardized fire resistance class of the floor beam - R60 (see table D.7).

Beam cross-sectional dimensions b = 300 mm, h = 600 mm, distance to the reinforcement axis a = 30 mm (Figure D.9). The cross-section of the beam is considered to be exposed to fire from three sides - from below and from the sides. The beam is considered to be freely supported. The load on the beam is taken according to table D.2, the step of the floor beams - 6 m.

Concrete class C 20/25 (f_{ck} = 18.5 MPa, γ_c = 1.3, $f_{cd} = f_{ck} / \gamma_c$= 18.5 / 1.5 = 14.5 MPa). Reinforcement class A500C (f_{uk} = 500 MPa,γ_s = 1,2, $f_{yd} = f_{yk} / \gamma_s$ = 500 / 1.15 = 435 MPa, E_s = 2,1x10^5 MPa), the cross-sectional area of the working reinforcemnt 4∅18 – A_s = 1018 mm^2.

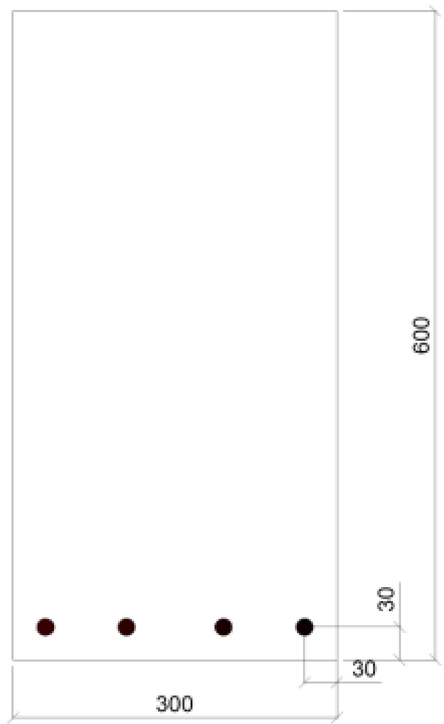


Figure D.10 – Cross-section of the beam

Calculation progress:

Calculation of the beam for fire resistance according to tabular data

To determine the internal forces in the beam to calculate the fire resistance take into account the permanent and temporary long-term (quasi-constant) value of the load on the floor with a coefficient of reliability for emergency situation responsibility.

Calculate the design value of the load on the beam:

- during a fire

$$q_{Ed,fi} = \left(q_{d.l} + q_{fl}^{char} \cdot step\ of\ beams\right) \cdot \gamma_n =$$

$$= (0{,}3 \cdot 0{,}6 \cdot 2{,}5 \cdot 9{,}81 + (4{,}99 + 0{,}35) \cdot 6) \cdot 0{,}975 = 35{,}54\ kN/m,\cdot$$

where $q_{fl}^{ch\text{ар}}$ - characteristic value of the load on the floor, including the weight of the floor materials and floor slabs, as well as the quasi-constant value of the temporary evenly distributed load, *kN / m²*;

6 m - step of floor beams;

γ_n= 0,975 - coefficient of reliability for liability for an emergency situation (fire) for the class of consequences SS2 according to DBN B.1.2-14: 2018 [2], see table. E.2;

- under normal conditions

$$q_{Ed} = \left(q_{\text{в.в}} + q_{\text{пер}}^{\text{гр}} \cdot \text{крок балок}\right) \cdot \gamma_n =$$

$$= (0{,}3 \cdot 0{,}6 \cdot 2{,}5 \cdot 9{,}81 \cdot 1{,}1 + 6 \cdot (5{,}79 + 1{,}95)) \cdot 1{,}05 = 53{,}86\ \text{кН/м},\cdot$$

where $q_{\text{пер}}^{\text{хар}}$ - maximum calculated value of the load on the floor, including the weight of floor materials and floor slabs, as well as short-term evenly distributed load, kN / m2;

6 m - step of floor beams;

γ_n= 1,05 - reliability coefficient for responsibility for the established situation, the class of consequences CC2 and the category of responsibility of the structure Б [2], see table. E.2;

Calculate the design values of bending moments in the beam:

- during a fire

$$M_{Ed,fi} = \frac{q_{d,fi} \cdot l^2}{8} = \frac{35{,}54 \cdot 5{,}4^2}{8} = 129{,}54\ \text{кН·м};$$

• under normal conditions

$$M_{Ed} = \frac{q_{Ed} \cdot l^2}{8} = \frac{53{,}86 \cdot 5{,}4^2}{8} = 196{,}32 \text{ кН·м.}$$

where l = 5,4 - the estimated length of the beam, m

Calculate the reduction factor η_{fi} that determines the level of load on the beam during a fire:

$$\eta_{fi} = \frac{M_{d,fi}}{M_{Ed}} = \frac{129{,}54}{196{,}32} = 0{,}66 \leq 0{,}7.$$

Because the load level of the beam during a fire $\eta_{fi} \leq 0.7$, for calculation of the fire resistance can be used tabular data for DSTU-N B B.2.6-196 [10], see table D.6.

Table D.6

Minimum dimensions and distances to the axis of reinforcement of freely supported beams for unstressed and prestressed reinforced concrete

Normalized fire resistance class	Minimal dimensions,mm				
	Possible combinations a and b_{min} where a - the average distance from the axis of the reinforcement, b_{min} - the width of the beam				Beam wall thickness,b_w
1	2	3	4	5	6
R30	b_{min}=80	120	160	200	80
	a=25	20	15*	15*	
R60	b_{min}=120	160	200	300	100
	a=40	35	30	25	
R90	b_{min}=150	200	300	400	110
	a=55	45	40	35	
R120	b_{min}=200	240	300	500	130
	a=65	60	55	50	
R180	b_{min}=240	300	400	600	150
	a=80	70	65	60	
R240	b_{min}=280	350	500	700	170
	a=90	80	75	70	
a_{sd}= a+10mm(see note)					
Note. For prestressed beams, increase the distance to the reinforcement axis in accordance with 8.2.4. a_{sd} - the distance to the side of the beam from the axis of the corner rods (prestressed reinforcement elements or wire) of the beams with only one row of reinforcement. For the values b_{min}, which are bigger than ones given in column 4, values a_{sd} are not increased. *) The protective layer of concrete must be at least determined in accordance with DBN V.2.6-98.					

Compare the geometric characteristics of the cross-section of the beam with the minimum required values in table D.6. The cross-sectional width of the beam b=300 mm is equal to the minimum value of b_{min}=300 mm; distance to the reinforcement axis a=30 mm exceeds the minimum value of a_{min}=25 mm.

