S. UZHEHOV, O. UZHEHOVA, S. ROTKO,
I. ZADOROZHNIKOVA, D. KYSLIUK, O. CHAPIUK

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Reviewers:
I. Voitiuk, first deputy general director of PJSC "Lutsksantekhmontazh No. 536";
V. Bilozir, candidate of technical sciences, associate professor of the Department of Building Structures of the Lviv National University of Nature Management;
V. Shvabiuk, doctor of technical sciences, professor of the Lutsk National Technical University.

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The study guide is compiled in accordance with the program of the compulsory discipline "Building structures". The manual provides data on materials used for building structures, their strength and deformation characteristics. The main theoretical provisions for the calculation of elements of building structures under various types of loads, the basics of designing these structures in accordance with current standards are outlined.

The manual is suitable for both classroom and independent study of the material by students of higher education, it contains the necessary reference materials for calculating elements of building structures, it is also recommended for use in course and diploma design.

The study guide is intended for students of higher education in the specialty 192 Construction and civil engineering, and can also be useful for engineering and technical workers of project and construction organizations.

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PREFACE

The study guide is designed for the study of the mandatory discipline "Building structures" for higher education degree holders of the bachelor's degree of the educational program "Construction and civil engineering" in the field of knowledge 19 Architecture and construction, specialty 192 Construction and civil engineering of full-time and part-time forms of education.

The material presented in the manual is based on current normative documents: DBN V.1.2-2:2006. Loads and influences. Design norms; DBN V.2.6-161:2017. Wooden structures. Substantive provisions; DBN V.1.2-14:2018. General principles of ensuring the reliability and structural safety of buildings and structures; DBN V.2.6-198:2014. Steel structures. Design standards. The manual provides basic information about materials for building structures, topics of calculation of stretched, compressed, bent structures made of wood and metal are disclosed.

Each chapter contains a statement of the problem, the order of calculations and brief explanations for the implementation of practical problems from the course.

The appendices contain reference material for calculating and designing building elements from wood and metal.

Section 1
Loads and impacts

1.1. Basic principles of calculation

Building structures and bases are calculated according to their bearing capacity (limit states of the first group) and suitability for normal operation (limit states of the second group).

The first group contains limit states, the transition through which leads to the complete unsuitability of the object or its element for operation and for which non-limit states can be [4, p. 6.2.2]:

- destruction of any nature (viscous, brittle, as a result of fatigue);
- loss of shape stability;
- loss of position stability;
- transition to a variable system;
- qualitative change of configuration;
- other phenomena that require the termination of operation (for example, perforation of the wall of a container with toxic substances or excessive movement of the base during subsidence or heaving of the soil).

The second group contains limit states that complicate the normal operation of the object or reduce its durability compared to the estimated service life and for which the non-limit states are [4, p. 6.2.3]:

- excessive movements or turns of some points of the structure;
- unacceptable fluctuations (excessive values of amplitude, frequency, speed, acceleration);

- the formation and opening of cracks, their achievement of maximum permissible values of opening or length;
- loss of shape stability in the form of local deformation or loss of cross-sectional shape stability;
- damage from corrosion or other types of physical deterioration, which lead to the need to limit operation due to a reduction in the object's service life;
- unacceptable heat losses due to enclosing structures, which leads to an increase in material costs for the operation of the building;
- failure of the building elements to achieve the required level of noise insulation.

The limit states of this group may be associated with the violation of the requirements for the use of the object without restrictions, the violation of the requirements for the level of comfort, the amenities of the staff, and the requirements for the appearance of the structures.

Reaching the limit state of the second group is classified as a failure-obstacle.

Limit states and calculation situations, according to which calculations should be performed, are given in the construction regulations.

The calculation of the limit states should ensure the reliability of the building or structure during the entire period of operation, as well as during the performance of works.

The conditions for ensuring reliability are that the calculated values of loads or forces caused by them, stresses, deformations, displacements, opening of cracks do not exceed the corresponding limit values established by the design standards of structures or foundations.

The calculation of structures and bases according to the limit states of the first and second groups must be performed taking into account the most unfavorable combinations of forces.

All building structures must meet the following requirements [4, p. 4.1.6]:

- perceive without destruction and unacceptable deformations the effects that occur during their erection and during the established period of operation;

- have sufficient performance under normal operating conditions during the entire estimated period of operation, namely: their operational parameters (movement, vibrations) with a given probability should not go beyond the limits set by the regulatory or design documentation, and their durability should be such that the deterioration of the material properties and structures due to rotting, corrosion, abrasion and other forms of physical wear and tear did not lead to an unacceptably high probability of failure;

- to have sufficient survivability in relation to local destruction and emergency impacts provided for by the standards (fire, explosions, vehicle collisions), while excluding the phenomenon of disproportionate destruction, when the total damage turns out to be significantly greater than the initial disturbance that caused them.

1.2. Classification of loads

When designing, it is necessary to consider the loads that occur during the construction and operation of structures, as well as during the manufacture, storage, and transportation of building structures in accordance with DBN B.1.2-2:2006 [1].

The accepted classification of loads provides the possibility of calculating building structures considering the necessary calculation situations and limit states, namely:

a) verification of strength, stability, and other load-bearing capacity criteria under one-time loading in extreme operating conditions (emergency design situation or stable or transient design situation that can be implemented a limited number of times during the service life);

b) checking of stiffness and crack resistance in the mode of normal operation (stable calculation situation);

c) verification of endurance under repeated loads (stable calculation situation);

d) considering the creep of materials and other rheological processes under the action of constant and long-term loads (stable design situation).

According to the norms [1], depending on the causes of the load and impacts, they are divided into main (which appear because of natural phenomena or human activity) and episodic (which are realized extremely rarely, once or several times during the service life of the structure, and duration of action which are limited in time to a short period. As a rule, emergency loads and impacts are episodic).

Depending on the variability over time, the impacts are divided into permanent (which act practically without changing during the service life of the structure) and variable (for which the change in its value over time relative to the average cannot be neglected).

Depending on the duration of continuous action, variable loads and impacts are divided into long-term, short-term, and episodic.

Loads that occur during the manufacture, storage, and transportation of structures, as well as during the construction of structures, should be considered in the calculations as short-term.

The basis for assigning loads is their characteristic values.

When calculating load-bearing structures and foundations, the reliability coefficient for responsibility (responsibility coefficient) should be taken into account γ_n [4, p. 7.6.4], which is determined depending on the class of consequences (responsibility) of the object and the type of calculation situation according to table 1.1 .

Table 1.1

The value of the reliability coefficient by responsibility γ_n [4, tab. 5]

Class of consequences (responsibility)	Design responsibility category	Values γ_n used in calculation situations				
		established		transitional		emergency
		the first group of limit states	friend group of limit states	the first group of limit states	friend group of limit states	the first group of limit states
CC3	A	1.250	1.00	1.050	0.975	1.050
	B	1.200		1.000		
	C	1.150		0.950		
CC2	A	1.100	0.975	0.975	0.950	0.975
	B	1.050		0.950		
	C	1.000		0.925		
CC1	A	1.000	0.950	0.950	0.925	0.950
	B	0.975		0.925		
	C	0.950		0.900		

Note 1. If the design regulations for certain types of buildings or structures do not provide specific recommendations on the distribution of structures by categories of responsibility according to classes of consequences (responsibility), they should be assigned to category B.

Note 2. For objects of new construction, constructed in the protection zone of monuments of cultural heritage of national and local importance, which according to all the characteristics of the possible consequences of their failure belong to the class of consequences (responsibility) CC1, the reliability coefficient provided for higher classes consequences, does not apply.

This coefficient is used as a multiplier to the action effect (deflection, effort, stress), except for those cases when such an effect is unloading. In calculations where the calculated value of the load is not used (for example, when evaluating test data), the coefficient of responsibility is taken as one.

The calculated load values are determined by multiplying the characteristic values by the load reliability coefficient γ_{fm} , which depends on the type of load.

Depending on the nature of the loads and the purpose of the calculation, four types of calculated values are used:

- marginal – the load value corresponding to an extreme situation that can occur no more than once during the life of the structure, and is used to check the limit states of the first group, exceeding the limits of which is equivalent to a complete loss of the structure's operability;
- e operational - the value of the load characterizing the conditions of normal operation of the structure. As a rule, the operational design value is used to check the limit states of the second group, associated with the difficulties of normal operation (occurrence of unacceptable movements of the structure, unacceptable vibration and unacceptably large opening of cracks in reinforced concrete structures, etc.);
- cyclic - the value of the load, which is used for calculations of structures for endurance and is defined as a harmonic process, equivalent in terms of the resulting effect on the structure to a real random process of variable load;
- quasi -permanent - calculated value of the load, which is used to take into account rheological processes occurring under the influence of variable loads, and is defined as the level of such a

constant influence, which is equivalent in terms of the resulting action to the actual random loading process.

To check the limit states of the first group, limit calculated values of loads are used.

To check the limit states of the second group of loads, they are set depending on the operating conditions of the structure under consideration, namely:

- if exiting the limit state can be allowed on average once per $\mathbf{T_n}$ years, then the check is performed using the limit calculation value corresponding to the period $\mathbf{T_n}$;
- if the departure from the limit state of the second group can be allowed during a certain fraction η (0< η <1) of the established service life of the structure T_{ef}, then the check is performed using the operational design value corresponding to this portion of the established service life (η $\mathbf{T_{ef}}$).

The transition to calculated values is carried out by multiplying by the load reliability coefficient γ_f **($\mathbf{T_n}$)** or γ_f **(η $\mathbf{T_{ef}}$)**. Value η are accepted according to the norms of design of structures depending on their purpose, responsibility and consequences of going beyond the limit state.

Permanent loads should include:

a) the weight of parts of buildings, including the weight of load-bearing and enclosing structures;

b) weight and pressure of soils (embankments, fills).

Variable long-term loads should include:

a) weight of temporary partitions, foundations and sub-concrete for equipment;

b) the weight of stationary equipment: machines, devices, motors, tanks, pipelines with fittings, supporting parts and insulation, belt conveyors, permanent lifting machines with their ropes and guides, as well as the weight of liquid and solid substances filling the equipment;

c) pressure of gases, liquids and loose bodies in tanks and pipelines, excess pressure and rarefaction of air that occurs during ventilation of mines;

d) load on the ceiling from stored materials and racking equipment in warehouses, refrigerators, granaries, book stores, archives and similar premises;

e) load from people, livestock, equipment on the ceiling of residential, public and agricultural buildings with quasi-constant calculated values;

e) vertical loads from bridge and overhead cranes with quasi-constant calculated values;

h) snow loads with quasi-constant calculated values; temperature climatic influences with quasi-constant calculated values;

k) impacts caused by deformations of the base, which are not accompanied by a radical change in the soil structure;

l) effects caused by changes in humidity, components of an aggressive environment, shrinkage and creep of materials.

Variable short-term loads should include:

a) the weight of people, repair materials in the areas of maintenance and repair of equipment with marginal or operational calculated values;

b) snow, wind, ice loads with limit or operational calculated values;

c) temperature climatic influences with marginal or operational calculated values.

Episodic loads include:

a) seismic and explosive effects;

b) loads caused by sudden disruptions of the technological process, temporary malfunction or destruction of equipment;

c) impacts caused by deformations of the base, which are accompanied by a fundamental change in the structure of the soil (when subsiding soils are soaked) or its subsidence in areas of mining and karst areas.

Characteristic and calculated values of episodic loads are determined by special regulatory documents.

1.3. Combination of loads

Combinations of loads are formed as a set of their design values or their corresponding forces and/or displacements, which is used to check the structure or base in a certain limit state and in a certain design situation. It is assumed that all loads in the selected combination simultaneously affect the calculation object.

The connection should include loads that most adversely affect the structures (foundations) from the point of view of the considered limit state. Influences that are mutually exclusive cannot be part of the same combination.

Two types of connections can be used in structural calculations - basic and emergency.

To check the limit states of the first group, basic connections are used, which contain constant loads with limit design values, limit design, cyclic or quasi-permanent values of variable loads.

To check the limit states of the second group, basic connections are used, which contain constant loads with operational design values, as well as operational design, cyclic or quasi-constant values of variable loads.

The emergency connection, in addition to constant and variable loads, can include only one episodic impact.

low probability of simultaneous implementation of the calculated values of several loads is taken into account by multiplying the calculated values of the loads included in the combination by the combination factor $\psi \leq 1$.

For basic connections including constant and at least two variable loads, the latter are accepted with a coefficient of connections ψ_1 **=0.95** for long-term loads and ψ_2 **=0.90** for short-term loads.

For emergency connections containing constant and at least two variable loads, the latter are accepted with a coupling factor of ψ_1 **=0.95** for long-term loads and ψ_2 **=0.80** for short-term loads. The emergency load is accepted with a coupling factor ψ_1 **=1.00**.

In basic combinations, taking into account three or more short-term loads, their calculated values are allowed to be multiplied by the combination factor ψ_2, which is accepted for the first (by degree of influence) short-term load – **1.0** , for the second – **0.8** , for the rest – **0.6**.

When determining the calculated combinations of loads for structures and foundations during the construction of buildings and structures, snow, wind, ice loads, as well as temperature and climatic influences, which were included in the calculated combinations, should be reduced by 20%.

1.4. Weight of structures and soils

The characteristic value of the weight of factory-made structures is determined according to standards, working drawings or passport data of manufacturing plants, and other building structures and soils - according to the design dimensions and specific weight of materials and soils, taking into account their humidity in the conditions of construction and operation of structures.

The own weight of 1 m^3 of stone masonry from solid large blocks made of concrete or natural stone is taken as equal to the density of these materials. The real average density of concretes, taking into account their humidity in operating conditions and natural stones from various mining rocks, is given in the table 1.2.

The average density of vibrating brick panels is taken as 1850 kg/m^3. The average density of walls and pillars made by hand masonry, taking into account the actually formed hollowness of the seams, is given in the table 1.3.

According to the data given in tables 1.2 and 1.3, the characteristic loads from the own weight of stone structures are determined.

The limit design load from self-weight is determined by multiplying the characteristic value by the reliability factor for the limit load $\gamma_{fm} = 1{,}1$ to take into account the probability of an increase in material density or wall thickening, within the thickness tolerance.

