MINISTRY OF EDUCATION AND SCIENCE OF UKRAINE

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Methodological guidelines

for the implementation of the course project
on the subject

TIMBER CONSTRUCTIONS

(for 3d-year full-time and part-time students education level «bachelor»
specialty 192 –Building and civil engineering)

Kharkiv – O. M. Beketov NUUE – 2019

Methodological guidelines for the implementation of course project on the subject «Timber constructions» (for 3d-year full-time and part-time students education level «bachelor» specialty 192 – Building and civil engineering) / O. M. Beketov National University of Urban Economy in Kharkiv; com. : A. M. Bidakov, O. M. Pustovoitova. – Kharkiv : O. M. Beketov NUUE, 2019. – 44 p.

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Recommended by the department of building construction record № 1 on 28.09.2018.

CONTENT

FOREWORD

Expansion of the timber structures in Western Europe countries is caused great advantages, compared metal and reinforced concrete structures, among which environmental friendliness, lightness, material recoverability, simple utilization, fire resistance and radio transparency of the timber.

Course project is an important part of educational process of student training at Building Construction Faculty, from which depends the training quality of specialists in construction industry. When doing their variant of course project, students gain skills of the methods of design of timber structures. This focuses the future engineer on certain design tasks, which would be happened while working at manufactures and design companies.

This course project consists of step-by-step and detailed computation of one-storey GLT civil or industrial building. It is also provided the design of the joint connection with appropriate guidelines for design rules.

1 ROOF SHEATING AND ROOF BEAMS

In this project need to design and calculate the boards roof sheating and roof beams the insulated covering of the industrial building in the size in the plan 15x48m in the city of Sumy. Slope of metall roof is 1/15 over mineral wool insulation with thickness 150мм and density 65кг/м³. Roof sheating comprise of softwood boards of strength class C24. Step of columns 4m. Service class – 1.

Constructive scheme choice

We accept a multi-span roof beams installed at a distance of 1,25 m along the slopes of the roof.

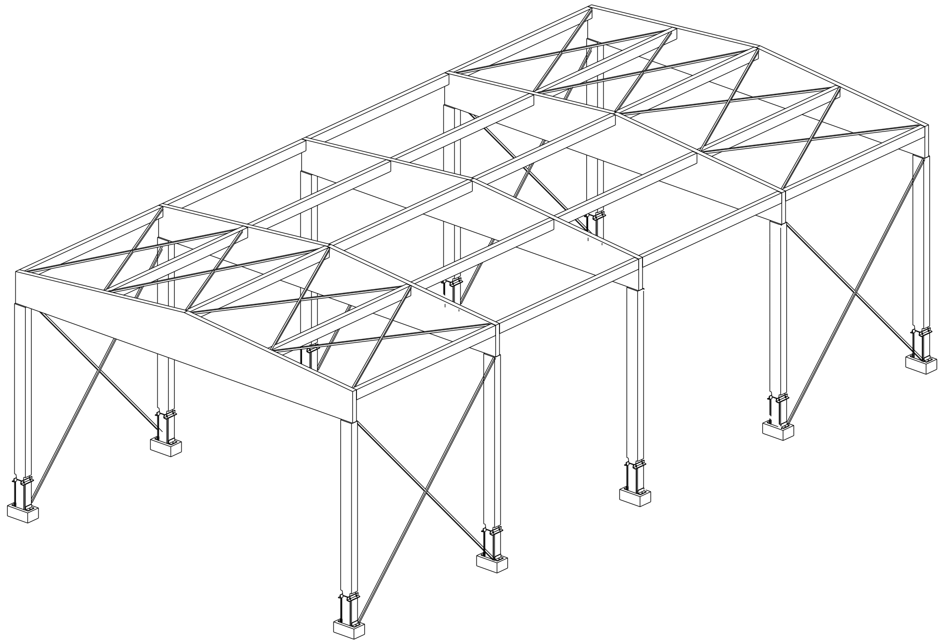


Figure 1 – General view of building

1.1 Design of roof sheating

Ultimate limit state design (ULS)

1.1.1 Static calculation

Table 1 – Loads (kN/m²)

	Calculation of loads	Characteristic value
1	Metall roof sheets	0,1
2	mineral wool insulation hard plates ρ = 60 кг/м³; δ = 150 mm	0,09
3	Vapor barrier	0,02
4	roof sheating boards (C24) δ = 22 mm, ρ_{mean} = 420 kg/ m³, $g_{r.s}$ = 0,022 m · 4,2 kN/m^3	0,1
		= 0,31
5	Snow load	1,67
	S_0 =1,67 kN/m^2 annex E, (ДБН В.1.2-2:2006)	

Explanation to table 1.

Characteristic value of snow (variable) load accepted:

$$S_0 = S_k = 1{,}67.$$

Constant load:

$$g_k = 0{,}31 \text{ kN/m}^2.$$

The schematic representation of the constituent elements of the roof construction is shown in Figure 2, where according to the scheme a) a roof with metal sheets is provided, and according to the scheme b) roof with soft coating on the cement-sand screed.

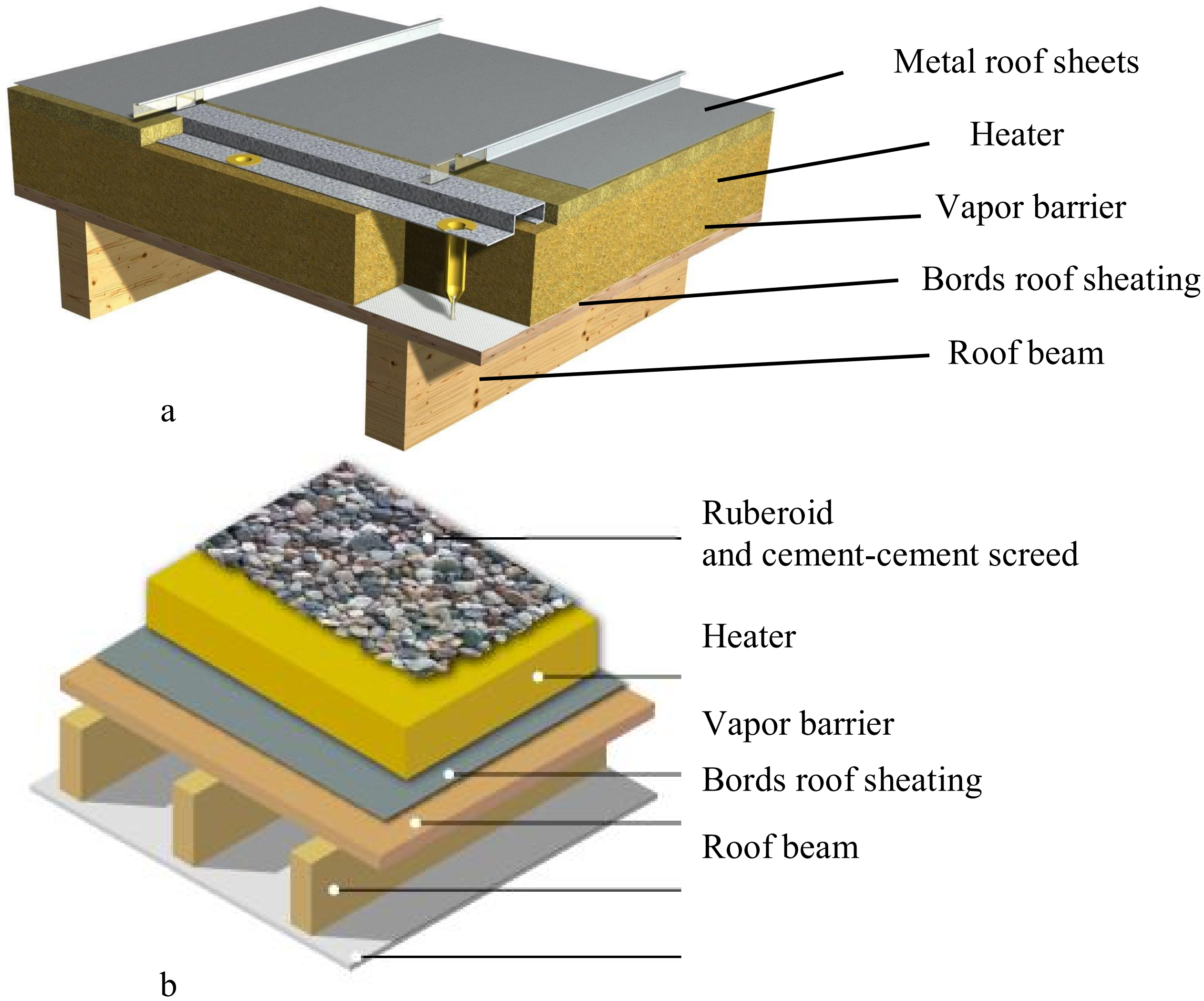


Figure 2 – Schema of roof layers
a – roof with metal sheets
b – roof with soft coating on the cement-sand screed

1.1.2 Load combinations

Limit value

$$\sum \gamma_G \cdot g_k + \cdot \gamma_Q \cdot S_k .$$

Service value

$$\sum g_k + S_k ,$$

where g_k – self weight;
S_k – snow load.

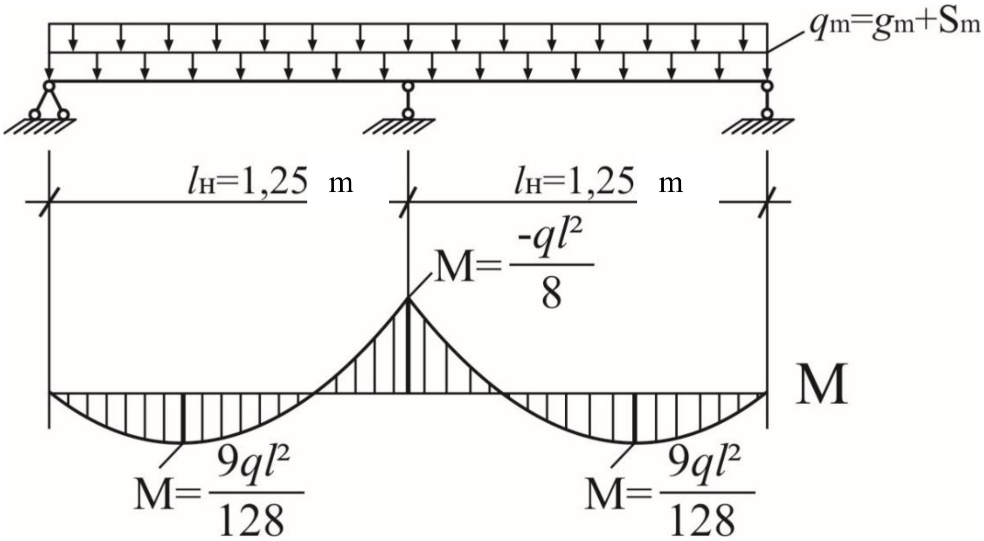


Figure 3 – Static schema of constant and snow loads

We accept the width of roof sheating b = 1 m.

Partial safety factors:

$$\gamma_G = 1{,}35 \qquad \gamma_Q = 1{,}5.$$

See coefficients values in ДСТУ-Н Б В.1.2-13:2008 or EN 1990:2002.

Design value of loads:

$$q_d = \sum \gamma_G \cdot G_k + \cdot \gamma_Q \cdot Q_k = g_d + \cdot s_d \,,$$

$$q_d = g_d + \cdot s_d = 1{,}35 \cdot 0{,}31 + 1{,}5 \cdot 1{,}67 = 2{,}93 \text{ kN/m}.$$

1.1.3 Constructive calculation

Slope component compresses or tensioned the roof sheating or decking (depending on where it is fixed). For small slopes ($\alpha \leq 30$ °) it can not be taken into account.

Thickness of the roof sheating:

Bending capacity

$$\sigma_{m,d} = \frac{M_d}{W_d} \leq f_{m,d};$$

$$W_d = \frac{M_d}{f_{m,d}} = \frac{0{,}57 \cdot 10^6 \text{ Nmm}}{16{,}6 \text{ N/mm}^2} = 34337{,}4 \text{ mm}^3.$$

Design bending strength

$$f_{m,d} = k_{\text{mod}} \cdot \frac{f_{m,k}}{\gamma_M} = 0{,}9 \cdot \frac{24}{1{,}3} = 16{,}6 \ \text{N/mm}^2,$$

where $f_{\dots,d}$ – design strength value;

$f_{\dots,k}$ – characteristic strength value;

k_{mod} – coefficient of modification, see. Annex A, table 1;

γ_M – partial factor for material properties, see. Annex A, table 3.

Load duaration class is «short-term», since the effect of the snow load, which is a short-acting effect on the building, is taken into account.