Thus, based on the analysis of tabular data, it was found that the normalized fire resistance class of the beam R60 is provided.

According to the tabular data, the distance from the axis of the angular rods to the side face of the beam a_{sd} is recommended to increase by 10 mm - a_{sd} = 40 mm.

Calculation of beams for fire resistance by zonal method

The zonal method of calculating the fire resistance involves dividing half of the cross-section of the beam into n ≥ 3 parallel zones of the same thickness, for which the average temperature is determined θ_m and the corresponding average compressive strength of $f_{cd}(\theta)$ [10].

Damaged during the fire, the cross-section of the beam is represented by the reduced cross-section. The reduction of the cross-section of the beam is based on determining the thickness a_z of the damaged area of the heating surface, see Figure D.11.

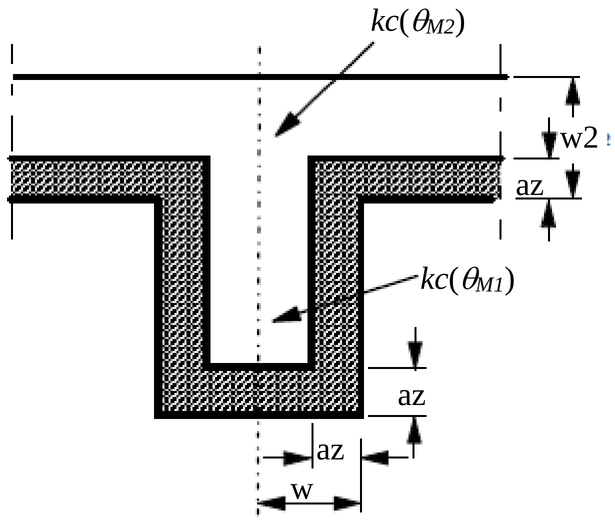


Figure D.11 – The reduced cross-section of a beam

Determine the width of the damaged zone a_z of the cross-section of the beam in the following sequence:

a) divide half of the cross-section of the beam into five parallel zones of equal thickness, see the division scheme on Fig. D.12;

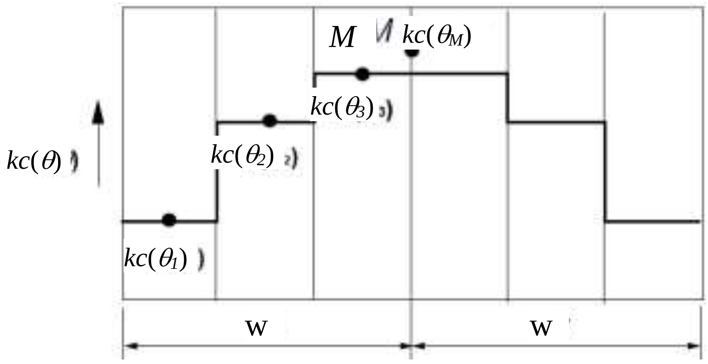
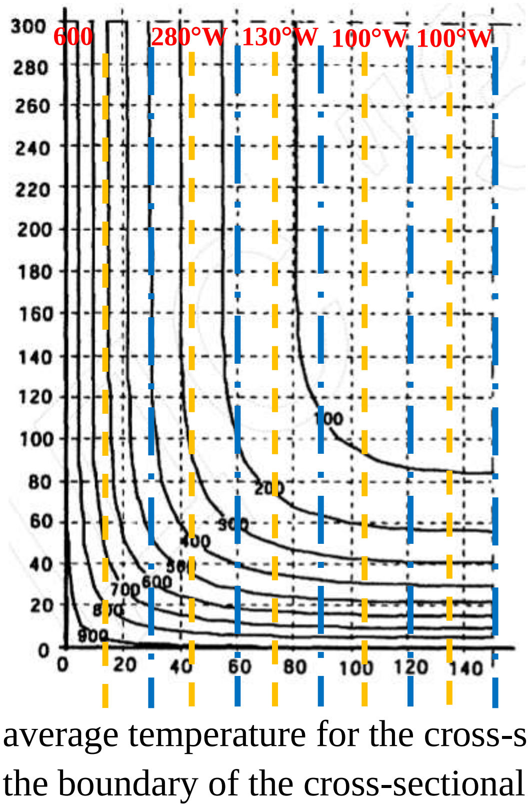


Figure D.12- The scheme of section division into zones of identical width

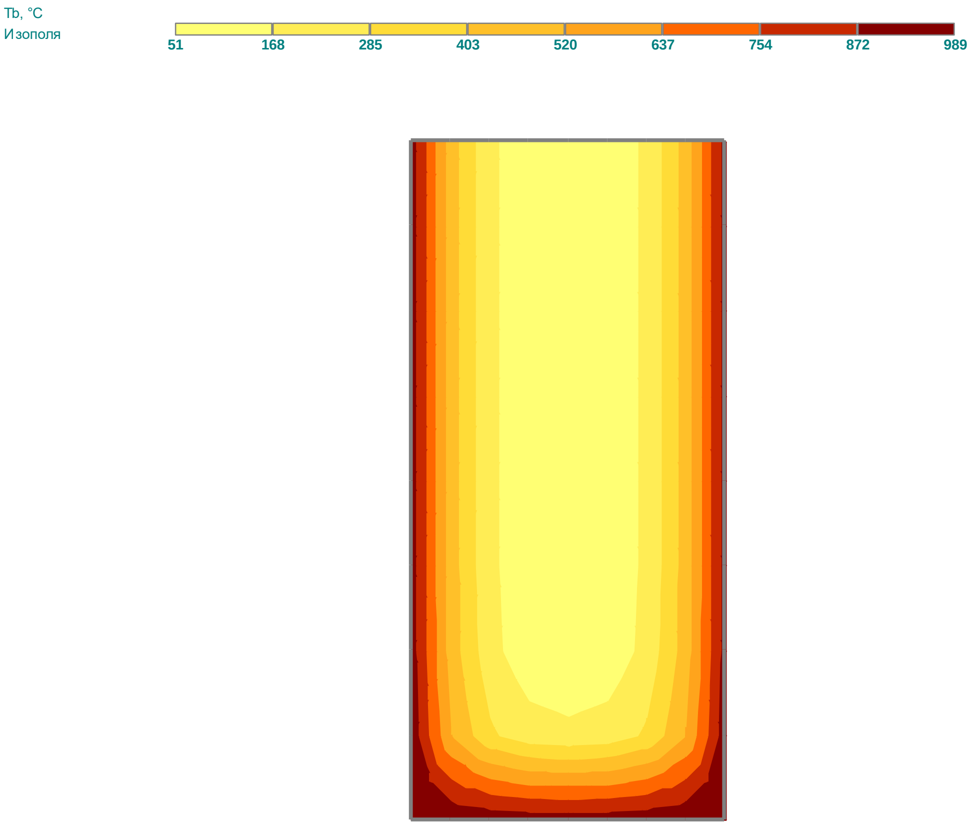
b) calculate the average temperature for each section zone. The calculation of the temperature in the cross-section of the beam can be performed using the isotherms listed in Appendix A [10], or using software packages such as LIRA-CAD 2019, Ansys Multiphysics [18] and others. Isotherms for a beam with a cross-section of 600x300 mm of fire resistance class R60 are shown in Fig. D.13.