Table 1.2

Density of concrete and natural stones

Name of the material	Density, kg/m^3
Heavy rubble concrete from erupted rocks	2400
The same, on limestone rubble	2300
Clay concrete	900 - 1800
Slag concrete	1400 - 1600
Structural lightweight concrete	700 - 1400
Dolomite	2200 - 2800
Limestone is dense, strong	2000 - 2600
Marble	2500 - 2800
Sandstone	2100 - 2800
Granite, diorite, gabbro	2500 - 3200
Basalt	2700 - 3300
Diabase	3000
Volcanic tuffs	900 - 1500
Porous limestones with compressive strength limit, MPa:	
0.4 - 3.5	900 - 1600
3.5 - 15.0	1500 - 2000

Table 1.3

The density of manual masonry of various types

Type of masonry	Stone density, kg/m^3	Masonry density, kg/m^3
From a natural stone of the correct shape on a heavy solution	2800	2680
The same	2000	1960
The same	1200	1260
Rubble	2800	2420
Rubble is made of limestone	2200 - 2500	2100
Solid brick on heavy mortar	1700 - 2000	1800
Solid brick on light mortar	1700 - 2000	1700
Made of hollow ceramics, hollow, porous-hole brick	1450	1500
The same	1300	1400
From solid slag concrete stones	1400 - 1600	1600
From slag-concrete stones with slotted cavities (stone voids 26%)	1040 - 1180	1250
From three-hollow slag concrete stones with through-hole voids and filling the voids with slag (voidness of stones 35%, slag density 1000 kg/m^3)	910 - 1040	1400

If the most unfavorable working conditions of the masonry will be for the smallest value of the longitudinal force, for example, when calculating overturning, then the calculated load from its own weight is determined by multiplying the characteristic loads by the load reliability factor $\gamma_{fm} = 0{,}9$

The density of soils is given in table 1.4.

Table 1.4

Soil density

Type of soil	Density, kg/m^3
Sand:	
- coarse-grained dry	1500
- fine-grained dry	1600
- coarse-grained moist	1800
- fine-grained, saturated with moisture	2000
Pebble:	
- angular	1800
- rounded	1900
Rubble	1600
Bulk soil:	
- loose dry	1400
- loose wet	1600
- loosened, saturated with moisture	1800
- compacted dry	1700
- compacted wet	1900
Loam:	
- loose dry	1500
- loose wet	1600
- loosened, saturated with moisture	2000
- compacted dry	1800
- compacted wet	1900
Clay:	
- loose dry	1600
- loose wet	2000
- dense moisture	2500

The reliability coefficient for the ultimate load γ_{fm} for soils is taken as follows:

- in natural occurrence - 1.1;
- bulk - 1.15.

When determining the loads from the soil, it is necessary to take into account the loads from stored materials, equipment and vehicles transferred to the soil.

1.5. Evenly distributed temporary loads

Characteristic and quasi-permanent values of uniformly distributed temporary loads on floor slabs, steps and floors on soils are given in table 1.5.

Table 1.5

Value of variable (temporary) evenly distributed loads on the floor, kPa [1, table 6.2, p. 13]

Buildings and premises	Characteristic values of loads, kPa (kgf/m^2)	Quasi-constant load values, kPa (kgf/m^2)
1. Apartments in residential buildings; bedrooms of children's preschool institutions and boarding schools; residential premises of holiday homes and boarding houses, hostels and hotels; wards of hospitals and sanatoriums; terraces	1.5 (150)	0.35 (35)
2. Service premises of administrative, engineering and technical, scientific personnel of organizations and institutions; classrooms of educational institutions; domestic premises (wardrobe rooms, showers, washrooms, toilets) of industrial enterprises and public buildings and structures	2.0 (200)	0.85 (85)
3. Offices and laboratories of health care institutions; laboratories of educational and scientific institutions; premises of electronic computing machines; kitchens of public buildings; technical floors; basement rooms	at least 2.0 (200)	at least 1.2 (120)

Buildings and premises	Characteristic values of loads, kPa (kgf/m^2)	Quasi-constant load values, kPa (kgf/m^2)
4. Halls:		
a) reading rooms	2.0 (200)	0.85 (85)
b) lunch (in cafes, restaurants, canteens)	3.0 (300)	1.2 (120)
c) gatherings and meetings, waiting, spectacular and concert, sports	4.0 (400)	1.7 (170)
d) trade, exhibition and exposition	at least 4.0 (400)	at least 1.7 (170)
5. Book repositories, archives	at least 5.0 (500)	at least 5.0 (500)
6. Scenes of spectacular institutions	at least 5.0 (500)	at least 2.1 (210)
7. Tribunes:		
a) with fixed seats	4.0 (400)	1.7 (170)
b) for spectators standing	5.0 (500)	1.8 (180)
8. Attic premises	0.7 (70)	-
9. Coverage in areas:		
a) with a possible crowding of people (coming out of production premises, halls, auditoriums, etc.)	4.0 (400)	1.7 (170)
b) used for recreation	1.5 (150)	0.6 (60)
c) others	0.5 (50)	–
10. Balconies (loggias) taking into account the load:		
a) strip of uniform width 0.8 malong the fence of the balcony (loggia)	4.0 (400)	1.7 (170)
b) a solid uniform balcony (loggia), the impact of which is more unfavorable than stipulated in item 10, a	2.0 (200)	0.85 (85)
11. Areas of maintenance and repair of equipment in production premises	at least 1.5 (150)	–
12. Vestibules, foyers, corridors, stairs (with passages to them) adjacent to the premises indicated in the items:		
a) 1, 2 and 3	3.0 (300)	1.0 (100)
b) 4, 5, 6 and 11	4.0 (400)	1.7 (170)
c) 7	5.0 (500)	2.1 (210)
13. Station platforms	4.0 (400)	1.7 (170)
14. Premises for livestock:		
small	at least 2.0	not less than

Buildings and premises	Characteristic values of loads, kPa (kgf/m^2)	Quasi-constant load values, kPa (kgf/m^2)
big	(200) at least 5.0 (500)	0.85 (85) at least 2.1 (210)

Note 1. Loads given in pos. 8, should be taken into account on the area not occupied by equipment and materials.
Note 2. The loads given in pos. 9, should be considered without snow load.
Note 3. The loads given in pos. 10, should be taken into account when calculating the load-bearing structures of balconies (loggias) and sections of walls in places where these structures are pinched. When calculating the lower sections of walls, foundations and foundations, the load on the balconies (loggias) should be taken as equal to the loads of the adjacent main premises of the buildings, and they should be reduced taking into account instructions 6.8 and 6.9.
Note 4. Characteristic and quasi-permanent values of loads for buildings and premises given in pos. 3, 4, d, 5, 6, 11 and 14, should be taken according to the construction task on the basis of technological decisions.

The characteristic values of the loads on the crossbars and floor slabs from the weight of the temporary partitions should be taken depending on their design, location and nature of bearing on the floors and walls.

These loads are allowed to be taken into account as evenly distributed additional loads, taking their characteristic values based on the calculation for the intended layouts of partitions, but not less than 0.5 kPa (50 kgf/m^2).

Reliability coefficients by load γ_{fm} are summarized in the table 1.6.

When calculating beams, crossbars, slabs, as well as columns and foundations, which perceive the load from one floor, the characteristic values of the loads are given in the table 1.5, should be reduced, depending on the cargo area of the calculated element A, m^2, by multiplying by the coupling coefficient ψ_A, which is equal to:

Table 1.6

Reliability coefficients regarding load action, γ_{fm}

No	Name of the load	γ_{fm}
1	2	3
1	The weight of metal structures in which the forces from their own weight are less than 50%	1.05 (0.95)
2	The weight of metal structures, in which the forces from their own weight are equal to or exceed 50%	1.10 (0.90)
3	The weight of concrete (with an average density of more than 1600 kg/m^3), reinforced concrete, stone, reinforced stone, wooden structures	1.10 (0.90)
4	The weight of concrete structures (with an average density of 1600 kg/m^3 and less), insulating, leveling and finishing layers (slabs, shells, backfill, screeds, materials in rolls, etc.) of factory production	1.20 (0.90)
5	The weight of concrete structures (with an average density of 1.600 kg/m^3 and less), insulating, leveling and furnishing layers (slabs, shells, backfill, screeds, materials in rolls, etc.) made on the construction site	1.30 (0.90)
6	The weight of soils in the natural setting	1.10 (0.90)
7	Weight of bulk soils	1.15 (0.90)
8	Stationary equipment (equipment)	1.05
9	Isolation of stationary equipment (equipment)	1.20
10	Filling equipment (equipment) (including tanks, pipelines) with liquids	1.00
11	Filling the equipment with suspensions, slags, loose bodies	1.10
12	Forklifts, electric trucks with loads	1.20
13	Evenly distributed load on the floor and stairs at $Q < 2000$ Pa	1.30
14	Evenly distributed load on the floor and stairs at $Q \geq 2000$ Pa	1.20
Note. Values in parentheses should be used to check the overturning resistance of the structure, as well as in other cases where the reduction of the weight of structures and soils may worsen the operating conditions of the structure		

a) for the premises specified in pos. 1, 2, 12, a (with load area $A > A_1 = = 9\ m^2$):

$$\psi_{A1} = 0{,}4 + \frac{0{,}6}{\sqrt{A / A_1}}; \tag{1.1}$$

b) for the premises specified in pos. 4, 11, 12, b (with load area $A > A_2 = = 36\ m^2$):

$$\psi_{A2} = 0{,}5 + \frac{0{,}5}{\sqrt{A / A_2}}. \tag{1.2}$$

When calculating the walls receiving the load from one floor, the load values should be reduced depending on the loading area A of the calculated elements (slabs, beams) that rest on the walls.

When determining the longitudinal forces for the calculation of columns, walls and foundations that receive loads from two or more floors, the characteristic values of the loads are given in the table 3.5, should be reduced by multiplying by the combination factor ψ_n :

a) for the premises specified in pos. 1, 2, 12, a,

$$\psi_{n1} = 0{,}4 + \frac{\psi_{A1} - 0{,}4}{\sqrt{n}}; \tag{1.3}$$

b) for the premises specified in pos. 4, 11, 12, b,

$$\psi_{n2} = 0{,}5 + \frac{\psi_{A2} - 0{,}5}{\sqrt{n}}, \tag{1.4}$$

where **n** is the total number of floors (for the premises listed in Table 1.5, items 1, 2, 4, 11, 12, a, 12, b), the load from which is taken into account when calculating the cross section of the column, wall, and foundation under consideration.

1.6. Snow loads

The snow load is a variable for which three calculated values are set: limit, operational, quasi-permanent.

The calculated limit value of the snow load on the horizontal projection of the covering (structure) is calculated according to the formula:

$$\boldsymbol{S_m = \gamma_{fm} S_0 C} \ , \qquad (1.5)$$

where γ_{fm} is the reliability coefficient for the limit value of the snow load, which is determined depending on the given average recurrence period **T** according to the table 1.7; intermediate values of the coefficient γ_{fm} determined by linear interpolation; for mass construction objects, an average recurrence period of **T** is allowed to be taken as equal to the established service life of the structure $\mathbf{T_{ef}}$;

Table 1.7

Reliability coefficients at the limit value snow load

T , years	1	5	10	20	40	50	60	80	100	150	200	300	500
γ_{fm}	0.24	0.55	0.69	0.83	0.96	1.00	1.04	1.10	1.14	1.22	1.26	1.34	1.44

$\mathbf{S_0}$ is the characteristic value of the snow load (in Pa), which is equal to the weight of the snow cover per 1 square meter of the soil surface, which can be exceeded on average once in 50 years; determined depending on the snow area on the map (Fig. 1.1) or according to Appendix E [1];

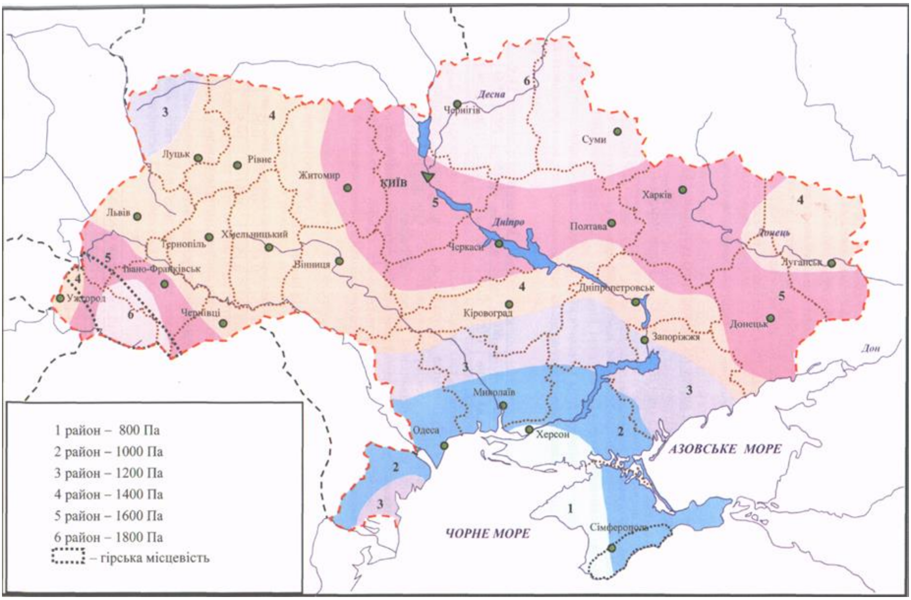


Fig. 1.1. Map of the zoning of the territory of Ukraine according to the characteristic values of the weight of the snow cover (Fig. 8.1, [1])

C is the coefficient determined by the formula:

$$C = \mu C_e C_{alt}\,, \tag{1.6}$$

where μ is the coefficient of transition from the weight of the snow cover on the soil surface to the snow load on the roof;

C_e is a coefficient that takes into account the influence of the operating mode on the accumulation of snow on the roof (cleaning, melting, etc.); for uninsulated roofs of shops with increased heat output with roof slopes of more than 3% and provision of proper drainage of melt water $C_e = 0.8$, in the absence of data on the mode of operation of the roof $C_e = 1$;

C_{alt} – geographical altitude coefficient; for objects located in mountainous terrain, it is determined by the formula:

$$\mathbf{C_{alt} = 1.4\,N + 0.3} \text{ (at H } \geq 0.5 \text{ km);} \tag{1.7}$$

$$\mathbf{C_{alt} = 1} \text{ (when H < 0.5 km).}$$

The operational calculated value is calculated according to the formula:

$$\boldsymbol{S_e = \gamma_{fe} S_0 C}\ , \tag{1.8}$$

where $\mathbf{S_0}$**,** **C** are the same as in formula (1.5);

γ_{fe} – the reliability coefficient according to the operational value of the snow load, which is determined according to the table 1.8 depending on the fraction of time η during which the conditions of the second limit state may be violated; value η they are accepted according to the norms of design of structures or set as design tasks depending on their purpose, responsibility and consequences of going beyond the limit state.

For objects of mass construction, it is allowed to take η **= 0.02.**

Table 1.8

Reliability coefficients according to the operational value of the snow load

η	0.002	0.005	0.01	0.02	0.03	0.04	0.05	0.1
γ_{fe}	0.88	0.74	0.62	0.49	0.40	0.34	0.28	0.10

The quasi-constant calculated value is calculated according to the formula:

$$\boldsymbol{S_p = \left(0{,}4 S_0 - \overline{S}\right) C}\ , \tag{1.9}$$

where $\overline{S}$ =160 Pa; $\mathbf{S_0}$**,** **C** are the same as in formula (1.5).