For roof sheating boards is accepted solid timber of a strength class C24. The width of the roof sheating $b_н$ is assumed to be equal 1000 mm. Required height of roof sheating boards:

$$W = \frac{b_H \cdot h_{\text{Н}}^{\ 2}}{6},$$

where

$$h_н = \sqrt{\frac{6W_d}{b_н}} = \sqrt{\frac{6 \times 34337{,}4 \ \text{mm}^3}{1000 \ \text{mm}}} = 14{,}4 \ \text{mm}.$$

We accept according to the assortment $h_н$ = 19 mm.

Since the height of the cross-section of the board is less than 150 mm, then design strength value need calculate by multiplying on the size factor:

$$k_h = \min\{(150/19)^{0,2}; 1{,}3\} = \min\{1{,}51;\ 1{,}3\} = 1{,}3$$

$$f_{m,y,d} = k_h \cdot f_{m,d} = 1{,}3 \cdot 16{,}6 = 21{,}6 \ \text{N/mm}^2.$$

1.1.4 Deflection:

k_{def}= 0,6 (Annex B, table. 1)

$\psi_{2,1}$ = 0,5 (ДСТУ-Н Б В.1.2-13:2008 or EN 1990:2002)

Module of elasticity for solid timber strength class C24

$E_{0,\text{mean}}$ = 11000 N/mm^2 (Annex Б, table. 1)

Moment of inertia

$$I = \frac{b \cdot h^3}{12} = \frac{1000 \cdot 19^3}{12} = 5{,}72 \cdot 10^6 \, \text{mm}^4.$$

The instantaneous deflections are:

$$w_{inst,G} = \frac{2{,}13 \cdot g_k \cdot l^4}{384 \cdot E_{0,mean} \cdot I} = \frac{2{,}13 \cdot 0{,}31 \cdot 1250^4}{384 \cdot 11000 \cdot 5{,}72 \cdot 10^6} = 0{,}07 \text{ mm},$$

$$w_{inst,Q} = \frac{q_k}{g_k} \cdot w_{inst,G} = \frac{1{,}67}{0{,}31} \cdot 0{,}07 = 0{,}38 \text{ mm},$$

$$w_{inst} = w_{inst,G} + w_{inst,Q,1} = 0{,}07 + 0{,}38 = 0{,}45 \text{ mm} < \frac{l}{300} = \frac{1250}{300} = 4{,}17 \text{ mm}.$$

Final deflection

$$w_{fin} = w_{inst} + \left(w_{inst,G} + \sum_{i=1}^{n} \psi_{2,i} \cdot w_{inst,Q,1} \right) \cdot k_{def} = 0{,}45 + (0{,}07 + 0{,}5 \cdot 0{,}38) \cdot 0{,}6 =$$

$$= 0{,}61 \text{ mm} < \frac{l}{150} = \frac{1250}{150} = 8{,}3 \text{ mm}.$$

$$w_{net,fin} = w_{fin} + w_S + w_{\Delta\omega} - w_c = 0{,}61 \text{ mm} < \frac{l}{250} = \frac{1250}{250} = 5 \text{ mm}.$$

Limit deflections for instantaneous deformation (w_{inst}) and final deformation of the beam on two supports, see. Annex B, Table. 2

Calculation according to second combination of loads

For roof sheathing accept solid wood boards with width $b_д$ = 150 mm. In this case load from man with a tool is 1кН and loaded two boards.

Static design

Self weigth of two boards:

P = 1 kN,

P = 1,2 kN · cos3°49' = 1 · 0,998 ≈ 1 kN.

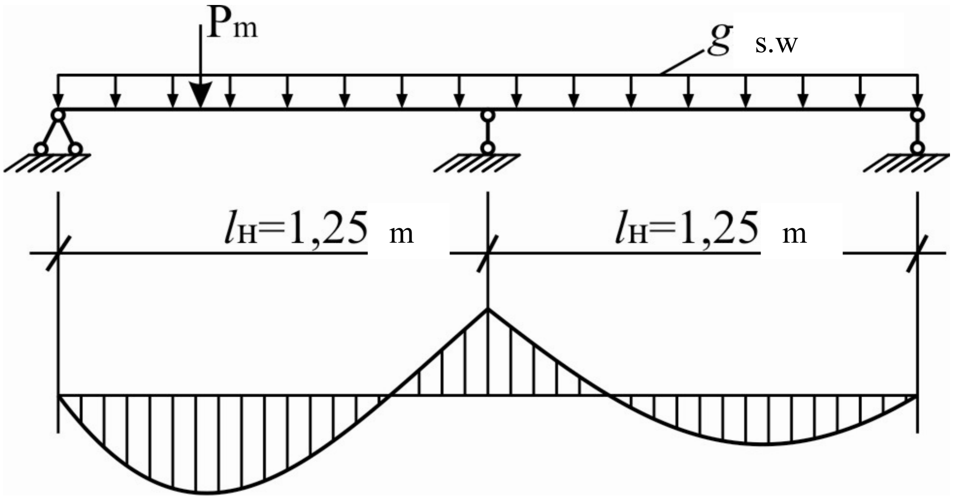


Figure 4 – Static schema of roof sheathing by acting mounting loads

Load from 2 boards δ = 19 mm; b_d = 150 mm

$g_{s.w}$= 2 · 0,019 · 0,15 · 4,2 kN/ m³ = 0,024 kN/m.

Bending moment (in the section under the concentrated load)

$$M_x = 0{,}07 \cdot q_{s.w} \cdot {l_H}^2 + 0{,}207 \cdot P \cdot l_H =$$

$$= 0{,}07 \cdot 0{,}024 \text{ kN/m } \cdot 1{,}25 \text{ m}^2 + 0{,}207 \cdot 1\, kN \cdot 1{,}25 \text{ m} = 0{,}261 \text{ kNm} .$$

Constructive calculation

$$f_{m,d} = k_{\text{mod}} \frac{f_{m,k}}{\gamma_m} = 1{,}1 \cdot \frac{24}{1{,}3} = 20{,}31 \text{ N/mm}^2,$$

k_{mod} =1,1 since the load on the boards from the installer (men with tools) is a accidental action.

$$\sigma = \frac{M(g+p)}{W_{2boards}} = \frac{26{,}1 \text{kN cm}}{18{,}05 \text{cm}^3} = 1{,}45 \text{ kN/cN}^2 = 14{,}5 \text{ N/mm}^2 < 20{,}31 \text{ N/mm}^2,$$

$$W = 2 \cdot \frac{15 \cdot 1{,}9^2}{6} = 18{,}05 \text{ cm}^3.$$

If the check is not perfomed, then you can reduce the step of roof beams or increase the thickness of flooring – which is more effectively determined by a feasibility study. Since the step of the roof beams is set, you must increase the thickness of the floor. According to the assortment, the thickness of the roof sheathing or work deck: 19mm; 22mm; 25mm; 32mm.

Plywood, fibreboard, chipboard and OSB or OSB can be used as roof sheating.

1.2 Design of roof beam (purlin)

1.2.1 Static calculation

Design scheme of roof beam is multi-span unbroken roof beam with even spans throughout the length.

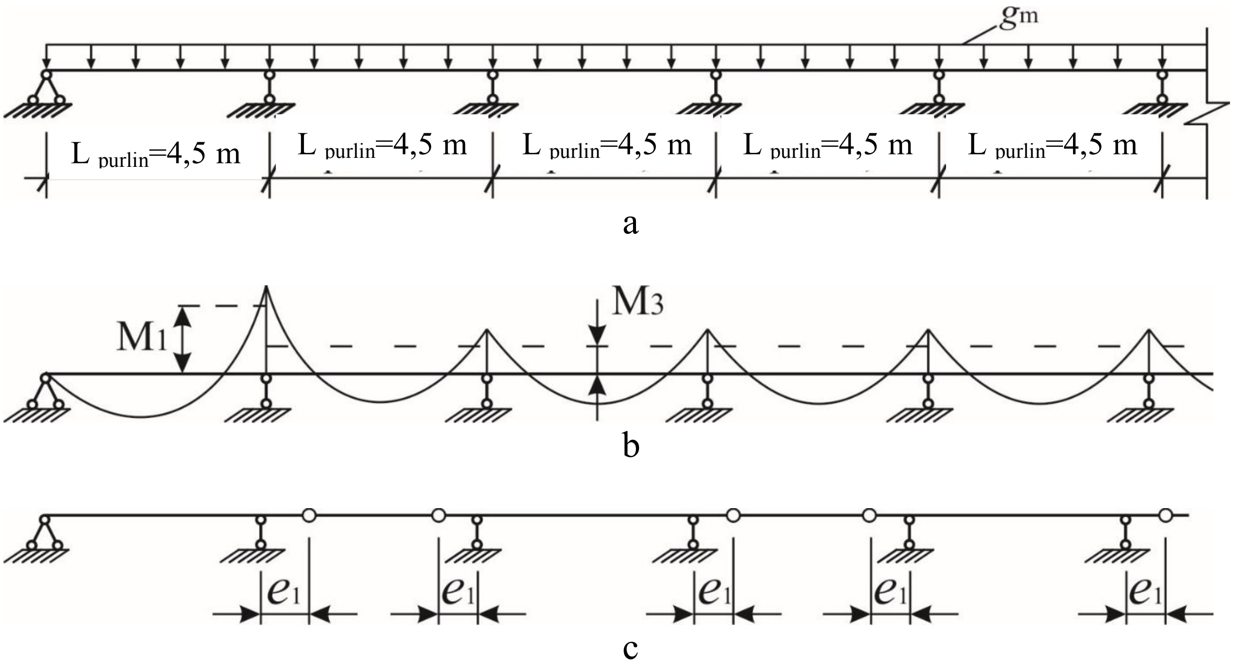


Figure 5 – Scheme of multi-span unbroken purlin and moment diagrams:
a – scheme of loads ;
b – moment diagrams;
c – arrangement of hinges

Load from roof layers (table 1):

$$g_k = 0{,}31\ \text{kN/m}^2.$$

Snow load:

$$S_k = 1{,}67\ \text{kN/m}^2.$$

We accept a roof beam which consists of two boards with cross section 50 * 150 mm. Strength class of the accepted boards is C30. The material for roof beams can be glued laminated timber (GLT), laminated veneer lumber or LVL, and beam type of GLT.

Estimated load from the self weight of the roof beams

Characteristic value:

$$g_k = 2 \cdot 0{,}05 \text{ m} \cdot 0{,}15 \text{ m} \cdot 4{,}6 \text{ kN/m}^3 / 1{,}25 \text{ m} = 0{,}07 \text{ кН/m}^2.$$

Permanent load on the roof beam based on it's self weight

$$g_k = 0{,}31 + 0{,}07 = 0{,}38 \text{ kN/m}^2,$$

$$q_d = (g_d + \cdot s_d) \cdot l_{пн} = (1{,}35 \cdot 0{,}38 + 1{,}5 \cdot 1{,}67) \cdot 1{,}25 = 3{,}76 \text{ kN/m}.$$

When the known value of the intensity of the total load on the purlin (running load) can be determined bending moments:

$$M_{оП} = \frac{q_в \cdot l_{purlin}^{\;2}}{12} = \frac{3{,}76 \cdot 4^2}{12} = 5{,}01 \; kNm.$$

$$M_1 = M_2 = \frac{ql^2}{24} \qquad l_{purlin} = B = 4 \text{ m}.$$

As indicated in Fig. 5 the maximum value of the moment occurs on the M_{on} support, and not in the middle span, except for the first span of the building.

1.2.2 Constructive calculation of roof beam

Strength of the normal stresses of the cross section:

$$\sigma_{m,d} = \frac{M_d}{W_d} \le f_{m,d};$$

where
$$W_d = \frac{M_d}{f_{m,d}} = \frac{5{,}01 \cdot 10^6 \text{Nmm}}{20{,}8 \text{ N/mm}^2} = 241 \cdot 10^3 \text{mm}^3.$$

Design value of the bending strength

$$f_{m,d} = k_{\text{mod}} \cdot \frac{f_{m,k}}{\gamma_M} = 0{,}9 \cdot \frac{30}{1{,}3} = 20{,}8 \text{N/mm}^2.$$

Required height of the roof beam cross section:

$$h_{req} = \sqrt{\frac{6W}{2b}} = \sqrt{\frac{6 \times 241 \cdot 10^3}{2 \cdot 50}} = 120{,}3 \text{ mm.}$$

We accept the roof beam: 2 x 50 x 125 mm.

The strength of the cross-section of the solid wood roof beam necessary be recounted because the height of the section is less than 150 mm.

$$k_h = \min\left\{(150/125)^{0,2}; 1{,}3\right\} = \min\left\{1{,}04;\ 1{,}3\right\} = 1{,}04,$$

$$f_{m,y,d} = k_h \cdot f_{m,d} = 1{,}04 \cdot 20{,}8 = 21{,}6 \text{ N/mm}^2.$$

The determined increase in strength through a scale factor increases the strength of the received cross section.