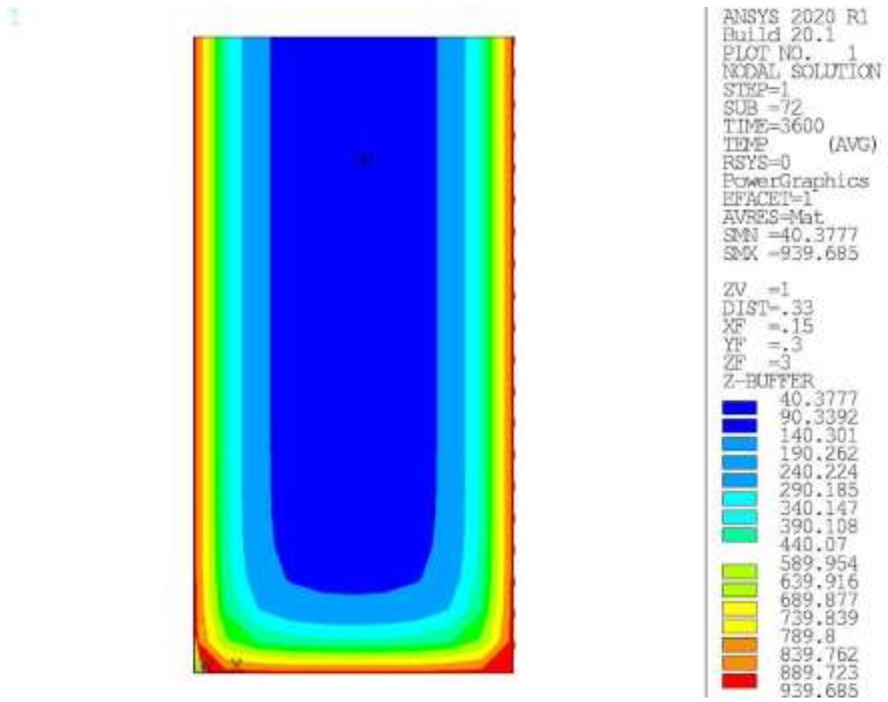
average temperature for the cross-sectional area

the boundary of the cross-sectional area

Figure D.13 - Temperature curves of section of a beam of 600x300 mm for R60 for ДСТУ-Н Б В.2.6-196



a)



b)

Figure D.14 - Temperature curves of section of a beam of 600x300 mm for R60 according to the calculation in the software packages: a)LIRA-CAD 2019; b) ANSYS 2020 R1

The average temperature for each section zone is determined graphically (Fig. D.13) and is:

$$\theta_1 = 600\ °C;\ \theta_2 = 280\ °C;\ \theta_3 = 130\ °C;\ \theta_4 = 100\ °C;\ \theta_5 = 100\ °C.$$

c) the coefficients of compressive strength reduction of concrete $k_c(\theta_i)$ for the corresponding temperature are determined graphically (Fig. D.13) or from table D.3 for DSTU-N B B.2.6-196 [10]. The values of the coefficients of reduction of concrete strength are given in table D.7.

Table D.7

The values of the coefficients of reduction of compressive strength of concrete for the cross-section of the beam

The coefficient of reduction of concrete strength	The average temperature of the cross-sectional area of the beam, θ_i, °C				
	600	280	130	100	100
$k_c(\theta_i)$	0.45	0.87	0.985	1.00	1.00

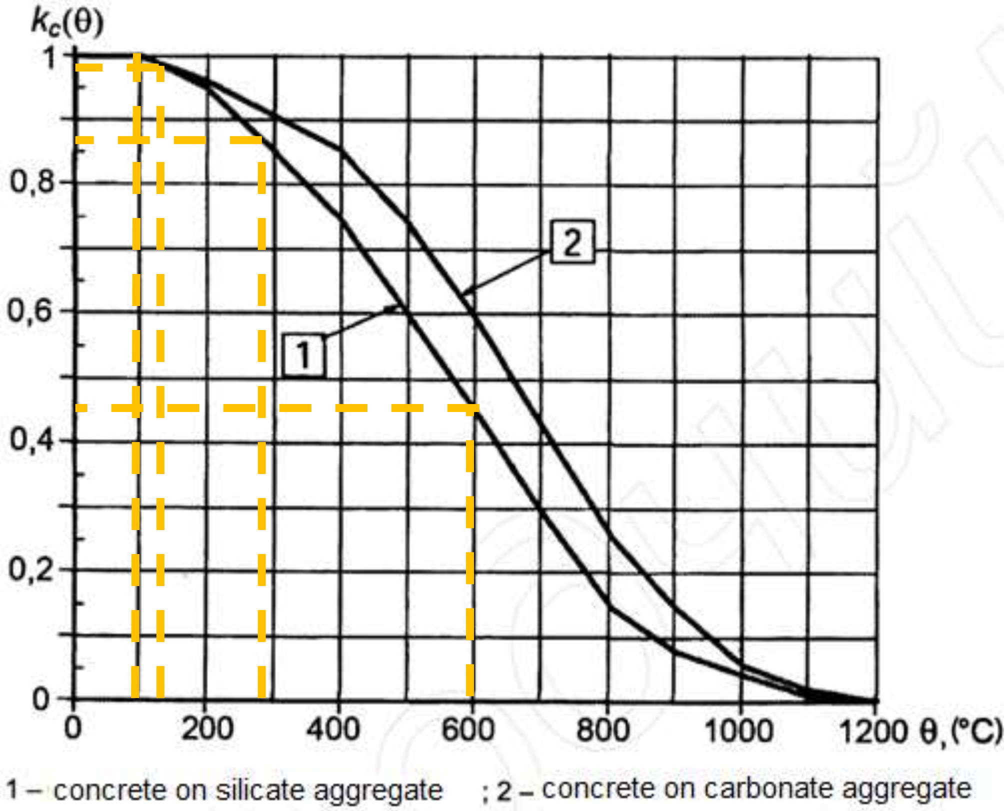


Figure D.15 - Coefficients of characteristic strength reduction of concrete

Determine the average coefficient of concrete strength reduction, which takes into account change in temperature of each cross-section zone while calculating, according to the formula:

$$k_{c,\,m} = \frac{(1-0{,}2/n)}{n} \cdot \sum_{i=1}^{n} k_{c,(\theta_i)} = \frac{(1-0{,}2/5)}{5} \cdot (0{,}45 + 0{,}87 + 0{,}985 + 1{,}00 + 1{,}00) = 0{,}827.$$