1.7. Wind loads

Wind load is a variable load for which two calculated values are set: limit and operational.

The limit calculated value of the wind load is determined by the formula:

$$W_m = \gamma_{fm} W_0 C \;, \tag{1.10}$$

where γ_{fm} is the reliability coefficient according to the maximum calculated value of the wind load, determined depending on the given average recurrence period **T** according to the table 1.9; intermediate coefficient values γ_{fm} are determined by linear interpolation. For objects of mass construction, an average recurrence period of **T** is allowed take as equal to the established service life of the structure $\mathbf{T_{ef}}$;

Table 1.9

Reliability coefficients according to the maximum calculated value of the wind load

T, years	5	10	15	25	40	50	70	100	150	200	300	500
γ_{fm}	0.55	0.69	0.77	0.87	0.96	1.00	1.07	1.14	1.22	1.28	1.35	1.45

$\mathbf{W_0}$ is the characteristic value of wind pressure, is equal to the average (static) component of wind pressure at a height 10 m above the ground surface, which can be exceeded on average once in 50 years; determined depending on the wind region on the map (Fig. 1.2) or according to Appendix E [1];

C is the coefficient determined by the formula:

$$C = C_{aer} C_h C_{alt} C_{rel} C_{dir} C_d, \qquad (1.11)$$

where C_{aer} – aerodynamic coefficient, which is determined according to Appendix I [1] depending on the shape of the structure or structural element;

Ch_h - the coefficient that takes into account the increase in wind load depending on the height of the structure or its part under consideration above the surface of the ground (Z), the type of surrounding terrain, is determined according to fig. 1.3;

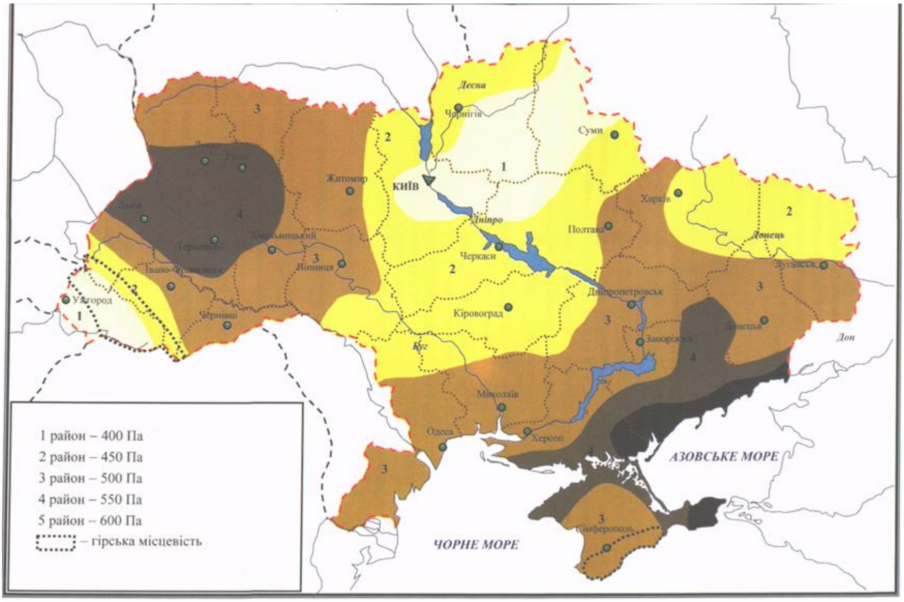


Fig. 1.2. Map of zoning of the territory of Ukraine according to characteristic values of wind pressure (Fig. 9.1, [1])

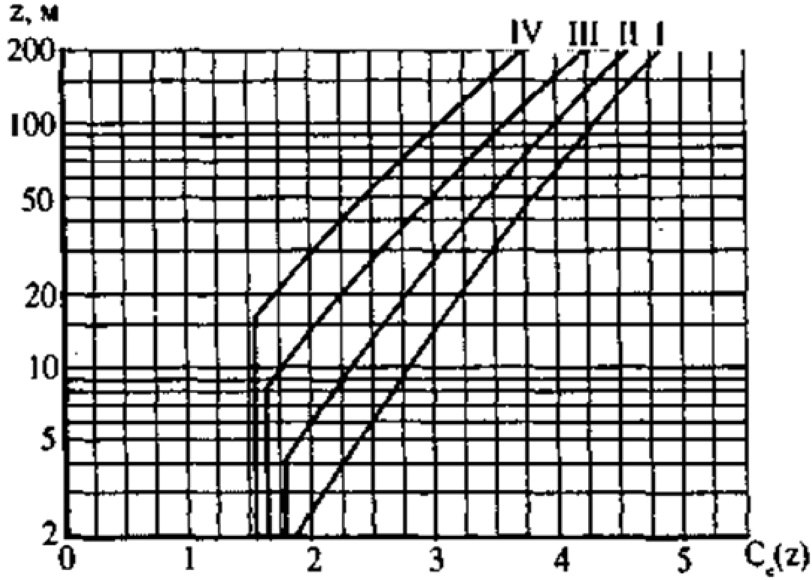


Fig. 1.3. Building height factor $\boldsymbol{C_h}$ (Fig. 9.2, [1])

$\mathbf{C_{alt}}$ – geographic height coefficient, which takes into account the height H (in kilometers) of the location of the building object above sea level; calculated according to the formula:

$$\mathbf{C_{alt} = 4\,H - 1} \;(\,H > 0.5\text{ km}); \qquad (1.12)$$

$$\mathbf{C_{alt} = 1} \;(\,H < 0.5\text{ km});$$

$\mathbf{C_{rel}}$ – a coefficient that takes into account the micro-relief of the area near the location of the construction site and is taken as equal to one, except when the construction site is located on a hill or slope;

$\mathbf{C_{dir}}$ - the direction coefficient, which takes into account the unevenness of the wind load along the wind directions and, as a rule, is taken to be equal to one. The value of $\mathbf{C_{di\,r}}$, which is different from one, is allowed to be taken into account for special justification only for open flat terrain and in the presence of sufficient statistical data;

C_d - the dynamism coefficient, which takes into account the influence of the pulsating component of the wind load and the spatial correlation of the wind pressure on the structure, is determined according to the graph in Fig. 9.5 [1].

The operational calculated value of the wind load is determined by the formula:

$$\boldsymbol{W_e = \gamma_{fe} W_0 C}\ , \tag{1.13}$$

where γ_{fe} is the reliability coefficient based on the operational calculated value of the wind load, determined according to the table 1.10 depending on the fraction of time η during which the conditions of the second limit state may be violated.

Table 1.10

Reliability coefficients based on the operational calculated value of the wind load

γ_{fe}	0.002	0.005	0.01	0.02	0.03	0.04	0.05	0.1
η	0.42	0.33	0.27	0.21	0.18	0.16	0.14	0.09

Intermediate values of the coefficient γ_{fe} should be determined by linear interpolation. The value η is accepted according to the norms of design of structures or set by the design task depending on their purpose, responsibility and consequences of going beyond the limit state. For objects of mass construction, it is allowed to take η **= 0.02.**

Section 2

STRUCTURES MADE OF WOOD AND PLASTICS

2.1. General information about structures made of wood and plastics

Wood is a valuable construction material.

Its **advantages:**

- high specific strength (the ratio of calculated resistance to density), close to the specific strength of steel, and therefore they have a small mass;
- low heat and sound conductivity, which allows you to use wood at the same time, both in load-bearing and enclosing elements;
- sufficient chemical resistance; in an aggressive environment, wood is more durable than metal and reinforced concrete;
- ease of processing and lack of restrictions on use for various construction works.

Disadvantages of wood:

- dependence of mechanical characteristics on many factors;
- hygroscopicity and its consequences - drying, swelling, cracking, etc.;
- heterogeneity of the structure and natural defects that affect strength;
- decay and low fire resistance.

The following types of wooden structures are used in modern construction:

- glued - the most progressive due to the low labor intensity of their production at highly mechanized factories;
- structures made of solid elements (beams, boards) are indispensable for temporary buildings, which can be made from local wood using hand tools and the simplest connections;
- folded section on pliable elms - are used if for some reason glued

structures cannot be used.

For the manufacture of wooden structures and parts, mainly softwood is used, which has a smooth tall trunk with a small number of knots and, thanks to the resin in the fibers, is less susceptible to decay.

2.2. Calculation of elements of wooden structures

Wooden structures are calculated according to the limit states of the first and second groups. The methodology for calculating wooden structures is based on the normative document: DBN V.2.6-161:2017. Wooden structures. Basic provisions [2].

To calculate the elements of wooden structures, based on experimental data, the characteristic values of the strength and stiffness of softwood were established, the values of which are given in Appendix 1. Moreover, it should be remembered that the values of strength in bending, in tension along the fibers, in compression along or across the fibers, as well as stiffness characteristics are different for different classes of wood.

Stretched elements

When solving problems on the calculation of stretched elements according to Appendix 1, it is necessary to establish the characteristic value of the strength of wood of a given class when stretched along the fibers. According to the condition of the task, determine which loads act on the given structure (by magnitude, by duration), identify them according to Appendix 3, and choose the conversion factor k_{mod} according to Appendix 2. According to Appendix 4, choose the γ_m coefficient. Knowing the characteristic value of the strength of wood of a given class when stretched along the fibers and using formula (6.1) [2] to determine the calculated value of wood strength:

$$f_{t,o,d} = k_{mod} \times f_{t,o,k} / \gamma_m \,. \tag{2.1}$$

According to formula (9.2) [2], determine the calculated value of the normal stress in the section of the stretched element:

$$\sigma_{t,o,d} = N_d / A_{net} \,, \tag{2.2}$$

where N_d is the axial tensile force,

A_{net} is the cross-sectional area of the element.

If $\sigma_{t,\,o,\,d} \leq f_{t,o,d}$, the strength of the stretched element is ensured. If this condition is not met, then strength is not guaranteed.

Example 2.1. Check the strength of the wooden element with cross-sectional dimensions of 125×125 mm, which works on tension. The element is weakened by a ∅20 mm bolt hole. The calculated axial force $N_d = 90\ kN$. The element is made of pine of the first grade, class C30, which works in class 2 conditions.

Solution:

1. From Appendix 1, we write out the characteristic value of the strength of C30 class wood when stretched along the fibers

$$f_{t,o,k} = 18MPa = 18N/mm^2 \,.$$

2. From appendix 2 for class 2 for constant load (see appendix 3), we select the conversion factor $k_{mod} = 0.6$.

3. For solid wood, according to Appendix 4, we choose $\gamma_m = 1.3$.

4. According to formula (2.1), we determine the estimated strength of C30 class wood:

$$f_{t,o,d} = k_{mod} \times f_{t,o,k} / \gamma_M = 0.6 \times 18/1.3 = 8.31\ N/mm^2 .$$

5. Cross-sectional area of the element, considering the cut hole

$$A_{net} = 125 \times 125 - 20 \times 125 = 13125\ mm^2 .$$

6. According to formula (2.2), we determine the calculated value of the normal stress in the section of the stretched element:

$$\sigma_{t,o,d} = N_d / A_{net} = 90 \times 10^3 / 13125 = 6.86\ N/mm^2 .$$

$$\sigma_{t,o,d} = 6.86\ N/mm^2 < f_{t,o,d} = 8.31\ N/mm^2 ,$$

therefore, the strength of the stretched element is ensured.

TASKS FOR INDEPENDENT WORK:

Check the strength of the wooden element by cross-sectional dimensions $b \times h$, working in tension. The element is weakened by a hole of a certain diameter d . Calculated axial force N_d . The element is made of pine of a certain grade.

Second last digit of the cipher	Size ***b***, mm	Axial force N_d, kN	Pine class	The last digit of the cipher	Size ***h***, mm	Number of holes	Hole diameter, mm
0	100	50	C20	0	150	1	20
1	125	60	C22	1	175	2	18
2	150	70	C24	2	100	3	16
3	175	80	C27	3	125	4	14
4	100	90	C30	4	150	1	22
5	125	80	C35	5	175	2	24
6	150	70	C27	6	100	3	14
7	175	60	C24	7	125	4	16
8	100	50	C22	8	150	1	18
9	125	90	C20	9	175	2	20

Example 2.2. Select the cross-section of the stretched bone of the farm, which works in the conditions of 1 operational class. The calculated axial force $N_d = 150\ kN$. The material is pine of the first grade, class C30.

Solution:

1. From Appendix 2 for class 1 for constant load (see Appendix 3), we select the conversion factor $k_{mod} = 0.6$.
2. From Appendix 1, we write out the characteristic value of the strength of C30 class wood when stretched along the fibers

$$f_{t,o,k} = 18MPa = 18N/mm^2 .$$

3. For solid wood, according to Appendix 4, we choose $\gamma_M = 1.3$.
4. According to formula (2.1), we determine the estimated strength of C30 class wood:

$$f_{t,o,d} = k_{mod} \times f_{t,o,k} / \gamma_M = 0.6 \times 18/1.3 = 8.31\ N/mm^2 .$$

5. Using formula (2.2), we determine the estimated cross-sectional area of the stretched element:

$$A_{net} = N_d / f_{t,o,d} = 150 \times 10^3 / 8.31 = 18050\ mm^2 .$$

6. If the section is square, then the side of the square:

$$a = \sqrt{A_{net}} = \sqrt{18050} = 134\ mm \approx 150\ mm,$$

$$A_{fact} = 150 \times 150 = 22500\ mm^2 > 18050\ mm^2 .$$

If we assume a rectangular section, set $b = 125\ mm,$ then: $h = A_{net} / b = 18050 / 125 = 144.4\ mm.$

We accept $\boldsymbol{b \times h = 125 \times 150}$ ***mm,***

then $\boldsymbol{A_{fact} = 125 \times 150 = 18750\ mm^2 > 18050\ mm^2}$.

This option turned out to be somewhat more economical.

TASKS FOR INDEPENDENT WORK:

Select the cross-section of the stretched bone of the farm, which works in the conditions of 1 operational class. Calculated axial force $\boldsymbol{N_d}$. The element is made of pine of a certain grade.

Second last digit of the cipher	Cross-sectional shape	Pine class	The last digit of the cipher	Axial force $\boldsymbol{N_d}$, kN
0	rectangle	C35	0	125
1	square	C27	1	135
2	rectangle	C24	2	145
3	square	C22	3	155
4	rectangle	C20	4	165
5	square	C35	5	155
6	rectangle	C27	6	145
7	square	C24	7	135
8	rectangle	C22	8	125
9	square	C20	9	115

Example 2.3. Determine the holding capacity of the wooden element of the covering truss - its stretched belt with cross-sectional dimensions of 150×150 mm, made of C35 class pine. Class of operation – 2. There are two holes ∅20 mm in the rod.