The moment of resistance and the moment of inertia of the accepted cross section of the roof beam:

$$W_x = 2 \cdot \frac{50 \cdot 125^2}{6} \cong 261 \cdot 10^3 \text{mm}^3,$$

$$J_x = 2 \cdot \frac{50 \cdot 125^3}{12} = 16{,}3 \cdot 10^6 \text{mm}^4.$$

Boards for the runs should take the following thicknesses according to the assortment: δ = 40; 44; 50; 60 mm. It is constructively recommended to take the height of the roof beam, depending on the span by the ratio: $h_{purlin.\ min} \approx B / 24$.

$B = 4$ m $\quad h_{purlin.\ min} = 125$ mm;

$B = 5$ m $\quad h_{purlin.\ min} = 150$ mm;

$B = 6$ m $\quad h_{purlin.\ min} = 175$ mm.

1.2.3 Strength of roof beam be two axes bending

Characteristic values of loads on the roof beam relative to local axes (see. fig. 6):

$$g_{y,k} = 0{,}38 \text{ kN/m}^2 \cdot \sin 3{,}8^\circ \cdot 1{,}25 \text{ m} = 0{,}032 \text{ kN/m},$$

$$g_{z,k} = 0{,}38 \text{ kN/m}^2 \cdot \cos 3{,}8^\circ \cdot 1{,}25 \text{ m} = 0{,}47 \text{ kN/m},$$

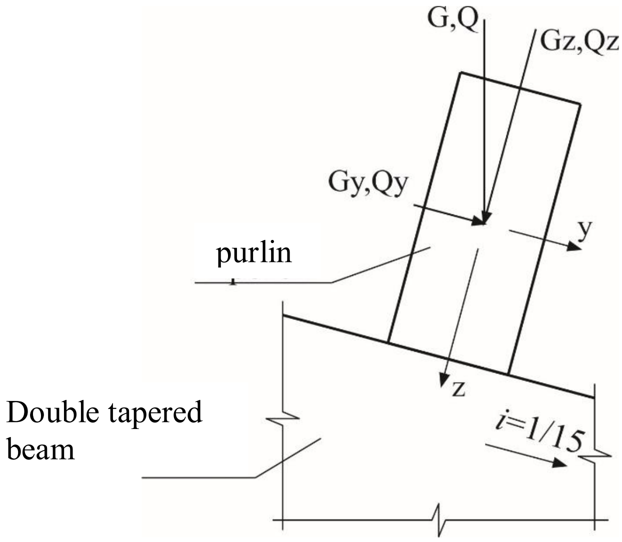


Figure 6 – Schema of the load components action on the roof beam

$$S_{y,k} = 1{,}67 \cdot \sin 3{,}8^{\circ} \cdot 1{,}25 \text{ m} = 0{,}138 \text{ kN/m},$$

$$S_{z,k} = 1{,}67 \cdot \cos 3{,}8^{\circ} \cdot 1{,}25 \text{ m} = 2{,}08 \text{ kN/m}.$$

Design values of the load:

$$q_d = 1{,}35 \cdot g_k + 1{,}5 \cdot S_k,$$

$$q_{y,d} = 1{,}35 \cdot 0{,}032 + 1{,}5 \cdot 0{,}138 = 0{,}25 \ kN/m,$$

$$q_{z,d} = 1{,}35 \cdot 0{,}47 + 1{,}5 \cdot 2{,}08 = 3{,}76 \text{ kN/m}.$$

Maximum bending moment

$$M_{1,y,d} = \frac{q_{z,d} \cdot l^2}{12} = \frac{3{,}76 \cdot 4^2}{12} = 5{,}01 \text{ kNm},$$

$$M_{1,z,d} = \frac{q_{y,d} \cdot l^2}{12} = \frac{0{,}25 \cdot 4^2}{12} = 0{,}33 \text{ kNm}.$$

Moments of the resistance

$$W_{y,d} = \frac{b \cdot h^2}{6} = \frac{100 \text{ mm} \cdot 125^2 \text{ mm}}{6} = 0{,}26 \cdot 10^6 \text{ mm}^3,$$

$$W_{z,d} = \frac{h \cdot b^2}{6} = \frac{125 \text{ mm} \cdot 100^2 \text{ mm}}{6} = 0{,}21 \cdot 10^6 \text{ mm}^3.$$

Bending stresses in the roof beam

$$\sigma_{m,y,d} = \frac{M_{1,y,d}}{W_{y,d}} = \frac{5{,}01 \cdot 10^6 \text{ N} \cdot \text{mm}}{0{,}26 \cdot 10^6 \text{ mm}^3} = 19{,}27 \text{ N/mm}^2,$$

$$\sigma_{m,z,d} = \frac{M_{1,z,d}}{W_{z,d}} = \frac{0{,}33 \cdot 10^6 \text{ N} \cdot \text{mm}}{0{,}21 \cdot 10^6 \text{ mm}^3} = 1{,}57 \text{ N/mm}^2.$$

Since the height of the cross-section of the board is less than 150 mm, then design strength value need calculate by multiplying on the size factor:

$$k_h = \min\{(150/125)^{0,2};1,3\} = \min\{1,04;\ 1,3\} = 1,04,$$

$$f_{m,y,d} = k_h \cdot f_{m,d} = 1,04 \cdot 20,8 = 21,6 \text{ N/mm}^2.$$

$$k_h = \min\{(150/100)^{0,2};1,3\} = \min\{1,09;\ 1,3\} = 1,09,$$

$$f_{m,y,d} = k_h \cdot f_{m,d} = 1,09 \cdot 20,8 = 22,7 \text{ N/mm}^2.$$

Checking of roof beam strength:

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \cdot \frac{\sigma_{m,z,d}}{f_{m,z,d}} \le 1 \quad \text{and} \quad k_m \cdot \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \le 1,$$

where $k_m = 0,7$ (for rectangular cross section).

$$\frac{19,27}{21,6} + 0,7 \cdot \frac{1,57}{22,7} = 0,892 + 0,7 \cdot 0,069 = 0,94 \le 1,$$

$$0,7 \cdot \frac{19,27}{21,6} + \frac{1,57}{22,7} = 0,7 \cdot 0,892 + 0,069 = 0,69 \le 1.$$

Condition is fulfilled. Strength is provided.

1.2.4 Deflection

$k_{def} = 0,6$ (see Annex B, table1),

$\psi_{2,1} = 0,2$

(ДСТУ-Н Б В.1.2-13:2008 or EN 1990:2002)

$E_{0,mean}$ = 12000 N/mm^2 (Module of elasticity for solid timber strength class C30, see Anex Б, table 1)

Moment of inertia:

$$I_y = \frac{b \cdot h^3}{12} = \frac{100 \cdot 125^3}{12} = 16,3 \cdot 10^6 \text{mm}^4,$$

$$I_z = \frac{h \cdot b^3}{12} = \frac{125 \cdot 100^3}{12} = 10,4 \cdot 10^6 \text{mm}^4.$$

The instantaneous deflections are:

$$w_{inst,G,y} = \frac{1 \cdot g_{y,k} \cdot l^4}{384 \cdot E \cdot I} = \frac{1 \cdot 0{,}143 \cdot 4000^4}{384 \cdot 12000 \cdot 16{,}3 \cdot 10^6} = 0{,}49\,\text{mm},$$

$$w_{inst,G,z} = \frac{1 \cdot g_{z,k} \cdot l^4}{384 \cdot E \cdot I_y} = \frac{1 \cdot 0{,}44 \cdot 4000^4}{384 \cdot 12000 \cdot 10{,}4 \cdot 10^6} = 2{,}35\,\text{mm},$$

$$w_{inst,G} = \sqrt{w_{inst,G,y}{}^2 + w_{inst,G,z}{}^2} = \sqrt{0{,}49^2 + 2{,}35^2} = 2{,}4\,\text{mm},$$

$$w_{inst,Q,1} = \frac{q_k}{g_k} \cdot w_{inst,G} = \frac{1{,}67}{0{,}37} \cdot 2{,}4 = 10{,}8\,\text{mm},$$

$$w_{inst} = w_{inst,G} + w_{inst,Q,1} = 2{,}4 + 10{,}8 = 13{,}2\,\text{mm} < \frac{l}{300} = \frac{4000}{300} = 13{,}3\,\text{mm}.$$

Final deflection

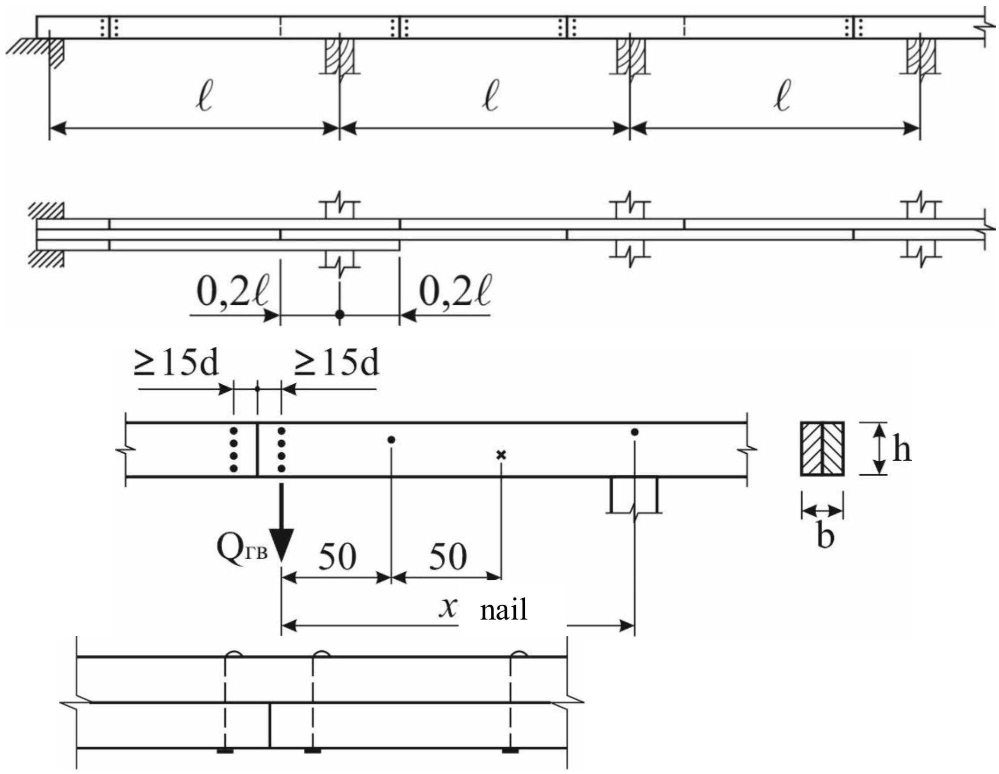
$$w_{fin} = w_{inst} + \left(w_{inst,G} + \sum_{i=1}^{n} \psi_{2,i} \cdot w_{inst,Q,1} \right) \cdot k_{def} = 13{,}2 + (2{,}4 + 0{,}2 \cdot 10{,}8) \cdot 0{,}6 =$$

$$= 15{,}9\,\text{mm} < \frac{l}{150} = \frac{4000}{150} = 26{,}7\,\text{mm}.$$

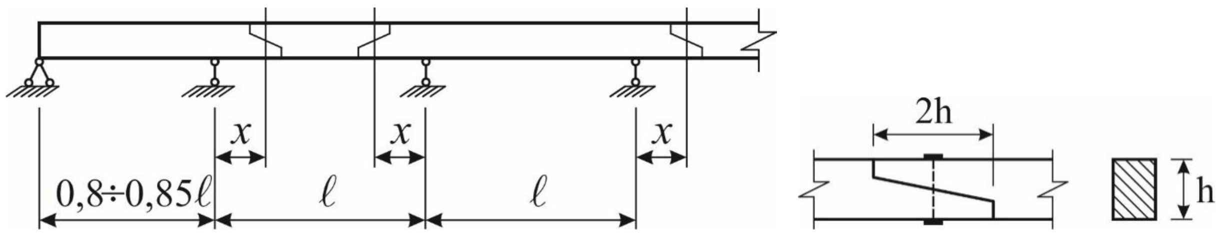
If the resulting deflection exceeds the limit, then it is necessary to accept beam with a higher cross-section height (150 mm for this case) and check one more rigidity (deflection).

Since in extreme spans there is a bending moment more than on support, then we install an additional board of the same section. On the length of the board joint with nails 120 x with distance in 500 mm, as shown in Figure 7-a.