Calculate the width of the damaged cross-sectional area of the beam by the formula:

$$a_z = w\left[1 - \frac{k_{c,m}}{k_{c,(\theta_M)}}\right] = 150 \cdot \left[1 - \frac{0{,}827}{1{,}00}\right] = 26 \text{ мм},$$

where w - half the width of the cross- section of the beam, mm;

$k_c\,(\theta_M)$ = 1.00 - the coefficient of compressive strength reduction of concrete at point M on the axis of symmetry of the beam cross-section.

Reduce the cross-sectional dimensions of the beam by the value a_z=26 mm between those sides that are exposed to fire in case of fire. The design values of the width and height of the beam cross-section are:

$$b_{fi} = b - 2 \cdot a_z = 300 - 2 \cdot 26 = 248 \text{ mm},$$

$$h_{fi} = h - a_z = 600 - 26 = 574 \text{ mm}.$$

d) calculate the temperature in the reinforcing bars of the beam. The calculation of the temperature in the reinforcement can be performed using the isotherm [10] and the software package LIRA-CAD 2019 [19].

The values of the temperature in the beam reinforcement are (Fig. D.16-D.17):

- for corner rods - θ_{angle} = 600 °C;
- for medium rods - θ_{sir} = 400 °C.

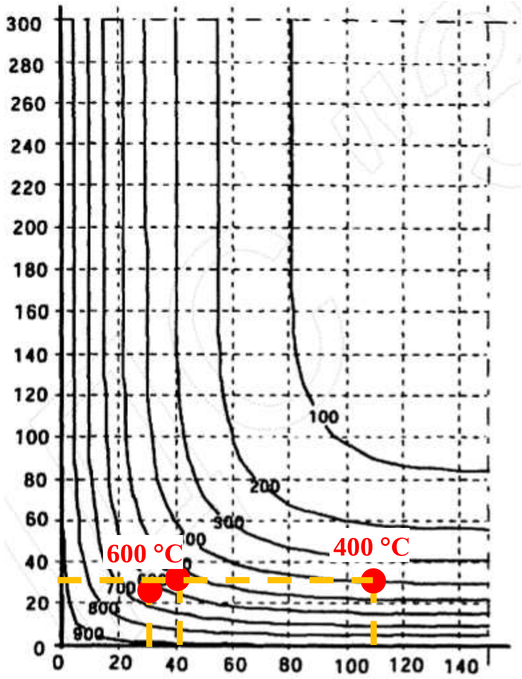


Figure D.16 - The temperature value in the beam reinforcement 600x300 mm for R60

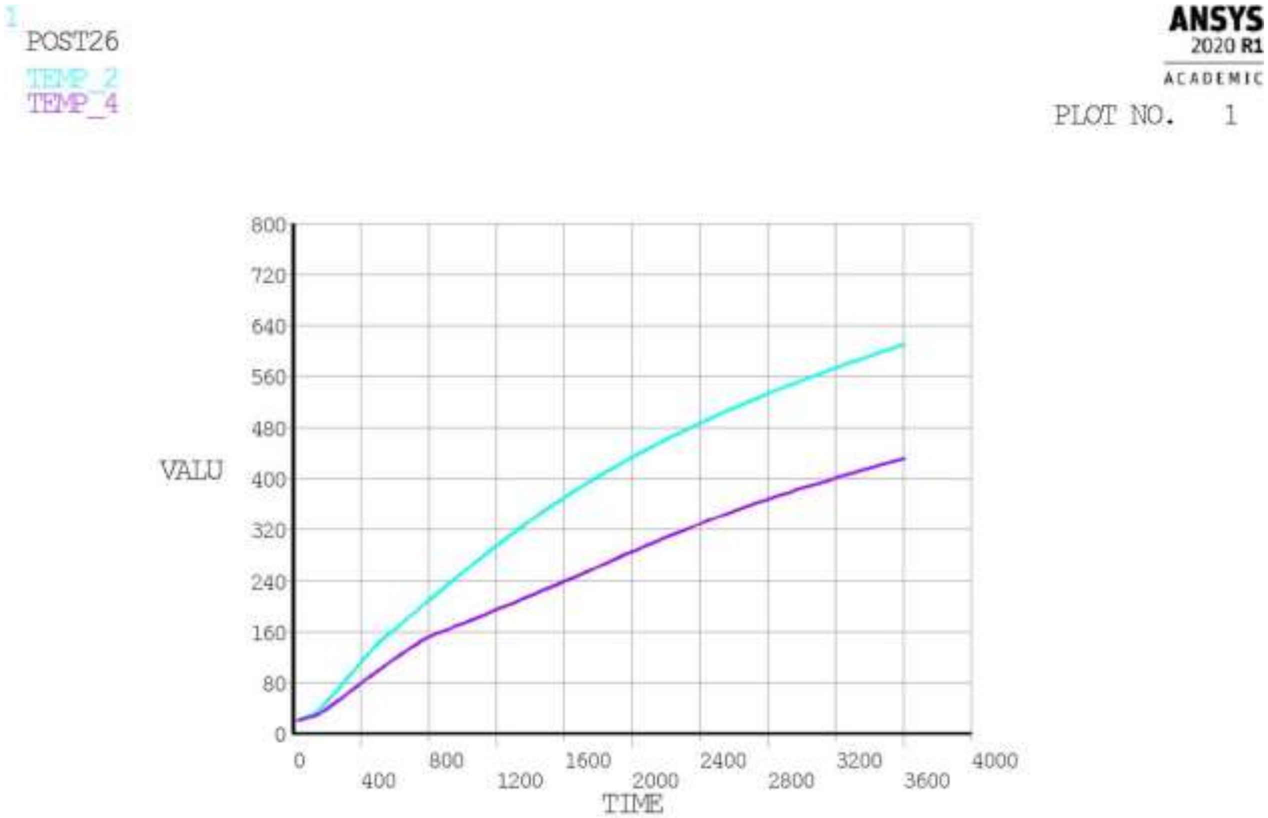


Figure D.17 – Graphs of temperature changing in beam reinforcement

The coefficients of reduction of the reinforcement strength are determined graphically (Fig. D.15) or from table D.9. The values of the reduction factors are taken as follows:

- for angular rods - $k_{s,\ (\theta\ =\ 550\,℃)}$= 0,45;
- for medium rods - $k_{s,\ (\theta\ =\ 400\,℃)}$= 0,70.