Solution:

1. Cross-sectional area of the element, considering two holes:

$$\boldsymbol{A_{net} = 150 \times 150 - 2 \times 20 \times 150 = 16500\ mm^2}\ .$$

2. From Appendix 1, we write out the characteristic value of the strength of C35 class wood when stretched along the fibers

$$f_{t,o,k} = 21\ MPa = 21\ N/mm^2\ .$$

3. The covering truss perceives constant loads (from its own weight and roof structure) and snow loads, then according to Appendix 3, it can be assumed that these loads are of medium duration, therefore, from Appendix 2, for operation class 2, the conversion factor is $k_{mod} = 0.8$.

4. For solid wood, according to Appendix 4, we choose $\gamma_M = 1.3$.

5. According to formula (2.1), we determine the estimated strength of C35 class wood:

$$f_{t,o,d} = k_{mod} \times f_{t,o,k} / \gamma_M = 0.8 \times 21/1.3 = 12.9\ N/mm^2\ .$$

6. Using formula (2.2), we determine the calculated axial force that can be applied to the stretched element:

$$N_d = f_{t,o,d} \times A_{net} = 12.9 \times 16500 = 213230\ N = 213.2\ kN\ .$$

Therefore, the load-bearing capacity of the truss belt made of C35 pine wood, with a cross-section of 150×150 mm, is 213 kN.

TASKS FOR INDEPENDENT WORK:

Determine the holding capacity of a stretched wooden element with the specified cross-sectional dimensions, made of pine of a certain class. The class of operation is set. There are holes in the rod.

Second last digit of the cipher	Section dimensions, mm	Pine class	The last digit of the cipher	Number of holes	Hole diameter, mm	Class of operation
0	100×150	C30	0	1	10	1
1	125×150	C20	1	2	12	2
2	150×150	C22	2	3	14	3
3	175×150	C24	3	4	16	1
4	100×100	C27	4	1	18	2
5	125×125	C20	5	2	10	3
6	175×175	C22	6	3	12	1
7	125×175	C24	7	4	14	2
8	125×100	C27	8	2	16	3
9	100×175	C30	9	3	18	1

Compressed elements

When solving problems for the calculation of compressed elements, it is necessary to check not only the strength (typical for short elements), but also the stability of compressed rods, because the phenomenon of longitudinal bending from the plane of the element (for long elements) may occur.

Stability is calculated according to formula (9.5) [2]:

$$\frac{\sigma_{c,o,d}}{k_c\, f_{c,o,d}} \leq 1\,. \tag{2.3}$$

According to Appendix 5, the estimated length of the element is determined, which depends on the conditions of fastening at the edges and the type of load.

Calculate the geometric characteristics of the cross-section of the element, its flexibility, according to Appendix 1, determine the modulus of elasticity of wood.

The reduced flexibility is determined by formula (9.9) [2].

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi}\sqrt{\frac{f_{c,o,k}}{E_{0,05}}}. \tag{2.4}$$

The coefficient is determined by formula (9.8) [2].

$$k_z = 0,5\left(1+\beta_c\left(\lambda_{rel,z}-0,3\right)+\lambda_{rel,z}^2\right). \tag{2.5}$$

According to the formula (9.7) [2] determine the coefficient of stability

$$k_{c,z} = 1\Big/\left(k_z+\sqrt{k_z^2-\lambda_{rel,z}^2}\right). \tag{2.6}$$

The normal stress is found from condition (9.5) [2].

$$\sigma_{c,o,d} = \frac{N_c}{k_c A} \tag{2.7}$$

and compare it with the limit value:

$$\sigma_{c,o,d} \le f_{c,o,d}\,. \tag{2.8}$$

If this condition is met, the strength and stability of the compressed element is ensured. If this condition is not met, stability is not guaranteed.

Example 2.4. Check the strength and stability of the compressed rod in section $\boldsymbol{b}\times\boldsymbol{h}$ = 125×175 mm, if 2,5 m. The rod is hinged at the edges. Axial compressive force – 175 kN from constant load. Class of operation - 1. Material - pine 1 grade C30.

Solution:

1. From Appendix 1, we write out the characteristic value of the strength of C30 class wood under compression along the fibers

$$f_{s,o,k} = 23\ MPa = 23\ N/mm^2 .$$

2. From Appendix 2 for class 1 for constant load, we select the conversion factor $k_{mod} = 0.6$.

3. For solid wood, according to Appendix 4, we choose $\gamma_M = 1.3$.

4. According to formula (2.1), we determine the calculated strength of C30 class wood under compression:

$$f_{c,o,d} = k_{mod} \times f_{c,o,k} / \gamma_M = 0.6 \times 23/1.3 = 10.6\ N/mm^2 .$$

5. According to formula (2.2), we determine the calculated value of the normal stress in the cross section of the compressed element:

$$\sigma_{s,o,d} = N_s / A_{net} = 175 \times 10^3 /(175 \times 125) = 8\ N/mm^2 .$$

$$\sigma_{s,o,d} = 8\ N/mm^2 < f_{c,o,d} = 10.6\ N/mm^2 ,$$

therefore, the strength of the compressed element is sufficient.

6. We calculate stability according to formula (2.3):

$$\frac{\sigma_{c,o,d}}{k_c\, f_{c,o,d}} \leq 1 .$$

7. According to Appendix 5, when hinged bearing $l_{ef} = l$ = 2,5 m.

8. For a rectangular section, the radius of inertia is determined approximately by the expression: $i \approx 0.29h$

$$and_{\,y} \approx 0.29h = 0.29 \times 175 = 50.75\ mm;$$

$$i_z \approx 0.29b = 0.29 \times 125 = 36.25\ mm.$$

9. The flexibility of the element is calculated as $\lambda = l_{ef} / i$:

$$\lambda_y = l_{ef} / i_y = 2500 / 50.75 = 49.26 \approx 49 \;;$$

$$\lambda_z = l_{ef} / i_z = 2500 / 36.25 = 68.96 \approx 69 \;.$$

10. From Appendix 1, we write out the C30 class pine modulus of elasticity:

$$E_{0,\,mean} = 12000\ N/mm^2\ .$$

11. From Appendix 1, the modulus of elasticity at a deformation of 0.05%:

$$E_{0,05} = \frac{2}{3} E_{0,mean} = \frac{2}{3} 12000 = 8000 N/mm^2\ .$$

12. Flexibility is given by formula (2.4).

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi} \sqrt{\frac{f_{c,o,k}}{E_{0,05}}} = \frac{69}{3,14} \sqrt{\frac{23}{8000}} = 1,178.$$

13. According to formula (2.5)

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

$$= 0,5\left(1 + 0,2\left(1,178 - 0,3\right) + 1,178^2\right) = 1,282,$$

$\beta_c = 0.2$ – for solid wood.

14. According to formula (2.6)

$$k_{c,z} = 1 \Big/ \left(k_z + \sqrt{k_z^2 - \lambda^2_{rel,z}}\right) = 1 \Big/ \left(1,282 + \sqrt{1,282^2 - 1,178^2}\right) = 0,56.$$

15. From condition (2.7):

$$\sigma_{c,o,d} = \frac{N_c}{k_c A} = \frac{175 \cdot 10^3}{0,56 \cdot 175 \times 125} = 14,28\ N/mm^2\ .$$

$$\sigma_{s,o,d} = 14.28\ N/mm^2 > f_{c,o,d} = 10.6\ N/mm^2\ ,$$

therefore, the stability of the compressed element **is not ensured** .

TASKS FOR INDEPENDENT WORK:

To test the strength and stability of a compressed rod of a certain section and length, made of pine of a certain class. The rod is hinged at the edges. Axial compressive force is optional. The class of operation is set

Second last digit of the cipher	Axial compressive force, kN	Class of operation	The last digit of the cipher	Section dimensions, mm	Pine class	Length , m
0	90	1	0	125×150	C20	2.6
1	95	2	1	150×150	C22	2.7
2	100	3	2	175×150	C24	2.9
3	105	1	3	100×100	C27	2.8
4	110	2	4	125×125	C20	2.7
5	115	3	5	175×175	C22	3.0
6	120	1	6	125×175	C24	2.9
7	115	2	7	125×100	C27	2.8
8	110	3	8	100×175	C30	2.7
9	100	1	9	100×150	C22	2.6

Example 2.5. Select the cross-section of a wooden rack made of pine of class C35, which is operated according to class 2 under the condition of constant and long-term loads $N_d = 195\ kN$. The length of the rack 3,2 m. The support on the edges is hinged.

Solution:

1. From appendix 2 for class 2 for solid wood and long-term load action, we select the conversion factor $k_{mod} = 0.7$.

2. From Appendix 1, we write out the characteristic value of the strength of C35 class wood under compression along the fibers

$$f_{s,o,k} = 25\ MPa = 23\ N/mm^2\ .$$

3. For solid wood, according to Appendix 4, we choose $\gamma_M = 1.3$.
4. According to formula (2.1), we determine the calculated strength of C30 class wood under compression:

$$f_{c,o,d} = k_{mod} \times f_{c,o,k} / \gamma_M = 0.7 \times 25/1.3 = 12.38\ N/mm^2\ .$$

5. From Appendix 1, we write out the C30 class pine modulus of elasticity:

$$E_{0,\ mean} = 13000\ N/mm^2\ .$$

6. From Appendix 1, the modulus of elasticity at a deformation of 0.05%:

$$E_{0,05} = \frac{2}{3} E_{0,mean} = \frac{2}{3} 13000 = 8667\ N/mm^2\ .$$

7. In the first approximation, we take $\lambda_y = 75$.
8. Flexibility is given by formula (2.4).

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi} \sqrt{\frac{f_{c,o,k}}{E_{0,05}}} = \frac{75}{3,14} \sqrt{\frac{25}{8667}} = 1,282.$$

9. According to formula (2.5)

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

$$= 0,5\left(1 + 0,2\left(1,282 - 0,3\right) + 1,282^2\right) = 1,42,$$

$\beta_c = 0.2$ – for solid wood.

10. According to formula (2.6), we determine the coefficient of longitudinal bending:

$$k_{c,y} = 1 \Big/ \left(k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}\right) = 1 \Big/ \left(1{,}42 + \sqrt{1{,}42^2 - 1{,}282^2}\right) = 0{,}49.$$

11. From the strength condition (2.7), $\sigma_{c,o,d} = \dfrac{N_d}{k_c A_{net}} \leq f_{c,o,d}$ we find the estimated cross-sectional area of the rack:

$$A_{net} = \frac{N_d}{k_c f_{c,o,d}} = \frac{195 \cdot 10^3}{0{,}49 \cdot 12{,}38} = 32145.33\ mm^2 .$$

12. If we take a rectangular section and in the first approximation set ***b=150 mm,*** then: ***h = Anet / b=32145.33 / 150 = 214.3 mm.*** We accept ***b×h = 150×225 mm.***
13. Geometric characteristics of the accepted section:

cross-sectional area:

$$A_{fact} = 150 \times 225 = 33750\ mm^2 > 32145.33\ mm^2 ;$$

minimum moment of inertia of the section:

$$I_{min} = hb^3/12 = 225 \times 150^3 / 12 = 63281250\ mm^4 ;$$

the smallest radius of inertia of the section:

$$i_{min} = \sqrt{I_{min}/A} = \sqrt{63281250 / 32145{,}33} = 43.3\ mm;$$

rack flexibility:

$$\lambda_{max} = l_{ef}/i_{min} = 3200/43{,}3 = 73{,}9.$$

14. Flexibility is given by formula (2.4).

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,o,k}}{E_{0,05}}} = \frac{73{,}9}{3{,}14}\sqrt{\frac{25}{8667}} = 1{,}26.$$

15. According to formula (2.5)

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

$$= 0,5\left(1+0,2\left(1,26-0,3\right)+1,26^2\right)= 1,39,$$

$\beta_c = 0.2$ – for solid wood.

16. According to formula (2.6), the coefficient of longitudinal bending:

$$k_{c,y} = 1\Big/\left(k_y + \sqrt{k_y^2-\lambda^2_{rel,y}}\right) = 1\Big/\left(1,39+\sqrt{1,39^2-1,26^2}\right) = 0,5.$$

17. From condition (2.7), we check the stability:

$$\sigma_{c,o,d} = \frac{N_c}{k_c A} = \frac{195\cdot 10^3}{0,5\cdot 150\times 225} = 11,55\ N/mm^2 .$$

$$\sigma_{s,o,d} = 11.55\ N/mm^2 < f_{c,o,d} = 12.38\ N/mm^2 ,$$

therefore, the stability (and strength) of the selected compressed element is ensured.

TASKS FOR INDEPENDENT WORK:

Choose a cross-section of a wooden rack made of pine of a certain class. Axial compressive force is optional. The class of operation is set. The length of the rack is set. The support on the edges is hinged.

Second last digit of the cipher	Pine class	Axial compressive force, kN	The last digit of the cipher	Class of operation	Length , m
0	C20	140	0	1	3.0
1	C22	130	1	1	2.9
2	C24	120	2	2	2.8
3	C27	105	3	3	2.7
4	C20	110	4	1	2.6
5	C22	115	5	2	2.6
6	C24	120	6	3	2.7
7	C27	115	7	1	2.8
8	C30	110	8	2	2.9
9	C22	130	9	3	2.9

Example 2.6. Determine the holding capacity of a wooden stand made of round pine logs of class C22 ∅15 cm. Class of operation - 2. Constant and long-term loads are applied. Hinged fasteners. Length 3.5 m.

Solution:

1. From Appendix 1, we write out the characteristic value of the strength of C22 class wood under compression along the fibers, we consider the coefficient ***1.2*** , since the rack is made of round logs:

$$f_{s,o,k} = 20\ MPa \times 1.2 = 24\ N/mm^2\ .$$

2. From appendix 2 for class 2 for solid wood and long-term load action, we select the conversion factor $k_{mod} = 0.7$.