Joint of the roof beam boards located at a distance of $X = 0{,}21 \cdot l$ from the support axis. Here l is distance between double tapered beams or roof beams supports. The roof beam can also be executed as a solid wood element of the same cross section as with the two boards, but jointed with two bolts, fig. 7, b), two bolts, (fig. 7, b).



a

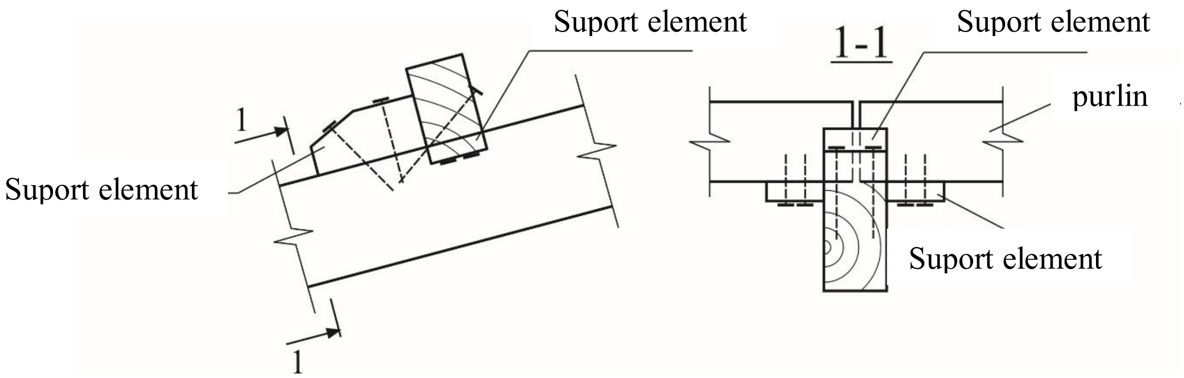


b

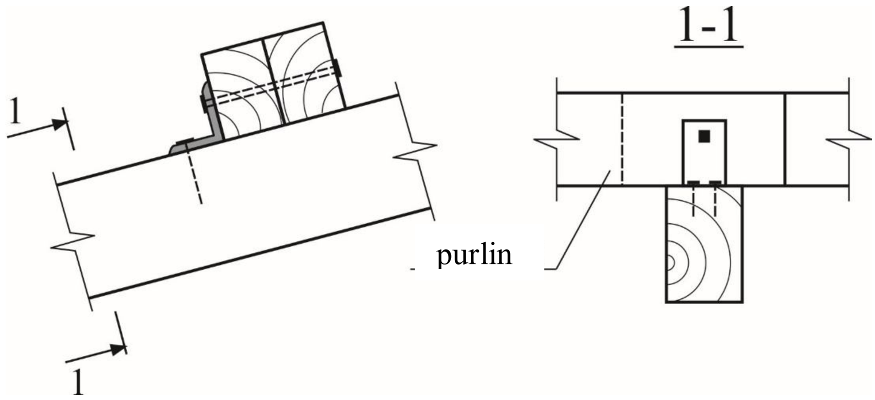
Figure 7 – Roof beam joints for cases of using boards and lumber:
a – nailed roof beam joint;
b – roof beam joint with bolts

Unbreakable multi-span roof beams can also be made of glued laminated timber as well as laminated veneer lumber (LVL). The length of such elements can reach 12–13 m (limited through the dimensions of vehicles), which significantly reduces the number of connections in the roof beams.

When installing roof beams on douple tapered beams with inclined upper edges, it is necessary to install wooden or metal constructive elements in the mounting areas, as shown in Figure 8.



a



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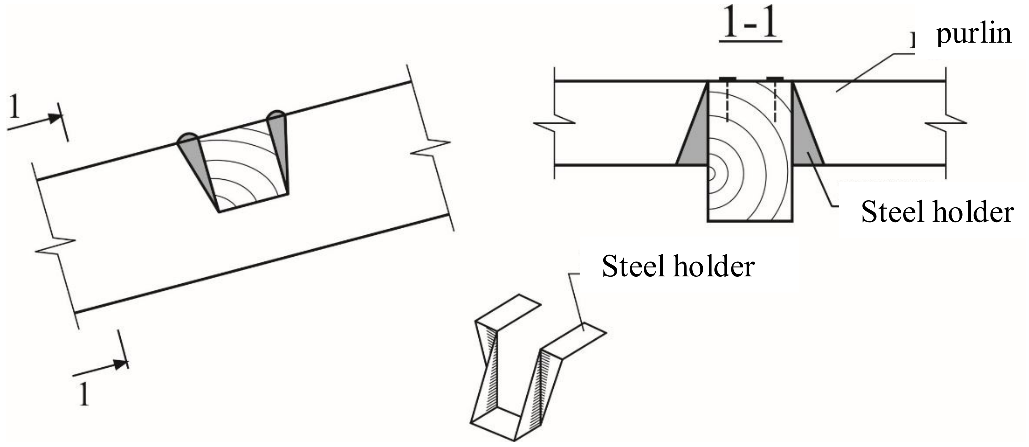


c

Figure 8 – Roof beam fixing:
a – constructive element;
b – metal L-type profile;
c – collar (or holder)

2 GLUED LAMINATED DOUBLE TAPERED BEAM

Need to design and calculate the double tapered glued laminated beam in the heated building in the city of Sumy (the span of beam is 15 m, the step of bearing structures B = 4 m, the slope of the upper edge i = 1/15). Material of the beam is a combined glued laminated timber (or GLT) of the strength class GL30c. Service class – 1.

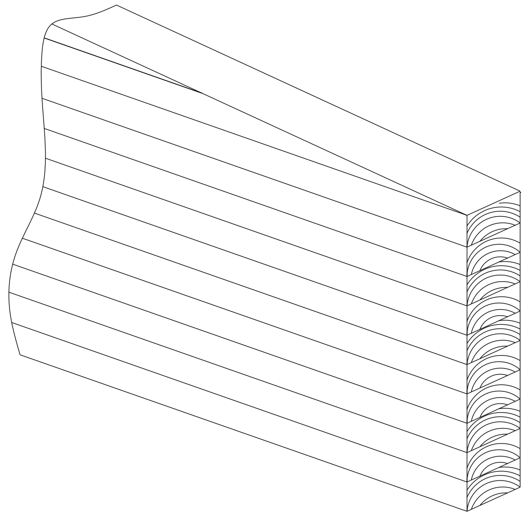


Figure 9 – View of the part of double tapered GLT beam

l – design value of the beam span

$$l = l^* - 2 \cdot 0{,}25\ \text{m} = l^* - 0{,}5\ \text{m}.$$

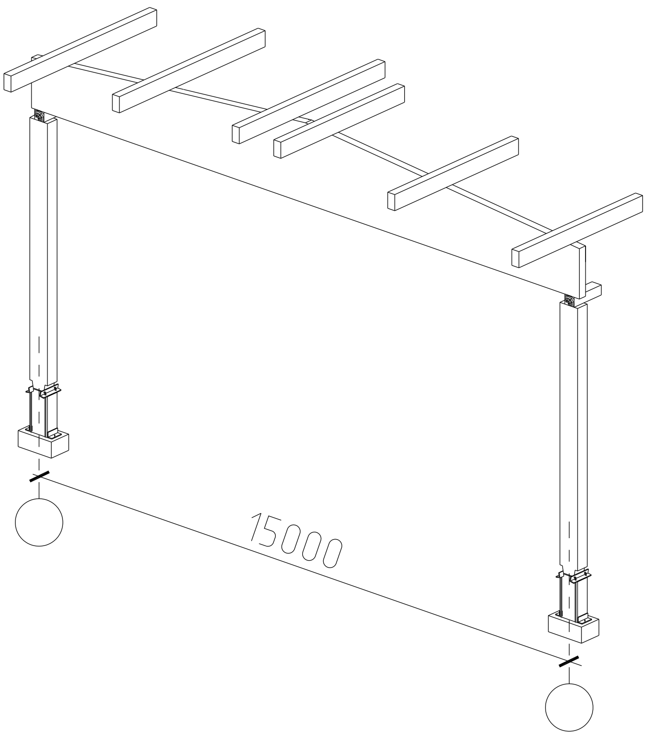


Figure 10 – Schema of roof beams location

2.1 Static calculation

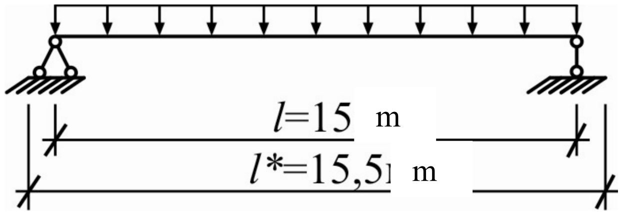


Figure 11 – Static schema

Constructive calculation

For beams $l \leq 18$ m b_{min} = 11–12 cm $\left[l = 21 - 24 \text{ m}; \quad b_{min} \geq 13{,}5 \text{ cm} \right]$

Recomendation b_{beards} = 150 175 200 225 250 mm;

b_{beam} = 140 165 185 210 230 mm.

l, m	9 – 10	11 – 13	14 – 16	17 – 18	19 – 21	22 – 24
b, mm	135	135 160	160 180	180 205	205 230	230

Accept b = 170 mm.

Height of beam is approximately $0{,}1 \cdot L = 0{,}1 \cdot 15000$ mm = 1500 mm/

2.2 Calculation of loads

Dead load (self weight)

The average density value of GLT of the strength class Gl30c according to table 4 of standart EN 14080:2013 is $\rho_{g,mean}$ = 430 kg/m^3.

Characteristic value of beam self weight:

$g_k = 0{,}17 \text{ m} \cdot 1{,}5 \text{ m} \cdot 15 \text{ m} \cdot 4{,}3 \text{ kN/m}^3 = 16{,}45 \text{ kN/m}$

The area of coverage from which the beam obtain the load is $4 \cdot 15$ m = 60 m^2. Load from self weight beam at 1 m^2 is

$$g_{k,beam} = g_k / S = 16{,}45 \text{ kN} / 60 \text{ m} = 0{,}27 \text{ kN/m}^2.$$

Permanent load on the beam, taking into account its self weight:

$$g_k = 0{,}37 + 0{,}27 = 0{,}64 \text{ kN/m}^3,$$

$$q_d = (g_d + \cdot s_d) \cdot l_{пн} = (1{,}35 \cdot 0{,}64 + 1{,}5 \cdot 1{,}67) \cdot 4 = 13{,}5 \text{ kN/m}.$$

Maximum bending moment in the middle of the span (in the apex zone)

$$M_{ap,d} = \frac{q_d \cdot l^2}{8} = \frac{13{,}5 \cdot 15^2}{8} = 379{,}7 \text{ kNm.}$$

Transverse force

$$Q_{\max,d} = \frac{q_d \cdot l}{2} = \frac{13{,}5 \cdot 15}{2} = 101{,}3 \text{ kN.}$$

Design values of the strength of beam material strength class GL30c.

$$f_{m,g,d,T22} = k_{\text{mod}} \cdot \frac{f_{m,g,k}}{\gamma_M} = 0{,}9 \cdot \frac{30}{1{,}25} = 21{,}6 \text{ N/mm}^2,$$

$$f_{t,90,g,d} = k_{\text{mod}} \cdot \frac{f_{t,90,g,k}}{\gamma_M} = 0{,}9 \cdot \frac{0{,}5}{1{,}25} = 0{,}36 \text{ N/mm}^2,$$

$$f_{v,g,d} = k_{\text{mod}} \cdot \frac{f_{v,g,k}}{\gamma_M} = 0{,}9 \cdot \frac{3{,}5}{1{,}25} = 2{,}52 \text{ N/mm}^2,$$

$$f_{c,0,g,d} = k_{\text{mod}} \cdot \frac{f_{c,0,g,k}}{\gamma_M} = 0{,}9 \cdot \frac{24{,}5}{1{,}25} = 17{,}64 \text{ N/mm}^2,$$

$$f_{c,90,g,d} = k_{\text{mod}} \cdot \frac{f_{c,90,g,k}}{\gamma_M} = 0{,}9 \cdot \frac{2{,}5}{1{,}25} = 1{,}8 \text{ N/mm}^2.$$

2.3 Geometrical dimensions of a double tapered GLT beam

The height of the beam on the support based on the maximum shear required height is determined as follows:

$$h_s \approx \frac{1{,}5 \cdot Q_d}{b_{ef} \cdot f_{v,g,d}} = \frac{1{,}5 \cdot Q_d}{b \cdot k_{cr} \cdot f_{v,g,d}} = \frac{1{,}5 \cdot 101{,}3 \cdot 10^{-3}}{0{,}17 \cdot 0{,}67 \cdot 2{,}52} = 0{,}53 \text{ m.}$$

We accept the height of the beams on the support $h_s = 0{,}55$ m.

Height of beams in the middle span:

$$h_{ap} = h_s + i \cdot \frac{l}{2} = 0{,}55 \text{ m} + \frac{1}{15} \cdot \frac{15 \text{ m}}{2} = 1{,}05 \text{ m.}$$

The height of the beam in the apex should be more than two heights of the beam on the support.