Calculate the reduced strength of the beam reinforcement by the formula:

$$f_{sd,fi}(\theta_m) = k_{v\,(\theta)} \times f_{sd} = 0{,}576 \times 435 = 250.67 \text{ MPa},$$

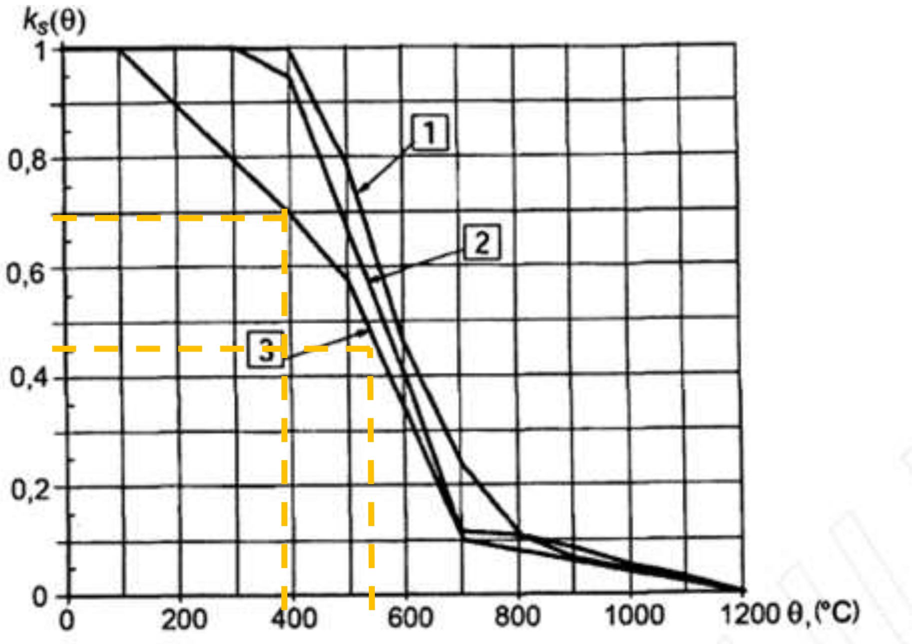
$$\text{where } k_v(\theta) = \frac{\sum k(\theta_i)}{n_v} = 2 \times (0{,}45 + 0{,}70) / 4 = 0{,}576,$$

where θ - the temperature of the *i*-th reinforcing rod;

$k_v(\theta)$ - the average coefficient of the strength reduction of the *v*-th reinforcing row;

$k(\theta i)$ - the coefficient of the strength reduction of the *i*-th rod (Fig. D.15, curve 3);

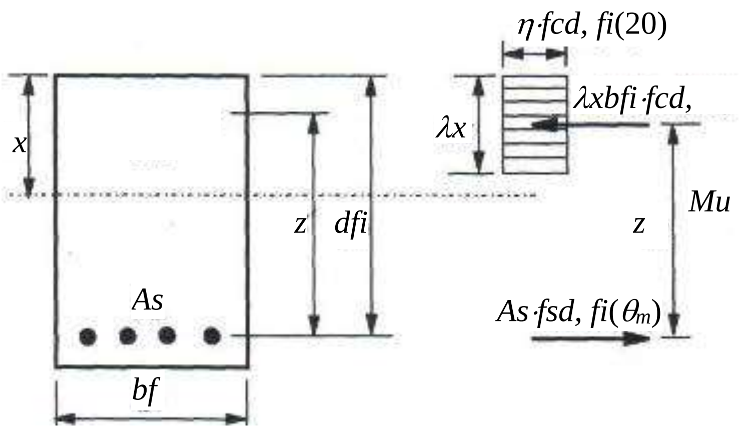
n_v - the number of reinforcing rods in the *v*-th reinforcing row.



1-tensioned reinforcement (hot-rolled) for deformations $\varepsilon_{s,fi} \geq 2\%$; 2- tensioned reinforcement (cold-deformed) for deformations $\varepsilon_{s,fi} \geq 2\%$; 3 – compressed and tensioned reinforcement for deformations $\varepsilon_{s,fi} < 2\%$

Figure D.18 - Coefficients of reinforcement durability reduction

Next, calculate the residual load-bearing capacity of the beam for a reduced cross-section with reduced reinforcement strength. The design scheme of the section and the forces acting in concrete and reinforcement are given in fig. D.19.



b_{fi} - the width of the reduced cross section;

d_{fi} - working height of the reduced cross section;

z - the distance between the stretched reinforcement and the compressed zone of concrete;

A_s - cross-sectional area of stretched reinforcement;

$f_{cd,\,fi\,(20)} = f_{ck} / \gamma_{c,\,fi}$ - design compressive strength of concrete at normal temperature;

$f_{sd,\,fi}\,(\theta_m)$ - design tensile strength of the reinforcement at elevated temperature θm;

λ, η and x - defined in DBN B.2.6-98

Figure D.19 - Design scheme of forces in the cross-section of the beam

From the equilibrium equation of the reduced section of the beam determine the height of the compressed zone of concrete:

$$\lambda x = A_s \times f_{sd,fi}(\theta_m) / f_{cd,fi(20)} \times b_{\mathrm{fi}} = 1018 \times 250{,}67 / 14{,}5 \times 248 = 71 \text{ mm},$$

Determine the shoulder of the inner pair of forces - compression in concrete and tension in reinforcement:

$$z = (\mathrm{d_{fi}} - 0.5 \times \lambda x) = (570 - 0.5 \times 71) = 534.5 \text{ mm}.$$

Determine the bearing capacity of the reduced cross-section of the beam:

$$M_u = A_s \times f_{sd,fi}(\theta_m) \times \mathrm{z} = 1018 \times 250{,}67 \times 534{,}5 = 136{,}4 \text{ kNm}.$$

e) Compare the bearing capacity of the reduced cross-section with the design bending moment in case of fire:

$$M_u = 136{,}4 \text{ кНм} > M_{Ed,fi} = 129{,}54 \text{ кНм}.$$

The bearing capacity of the reduced cross-section exceeds the calculated bending moment in case of fire. Thus, the limit of fire resistance of the beam on the basis of loss of bearing capacity exceeds 60 minutes.