3. For solid wood, according to Appendix 4, we choose $\gamma_m = 1.3$.

4. According to formula (2.1), we determine the calculated value of the strength of C22 class wood under compression:

$$f_{c,o,d} = k_{mod} \times f_{c,o,k} / \gamma_M = 0.7 \times 24/1.3 = 12.92\ N/mm^2\ .$$

5. Cross-sectional area of a log:

$$A = \pi d^2 / 4 = 3{,}14 \times 150^2 / 4 = 17671\ mm^2\ .$$

6. Radius of inertia for elements of circular section:

$$i = 0{,}5R = 0{,}5 \times 150/2 = 37{,}5\ mm$$

7. According to Appendix 5, when hinged bearing $l_{ef} = l = 3{,}5\ m$.

8. The flexibility of the element is calculated as $\lambda = l_{ef} / i$:

$$\lambda = l_{ef} / i = 3500 / 37.5 = 93.3.$$

9. From Appendix 1, we determine the modulus of elasticity of pine

of class C22, considering the coefficient of ***1.2 for a log*** :

$$E_{0,mean} = 1.2 \times 10000 = 12000\ N/mm^2 .$$

10. From Appendix 1, the modulus of elasticity at a deformation of 0.05%:

$$E_{0,05} = \frac{2}{3} E_{0,mean} = \frac{2}{3} 12000 = 8000\ N/mm^2 .$$

11. Flexibility is given by formula (2.4).

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi} \sqrt{\frac{f_{c,o,k}}{E_{0,05}}} = \frac{93,3}{3,14} \sqrt{\frac{24}{8000}} = 1,63.$$

12. According to formula (2.5)

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

$$= 0,5\left(1 + 0,2\left(1,63 - 0,3\right) + 1,63^2\right) = 1,96,$$

$\beta_c = 0.2$ – for solid wood.

13. According to formula (2.6), the coefficient of longitudinal bending:

$$k_{c,y} = 1 \Big/ \left(k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}\right) = 1 \Big/ \left(1,96 + \sqrt{1,96^2 - 1,63^2}\right) = 0,33.$$

14. Under the condition of strength, considering the stability of the structure of the rack

$$N = k_c A_{net} f_{c,o,d} = 0,33 \times 17671 \times 12,92 = 75342\ H \approx 75 kN.$$

Therefore, the holding capacity of the rack is ***75 kN*** .

TASKS FOR INDEPENDENT WORK:

Determine the holding capacity of a wooden rack made of round pine logs of a certain class and the specified diameter. The class of operation is set.

There are constant and long-term loads. The length of the rack is set. The support on the edges is hinged.

Second last digit of the cipher	Class pine trees	Class operation	The last digit of the cipher	Diameter round, see	Length , m
0	C20	3	0	10	2.6
1	C22	3	1	12	2.7
2	C24	1	2	14	2.8
3	C27	2	3	15	2.9
4	C20	3	4	16	3.0
5	C22	1	5	18	3.0
6	C24	2	6	20	2.6
7	C27	3	7	22	2.7
8	C30	1	8	24	2.8
9	C22	2	9	25	2.9

Bending elements

Bending elements are most often flooring elements, floor beams, rafters, staircase elements, etc. Before solving the problem, the operating conditions of the structure, the nature of fixing on the supports, and the acting loads should be clearly established. The maximum bending moment in the most dangerous cross-section is set according to the conditions of resistance and load.

The strength of the bending element is checked according to (9.16) [2]:

$$\sigma_{m,y,d} = \frac{M_{y,d}}{W_{yd}} \leq f\,m,\,d\,. \tag{2.9}$$

If this condition is met, the strength of the bending element is ensured. If this condition is not met, then strength is not guaranteed.

Here , $f_{t,d}$ is the calculated value of the bending strength of wood, which is calculated according to the formula:

$$f_{t,d} = k_{mod} \times f_{t,k} / \gamma_m \,. \tag{2.10}$$

We check the rigidity of the floor beam according to the data of appendices 7 and 8:

$$w = \frac{5ql_{ef}^4}{32E_{o,mean}bh^3} \leq w_{net,fin} = \frac{1}{250}l \,. \tag{2.11}$$

If this condition is met, the stiffness of the bending element is ensured. If this condition is not fulfilled, then rigidity is not ensured.

The calculated deflection for the rafter (as an element located at an angle) is determined according to Appendix 8, considering the shear:

$$w = \frac{5ql_{ef}^4}{32E_{o,mean}bh^3}\left[1 + 0{,}96\,\frac{E_{o,mean}}{G_{o,mean}}\left(\frac{h}{l_{ef}}\right)^2\right] \leq w_{net,fin} \,. \tag{2.12}$$

If this condition is met, the stiffness of the bending element is ensured. If this condition is not fulfilled, then rigidity is not ensured.

Example 2.7. Design a floor beam in a residential building. The length of the beam 5 m. Step beams 0.9 m. The floor is laminate. The beam material is pine.

Solution:

1. Architectural solution of the ceiling on wooden beams in the case of designing a laminate floor (*as an option*):

laminate thickness $\delta = 8$ ***mm***, approximate density $\rho = 600$ ***kg/m³*** ;

leveling layer, $\delta = 20$ ***mm***, $\rho = 1700$ ***kg/m³*** ;

sound insulation, $\delta = 60$ ***mm***, $\rho = 550$ ***kg/m³*** ;

flooring (boards), δ= ***40 mm***, ρ= ***500 kg/m³*** .

2. According to the table 1.5 select the variable load value: for residential buildings it is ***1500 Pa*** .
3. We collect the load on the floor in tabular form:

Collection of loads on the floor

No	Type of load	Operating (characteristic) load, Pa	Coefficient γ_{fm}	Estimated (limit) load, Pa
1.	Permanent			
1.1.	laminate floor	0.008×600×10 = 48	1.2	48×1.2 = 57.6
1.2.	leveling layer	0.02×1700×10 = 340	1.2	340×1.2 = 408
1.3.	soundproofing	0.06×550×10 = 330	1.3	330×1.3 = 396
1.4.	planking	0.04×500×10 = 200	1.1	200×1.1 = 220
	In total	918	-	1081.6
2.	variable	1500	1.3	1950
3.	Complete	2418	-	3031.6

In this table, the reliability coefficients for the action of the load **,** γ_{fm} **,** are chosen according to table 1.6.

4. Determination of the load on the floor beam at a beam pitch of 0.9 m:

operating load:

$$q_e = q \times s = 2418 \times 0.9 = 2176.2\ N/m;$$

calculated (limit) load on the beam:

$$q_m = q_p \times s = 3031.6 \times 0.9 = 2728.44\ N/m.$$

5. According to Appendix 6 for the distributed load $l_{ef} / l = 0.9$, so the calculated length of the beam $l_{ef} = 0.9 \times 5 = 4{,}5\ m$.
6. Calculated bending moment for a single-span beam with hinged support at the edges:

$$M_{yd} = \frac{q_m l_{ef}^2}{8} = \frac{2728{,}44 \times 4{,}5^2}{8} = 6906\ Nm.$$

7. From Appendix 1, we write out the characteristic value of the bending strength of C24 class wood (*accepted in the first approximation*)

$$f_{t,k} = 24\ MPa = 24\ N/mm^2 .$$

8. The floor beam perceives constant (from its own weight and the structure of the floor) and variable (useful) loads, then according to Appendix 3 it can be assumed that these loads are of medium duration, therefore, from Appendix 2 for operation class 1, the conversion factor is $k_{mod} = 0.8$.

9. For solid wood, according to Appendix 4, we choose $\gamma_M = 1.3$.

10. According to the formula (2.10), we determine the calculated strength of C22 class wood during bending:

$$f_{t,d} = k_{mod} \times f_{t,k} / \gamma_M = 0.8 \times 24/1.3 = 14.77\ N/mm^2 .$$

11. From the strength condition (2.9), we determine the calculated moment of resistance of the beam section

$$W = \frac{M}{f_{m,d}} = \frac{6906 \times 10^3}{14{,}77} = 467594\ mm^3 .$$

12. In the first approximation, we take $b = 100\ mm$, then

$$h = \sqrt{\frac{6W}{b}} = \sqrt{\frac{6 \times 467594}{100}} = 167.5\ mm .$$

13. We accept $b \times h = 100 \times 175\ mm.$

14. Actual moment of section resistance:

$$W = \frac{bh^2}{6} = \frac{100 \times 175^2}{6} = 510416.7\ mm^3 .$$

15. Let's check the strength of the selected beam according to (2.9):

$$\sigma_{m,y,d} = \frac{M_{y,d}}{W_{yd}} = \frac{6906 \times 10^3}{510416{,}7} = 13.53\ N/mm^2$$

$$\sigma_{m,y,d} = 13.53\ N/mm^2 < f_{m,d} = 14.77\ N/mm^2 ,$$

therefore, the strength of the selected beam is ensured.

16. We check the rigidity of the floor beam according to (2.11):

$$w = \frac{5ql_{ef}^4}{32E_{o,mean}bh^3} \leq w_{net,fin} = \frac{1}{250}l .$$

17. From Appendix 1, we determine the C24 class pine modulus of elasticity:

$$E_{0,\ mean} = 11000\ N/mm^2 .$$

18. The maximum allowable deflection for a given ceiling beam according to Appendix 7:

$$w_{net,fin} = \frac{1}{250}l = \frac{5000}{250} = 20\ mm.$$

19. Calculated deflection for a given floor beam according to Appendix 8:

$$w = \frac{5ql_{ef}^4}{32E_{o,mean}bh^3} = \frac{5 \times 2176{,}2 \times 4{,}5^4}{32 \times 11000 \times 10^6 \times 0{,}1 \times 0{,}175^3} =$$

$$= 0.0236\ m = 23.6\ mm > 20\ mm.$$

Therefore, the rigidity of the cross-section is not ensured, it is necessary to increase the cross-section of the beam.

20. We accept $b \times h = 125 \times 175\ mm$, then

$$w = \frac{5ql_{ef}^4}{32E_{o,mean}bh^3} = \frac{5 \times 2176,2 \times 4,5^4}{32 \times 11000 \times 10^6 \times 0,125 \times 0,175^3} =$$

= 0.01892 m = 18.9 mm.

$$w = 18.9mm < w_{net,fin} = 20\ mm.$$

So, beam ***b is selected×h = 125×175 mm*** satisfies the stiffness conditions.

TASKS FOR INDEPENDENT WORK:

Design a floor beam in a residential building. The length of the beam is given. Take the steps of the beams as an option. The material of the beam is pine of a certain class. The floor is optional.

Second last digit of the cipher	Step of beam, m	The last digit of the cipher	Class pine trees	Material floors	Beam length , m
0	1.2	0	C22	parquet	4.0
1	1.2	1	C24	laminate	4.1
2	1.1	2	C27	linoleum	4.2
3	1.1	3	C20	board	4.3
4	1.0	4	C22	carpet	4.4
5	0.9	5	C24	parquet	4.5
6	0.9	6	C27	laminate	4.6
7	0.9	7	C20	linoleum	4.7
8	0.8	8	C22	board	4.8
9	0.8	9	C24	carpet	4.9

Example 2.8. Choose a rafter cross-section for installing a roof made of natural tiles above the attic. The slope of the roof is 40 ^{o} . The rafter material is C22 grade pine. The pitch of the rafters is 1.1 m. The distance between the supports in the plan 5 m. The city of construction is Lutsk.

Solution:

1. Determination of snow load for the construction area - Lutsk

1.1. The maximum calculated snow load on a horizontal surface is determined by formula (1.5):

$$S_m = \gamma_{fm} \times So \times C,$$

where γ_{fm} – the reliability coefficient at the limit calculated value

snow load and is determined depending on the given

average recurrence period T according to table 1.6.

For a period of 50 years (accepted for low-rise buildings)

$\gamma_{fm} = 1$;

S_o is the characteristic value of the snow load (in Pa), which

determined according to 8.5 [1]. For the city of Lutsk $S_o = 1240$ *Pa* .

C is the coefficient determined by formula (1.6) :

$$C = \mu C_e C_{alt} ,$$

here μ is the coefficient of transition from the weight of the snow cover on the surface

of the soil to the snow load on the roof, which

is determined according to 8.7, 8.8 [1]. For a given roof slope angle

40 by interpolation $\mu = 0.6$.

C_e is a coefficient that takes into account the mode of operation of the roof and

is determined according to 8.9 [1]. It is allowed to take $C_e = 1$.

C_{alt} is the geographical altitude coefficient determined according to 8.10 [1].

It is allowed to take $C_{alt} = 1$.

Therefore, $C = \mu C_e C_{alt} = 0.6 \times 1 \times 1 = 0.6$.

Then $S_m = \gamma_{fm} \times S_o \times C = 1 \times 1240 \times 0.6 = 744$ *Pa* .

1.2. The operational snow load on a horizontal surface is determined by the formula (1.8):

$$S_e = \gamma_{fe} \times So \times C,$$

where γ_{fe} – the reliability coefficient according to the operational calculation

value of snow load, determined according to the table 1.7

depending on the share of time η during which they can

the conditions of the second limit state are violated. For objects

mass construction is allowed to be accepted $\eta = 0.02$, for

which $\gamma_{fe} = 0.49$.

Therefore , $S_e = \gamma_{fe} \times S_o C = 0.49 \times 1240 \times 0.6 = 365$ ***Pa.***

2. Collection of loads from the roof (according to reference books, internet sources).

Natural tiles: ***13 pieces*** are laid on $1m^2$.

Weight ***1 pc*** . ***- 3.2...3.6 kg .3,6 kg×13 pcs. = 46,8 kg.***

Therefore, the load from the tiles ***is 468N/m²*** **.**

Crate: bar size ***60×60 mm,*** step ***360 mm***, therefore in

1 m/run has 3 pieces. crate bars

Dry wood has a density of $500\ kg/m^3$.

Therefore, the load from the crate:

$$0.06 \times 0.06 \times 3 \text{ pcs.} \times 500 \text{ kg/m}^3 \times 10 = 54 \text{ N/m}^2 .$$

Thermal insulation: accepted according to thermal engineering calculation

thermal insulation ***180...200 mm thick*** , its density ***30 kg/m³*** .

Therefore, the load from thermal insulation:

$$0.2 \times 30 \text{ kg/m}^3 \times 10 = 60 \text{ N/m}^2.$$

Plasterboard on a metal frame ***15 kg/m² = 150 N/m²***.

3. We collect the load from the roof above the attic floor in tabular form (Table 1):

Table 1

Collection of loads from the roof

No	Type of load	Operating (characteristic) load, Pa	Coefficient γ_{fm}	Estimated (limit) load, Pa
1.	Permanent			
1.1.	tiling	468	1.1	468×1.1 = 515
1.2.	crate	54	1.1	54×1.1 = 59
1.3.	thermal insulation	60	1.2	60×1.2 = 72
1.4.	plasterboard	150	1.1	150×1.1 = 165
	In total	732	-	811
2.	Changeable (snow)	365	-	744
3.	Complete	1097	-	1555

4. Collection of loads on the rafter.

In the first approximation, we take the dimensions of the cross section of the rafter to be ***100×150 mm.*** Dry pine wood of class C22 has a density of ***340 kg/m³*** (according to Appendix 1).