We accept the height of the beams in the middle h_{ap}=1,1 m (Fig. 12).

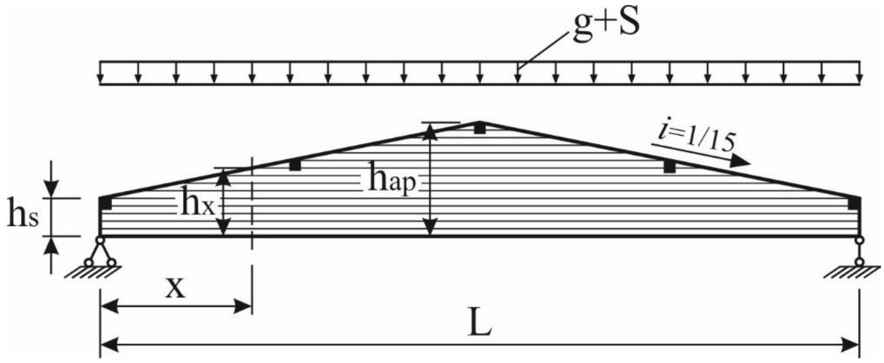


Figure 12 – Scheme of beam loading and place of attachment bracing system.

2.4 Calculation of the beam strength by bending

For a symmetrical double tapered beam with constant uniformly distributed load q_d, the location of the critical cross section – that is the abscissa where the maximum bending stress occurs – can be calculated as follows:

$$X = \frac{l}{2} \cdot \frac{h_s}{h_{ap}} = \frac{15\,\text{m}}{2} \cdot \frac{0{,}55}{1{,}1} = 3{,}25\,\text{m}.$$

The corresponding depth of the beam is:

$$h_x = h_s + x \cdot \tan\delta = 0{,}55 + 3{,}25 \cdot 0{,}07 = 0{,}778\ \text{m},$$

$$h_x = 0{,}778\,\text{m}\ >\ 0{,}6\,\text{m} \rightarrow k_h = 1,$$

$$M_{x,d} = Q_d \cdot x - \frac{q_d \cdot x^2}{2} = 101{,}3\text{kN} \cdot 0{,}778\ \text{m} - \frac{13{,}5\ \text{kN/m} \cdot (0{,}778\ \text{m})^2}{2} = 74{,}7\ \text{kNm},$$

$$W_{y,x} = \frac{b \cdot h_x^2}{6} = \frac{0{,}17 \cdot 0{,}778^2}{6} = 0{,}017\ \text{m}^3.$$

The corresponding bending stress at the critical cross section is (bending and tension):

$$\sigma_{m,0,d} = \frac{M_{x,d}}{W_{y,x}} = \frac{74{,}7\text{kNm}}{0{,}017\text{m}^3} = 4{,}4\ \text{N/mm}^2,$$

$$\frac{\sigma_{m,0,d}}{f_{m,g,d}} = \frac{4{,}4}{21{,}6} = 0{,}2 < 1.$$

Bending stress at the critical cross section is (bending and compression):

$$\sigma_{m,\alpha,d} = \sigma_{m,0,d} = 4{,}4\ \text{N/mm}^2,$$

$$k_{m,\alpha} = \frac{1}{\sqrt{1+\left(\dfrac{f_{m,g,d}}{0{,}75\cdot f_{v,g,d}}\cdot\tan\alpha\right)^2+\left(\dfrac{f_{m,g,d}}{f_{t,90,g,d}}\cdot\tan^2\alpha\right)^2}} =$$

$$= \frac{1}{\sqrt{1+\left(\dfrac{21{,}6}{0{,}75\cdot 2{,}52}\cdot\dfrac{1}{15}\right)^2+\left(\dfrac{21{,}6}{1{,}8}\cdot\left(\dfrac{1}{15}\right)^2\right)^2}} = \frac{1}{\sqrt{1+0{,}59+0{,}003}} = 0{,}794$$

$$\frac{\sigma_{m,\alpha,d}}{k_h\cdot k_{m,\alpha}\cdot f_{m,d}} = \frac{4{,}4}{1\cdot 0{,}794\cdot 21{,}6} = 0{,}26 < 1.$$

2.5 The bending moment at mid-span

Moment of resistance

$$W_{y,ap,d} = \frac{b\cdot h_{ap}^2}{6} = \frac{0{,}17\cdot 1{,}1^2}{6} = 0{,}034\,\text{m}^3,$$

$$k_l = 1+1{,}4\cdot\tan\alpha+5{,}4\cdot\tan^2\alpha = 1+1{,}4\cdot\frac{1}{15}+5{,}4\cdot\left(\frac{1}{15}\right)^2 = 1{,}12,$$

$$k_p = 0{,}2\cdot\tan\alpha = 0{,}2\cdot\frac{1}{15} = 0{,}013,$$

$$\sigma_{m,d} = k_l\cdot\frac{M_{ap,d}}{W_{y,ap,d}} = 1{,}12\cdot\frac{379{,}7\ \text{kNm}}{0{,}034\ \text{m}^3} = 12{,}51\ \text{N/mm}^2.$$

The tensile stress perpendicular to the grain can be evaluated by multiplying the bending stress at mid-span by the factor k_p, which can be taken from volume

$$\sigma_{t,90,d} = k_p\cdot\frac{M_{ap,d}}{W_{y,ap,d}} = 0{,}013\cdot\frac{379{,}7\,\text{kNm}}{0{,}034\,\text{m}^3} = 0{,}16\,\text{N/mm}^2.$$

Factor k_{dis} is takes into account that the tension stress perpendicular to the grain is not uniformly distributed in the loaded timber volume V

$$k_{dis} = 1{,}4, \qquad k_{vol} = \left(\frac{V_0}{V}\right)^{0{,}2}.$$

where $V_0 = 0{,}01$ m^3 is reference volume.

$$V = h_{ap} \cdot \left(h_{ap} - 0{,}25 \cdot h_{ap} \cdot \tan\alpha\right) \cdot b = 1{,}1 \cdot \left(1{,}1 - 0{,}25 \cdot 1{,}1 \cdot \frac{1}{15}\right) \cdot 0{,}17 = 0{,}2 \text{ m}^3.$$

The volume of wood which is loaded in tension can be estimated as follows

$$V \le 2/3 \cdot \left(l \cdot h_s + l/3 \cdot \left(h_{ap} - h_s\right)\right) \cdot b = 2/3 \cdot \left(15 \cdot 0{,}55 + 15/3 \cdot (1{,}1 - 0{,}55)\right) \cdot 0{,}17 = 1{,}25 \text{ m}^3.$$

$$k_{vol} = \left(\frac{0{,}01}{0{,}2}\right)^{0{,}2} = 0{,}549.$$

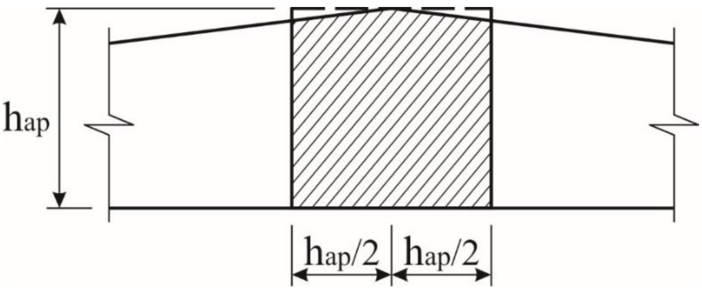


Figure 13 – Geometry of the tensed volume in the apex zone of the beam

Verification:

$$\frac{\sigma_{m,d}}{k_r \cdot f_{m,g,d}} = \frac{\sigma_{m,d}}{k_r \cdot f_{m,g,d,T15}} = \frac{12{,}51}{1 \cdot 19{,}4} = 0{,}64 < 1$$

$$f_{m,g,d,T15} = k_{\text{mod}} \cdot \frac{f_{m,g,k}}{\gamma_M} = 0{,}9 \cdot \frac{27}{1{,}25} = 19{,}4 \text{ N/mm}^2.$$

The tension strength perpendicular to grain shall be reduced in order to take into account the volume effect.

$$\frac{\sigma_{t,90,d}}{k_{dis} \cdot k_{vol} \cdot f_{t,90,g,d}} = \frac{0{,}16}{1{,}4 \cdot 0{,}549 \cdot 0{,}36} = 0{,}57 < 1.$$

2.6 Lateral torsional buckling

Lateral torsional buckling in the double tapered beam (primary beams) may only occur between two adjacent purlins (roof beams), under the condition that 1) the roof is braced in the transverse direction and 2) the purlins are adequately fastened to the primary beams.

The height of the cross-section of the beam to check its buckling stability is taken at a distance 0,65·*L*/2.

$$l_{ef} = \frac{L}{2} = \frac{15\ \text{m}}{2} = 7{,}5\ \text{m},$$

$$E_{0,g,05} = 10800\ \text{N/mm}^2,$$

$$G_{g.05} = 540\ \text{N/mm}^2,$$

$$h_{0,65} = h_s + 0{,}65(h_{ap} - h_s) = 0{,}55 + 0{,}65(1{,}1 - 0{,}55) = 0{,}908\ \text{m},$$

$$I_z = \frac{b^3 \cdot h_{0,65}}{12} = \frac{0{,}17^3 \cdot 0{,}908}{12} = 3{,}72 \cdot 10^{-4}\,\text{m}^4,$$

$$\frac{h_{0,65}}{b} = \frac{0{,}908}{0{,}17} = 5{,}34 \rightarrow \alpha = 0{,}293,$$

$$I_{tor} = \alpha \cdot b^3 \cdot h_{0,65} = 0{,}293 \cdot 0{,}17^3 \cdot 0{,}908 = 1{,}31 \cdot 10^{-3}\,\text{m}^4,$$

$$W_y = \frac{b \cdot h_{0,65}^2}{6} = \frac{0{,}17 \cdot 0{,}908^2}{6} = 2{,}34 \cdot 10^{-2}\,\text{m}^3.$$

The critical bending stress can be calculated

$$\sigma_{m,crit} = \frac{M_{y,crit}}{W_y} = \frac{\pi\sqrt{E_{0,g,05} \cdot I_z \cdot G_{g,05} \cdot I_{tor} \cdot 1{,}4}}{l_{ef} \cdot W_y} =$$

$$= \frac{3{,}14 \cdot \sqrt{10800\ \text{MN/m}^2 \cdot 3{,}72 \cdot 10^{-4}\,\text{m}^4 \cdot 540\ \text{MN/m}^2 \cdot 1{,}31 \cdot 10^{-3}\,\text{m}^4 \cdot 1{,}4}}{7{,}5\ \text{m} \cdot 2{,}34 \cdot 10^{-2}\,\text{m}^3} =$$

$$= 35{,}7\ \text{MN/m}^2 \cong 35{,}7\ \text{N/mm}^2.$$

The relative slenderness ratio for bending

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,g,k}}{\sigma_{m,crit}}} = \sqrt{\frac{30}{35,7}} = 0,92 \,,$$

$$0,75 < \lambda_{rel,m} \leq 1,4 \,,$$

$$k_{crit} = 1,56 - 0,75 \cdot \lambda_{rel,m} = 0,87 \,.$$

Verification of beam narrow edge which parallel to the grain direction (bending with tension) in the section x:

$$\sigma_{m,0,d} = 4,44, \quad \text{GL30c}$$

$$\frac{\sigma_{m,0,d}}{k_{crit} \cdot k_h \cdot f_{m,g,d}} = \frac{4,44}{0,87 \cdot 1 \cdot 21,6} = 0,24 < 1 \,.$$

Verification of beam upper slope edge (bending with compression) in the section x:

$$\sigma_{m,0,d} = 4,44, \quad \text{GL30c}$$

$$\frac{\sigma_{m,\alpha,d}}{k_{crit} \cdot k_{m,\alpha,c} \cdot f_{m,g,d}} = \frac{4,44}{0,87 \cdot 0,794 \cdot 21,6} = 0,30 < 1 \,.$$

2.7 Deflection

Components of the vertical deflection of the beam, taking into account the creep and precamber are shown in Figure 14.