Conclusion: According to the results of the calculation of fire resistance according to the tabular data and the zonal method, the normalized fire resistance class of the beam **R 60** is provided.

Appendix E

Tabular data for the calculation of building structures

Table E.1

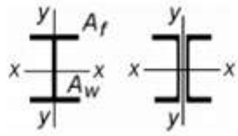
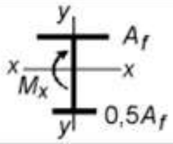
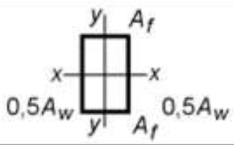
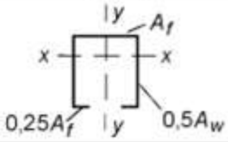
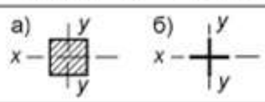
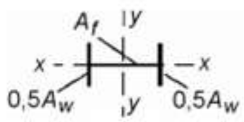
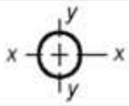
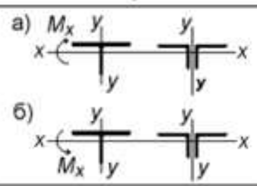
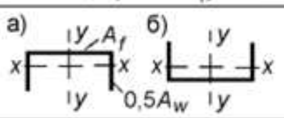
The value of the safety factor for responsibility γ_n

for ДБН В.1.2-14:2018

Class of consequences	Category of responsibility of structures	Values γ_n used in calculation situations				
		Established		Transitional		Emergency
		First group of limit states	Second group of limit states	First group of limit states	Second group of limit states	First group of border states
CC3	А	1,250	1,000	1,050	0,975	1,050
	Б	1,200		1,000		
	В	1,150		0,950		
CC2	А	1,100	0,975	0,975	0,950	0,975
	Б	1,050		0,950		
	В	1,000		0,925		
CC1	А	1,000	0,950	0,950	0,925	0,950
	Б	0,975		0,925		
	В	0,950		0,900		

Table E.2

The values of the coefficients for the calculation of the elements taking into account the development of plastic deformations according to ДБН В.2.6-198:2014, Appendix M

Type of cross-section	Scheme of cross-section	A_f/A_w	Maximum coefficients' values		
			c_x	c_y	n при $M_y = 0$*)
1-st		0,25	1,19	1,47	1,5
		0,50	1,12		
		1,00	1,07		
		2,00	1,04		
2-nd		0,5	1,40	1,47	2,0
		1,0	1,28		
		2,0	1,18		
3-rd		0,25	1,19	1,07	1,5
		0,50	1,12	1,12	
		1,00	1,07	1,20	
		2,00	1,04	1,26	
4-th		0,50	1,40	1,12	2,0
		1,00	1,28	1,20	
		2,00	1,18	1,31	
5-th		–	1,47	1,47	а) 2,0 б) 3,0
6-th		0,25	1,47	1,04	3,0
		0,50		1,07	
		1,00		1,12	
		2,00		1,19	
7-th		–	1,26	1,26	1,5
8-th		–	1,60	1,47	а) 3,0 б) 1,0
9-th		0,5	1,6	1,07	а) 3,0 б) 1,0
		1,0		1,1	
		2,0		1,19	

*) At $M_y \neq 0$ apply n = 1,5, except for cross- sections of type 5, a) for which n = 2 and type 5, b) for which n = 3.

Note 1. When determining the coefficients for intermediate values A_f/A_w, linear interpolation is allowed.

Note 2. The values of the coefficients c_x and c_y are taken not more than $1.15\gamma_f$, where γ_f - is the reliability factor for the load, calculated as the ratio of the design value of the equivalent (for the value of bending moment) load to the characteristic.

Table E.3

Characteristic values of strength, rigidity and density for coniferous wood according to ДБН В.2.6-161:2017, Appendix Б

№	Strength classes	C14	C16	C18	C20	C22	C24	C27	C30	C35	C40	C45	C50
Strength values, N/mm^2													
1	Bending $f_{m,k}$ ^{a}	14	16	18	20	22	24	27	30	35	40	45	50
2	Tension along $f_{t,0,k}{}^a$	8	10	11	12	13	14	16	18	21	24	27	30
3	Tension across $f_{t,90,k}$	0,4											
4	Compression along $f_{c,0,k}{}^a$	16	17	18	19	20	21	22	23	25	26	27	29
5	Compression across $f_{c,90,k}$	2,0	2,2	2,2	2,3	2,4	2,5	2,6	2,7	2,8	2,9	3,1	3,2
6	Chipping and torsion $f_{v,k}{}^c$	2,0											
Rigidity values, N/mm^2													
7	Elasticity modulus along $E_{0,mean}{}^{a,b}$	7000	8000	9000	9500	10000	11000	11500	12000	13000	14000	15000	16000
8	Elasticity modulus across $E_{90,mean}{}^{,b}$	230	270	300	320	330	370	380	400	430	470	500	530
9	Shear modulus $G_{mean}{}^{b,c}$	440	500	560	590	630	690	720	750	810	880	940	1000
Density values, kg/m^3													
10	Density ρ_k	290	310	320	330	340	350	370	380	400	420	440	460

Note. The value of the characteristic tensile strength across the fibers $f_{t,90,k}$, characteristic strength under the action of chipping and torsion differ from the design values according to ДСТУ EN 338, but only the values presented here should be used in the calculation.

[a] The estimated value for the log is increased by 20% in the conditions of absence of bark and bast without weakening the edge zone.

[b] Characteristic value of the shear modulus G_{Rk} of all strength classes can be accepted 1,0 N/mm^2 when calculating. At chipping stresses it is necessary to take the value of the shear modulus, which is equal to $G_{R,mean} = 0{,}10 \cdot G_{mean}$.