The linear load on the rafter with a rafter pitch of ***1.1 m*** is calculated in the form of a table (Table 2):

Table 2

The load on the rafter

No	Type of load	Operating (characteristic) load	Design (limit) load
1.	Permanent		
1.1.	from the roof	***732 Pa×1.1 m = 805.2 N/m***	***811 Pa×1.1 m = 892.1 N/m***
1.2.	own weight of the rafter	***0.1m×0.15m×340kg/***m^3***×10 = = 51 N/m***	***51 N/m×1.1 = 56.1 N/m*** here 1.1 is the γ_{fm} ***coefficient*** for wood
2.	Changeable (snow)	***365 Pa×1.1 m = 401.5 N/m***	***744 Pa×1.1 m = 818.4 N/m***
3.	Complete	***1257.7 N/m***	***1766.7 N/m***

5. The bending of the rafter will cause a load perpendicular to the axis of the rafter. It is necessary to consider the angle of inclination of the rafters to the horizon of 40 ^{o}, $\cos 40^o = 0.766$.

5.1. Operating load:

$$q_e = 1257.7 \times 0.766 = 963.4\ N/m\ .$$

5.2. Estimated (limit) load:

$$q_m = 1766.7 \times 0.766 = 1353.3\ N/m\ .$$

6. The distance between the supports in the plan - 4 m(by condition). The slope of the rafters is 40 ^{o}. The length of the rafter will be:

$$4\ m / \cos 40^o = 4000 / 0.766 = 5222\ mm.$$

7. According to Appendix 6 for a uniformly distributed load $l_{ef} / l = 0.9$, therefore the calculated length of the rafter:

$$l_{ef} = 0.9 \times 5222 = 4700\ mm = 4.7\ m\ .$$

8. Calculated bending moment for the rafter as a single-span beam with hinged support at the edges:

$$M_{yd} = \frac{q_m l_{ef}^2}{8} = \frac{1353,3 \times 4,7^2}{8} = 3736.8\ Nm.$$

9. From Appendix 1, we write down the characteristic value of the bending strength of C22 class pine

$$f_{t,k} = 22\ MPa = 22\ N/mm^2\ .$$

10. The rafter perceives constant (from its own weight and construction of the covering) and variable (snow) loads, then according to Appendix 3 it can be assumed that these loads are of medium duration, therefore, from Appendix 2 for operation class 2, the conversion factor is $k_{mod} = 0.8$.

11. For solid wood, according to Appendix 4, we choose $\gamma_M = 1.3$.

12. According to the formula (2.10), we determine the calculated strength of C22 class wood during bending:

$$f_{t,d} = k_{mod} \times f_{t,k} / \gamma_M = 0.8 \times 22/1.3 = 13.54\ N/mm^2\ .$$

13. As cross section of rafter was previously accepted ***100×150 mm,*** then the moment of resistance of the section:

$$W = \frac{bh^2}{6} = \frac{0,1 \times 0,15^2}{6} = 0.000375\ m^3\ .$$

14. Let's check the strength of the selected rafter according to the formula (2.9):

$$\sigma_{m,y,d} = \frac{M_{y,d}}{W_{y,d}} = \frac{3736,8}{0,000375} = 9964741.6\ N/\mathrm{m}^2 = 9.96\ N/mm^2$$

$$\sigma_{m,y,d} = 9.96\ N/mm^2 < f_{m,d} = 13.54\ N/mm^2\ ,$$

therefore, the strength of the selected rafter is ensured.

15. Checking the stiffness of the rafters.

From Appendix 1, we determine the modulus of elasticity of pine class C22 $\boldsymbol{E_{0,mean} = 11000\ N/mm^2}$ and the shear modulus $\boldsymbol{G_{mean} = 630\ N/mm^2}$.

Limit deflection for the rafter according to Appendix 7:

$$w_{net,fin} = \frac{1}{250} l = \frac{5222}{250} = 20.9\ mm.$$

Calculated deflection for the rafter according to the formula (2.12):

$$w = \frac{5ql_{ef}^4}{32E_{o,mean}bh^3}\left[1 + 0,96\frac{E_{o,mean}}{G_{o,mean}}\left(\frac{h}{l_{ef}}\right)^2\right] =$$

$$= \frac{5 \times 963,4 \times 4,7^4}{32 \times 10000 \times 10^6 \times 0,1 \times 0,15^3}\left[1 + 0,96\frac{10000}{630}\left(\frac{0,15}{4,7}\right)^2\right] =$$

$= 0.0221\ m = 22.1\ mm > w_{net,fin} = 20.9\ mm.$

Therefore, the rigidity of the cross-section is not ensured, it is necessary to increase the cross-section of the rafters.

We accept $\boldsymbol{b \times h = 100 \times 160\ mm}$, then

$$w = \frac{5 \times 963,4 \times 4,7^4}{32 \times 10000 \times 10^6 \times 0,1 \times 0,16^3}\left[1 + 0,96\frac{10000}{630}\left(\frac{0,16}{4,7}\right)^2\right] =$$

$= 0.018\ 2499\ m = 18.2\ mm.$

$$w = 18.2\ mm < w_{net,fin} = 20.9\ mm.$$

So, the selected rafter cross-section $\boldsymbol{b \times h = 100 \times 160\ mm}$ satisfies the stiffness conditions.

TASKS FOR INDEPENDENT WORK:

Choose a cross-section of the rafter for the arrangement of the roof of the specified material above the attic or above the cold attic. The slope of the roof is set. The material of the rafters is pine of a certain class. The pitch of the rafters and the distance between the supports are planned to be adopted according to the option. The city of construction is optional.

The Second last digit of the code and the city of construction	Step rafter, m	The presence of an attic	The last digit of the cipher	Class pine trees	Material the roof	Slope of the roof	Distance between supports in plan, m
0, Rivne	1.2	+	0	C22	natural tiles	46 º	4.5
1, Dnipro	1.2	-	1	C24	metal tile	22 º	4.6
2, Kalush	1.1	+	2	C27	bituminous tiles	24 º	4.7
3, Lutsk	1.1	-	3	C20	ondulin	36 º	4.8
4, Sumy	1.0	+	4	C22	metal profile	28 º	4.9
5, Lviv	0.9	-	5	C24	natural tiles	42 º	4.5
6, Dubno	0.9	+	6	C27	metal profile	17 º	4.6
7, Kaniv	0.9	-	7	C20	bituminous tiles	19 º	4.7
8, Kovel	0.8	+	8	C22	ondulin	35 º	4.8
9, Odesa	0.8	-	9	C24	metal tile	29 º	4.9

Section 3

METAL STRUCTURES

3.1. General information about metal structures

Metal structures are used today in almost all types of buildings and engineering structures, especially in the presence of significant spans, heights, and loads.

Depending on the structural form and purpose, metal structures are divided into: structures of one-story industrial buildings (in the form of all-metal or mixed frames); in heli-span coverings (beam, frame, arch, hanging, combined, etc.); bridges , overpasses; sheet structures (elements of tanks, bunkers, large-diameter pipelines and various structures of chemical production and oil refining); shafts and masts (for radio and television structures, in the supports of power lines, oil towers, smoke and ventilation pipes); to arches of multi-story buildings; crane structures and much more.

Compared to others, metal structures have **advantages** :

- reliability in work, which is ensured by the homogeneity of the structure, and quite closely corresponds to the calculation prerequisites for elastic or elastic-plastic work of the material;

- lightness - of all building structures (reinforced concrete, stone, wood), metal ones are the lightest;

- industriality - metal structures in their total mass are manufactured in factories equipped with modern equipment, which ensures a high degree of industrialization of their production. Installation of metal structures, as a rule, is carried out by specialized organizations using high-performance equipment;

- impermeability - metals have not only significant strength, but also high density, which ensures impermeability for gases and liquids, high protective properties against the influence of harmful radiation. The density of the metal and its joints is achieved by welding, which is a necessary condition to produce tanks.

The **disadvantages of** metal structures include:

- corrosion caused by the high chemical activity of the metal because of interaction with the environment and aggressive gases, which leads to its complete destruction. Under unfavorable conditions, this can happen in two...three years. Although aluminum alloys are much more resistant to corrosion, they also corrode under adverse conditions. Cast iron resists corrosion well. Increasing the corrosion resistance of metal structures is achieved by adding special alloying elements to steel, periodically coating structures with protective films (varnishes, paints, etc.), as well as choosing a rational structural form without gaps where moisture and dust can accumulate;

- low fire resistance - at a temperature of +200°C, a decrease in the modulus of elasticity is observed in steels, which leads to an increase in structural deformations, and at +600°C, the material completely transitions into a plastic state. Aluminum alloys become plastic at temperatures close to +300°C. Because of this, fire-hazardous metal constructions of buildings must be protected by fire-resistant facings (concrete, ceramics, special coatings, etc.).

For steel building structures, rolled steel should be used (sheet, shaped, wide-strip, universal, graded), bent profiles and pipes made of low-carbon and low-alloy steel, as well as steel ropes [5, item 6.1.1, p. 18].

General-purpose rolled steel, rolled steel for welded constructions and steel rolled steel manufactured by the technology of hot rolling, thermomechanical rolling and after heat treatment (annealing, normalizing, quenching, and tempering, tempering, etc.) are used.

It is also allowed to use reinforcing hot-rolled weldable steel, bundles, and strands of wire.

The calculated supports [5, p. 7.2, p. 22] of sheet, wide-band universal, shaped rolled products and pipes of mass application are given in appendix 9.

Physical characteristics of materials for steel structures are given in [5, appendix B, p. 127] the density of rolled steel and steel castings ρ= 7850 kg/m^3 , the density of iron castingsρ = 7200 kg/m^3 ; modulus of elasticity of rolled steel E = 2.06×10 ^{5} N/mm^2 ; shear modulus of rolled steel G = 0.79×10 ^{5} N/mm^2 ; coefficient of transverse deformation (Poisson's ratio) ν= 0.3.

3.2. Calculation of elements of steel structures

Steel structures are calculated according to the limit states of the first and second groups. The method of calculating steel structures is based on the normative document: DBN V.2.6-198:2014. Steel structures. Design norms [5].

Stretched elements

Calculation of the strength of steel elements with a characteristic resistance of up to 440 N/mm^2 in central tension is performed according to formula (8.1) [5]:

$$\frac{N\gamma_n}{A_n R_y \gamma_c} \le 1\,, \qquad (3.1)$$

where ***N*** is the axial tensile force;

γ_n is the reliability coefficient by responsibility, determined according to DBN B.1.2-14;

R_y is the calculated resistance of steel beyond the yield point, determined by Appendix 9;

A_n is the cross-sectional area of the element;

γ_c is the coefficient of working conditions, determined according to Appendix 10.

Example 3.1. Choose a rolled profile for the stretched rod of the covering truss if the known axial force $N = 190\ kN$. **C235** steel is used . Reliability coefficient for the responsibility of the building to accept $\gamma_n = 1.0$.

Solution:

1. According to Appendix 9, the calculated resistance of the shaped part is chosen

rolled steel C235: R_y = 230 N/mm^2 .

2. According to Appendix 10, note 5, the coefficient of the working conditions of steel structures γ_c = 1.0.

3. From the condition of the strength of the stretched elements (3.1), we determine the calculated cross-sectional area of the rod:

$$A_n = \frac{N\gamma_n}{R_y\gamma_c} = \frac{190 \cdot 10^3 \cdot 1,0}{230 \cdot 1,0} = 847,8\ mm^2 = 8.48\ cm^2 .$$

4. From Appendix 11 (assortment of profiles) we choose an uneven angle L according to the calculated area A_n = ***8.48 cm²*** **90**×**56**×**6** , (GOST 8510-86) whose actual area is ***8.54 cm²*** .

5. Let's check how the condition of strength of the stretched rod is fulfilled according to formula (3.1):

$$\frac{N\gamma_n}{A_n R_y\gamma_c} = \frac{190 \cdot 10^3 \cdot 1,0}{854 \cdot 230 \cdot 1,0} = 0,967 < 1.0 ,$$

therefore, the strength of the rod from the angle L **90**×**56**×**6** is provided.

Compressed elements

Calculation of the strength of centrally compressed elements made of steel with a characteristic resistance of up to 440 N/mm^2 is performed according to the same formula (3.1) as for tension. In addition, the stability calculation (8.3) [5] should be performed:

$$\frac{N\gamma_n}{\varphi A R_y\gamma_c} \le 1 , \qquad (3.2)$$

where φ is the coefficient of stability under central compression, the value of which

with conditional flexibility $\overline{\lambda}$ <0.4 are accepted $\varphi = 1.0$, and at

$\overline{\lambda}$ ≥ 0.4 must be calculated according to formula (8.4) [5]:

$$\varphi = \frac{0,5}{\overline{\lambda}^2}\left(\delta - \sqrt{\delta^2 - 39,48\overline{\lambda}^2}\right). \tag{3.3}$$

Conditional flexibility $\overline{\lambda}$ is determined by the condition:

$$\overline{\lambda} = \lambda\sqrt{R_y/E}\ , \tag{3.4}$$

here the flexibility of the element $\lambda = l_{ef} / i$.

The radius of inertia of the section i determined for each specific cross-section of the element according to the tables of assortments of rolling profiles (appendix 11), choose a smaller value at which the flexibility will be maximum.

The calculated length of the element l_{ef} steel structure is determined according to [5, chapter 13].

In particular, the calculated lengths l_{ef} of compressed columns (risers) of constant cross-section length or individual sections of stepped columns should be determined by formula (13.2) [5]:

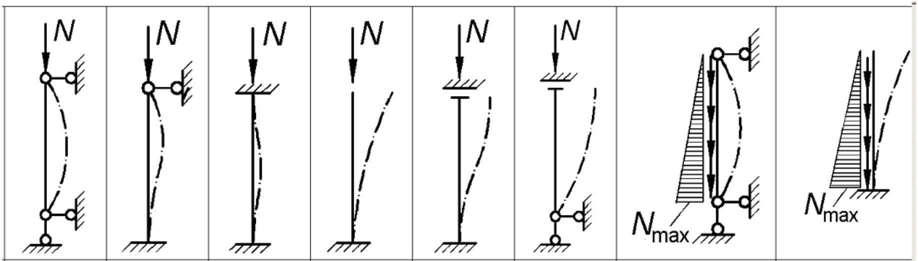
$$l_{ef} = \mu l_c\ , \tag{3.5}$$

where μ is coefficient of calculated column length, determined depending on the conditions of securing their ends and the type of load (Table 3.1);

l_c is the geometric length of a column, its separate section or floor height

Table 3.1

Coefficients of calculated length of columns (risers) [5, table 13.7]



1,0	0,7	0,5	2,0	1,0	2,0	0,725	1,12

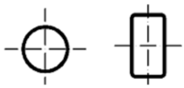
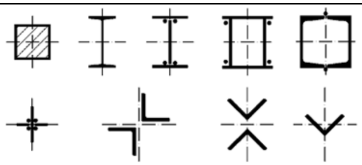
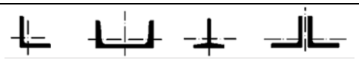
The value of the coefficient δ calculated according to formula (8.5) [5]:

$$\delta = 9{,}87\left(1 - \alpha + \beta\bar{\lambda}\right) + \bar{\lambda}^2, \qquad (3.6)$$

where α and β – coefficients characterizing the initial irregularities shapes and residual stresses and are determined by table 3.2 depending on the type of cross section rod and type of resistance curve ***a, b*** and ***c*** , which are given in Appendix 12.