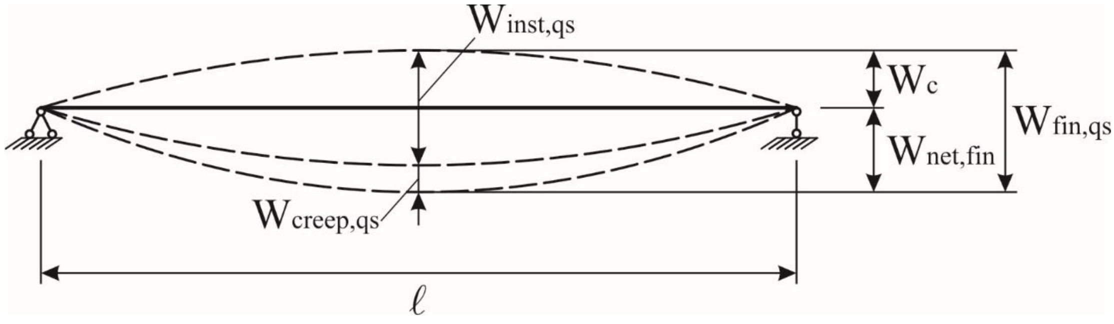


Figure 14 – Components of beam deflection

Elastic characteristics of timber beam strength class GL30c:

$$E_{0,g,mean} = 13000\,\text{N/mm}^2 = 13000\,\text{MN/m}^2,$$

$$G_{g.05} = 650\,\text{N/mm}^2 = 650\,\text{MN/m}^2.$$

Since the service class of the building – 1 and the shortest action on the building (snow load) is a "short-term effect" on the class of the duration of the load, the coefficient k_{def} = 0,6.

Instantaneous deformation:

$$I_s = \frac{b \cdot h_s^3}{12} = \frac{0{,}17 \cdot 0{,}55^3}{12} = 2{,}36 \cdot 10^{-3}\,\text{m}^4,$$

$$M_{\max} = \frac{q_{g,k} \cdot L^2}{8} = \frac{(0{,}64 + 1{,}67) \cdot 15^2}{8} = 36{,}84\,\text{kNm} = 3{,}68 \cdot 10^{-2}\ \text{MNm},$$

$$k_m = \frac{(h_s / h_{ap})^3}{0{,}15 + 0{,}85 \cdot (h_s / h_{ap})} = \frac{(0{,}55/1{,}1)^3}{0{,}15 + 0{,}85 \cdot (0{,}55/1{,}1)} = 0{,}22,$$

$$k_v = \frac{2}{1 + (h_{ap} / h_s)^{2/3}} = \frac{2}{1 + (1{,}1/0{,}55)^{2/3}} = 0{,}77.$$

2.8 Beam deflection

The instantaneous deflections due to self weight:

$$w_{inst,G} = \frac{M_{\max} \cdot l^2}{9{,}6 \cdot E_{0,g,mean} \cdot I_s} \cdot k_m + \frac{1{,}2 \cdot M_{\max}}{G_{g,mean} \cdot A_s} \cdot k_v =$$

$$= \frac{3{,}68 \cdot 10^{-2}\ \text{MNm} \cdot 15\ \text{m}^2}{9{,}6 \cdot 13000\ \text{MN/m}^2 \cdot 2{,}36 \cdot 10^{-3}\text{m}^4} \cdot 0{,}22 + \frac{1{,}2 \cdot 3{,}68 \cdot 10^{-2}\ \text{MNm}}{650\ \text{MN/m}^2 \cdot (0{,}55\ \text{m} \cdot 0{,}17\ \text{m})} \cdot 0{,}77 =$$

$$= 0{,}0062 + 0{,}0006 = 0{,}0068\ \text{m} = 6{,}8\ \text{mm}.$$

The instantaneous deflections due to the variable action (snow load):

$$w_{inst,Q} = \frac{q_k}{g_k} \cdot w_{inst,G} = \frac{1{,}67}{0{,}64} \cdot 6{,}8\ \text{mm} = 17{,}7\ \text{mm}.$$

The creep deflection for the variable load is determined from $\psi_2 \cdot q_k$. ($\psi_2 = 0$ for the snow load, as the location of the building is less than 1000 m above sea level).

Corresponding to the selfweight the deflection for the variable load can be calculated as

$$w_{inst} = w_{inst,G} + w_{inst,Q} = 6{,}8\ \text{mm} + 17{,}7\ \text{mm} = 24{,}5\ \text{mm} < \frac{1}{300} = \frac{15000}{300} = 50\ \text{mm}.$$

Final deflection with taken into account of creep:

$$w_{fin} = w_{inst} + \left(w_{inst,G} + \sum_{i=1}^{n} \psi_{2,i} \cdot w_{inst,Q} \right) \cdot k_{def} = 24{,}5 + (6{,}8 + 0 \cdot 17{,}7) \cdot 0{,}6 =$$

$$= 28{,}6\ \text{mm} < \frac{1}{200} = \frac{15000}{200} = 75\ \text{mm}.$$

Final long term deflection from a quasi-constant load without precamber $w_c = 0$:

$$w_{net,fin} = \left(w_{inst,G} + \sum_{i=1}^{n} \psi_{2,i} \cdot w_{inst,Q} \right) \cdot (1 + k_{def}) - w_c = (6{,}8 + 0 \cdot 17{,}7) \cdot (1 + 0{,}6) - 0 =$$

$$= 10{,}88\ \text{mm} < \frac{1}{300} = \frac{15000}{300} = 50\ \text{mm}.$$

2.9 Compression perpendicular to the grain at the supports

2.9.1 Determination of stresses

The wood of the beam on the support is compressed perpendicular to the grain on the support (lining beam), see Figure 15. The cross section of the lining beam is taken from solid timber with cross section parameters 150 × 150 mm.

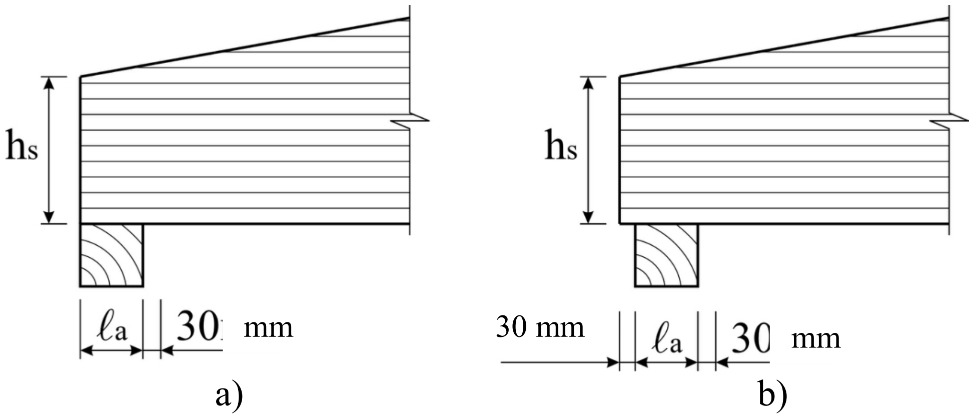


Figure 15 – Schemes of the beam support on the lining beam

Effective area of the beam which loaded perpendicular to the grain (diagram a, Fig. 15):

$$l_{ef} = l_A = 150\,\text{mm} + 30\,\text{mm} = 180\,\text{mm},$$

$$A_{ef} = l_{ef} \cdot b = 180\,\text{mm} \cdot 170\,\text{mm} = 30600\,\text{mm}^2,$$

$$k_{c,90} = 1{,}75 \qquad (l_1 > 2h),$$

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{ef}} = \frac{Q_{\max,d}}{A_{ef}} = \frac{101{,}3\ \text{kN}}{30600\ \text{mm}^2} = 3{,}31\ \text{N/mm}^2.$$

Strength verification:

$$\frac{\sigma_{c,90,d}}{k_{c,90} \cdot f_{c,90g,d}} = \frac{3{,}31\ \text{N/mm}^2}{1{,}75 \cdot 1{,}8\ \text{N/mm}^2} = 1{,}05 > 1$$

verification is not performed.

For increasing the effective area, we adopt the support (scheme b, fig. 15). Effective area of the beam perpendicular to the grain (scheme b, fig. 15):

$$l_{ef} = l_A = 150\,\text{mm} + 2 \cdot 30\,\text{mm} = 210\,\text{mm},$$

$$A_{ef} = l_{ef} \cdot b = 210\,\text{mm} \cdot 170\,\text{mm} = 35700\,\text{mm}^2,$$

$$k_{c,90} = 1{,}75 \qquad (l_1 > 2h),$$

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{ef}} = \frac{Q_{\max,d}}{A_{ef}} = \frac{101{,}3\ \text{kN}}{35700\ \text{mm}^2} = 2{,}84\ \text{N/mm}^2.$$

Verification

$$\frac{\sigma_{c,90,d}}{k_{c,90} \cdot f_{c,90g,d}} = \frac{2{,}84 \text{ N/mm}^2}{1{,}75 \cdot 1{,}8 \text{ N/mm}^2} = 0{,}9 < 1$$

verification is performed.

2.9.2 Shear at the supports

Reduced transverse force:

$$V_d^{/} = \max V_d - q_d\left(h_s + \frac{l_A}{2}\right) = Q_d - q_d\left(h_s + \frac{l_A}{2}\right) =$$

$$= 101{,}3 \text{ kN} - 13{,}5 \text{ kN/m} \cdot \left(0{,}55 \text{ m} + \frac{0{,}15 \text{ m}}{2}\right) = 92{,}9 \text{ kN}.$$

$$h_s^{/} = h_s + (l_A + h_s) \cdot \tan\alpha = 0{,}55 \text{ m} + (0{,}15 \text{ m} + 0{,}55 \text{ m}) \cdot \left(\frac{1}{15}\right) = 0{,}6 \text{ m}.$$

Shear stress:

$$\tau_d = 1{,}5 \cdot \frac{V_d^{/}}{b_{ef} \cdot h_s^{/}} = 1{,}5 \cdot \frac{V_d^{/}}{k_{cr} \cdot b \cdot h_s^{/}} = 1{,}5 \cdot \frac{92{,}9 \text{ kN}}{0{,}67 \cdot 0{,}17 \text{ m} \cdot 0{,}6 \text{ m}} = 2{,}04 \text{ MN/m}^2.$$

Strength verification:

$$\frac{\tau_d}{f_{v,g,d}} = \frac{2{,}04}{2{,}52} = 0{,}81 < 1.$$

The selected strength class GL30c is combined and consists of boards of different strength classes, where in the extreme zones of the cross section there is a higher (strong) timber, figure 16.

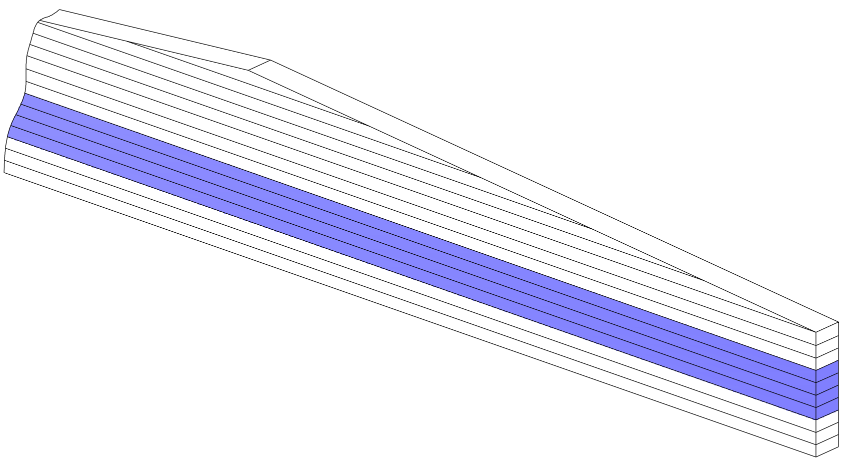


Figure 16 – Scheme of location of low-grade timber boards in the middle of a double tapered beam

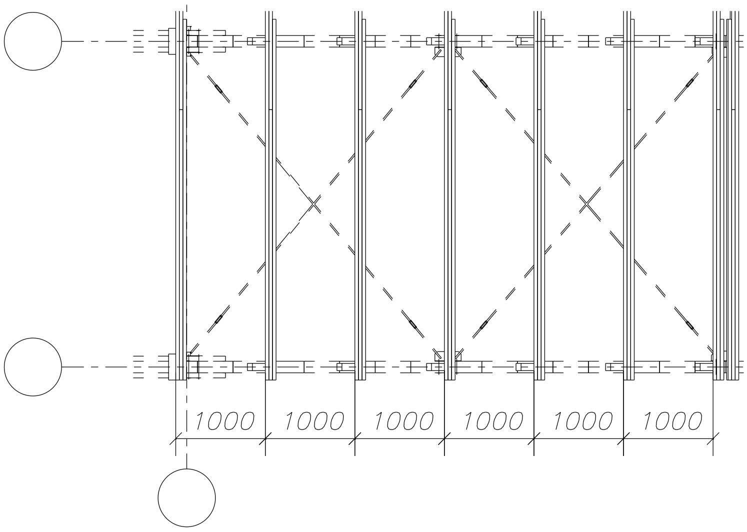


Figure 17 – Bracing system for wind load

3 DESIGN OF CROSS FRAME WITH GLT COLUMNS

Design and calculate the column of one-store building in Sumy city. Span is l = 15 m, bearing structure distance is B = 4 m, mark of the column top is H = 4 m. Structure of roof sheating is continuous beam; walls – three-hinged sandwich panels with mineral wool insulation and metal sheets; bearing structure of roof sheating is glued laminated double tapered beams. Column material is the second grade pine board, service class 2.