[c] For the characteristic value of stiffness E_{005} , $E_{90{,}05}$ та $G_{0{,}05}$ the calculated values are:

Table E.4

Characteristic values of strength, rigidity and density for hardwood according to ДБН В.2.6-161:2017, Appendix Б

№	Strength classes	D30	D35	D40	D50	D60	D70
Strength values, N/mm^2							
1	Bending $f_{m,k}$	30	35	40	50	60	70
2	Tension along $f_{t,0,k}$	18	21	24	30	36	42
3	Tension across $f_{t,90,k}$	0,5					
4	Стиск вздовж $f_{c,0k}$	23	25	26	29	32	34
5	Compression across $f_{c,90,k}$	8,0	8,4	8,8	9,7	10,5	13,5
6	Chipping and torsion $f_{v,k}$	3,0	3,4	3,8	4,6	5,3	6,0
Rigidity values, N/mm^2							
7	Elasticity modulus along $E_{0mean}{}^{a}$	10 000	10 000	11 000	14 000	17 000	20 000
8	Elasticity modulus across $E_{90mean}{}^{a}$	640	690	750	930	1130	1330
9	Shear modulus $G_{mean}{}^{a}$	600	650	700	880	1060	1250
Density values, kg/m^3							
10	Density ρ_k	530	560	590	650	700	900

Note. The value of the characteristic tensile strength across the fibers $f_{t,90,k}$, differs from design values according to ДСТУ EN 338, but only the values presented here should be used in the calculation.

[a] For the characteristic value of rigidity E_{005} , $E_{90,05}$ та $G_{0,05}$ the calculated values are:

$E_{0,05} = 5/6 \bullet E_{omean}$, $E_{90,05} = 5/6 \bullet E_{90mean}$, $G_{0,05} = 5/6 \bullet G_{mean}$.

Table E.5

Characteristic values of strength, rigidity and density for homogeneous glued wood according to ДБН В.2.6-161:2017, Appendix Б

Strength classes of glued wood		GL24h	GL28h	GL32h	GL36h
Strength values, N/mm^2					
Bending strength	$f_{m,g,k}$	24	28	32	36
Tension strength	$f_{t,0,g,k}$	16,5	19,5	22,5	26
	$f_{t,90,g,k}$	0,4	0,45	0,5	0,6
Compression strength	$f_{c,0,g,k}$	24	26,5	29	31
	$f_{c,90,g,k}$	2,7	3,0	3,3	3,6
Chipping strength	$f_{v,g,k}$	2,7	3,2	3,8	4,3
Rigidity values, N/mm^2					
Elasticity modules	$E_{0,g,mean}$	11 600	12 600	13 700	14 700
	$E_{0,g,05}$	9 400	10 200	11 100	11 900
	$E_{90,g,mean}$	390	420	460	490
Shear modulus	$G_{g,mean}$	720	780	850	910
Density values, kg/m^3					
Density	$\rho_{g,k}$	380	410	430	450

Table E.6

Safety factors for material γ_M for wood

Main combinations	γ_M
Solid wood	1,3
Glued wood	1,25
glued veneer, plywood, OSB	1,2
Chipboard, fiberboard, MDF	1,3
Joints	1,3
Metal gear parts	1,25
Accidential combinations	1,0

Table E.7

The degree of fire resistance of the house and the classes of fire resistance of building structures according to ДБН В.1.1-7:2016

Degree of fire resistance	Minimum values of fire resistance classes of building structures and maximum values of fire distribution groups on them								
	Walls				columns	stair landings, ladders, steps, stairs, beams, marches of stairwells	Overlapping between floors (including attics and above basements)	elements of combined floors	
	bearing and stairwells	self-supporting	external non-bearing	internal non-bearing (partitions)				slabs, flooring, girders	beams, trusses, arches, frames
I	REI 150 M0	REI 90 M0	E 30 M0	EI 30 M0	R 150 M0	R 60 M0	REI 60 M0	RE 30 M0	R 30 M0
II	REI 120 M0	REI 60 M0	E 15 M0	EI 15 M0	R 120 M0	R 60 M0	REI 45 M0	RE 15 M0	R 30 M0
III	REI 120 M0	REI 60 M0	E 15, M0 E 30, M1	EI 15 M1	R 120 M0	R 60 M0	REI 45 M1	Not determined	
IIIa	REI 60 M0	REI 30 M0	E 15 M1	EI 15 M1	R 15 M0	R 60 M0	REI 15 M0	RE 15 M1	R 15 M0
IIIb	REI 60 M1	REI 30 M1	E 15, M0 E 30, M1	EI 15 M1	R 60 M1	R 45 M0	REI 45 M1	RE 15, M0 RE 30, M1	R 45 M1
IV	REI 30 M1	REI 15 M1	E 15 M1	EI 15 M1	R 30 M1	R 15 M1	REI 15 M1	Not determined	
IVa	REI 30 M1	REI 15 M1	E 15 M2	EI 15 M1	R 15 M0	R 15 M0	REI 15 M0	RE 15 M2	R 15 M0
V	Not determined								

Note 1. Classes of fire resistance of building structures are determined depending on the limit states and the limit of fire resistance in accordance with ДБН В.1.2-7, ДСТУ Б В.1.1-4 , defined in Appendix Г.

Note 2. The class of fire resistance of self-supporting walls, which are taken into account in the calculations of rigidity and stability of the building, is accepted as for load-bearing walls.

Note 3. Groups of fire propagation of building structures are determined according to the method given in Appendx Д to these Codes.

Table E.8

The value of the coefficients of reduction of compressive strength of concrete at higher temperatures in accordance with ДСТУ-Н Б В.2.6-196:2014

Concrete temperature, θ, °C	Siliceous aggregates			Calcareous aggregates		
	$f_{c,\theta}/f_{ck}$	$\varepsilon_{c1,\theta}$	$\varepsilon_{cu1,\theta}$	$f_{c,\theta}/f_{ck}$	$\varepsilon_{c1,\theta}$	$\varepsilon_{cu1,\theta}$
1	2	3	4	5	6	7
100	1,00	0,0040	0,0225	1,00	0,0040	0,0225
200	0,95	0,0055	0,0250	0,97	0,0055	0,0250
300	0,85	0,0070	0,0275	0,91	0,0070	0,0275
400	0,75	0,0100	0,0300	0,85	0,0100	0,0300
500	0,60	0,0150	0,0325	0,74	0,0150	0,0325
600	0,45	0,0250	0,0350	0,60	0,0250	0,0350
700	0,30	0,0250	0,0375	0,43	0,0250	0,0375
800	0,15	0,0250	0,0400	0,27	0,0250	0,0400
900	0,08	0,0250	0,0425	0,15	0,0250	0,0425
1000	0,04	0,0250	0,0450	0,06	0,0250	0,0450
1100	0,01	0,0250	0,0475	0,02	0,0250	0,0475
1200	0,00	-	-	0,00	-	-