Table 3.2

The values of the coefficients α and β([5, table 8.1, p. 25]

	*a*	$\alpha = 0{,}03$	$\beta = 0{,}06$
	*b*	$\alpha = 0{,}04$	$\beta = 0{,}09$
	*c*	$\alpha = 0{,}04$	$\beta = 0{,}14$

The value of the coefficient φ calculated by ***formula*** (3.3) should be

$$\varphi \leq 7{,}6/\bar{\lambda}^2 \qquad (3.7)$$

for the type of stability curve ***and*** at $\bar{\lambda}$ ***>3.8*** , stability curve ***b*** at $\bar{\lambda}$ ***>4,4*** , stability curve ***p*** at $\bar{\lambda}$ ***>5.8*** (Appendix 12).

Example 3.2. Determine the holding capacity of a compressed rack made of I-beam No. 20, made of C235 steel, length 2,8 m, which has hinged ends. Reliability coefficient for the responsibility of the building to accept $\gamma_n = 1.0$.

Solution:

1. According to Appendix 9, the calculated resistance of the shaped part is chosen

rolled steel C235: R_y = 230 N/mm^2 .

2. According to Appendix 10, note 5, the coefficient of the working conditions of steel structures γ_c = 1.0.
3. From Appendix 11 (assortment of profiles), we write out some characteristics of I-beam No. 20: cross-sectional area $A_n = 26.8$ cm^2 , a smaller radius of inertia of the cross-section $i_{in} = 2{,}07$ cm.
4. According to the table 3.1 for hinged edges $\mu = 1.0$, therefore

$$l_{ef} = \mu l_c = 1.0 \times 2{,}8\ m = 2{,}8\ m = 280\ cm.$$

5. Flexibility $\lambda = l_{ef} / i = 280 / 2.07 = 135.36$.
6. Conditional flexibility is calculated according to (3.4):

$$\bar{\lambda} = \lambda\sqrt{R_y/E} = 135{,}36\sqrt{230/2{,}06\cdot 10^5} = 4.52.$$

7. According to the table 3.2 for a post of a I-beam profile and a stability curve of the type ***b*** value $\alpha = 0.04, \beta = 0.09$.

8. We calculate the coefficient using formula (3.6). δ

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

$$= 9{,}87\left(1 - 0{,}04 + 0{,}09\times 4{,}52\right) + 4{,}52^2 = 33.92.$$

9. The coefficient of stability is found by (3.3):

$$\varphi = \frac{0{,}5}{\bar{\lambda}^2}\left(\delta - \sqrt{\delta^2 - 39{,}48\bar{\lambda}^2}\right) =$$

$$= \frac{0{,}5}{4{,}52^2}\left(33{,}92 - \sqrt{33{,}92^2 - 39{,}48\times 4{,}52^2}\right) = 0{,}376.$$

10. Let's check the fulfillment of condition (3.7): $\varphi \le 7{,}6/\bar{\lambda}^2$

$$7{,}6/4{,}52^2 = 0{,}372\,.$$

So, we finally accept $\varphi = 0.372$.

11. According to formula (3.2), we determine the holding capacity of a post made of I-beam No. 20:

$$N = \varphi A_n R_y \gamma_c \gamma_n =$$

$$= 0.372\times 26.8\times 100\times 230\times 1.0\times 1.0 = 229300.8\ N = 229.3\ kN,$$

therefore, the holding capacity of the given rack is ***229.3 kN*** .

TASKS FOR INDEPENDENT WORK:

Determine the bearing capacity of a compressed I-beam rack made of steel of a certain class. The length of the rack is optional. The fixing conditions are specified and determined by the coefficient μ(see Table 3.1). Reliability coefficient for the responsibility of the building to accept γ_n ***= 1.0*** .

Second last digit of the cipher	Strength class of rolled steel	The last digit of the cipher	Double-decker room	Coefficient μ	Rack length , m
0	C235	0	20	2.0	3.0
1	C245	1	24	0.7	3.1
2	C255	2	27	1.0	3.2
3	C275	3	33	0.7	3.3
4	C235	4	36	0.5	3,4
5	C245	5	40	2.0	3.5
6	C255	6	45	1.0	3.6
7	C235	7	24	0.7	3,4
8	C245	8	27	0.5	3.5
9	C255	9	20	1.0	2.9

Calculation of elements of steel structures during bending

Depending on the purpose, operating conditions and technical and economic justification, the calculation of bending elements (beams) should be performed without taking into account or taking into account the development of limited plastic deformations according to the division of the cross sections of the elements into three classes, in accordance with p. 5.3.6 [5]. Beams of the 1st class should be used for all types of loads and calculated within the limits of elastic deformations; beams of the 2nd and 3rd classes are recommended to be used under the action of static loads and calculated taking into account the development of limited plastic deformations.

Calculation of the strength of split beams of the 2nd and 3rd classes of I-beam and box sections made of steel with a characteristic resistance of up to 440 N/mm^2 under tangential stresses

$$\tau_y = |Q_y| / A_w \leq 0{,}9 R_s \qquad (3.8)$$

(except for supporting cross-sections) when bending in the plane of greatest stiffness, perform according to formula (9.10) [5]:

$$\frac{M_x \gamma_n}{c_x \beta_r W_{xn,\min} R_y \gamma_c} \leq 1 \, . \qquad (3.9)$$

A_W is the area cross section of the beam wall (determined by Appendix 11 according to the geometric characteristics of the selected profile);

R_S is the calculated shear resistance of steel, determined according to [5, table 7.1]:

$$R_S = 0.58 \, R_{yn} / \gamma_m \, , \qquad (3.10)$$

here R_{yn} is the characteristic resistance of rolled steel, equal to the limit fluidity of steel, determined according to Appendix 9;

γ_m is reliability factor by material, $\gamma_m = 1.05$ [5, tab. 7.2];

β_r is the coefficient. P ry $\tau_y \leq 0{,}5 R_s$, as well as under the action of calculated cross-section of the beam of the bending moment for in the absence of a transverse force, $\beta_r = 1$ ***is taken*** ; in this case in formula (3.9) instead of the coefficient c_x is used

$$c_{xt} = 0.5 \, (1.0 + c_x) \, ; \qquad (3.11)$$

c_x is a coefficient taken according to table 3.3 .

Calculation of the strength in the supporting section of the beams (when $M_x = 0$ and $M_y = 0$) should be performed according to formulas (9.14) [5]:

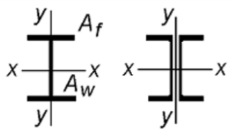
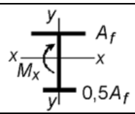
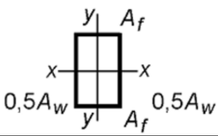
$$\frac{Q_y \gamma_n}{A_w R_s \gamma_c} \le 1\,; \qquad (3.12)$$

$$\frac{Q_x \gamma_n}{2A_f R_s \gamma_c} \le 1\,, \qquad (3.13)$$

A_f is a square cross section of the beam belt. For box cross-section A_f is the total cross-sectional area of two walls.

Table 3.3

Coefficients for calculating elements taking into account the development of plastic deformations [5, table M.1, *fragment*]

<table>
<tr><td></td><td></td><td>A_f / A_w</td><td>c_x</td><td>c_y</td><td>n</td></tr>
<tr><td rowspan="4">1</td><td rowspan="4"></td><td>0,25</td><td>1,19</td><td rowspan="4">1,47</td><td rowspan="4">1,5</td></tr>
<tr><td>0,50</td><td>1,12</td></tr>
<tr><td>1,00</td><td>1,07</td></tr>
<tr><td>2,00</td><td>1,04</td></tr>
<tr><td rowspan="3">2</td><td rowspan="3"></td><td>0,5</td><td>1,40</td><td rowspan="3">1,47</td><td rowspan="3">2,0</td></tr>
<tr><td>1,0</td><td>1,28</td></tr>
<tr><td>2,0</td><td>1,18</td></tr>
<tr><td rowspan="4">3</td><td rowspan="4"></td><td>0,25</td><td>1,19</td><td>1,07</td><td rowspan="4">1,5</td></tr>
<tr><td>0,50</td><td>1,12</td><td>1,12</td></tr>
<tr><td>1,00</td><td>1,07</td><td>1,20</td></tr>
<tr><td>2,00</td><td>1,04</td><td>1,26</td></tr>
</table>

Flexural elements should be calculated for general stability [5, p. 9.4] with a check of the local stability of the walls and waist sheets [5, p. 9.5]. However, there is no need to check the stability of the bending elements if the compressed belt is continuously and reliably fixed with a hard floor.

Calculation of bending elements for stiffness is performed according to the general rules of resistance of materials. In particular, the deflection of a single-span beam loaded with a uniformly distributed linear load is determined from the condition:

$$f = \frac{5}{384}\frac{q_n l^4}{EI} \leq f_u, \quad (3.14)$$

f_u – limit deflection, depends on the size of the span, type design, determined according to Appendix 13.

Example 3.3. Choose a rolled profile made of **C235** steel for a single-span ceiling beam of length 5 m. The beam is subjected to an operational (normative) uniformly distributed load of 30 kN/m, a limit (calculated) load of 36 kN/m. Reliability coefficient for the responsibility of the building to accept $\gamma_n = 1.0$.

Solution:

1. Let's determine the maximum calculated internal forces that arise in a single-span beam from the action of an external load:

$$M_{\max} = \frac{ql^2}{8} = \frac{36 \times 5^2}{8} = 112,5 \; kNm;$$

$$Q_{\max} = \frac{ql}{2} = \frac{36 \times 5}{2} = 90 \; kN$$

2. Calculation of the strength of bending elements is performed according to formula (3.9) :

$$\frac{M_x \gamma_n}{c_x \beta_r W_{xn,\min} R_y \gamma_c} \leq 1,$$

   where you can find the calculated value of the moment of resistance of the beam section:

$$W_{xn,\min} = \frac{M_x \gamma_n}{c_x \beta_r R_y \gamma_c}.$$

3. In the first approximation, we take $c_x = 1.1$; $\beta_r = 1.0$. The calculated resistance of shaped rolled steel C235 is chosen according to Appendix 9: R_y = 230 N/mm^2 . Coefficient of operating conditions of steel structures $\gamma_C = 1.0$ according to Appendix 10, note 5.
4. The calculated value of the moment of resistance of the section:

$$W_{xn,\min} = \frac{112{,}5 \times 10^3 \times 1{,}0}{1{,}1 \times 1{,}0 \times 230 \times 1{,}0} = 444664\ mm^2 = 444.7\ cm^3.$$

5. According to the assortment (appendix 11), we choose I-beam No. 30 with the actual value of the moment of resistance of the section $W_x = 472$ cm^2. The mass of one meter of running I-beam No. 30 is 36,5 kg.
6. Operating load taking into account own weight:

$$q_n = 30 + 0.365 = 30.365\ kN/m.$$

7. Calculated load taking into account own weight:

$$q = 36 + 0.365 \times 1.05 = 36.38\ kN/m\ ,$$

here $\gamma_{fm} = 1.05$ is the reliability factor for the material.

8. The maximum calculated internal forces from the load, taking into account the self-weight of the selected beam:

$$M_{\max} = \frac{ql^2}{8} = \frac{36{,}38 \times 5^2}{8} = 113{,}7\ kNm;$$

$$Q_{\max} = \frac{ql}{2} = \frac{36{,}38 \times 5}{2} = 90{,}95\ kN.$$

9. For I-beam No. 30 (according to Appendix 11), the cross-sectional area of the belt:

$A_f = 135 \times 10.2 = 1377\ mm^2$,

cross-sectional area of the beam wall:

$A_w = 6.5(300 - 2 \times 10.2 - 2 \times 12) = 1661.4\ mm^2$,

10. Ratio $A_f / A_w = 1377 / 1661.4 = 0.83.$

By interpolation according to table 3.3, we find $c_x = 1.1$.

11. Using formula (3.11), we find

$$c_{xt} = 0.5\ (\ 1.0 + c_x\ ) = 0.5\ (\ 1.0 + 1.1\ ) = 1.05\ .$$

12. Let's check the strength according to (3.9):

$$\frac{M_x \gamma_n}{c_x \beta_r W_{xn,\min} R_y \gamma_c} = \frac{113{,}7 \cdot 10^6 \cdot 1{,}0}{1{,}05 \cdot 1{,}0 \cdot 472 \cdot 10^3 \cdot 230 \cdot 1{,}0} = 0{,}997 < 1.0\ ,$$

therefore, the strength of the selected beam made of rolled I-beam No. 30 is span is provided.

13. Calculation of strength in support sections is performed according to (3.12):

$$\frac{Q_y \gamma_n}{A_w R_s \gamma_c} = \frac{90{,}95 \cdot 10^3 \cdot 1{,}0}{1661{,}4 \cdot 129{,}8 \cdot 1{,}0} = 0{,}42 < 1.0\ ,$$

therefore, the strength of the support section of the selected beam is rolled I-beam No. 30 is provided for shear.

14. We calculate the calculated deflection of the beam according to the formula (3.14):

$$f = \frac{5}{384} \frac{q_n l^4}{EI} = \frac{5}{384} \times \frac{30{,}365 \cdot 10^3 \cdot 5^4}{2{,}06 \cdot 10^5 \cdot 7080 \cdot 10^{-8}} = 0.0169\ m = 16.9\ mm\ .$$

15. Limit deflection f_u we determine according to Appendix 13 by

interpolation. For span 5 m

$$f_u = L / 183 = 5000 / 183 = 27 \text{ mm.}$$

$$f = 16{,}9 \text{ mm} < f_u = 27 \text{ mm.}$$

Therefore, the stiffness of the selected beam made of rolled I-beam No. 30 is secured

16. There is no need to check the stability of the beam, since the compressed upper belt is continuously and reliably fixed by the rigid floor covering.

TASKS FOR INDEPENDENT WORK:

Choose a rolling profile made of steel of a certain class for a single-span floor beam. The length of the beam can be taken according to the option. A specified operational (normative) uniformly distributed load and a limit (design) load act on the beam. Reliability coefficient for the responsibility of the building to accept $\gamma_n = 1.0$.