3.1 Geometry parameters of cross section of column

The height of column section is determined using slenderness ratio and height of the column as shown below.

Notation conventions:

H – the height of column;

h – the height of cross section of column;

b – the width of cross section of column;

$$h = 3{,}3 \cdot n \left[h = \left(\frac{1}{10} \div \frac{1}{20} \right) H \right];$$

$b = 20{,}5$ cm $\quad H = 7$ m

$b = 18$ cm $\quad H = 6$ m

$$\lambda_x = \frac{2{,}2H}{0{,}289h} \le 120$$

$b = 16$ cm $\quad H = 5$ m

$$h_{\min} = \frac{2{,}2H\,(\text{cm})}{0{,}289 \cdot 120}$$

$b = 13{,}5$ cm $\quad H = 4$ m

$$\left(h_{\min} = \frac{1}{15{,}76} \approx \frac{1}{16} H \right)$$

round up

B, m	$1/k$
6	1/14
5	1/15
4	1/16

$$h^* = \frac{1}{k} \cdot H \text{ round up}$$

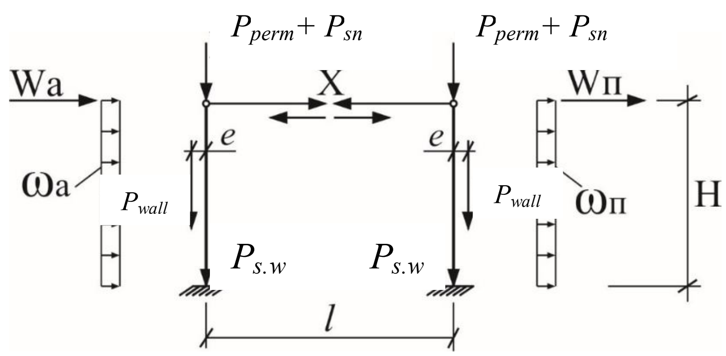
$$h^* = \frac{1}{k} \cdot H \qquad h = 3{,}3 \cdot n \ge h^*.$$

Thus:

$$h^* = \frac{1}{16} \cdot H = \frac{1}{16} \cdot 400 \text{ cm} = 25 \text{ cm} \qquad h = 3{,}3 \text{ cm} \cdot 8 = 26{,}4 \text{ cm}.$$

3.2 Static design

3.2.1 Design scheme



where P_{wall} – wall load;

P_{perm} – permanent load;

P_{sn} – snow load;

$P_{s.w}$ – dead weight of column;

w – wind load.

3.2.2 Loads acting to the column:

– permanent load from coating

$$P_{perm} = (g_{roof} + g_{s.w.beam})B \cdot \frac{l}{2} = (0{,}64\ kH/m^2) \cdot 4\ m \cdot \frac{15\ m}{2} = 19{,}2\ kH;$$

– snow load

$$P_{sn} = S_k \cdot B \cdot \frac{l}{2} = 1{,}67\ kH/m^2 \cdot 4\ m \cdot \frac{15\ m}{2} = 50{,}1\ kH;$$

– wall load by taking into account wall beams is conditionally accept as:

$$g_{wall} = 0{,}8\ kN/m^2,$$

$$P_{wall} = g_{sn} \cdot B \cdot (H + h_s) = 0{,}8\ kN/m^2 \cdot 4\ m \cdot (4\ m + 0{,}85\ m) = 15{,}5\ kN,$$

$$h_s = h_{\sup} + h_{s.beam.} + h_{purlin} = 0{,}15\ m \cdot 0{,}55\ m + 0{,}15\ m = 0{,}85\ kN;$$

– dead weight of column:

$$P_{s.w} = b \cdot h \cdot H \cdot \rho_{mean,Gl22h} = 0{,}135\ m \cdot 0{,}264\ m \cdot 4\ m \cdot 4{,}1\ kN/m^2 = 0{,}59\ kN;$$

– wind load:

$$w_0 = 0{,}42\ \text{kN/m}^2;$$

$$w_a = 0{,}56\ \text{kN/m}^2 \cdot B \cdot w_0 = 0{,}56\ \text{kN/m}^2 \cdot 4\ \text{m} \cdot 0{,}42\ \text{kN/m}^2 = 0{,}94\ \text{kN/m};$$

$$w_a = 0{,}42\ \text{kN/m}^2 \cdot B \cdot w_0 = 0{,}42\ \text{kN/m}^2 \cdot 4\ \text{m} \cdot 0{,}42\ \text{kN/m}^2 = 0{,}71\ \text{kN/m};$$

$$W_a = w_a \cdot h_1 = 0{,}94\ \text{kN/m} \cdot 1{,}52\ \text{m} = 1{,}43\ \text{kN};$$

$$W_n = w_n \cdot h_1 = 0{,}71\ \text{kN/m} \cdot 1{,}52\ \text{m} = 1{,}08\ \text{kN};$$

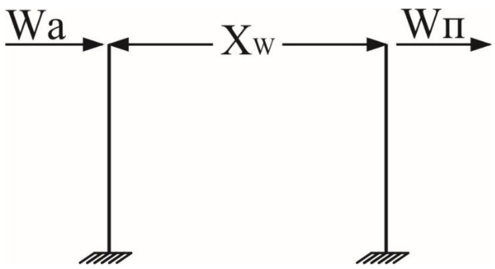
$$h_1 = h_{\sup} + h_{s.beam} + h_{purlin} = 0{,}15\ m + 1{,}1\ m + 0{,}27\ m = 1{,}52\ m;$$

$$h_{roof.} = h_{sheating.} + h_{insul.} + h_{purlin.} + h_{coat.} = 0{,}019 + 0{,}1\ m + 1{,}15\ m + 0{,}001\ m = 0{,}27\ m.$$

Forces in the columns of frame as one time static indeterminate system are determined for each load separately taking into account rigidity of the cross-bar $EI = \infty$.

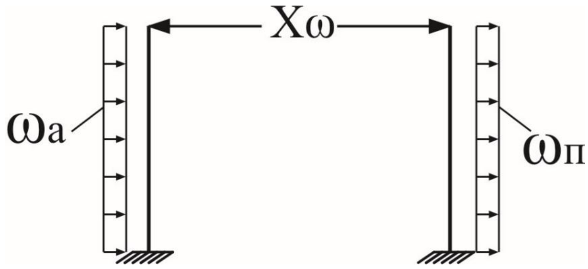
Determine the X:

– from wind load acting to the top of column:



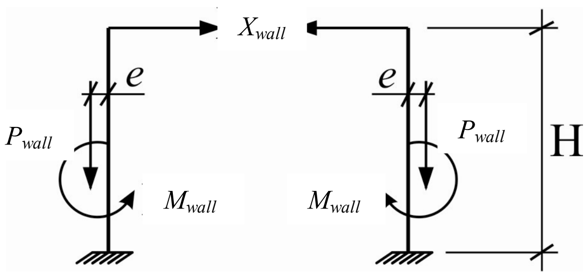
$$X_W = \frac{W_a - W_n}{2} = \frac{1{,}43\ \text{kN} - 1{,}08\ \text{kN}}{2} = 0{,}175\ \text{kN};$$

– from wind load acting to the walls:



$$X_W = \frac{3}{16}\left(W_a - W_n\right)\cdot H = \frac{3}{16}\left(0{,}94\ \text{kN/m} - 1{,}08\ \text{kN/m}\right)\cdot 4\ m = 0{,}17\ \text{kN};$$

– from walls:



$$M^{wall} = P^{wall}\cdot e \qquad e \approx 0{,}2,$$

$$X_{wall} = \frac{9M_{wall}}{8H} = \frac{9\cdot 15{,}5\ kNm}{8\cdot 4\ m} = 4{,}36\ kN,$$

$$M_{wall} = 4{,}36\ kN\cdot 0{,}2\ m = 8{,}72\ kNm.$$

Boundary design forces in the bottom of cross section of the column.

$$M_{left} = \left[\left(W_a - X_W - W_n \right) \cdot H + \frac{w_a H^2}{2} \right] \cdot \psi - M^{CT} + X_{CT} \cdot H =$$

$$= \left[\left(1{,}43 \text{ kN} - 0{,}175 \text{ kN} - 0{,}17 \text{ kN} \right) \cdot 4\, m + \frac{0{,}94 \text{ kN/m} \cdot 4 \text{ m}^2}{2} \right] \cdot 0{,}9 - 8{,}72 \text{ kNm} +$$

$$+ 4{,}36 \text{ kN} \cdot 4 \text{ m} = 19{,}61 \text{ kNm};$$

$$M_{right} = \left[\left(W_n - X_W + X_w \right) \cdot H + \frac{w_n H^2}{2} \right] \cdot \psi + M^{CT} - X_{CT} \cdot H =$$

$$= \left[\left(1{,}08 \text{ kN} - 0{,}175 \text{ kN} + 0{,}17 \text{ kN} \right) \cdot 4\, m + \frac{0{,}71 \text{ kN/m} \cdot 4 \text{ m}^2}{2} \right] \cdot 0{,}9 + 8{,}72 \text{ kNm} -$$

$$- 4{,}36 \text{ kN} \cdot 4 \text{ m} = 1{,}52 \text{ kNm};$$

$\Psi = 0{,}9$

$N_{ref} = P_{perm} + P_{wall} + P_{s.w} + P_{s.n} \cdot \psi = 19{,}2 + 15{,}5 + 0{,}59 + 50{,}1 \cdot 0{,}9 = 80{,}4\ kN$;

$N_{left} = N_{right} = 80{,}4\ kN;$

$M_{design} = M_{\max} = 19{,}61\ kNm,$

$Q_{right} = W_n + X_W + X_w + w_n \cdot H + X_{cm} = 1{,}08 + 0{,}175 + 0{,}17 + 0{,}71 \cdot 4 + 4{,}36 = 8{,}63\ kN$;

$Q_{left} = W_a - X_W - X_w + w_a \cdot H + X_{cm} = 1{,}43 - 0{,}175 - 0{,}17 + 0{,}94 \cdot 4 + 4{,}36 = 9{,}21\ kN$;

$Q_{design} = Q_{\max} = 9{,}21\ kN.$

3.3 Checking calculations of the column

3.3.1 Design strengths of the beam

The column is made of glulam GL22h, service class 1. Load duration class is "short term" load.

$$f_{m,g,d} = k_{\text{mod}} \cdot \frac{f_{m,g,k}}{\gamma_M} = 0{,}9 \cdot \frac{22}{1{,}25} = 15{,}2 \text{ N/mm}^2,$$

$$f_{v,g,d} = k_{\text{mod}} \cdot \frac{f_{v,g,k}}{\gamma_M} = 0{,}9 \cdot \frac{3{,}5}{1{,}25} = 2{,}52 \text{ N/mm}^2,$$

$$f_{c,0,g,d} = k_{\text{mod}} \cdot \frac{f_{c,0,g,k}}{\gamma_M} = 0{,}9 \cdot \frac{22}{1{,}25} = 15{,}2 \text{ N/mm}^2,$$

$$f_{c,90,g,d} = k_{\text{mod}} \cdot \frac{f_{c,90,g,k}}{\gamma_M} = 0{,}9 \cdot \frac{2{,}5}{1{,}25} = 1{,}8 \text{ N/mm}^2.$$

Elastic modulus:

$E_{0,05}$ = 8800 N/mm^2.