Table E.9

The value of the coefficients of reduction of compressive strength of reinforcement at higher temperatures in accordance with ДСТУ-Н Б В.2.6-196:2014

Steel temperature, θ, °C	$f_{sy.\theta}/f_{yk}$		$f_{sp.\theta}/f_{yk}$		$E_{s,\theta}/E_s$	
	hot-rolled	cold-deformed	hot-rolled	cold-deformed	hot-rolled	cold-deformed
1	2	3	4	5	6	7
100	1,00	1,00	1,00	0,96	1,00	1,00
200	1,00	1,00	0,81	0,92	0,90	0,87
300	1,00	1,00	0,61	0,81	0,80	0,72
400	1,00	0,94	0,42	0,63	0,70	0,56
500	0,78	0,67	0,36	0,44	0,60	0,40
600	0,47	0,40	0,18	0,26	0,31	0,24
700	0,23	0,12	0,07	0,08	0,13	0,08
800	0,11	0,11	0,05	0,06	0,09	0,06
900	0,06	0,08	0,04	0,05	0,07	0,05
1000	0,04	0,05	0,02	0,03	0,04	0,03
1100	0,02	0,03	0,01	0,02	0,02	0,02
1200	0,00	0,00	0,00	0,00	0,00	0,00

List of references

1. Навантаження і впливи. Норми проектування : DBN V.1.2-2:2006. – [Чинні від 2007-01-01]. – К. : Мінбуд України, Державне підприємство «Укрархбудінформ», 2006. – 75 с. – (Державні будівельні норми)
2. Загальні принципи забезпечення надійності та конструктивної безпеки будівель і споруд : DBN V.1.2-14:2018 – [Чинні від 2019-01-01]. – К.: Мінрегіон України, Державне підприємство «Укрархбудінформ», 2018. – 36 с. – (Державні будівельні норми)
3. Сталеві конструкції. Норми проектування : DBN V.2.6-198:2014 – [Чинні від 2015-01-01]. – К.: Мінрегіон України, Державне підприємство «Укрархбудінформ», 2014. – 205 с. – (Державні будівельні норми)
4. Дерев’яні конструкції. Основні положення : DBN V.2.6-161:2017 – [Чинні від 2018-02-01]. – К.: Мінрегіон України, Державне підприємство «Укрархбудінформ», 2017. – 117 с. – (Державні будівельні норми)
5. Кам'яні та армокам'яні конструкції. Основні положення: DBN V.2.6-162:2010 – [Чинні від 2011-09-01]. – К. : Мінрегіонбуд України, Державне підприємство «Укрархбудінформ», 2011. – 100 с. – (Державні будівельні норми)
6. Пожежна безпека об’єктів будівництва. Загальні вимоги: DBN V.1.1-7:2016 – [Чинні від 2017-06-01]. – К. : Мінрегіон України, Державне підприємство «Укрархбудінформ», 2017. – 47 с. – (Державні будівельні норми)
7. Бетонні та залізобетонні конструкції. Основні положення: DBN V.2.6-98:2009. – [Чинні від 2011-06-01]. – К. : Мінрегіонбуд України, Державне підприємство «Укрархбудінформ», 2011. – 71 с. – (Державні будівельні норми)
8. Бетонні та залізобетонні конструкції з важкого бетону. Правила проектування : ДСТУ Б.В.2.6–156:2010. – [Чинний з 2011-06-01]. – К. : Мінрегіонбуд України, 2011. – 118 с. – (Національний стандарт України)
9. Прогини і переміщення. Вимоги проектування : ДСТУ Б В.1.2-3:2006 –

[Чинний з 2007-01-01]. – К. : Мінбуд України, 2006. – 15 с. – (Національний стандарт України)

10. Настанова з проектування залізобетонних балок. Розрахунок на вогнестійкість: ДСТУ-Н Б В.2.6-196:2014 – [Чинний з 2015-07-01]. – К. : Мінгеріонбуд України, 2015. – 42 с. – (Національний стандарт України)
11. Металеві конструкції: Загальний курс: Підручник для вищих навчальних закладів /О.О. Нілов, В.О. Пермяков, О.В. Шимановський та ін. / за заг. ред. О.О. Нілова і О.В. Шимановського, – Вид. 2-е перероб. і доп. – К.: Сталь, 2010. – 869 с.
12. Покриття будівлі по сталевих фермах: методичні вказівки до виконання курсового проекту / уклад.: В.О. Володимирський, С.І. Білик, Т.О. Клюшниченко та ін. – К.: КНУБА, 2014. – 68 с.
13. Обстеження та підсилення металевих конструкцій: методичні рекомендації до виконання курсового проекту / уклад.: А.С. Білик, М.О. Бут, В.М. Адаменко. – К.: КНУБА, 2013. – 40 с
14. Будівельні конструкції: Основи розрахунку: Підручник / Бучок Ю. Ф. – К.: Вища шк., 1994. – 447 с.
15. Комп'ютерні технології проектування металевих конструкцій: навч. посіб./ М.С. Барабаш, С.В. Козлов, Д.В. Медведенко. – К.: НАУ, 2012. – 572 с.
16. Розрахунок залізобетонних конструкцій на вогнестійкість відповідно до Єврокоду 2. Практичний посібник / В.Г. Поклонський, О.А. Фесенко, В.Г. Тарасюк та ін. – К.: Інтертехнологія, 2016. – 83 с.
17. Методичні вказівки до виконання курсової роботи з дисципліни «Залізобетонні конструкції» (для слухачів другої вищої освіти спеціальності 7.06010103 – «Міське будівництво та господарство») / Харків. нац. унт міськ. госп-ва ім. О.М. Бекетова; уклад.: Є.С. Сєдишев. – Харків: ХНУМГ ім. О.М. Бекетова, 2015. – 52 с.
18. Основи роботи в скінченно-елементному програмному комплексі ANSYS. / Калінін Є.І. – Конспект лекцій. Частина 2 – Харків: Видавництво ХНАДУ, 2013. – 135 с.

19. Программный комплекс ЛИРА-САПР. Руководство пользователя. Обучающие примеры / Водопьянов Р.Ю., Титок В.П., Артамонова А.Е, Ромашкина М.А. Под редакцией академика РААСН Городецкого А.С. // Электронное издание, 2017 г., – 535 с.

Contents