3.3.2 Geometry of column section

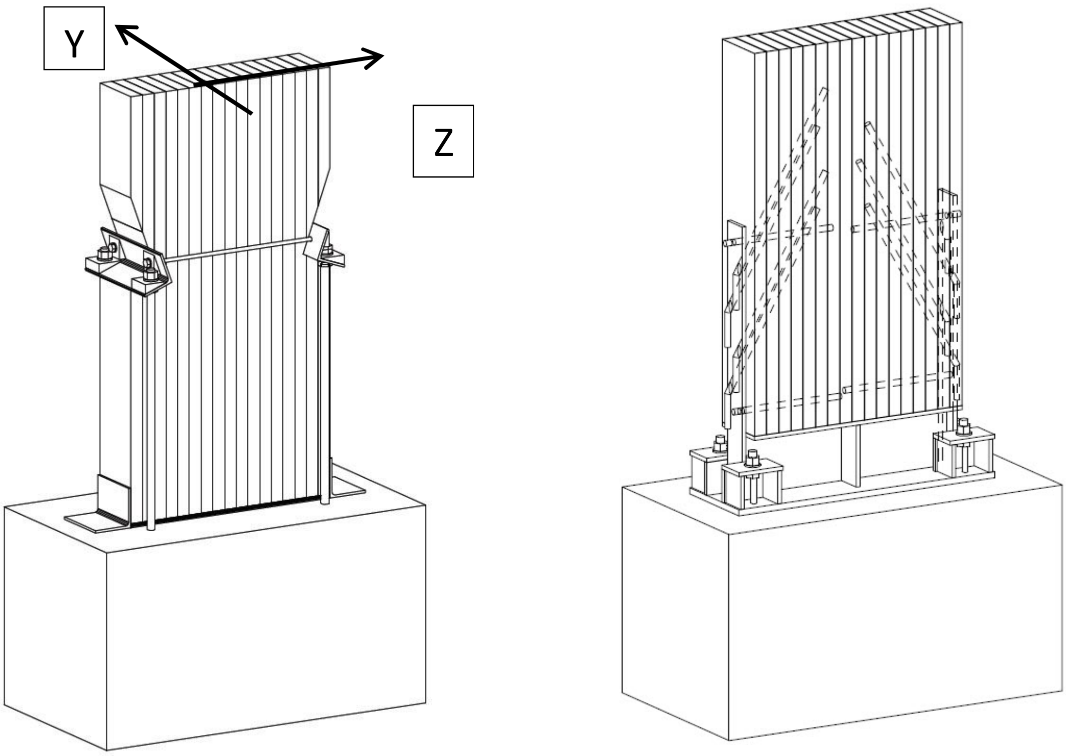


Figure 18 – Variants of rigid joint of the column with the foundation

Radius of inertia of the column:

$$i_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{\frac{b \cdot h^3}{12}}{b \cdot h}} = \sqrt{\frac{h^2}{12}} = \frac{h}{\sqrt{12}} = 0{,}289 \cdot h = 0{,}289 \cdot 264 = 76{,}3 \text{ mm},$$

$$W_y = \frac{bh^2}{6} = \frac{135 \cdot 264^2}{6} = 1{,}57 \cdot 10^6 \text{mm}^3.$$

The coefficient $k_{c,y}$ is determined in accordance with the following slenderness ratio λ_y (steps: $l_{ef,y} \rightarrow \lambda_y \rightarrow \lambda_{rel,y} \rightarrow k_y \rightarrow k_{c,y}$):

$$l_{ef,y} = l_{ef,z} = \beta \cdot l = 2{,}2 \cdot 4000 = 8800 \text{ mm.}$$

Slenderness ratio:

$$\lambda_y = \frac{l_{ef}}{i_y} = \frac{8800}{76{,}3} = 115{,}3 .$$

Relative slenderness ratio:

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c,0,k}}{E_{0,05}}} = \frac{115{,}3}{3{,}14} \cdot \sqrt{\frac{22}{8800}} = 1{,}84$$

$$k_y = 0{,}5\left[1 + \beta_c \cdot \left(\lambda_{rel,y} - 0{,}3\right) + \lambda_{rel,y}^2\right] = 0{,}5\left[1 + 0{,}1 \cdot \left(1{,}84 - 0{,}3\right) + 1{,}84^2\right] = 2{,}302$$

$$k_{c,y} = \min\left\{\frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}}; 1\right\} = \left\{\frac{1}{2{,}302 + \sqrt{2{,}302^2 - 1{,}84^2}}; 1\right\} = 0{,}272.$$

Bending and compressive stresses:

$$\sigma_{m,y,d} = \frac{M_{y,d}}{W_y} = \frac{19{,}61 \cdot 10^6 \text{ Nmm}}{1{,}57 \cdot 10^6 \text{ mm}^3} = 12{,}5 \text{ N/mm}^2,$$

$$\sigma_{c,0,d} = \frac{\text{N}}{\text{A}} = \frac{80{,}41 \cdot 10^3 \text{ N}}{135 \text{ mm} \cdot 264 \text{ mm}} = 2{,}26 \text{ N/mm}^2.$$

The height of beam cross section is not higher then 600 mm, so it is need to raise the bending strength.

$$(h = 264 \text{ mm}) < 600 \text{ mm}$$

$$k_{h,y} = \min\left[\left(\frac{600}{h}\right)^{0,1}; 1{,}1\right] = \min\left[\left(\frac{600}{264}\right)^{0,1}; 1{,}1\right] = 1{,}09,$$

$$f_{m,y,d} \cdot k_{h,y} = 1{,}09 \cdot 15{,}2 = 16{,}6 \text{ N/mm}^2.$$

Check the strength:

$$\frac{\sigma_{c,0,d}}{k_{c,y} \cdot f_{c,0,d}} + k_m \cdot \frac{\sigma_{m,y,d}}{f_{m,y,d}} = \frac{2{,}26}{0{,}272 \cdot 15{,}2} + 0{,}7 \cdot \frac{12{,}5}{16{,}6} = 0{,}547 + 0{,}527 = 1{,}074 > 1$$

it is not enough.

It is need to increase the height of cross section of the column and check the strength and stability as shown above once more.

3.4 Algorithm of the design of joint of the column and foundation with glued-in rods

Methods of the design in accordance with СП 64.13330.2011

$$M_d = \frac{M}{\xi};$$

$$\xi = 1 - \frac{N}{\varphi \cdot R_c \cdot F_{бр}},$$

φ – determine by the equation 7 and 8 СП 64.13330.2011, depending on λ;

M_d, N, Q – forces.

Section dimensions $b \cdot h_k$

$$N_0 = N_{tum} + N_n,$$

N_{tum} – short term load to the column (snow);

N_n – permanent load to the column;

N_0 – total load.

Determine the edge stresses in the cross section of the column:

$$\sigma_p = \frac{M}{W_{бр} \cdot \xi_p} - \frac{N_n}{F_{бр}} \le R_p \text{ (table 3 СП 64.13330.2011)}$$

$$\sigma_{cm} = \frac{M}{W_{бр} \cdot \xi_c} - \frac{N_0}{F_{бр}} \le R_c \text{ (table 3 СП 64.13330.2011)}$$

$$\xi_p = 1 - \frac{N_n}{\varphi \cdot R_c \cdot F_{бр}}; \qquad \xi_c = 1 - \frac{N_0}{\varphi \cdot R_c \cdot F_{бр}}.$$

Dimension of the compression zone:

$$k = \frac{\sigma_c \cdot h_k}{\sigma_c + \sigma_p}.$$

Maximum tensile force in the anchor:

$$N_p = \frac{M}{(S+c)\cdot \xi_p} - \frac{N_p \cdot c}{S+c}.$$

According to the N_p select two anchor plates.

Determine N_p^{plast} by the formula shown above, but with your values *S* and *c*. Determine the cross section of anchor plates.

Accept angle of slope α =30°.

Force in the rods:

$$N_{cm} = \frac{N_p}{\cos\alpha}.$$

Further accept the rod diameter and the depth of glued-in (P. 7.39–7.49 СП 64.13330.2011)

$$k_c = 1{,}2 - 0{,}02\frac{l_p}{d}.$$

Load bearing capacity of the rod:

$$T = R_{ck} \cdot \pi \cdot d_1 \cdot l_p \cdot k_c \cdot m_d \le F_a \cdot R_a;$$

R_{ck} – shear resistance (formula 3 СП 64.13330.2011, in accordance with P. 5.2.);

d_1 – hole diameter;

$l_p = l - l_0$; $l_0 = 3d -$ when welding the rods

$* k_\sigma = 1 - 0{,}01\sigma$ (MPa) – withdrawal capacity of rods in tensile;

σ – maximum tensile stress.

$* m_d = 1{,}12 - 10d$ (m)

P. 7.45

k_{cp} – (p.7.45 СП 64.13330.2011) coefficient, which is taking into account non-uniform inclusion the rods.

RECOMMENDED LITERATURE

1. Structural Timber Design to Eurocode 5, 2nd Edition. Jack Porteous, Abdy Kermani. ISBN: 978-0-470-67500-7. 638 pages. June 2013, Wiley-Blackwell.

2. J. Porteous, P. Ross. Designers' Guide to Eurocode 5: Design of Timber Buildings. ICE Publishing, 176 pages, 2013.

3. Borgström E. Design of timber structures. Structural aspects of timber construction. Volume 1. EDITION 2:2016, Swedish Forest Industries Federation, 316 p. 2016.

ANNEX A

Table A.1 – Value of the coefficient k_{mod} depending on temperature and moisture (service class) and load term duration

Material	Service class	Load term duration				
		Permanent (> 10 years)	Long term (6months-10years)	Medium term (1week-6months)	Short term (< 1 week)	Instantaneous
Solid timber, GLT, LVL, plywood	1 2 3	0,6 0,6 0,5	0,7 0,7 0,55	0,8 0,8 0,65	0,9 0,9 0,7	1,1 1,1 0,9
OSB	1 (OSB/2)	0,3	0,45	0,65	0,85	1,1
	1 (OSB/3) (OSB/4)	0,4	0,5	0,7	0,9	1,1
	2 (OSB/3) (OSB/4)	0,3	0,4	0,55	0,7	0,9

Table A.2 – Service classes

Service class	Notation		
	Moisture of the structure elements	Service temperature	Relative air humidity
1 (closed premises; no direct effect of climatic factors)	≤ 12%	+20°C	≤ 65%
2 (partially premises; no direct effect of climatic factors)	≤ 20%	+20°C	≤ 85%
3 (direct precipitation)	> 20%	>20°C	> 85%

Table A.3 – Coefficient γ_M

	γ_M
Solid timber	1,3
GLT	1,25
LVL, plywood, OSB	1,2

ANNEX B

Table B.1 – Characteristic values of the strength of the solid softwood (N/mm^2) according to the EN 338:2009

		Strength classes								
		C14	C16	C18	C20	C22	C24	C27	C30	C35
(N/mm^2)										
Bending	$f_{m.g,k}$	14	16	18	20	22	24	27	30	35
Tension perpendicular	$f_{t,90,g,.k}$	0,4								
Compression parallel	$f_{c,0,g,.k}$	16	17	18	19	20	21	22	23	25
Compression perpendicular	$f_{c,90,g.k}$	2,0	2,2		2,3	2,4	2,5	2,6	2,7	2,8
Shear	$f_{v,g,.k}$	3,0	3,2	3,4	3,6	3,8	4,0			
(kN/mm^2)										
Mean modulus of elasticity parallel	$E_{0,mean}$	7	8	9	9,5	10	11	11,5	12	13
Density (kg/m^3)										
Density	$\rho_{g,mean}$	350	370	380	390	410	420	450	450	460

Table B.2 – Characteristic strength and stiffness properties of GLT (N/mm^2) according to the EN 14080:2013

		Strength classes							
		Gl24h	**Gl24c**	**Gl28h**	**Gl28c**	**Gl30h**	**Gl30c**	**Gl32h**	**Gl32c**
(N/mm^2)									
Bending	$f_{m.g,k}$	24	24	28	28	30	30	32	32
Tension perpendicular	$f_{t,90,g,.k}$	0,5							
Compression parallel	$f_{c,0,g,.k}$	24	21,5	28	24	30	24,5	30	24,5
Compression perpendicular	$f_{c,90,g.k}$	2,5							
Shear	$f_{v,g,.k}$	3,5							
(kN/mm^2)									
Mean modulus of elasticity parallel	$E_{0,mean}$	11,5	11,0	12,6	12,5	13,6	10,8	14,2	13,5
Shear modulus	$G_{0,5}$	0,54							
Density (kg/m^3)									
Density	$\rho_{g,mean}$	420	400	460	420	480	430	490	440

ANNEX C

Table C.1 – Deformability coefficient k_{def}

Material	Service class		
	1	2	3
Solid timber, GLT, LVL	0,6	0,8	2,0
OSB (OSB/2) (OSB/3, OSB/4)	 2,25 1,5	 – 2,25	 – –

Table C.2 – Boundary values of spans

	w_{inst}	$w_{net,fin}$	w_{fin}
Beam on two supports	l/300–l/500	l/250–l/350	l/150–l/300
Console	l/150–l/250	l/125–l/175	l/75–l/150

Виробничо-практичне видання

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(*для студентів 3 курсу денної та заочної форм навчання,
спеціальності 192 – Будівництво та цивільна інженерія,
професійне спрямування «Промислове і цивільне будівництво»)*

(Англ. мовою)

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Відповідальний за випуск *А. В. Набока*

За авторською редакцією

Комп’ютерне верстання *О. М. Пустовойтова*

План 2019, поз. 11М

Підп. до друку 13.03.2019. Формат 60 × 84/16
Друк на ризографі. Ум. друк. арк. 1,0
Тираж 50 пр. Зам. №

Видавець і виготовлювач:
Харківський національний університет
міського господарства імені О. М. Бекетова,
вул. Маршала Бажанова, 17, Харків, 61002.
Електронна адреса: rectorat@kname.edu.ua
Свідоцтво суб’єкта видавничої справи:
ДК № 5328 від 11.04.2017.