Second last digit cipher	Strength class of rolled steel	The last digit cipher	Beam length, m	Uniformly distributed load, kN/m	
				operational (normative)	marginal (calculated)
0	C245	0	6.0	20	25
1	C255	1	5.1	21	26
2	C235	2	5.2	22	27
3	C235	3	5.3	23	28
4	C245	4	5.4	24	29
5	C255	5	5.5	25	30
6	C245	6	5,6	26	32
7	C255	7	5.7	27	33
8	C235	8	5.8	28	34
9	C245	9	5.9	29	35

3.3. Design of connections of steel structures

Steel building structures are connected by welding, bolts, and rivets.

Welded joints

The most common, about 95%, are welded joints [6, item 4.1] . Welding was invented at the end of the 9th century, improved and introduced by the Ukrainian scientist, academician Yevhen Paton. Further development of welding takes place at the Ukrainian Institute of Electric Welding named after Ye.O. Paton and of world importance.

Advantages of welded joints: high strength and reliability, absence of intermediate parts and holes, ease of execution, saving of metal by 10...20% compared to other types of joints, high level of mechanization and automation of welding processes.

Disadvantages of welded joints: residual deformations and stresses due to uneven heating and cooling of the metal, significant concentration of stresses in the seams and near the seams, which can impair strength under repeated and sign-changing effects, the need for special equipment for welding work.

According to the structural features, there are butt joints (connection of elements placed in the same plane), angular (when welding a seam in the angle formed by the faces of the connected elements) and slotted (when filling the slots in the connected elements with metal) seams.

According to the length, continuous and discontinuous seams are distinguished.

Standards for electric arc welding provide [6, p. 4.2.2] . the following types of welded joints:

- butt joints, performed using butt joints in beams, crossbars, columns of buildings, tanks, gas holders, bunkers, silos, pipelines and other sheet structures. Butt seams in welded joints can be placed perpendicularly or at an angle to the line of action of the force;
- angular, used for fastening mutually perpendicular elements, for example, waist seams of beams and columns, joining ribs, etc.;
- branded, used for fastening mutually perpendicular elements, for example, waist seams of beams and columns, joining ribs, etc.;
- on the run Such connections are formed with the help of angle seams and are used in most assembly joints and quite often in factory conditions.

When designing steel structures with welded joints, the minimum necessary number of welds should be used and their minimum sizes should be assigned. It is necessary to ensure free access to the places where welded joints are made, taking into account the selected type and technology of welding [5, p. 16.1.1] .

Welded butt joints of sheet parts are designed straight with full penetration using lead bars made of the same metal as the main parts [5, item 16.1.4] .

The dimensions of welded angle seams and the design of the connection must meet the requirements [5, p. 16.1.5]:

a) the leg of the angle seam $\boldsymbol{k}_f$ (Fig. 3.1) must satisfy the calculation conditions and be no less than specified in table 3.4 and no less than 4 mm;

b) the leg of the angle seam $\boldsymbol{k}_f$ (Fig. 3.1, a) should not exceed ***1.2t*** , where ***t*** is the smallest of the thicknesses of the welded elements; the leg of the seam laid along the rounded edge of the shaped rolled product with a thickness of ***t*** should not exceed ***0.9t*** ;

c) the estimated length of the angle seam must be at least $\boldsymbol{4k_f}$ and not less than 50 mm.

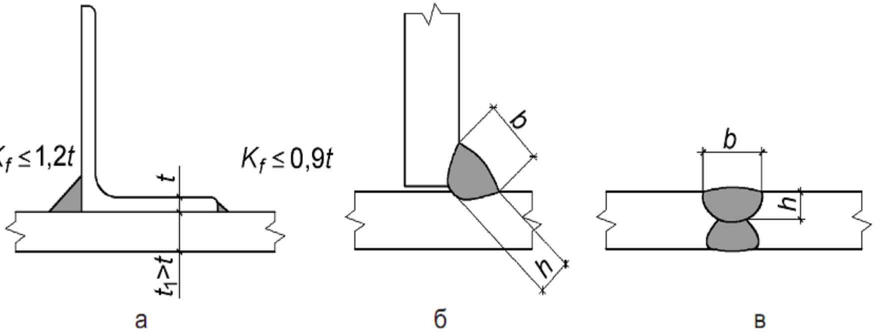


Fig. 3.1. The dimensions of the welds [5, fig. 16.1]

Table 3.4

Minimum legs of welds [5, table 16.1]

Connection type	Type of welding	Characteristic rolling resistance R_{yn} , N/mm^2	The minimum leg of the seam $k_{f,min}$, mm, at the thickness of the thicker element in the joint t_{max}, mm						
			4…5	6…10	11…16	17…22	23…32	33…40	41…80
Branded with two-sided corner seams; false and angular	Manual and mechanized in a mixture of gases	$R_{yn} \le 290$	4	5	6	7	8	9	10
		$290 < R_{yn} \le 390$	4	5	6	7	8	9	10
		$390 < R_{yn} \le 590$	5	6	7	8	9	10	12
	Mechanized in carbon dioxide	$R_{yn} \le 290$	3	4	4	5	5	6	6
		$290 < R_{yn} \le 390$	3	4	5	6	7	8	9
		$390 < R_{yn} \le 590$	4	5	6	7	8	9	10
Branded with one-sided corner seams	Manual and mechanized in a mixture of gases	$R_{yn} \le 390$	5	6	7	8	9	10	12
	Automatic and mechanized in carbon dioxide	$R_{yn} \le 390$	4	5	6	7	8	9	10

Calculation of welded butt joints under the condition of longitudinal force $\boldsymbol{N}$ passing through the center of gravity of the joint is performed according to formula (16.1) [5]:

$$\frac{N\gamma_n}{t_{\min} l_w R_{wy} \gamma_c} \le 1 \,, \qquad (3.15)$$

where t_{min} is the smallest of the thicknesses of the elements in the connection;

R_{wy} – calculated tensile, compressive and bending resistance of butt welds beyond the yield point (see Table 3.5);

l_w is the calculated length of the butt seam, which is equal to its geometric length minus 2 t . In the case when the ends of the seam are removed from the joint (use of removal strips), the estimated length of the seam is equal to its geometric length.

Table 3.5

Formulas for determining calculated resistances
welds [5, tab. 7.3]

Welded connection	A tense state	Characteristics of calculated resistance	Conventional sign	Calculated resistance
Contiguous	Compression Stretching and bending during mechanized or manual welding with physical control of the quality of the seam	Beyond the liquidity limit	R_{wy}	$R_{wy} = R_y$
		By temporary resistance	R_{wu}	$R_{wu} = R_u$
	Stretching and bending during mechanized or manual welding	Beyond the liquidity limit	R_{wy}	$R_{wy} = 0{,}85R_y$
	Landslide		R_{ws}	$R_{ws} = R_s$
With corner seams	Shift (conditional)	In the plane of the deposited metal	R_{wf}	$R_{wf}=0{,}55R_{wun}/\gamma_{wm}$
		Fusion boundaries in the plane of the metal	R_{wz}	$R_{wz} = 0{,}45R_{un}$
Note. The value of the reliability coefficient according to the material of the seam γ_{wm} is taken to be equal to: 1,25 – at $R_{wun} \le 490$ N/mm^2; 1,35 – при 490 N/mm^2 < $R_{wun} \le 620$ N/mm^2.				

Example 3.4. Calculate the butt-welded connection without lead strips of two sheets of **C235** steel in thickness 6 mm and 8 mm width 200 mm. An axial tensile force of 300 kN acts in the connection. Manual welding. Reliability coefficient for the responsibility of the building to accept $\boldsymbol{\gamma_n = 1.0}$

Solution:

1. We perform the calculation according to the formula (3.15):

$$\frac{N\gamma_n}{t_{\min} l_w R_{wy} \gamma_c} \le 1 .$$

2. According to Appendix 9, the calculated resistance of the sheet is chosen

rolled steel C235: $\boldsymbol{R_y}$ = 230 N/mm^2.

According to the table 3.5 $\boldsymbol{R_{wy} = 0.85\, R_y = 0.85 \times 230 = 195.5}$ N/mm^2

3. According to Appendix 10, note 5, the coefficient of the working conditions of steel structures $\boldsymbol{\gamma_c}$ = 1.0.

4. If the seam is straight and without lead strips, then the calculated length of the butt seam: $\boldsymbol{l_w = 200\text{ mm} - 2 \times 6\text{ mm} = 188\text{ mm}}$.

5. Let's check the strength of the seam:

$$\frac{N\gamma_n}{t_{\min} l_w R_{wy} \gamma_c} = \frac{300 \cdot 10^3 \cdot 1{,}0}{6 \cdot 188 \cdot 195{,}5 \cdot 1{,}0} = 1{,}36 > 1.0 ,$$

therefore, the strength of such a connection is insufficient, therefore you need to increase the length of the butt seam and perform it not straight, but oblique.

6. Let 's take a diagonal seam lengthwise 240 mm. A "triangle" was formed, one leg of which 200 mm is the hypotenuse 240 mm, the other leg: $\sqrt{240^2 - 200^2} = 132{,}7$ mm. The angle of inclination of the oblique seam has

   $\sin\alpha = 200/240 = 0{,}833$, $\cos\alpha = 132{,}7 / 240 = 0{,}553$.

7. Estimated length of the seam without output strips:

$$l_w = 240\ mm - 2\times 6\ mm = 228\ mm.$$

8. The tensile force is now applied at an angle to the butt seam and will cause tension (normal) and shear (tangential) in it. Let's check the tensile strength:

$$\frac{N\cdot \sin\alpha \cdot \gamma_n}{t_{\min} l_w R_{wy} \gamma_c} = \frac{300\cdot 10^3 \cdot 0{,}833\cdot 1{,}0}{6\cdot 228\cdot 195{,}5\cdot 1{,}0} = 0{,}934 < 1.0,$$

   therefore, the tensile strength of the butt seam is ensured.

9. Let's check the shear strength:

$$\frac{N\cdot \cos\alpha \cdot \gamma_n}{t_{\min} l_w R_{ws} \gamma_c} < 1.0.$$

   Here $R_{ws} = R_s = 0.58\, R_{yn} / \gamma_m$ according to the formula (3.10).

For **C235** according to Appendix 9 $R_{yn} = 235\ N/mm^2$. $\gamma_m = 1.05$.

$$R_{ws} = R_s = 0.58\, R_{yn} / \gamma_m = 0.58\times 235 / 1.05 = 129.8\ N/mm^2.$$

$$\frac{N\cdot \cos\alpha \cdot \gamma_n}{t_{\min} l_w R_{ws} \gamma_c} = \frac{300\cdot 10^3 \cdot 0{,}553\cdot 1{,}0}{6\cdot 228\cdot 129{,}8\cdot 1{,}0} = 0{,}934 < 1.0,$$

therefore, the shear strength of the oblique butt seam is ensured.

TASKS FOR INDEPENDENT WORK:

Calculate the butt-welded connection of two steel sheets of a certain class of given thickness and width. An axial tensile force acts in the joint. Manual welding. Reliability coefficient for the responsibility of the building to accept $\gamma_n = 1.0$.

Second last digit cipher	Sheet width, mm	Strength class of rolled steel	The presence of output strips	Sheet thickness 1, mm	The last digit cipher	Sheet thickness2, mm	Axial tensile force, kN
0	180	C235	+	6	0	6	200
1	190	C245	–	8	1	8	210
2	200	C255	–	10	2	10	220
3	210	C245	–	12	3	12	230
4	220	C255	+	14	4	14	240
5	230	C235	+	16	5	16	250
6	240	C235	–	14	6	14	260
7	250	C245	–	12	7	12	270
8	260	C255	+	10	8	10	280
9	270	C245	+	8	9	8	290

Bolted connections

Bolted connections are most often used when assembling structures [6, p. 4.1] .

Advantages of bolted connections:

- do not require special equipment;
- do not have a thermal effect on the connected parts,
- bolt holes are relatively small stress concentrators,
- work reliably under conditions of repeated and sign-changing loads.

According to the principle of operation, bolted connections are divided into two types:

1) connections in which there is a shift between the connecting elements (connections on ordinary bolts);

2) connections in which there is no shift between the elements (connections on high-strength bolts).

Connections of the first type include bolted connections of increased accuracy (accuracy class A), normal accuracy (B), rough accuracy (C).

The difference between them lies in the requirements for manufacturing accuracy (quality of processing or processing accuracy), the density of installation in the holes, and the methods of forming the holes for the bolts.

Accuracy class A bolts should be used for connections in which holes are drilled to the design diameter in assembled elements or conductors, or drilled or pressed to a smaller diameter in individual elements followed by drilling to the design diameter in assembled elements. This makes it possible to make a connection with a minimum gap between the bolt rod and the wall of the hole. Therefore, bolts of accuracy class A are sometimes called "clean" bolts. They are made with a diameter tolerance of up to -0.3 mm and are inserted into holes with a gap of up to 0.5 mm.

Bolts of accuracy class B are manufactured with a deviation from the nominal diameter of up to -0.52 mm, and the diameter of the holes for them is taken to be 2 mm larger than the nominal diameter of the bolt (the size of the gap can be up to 2,52 mm).

Bolts of accuracy class C have a diameter tolerance 1 mm. They are installed in holes with a gap of up to 3 mm. The largest discrepancy between the diameter of the hole and the diameter of the bolt is 6 mm. This discrepancy is called "blackness". Therefore, connections on bolts of accuracy classes B and C are also called connections on "black" bolts. Such connections are very deformable and not all bolts are included in the work at the same time

Bolts of coarse and normal accuracy are mostly used in assembly joints for fixing elements.

When shear forces are applied, high-precision bolts and high-strength bolts are used [6, p. 4.3.1] .

The bolts are placed in accordance with the requirements given in the table 3.6.

Table 3.6

Requirements for placement of bolts [5, tab. 16.3]

Characteristics of distance and characteristic resistance of rolling steel of connecting elements	Distance between bolts when placing bolts
1 The distance between the centers of the bolt holes in any direction: a) minimal: at $R_{yn} \le 390$ N/mm^2;	2d
at $R_{yn} > 390$ N/mm^2;	3d
b) maximum in the extreme rows during tension and compression in the absence of corners located along the edge of the connection elements	8d or 12t
c) the maximum in the middle rows, as well as in the extreme rows in the presence of corners located along the edge of the connecting elements: when stretching	16d or 24t
under compression	12d or 18t
The distance from the center of the bolt hole to the edge of the element: a) minimum along the line of action of the force: at $R_{yn} \le 390$ N/mm^2;	1,5d
at $R_{yn} > 390$ N/mm^2;	2,5d
b) across the line of action of the force: with cut edges;	1,5d
with rolling edges;	1,2d
c) maximum;	4d or 8t
d) minimal in a frictional joint at any edge and any effort	1,3d
3 The minimum distance between the centers of the holes along the line of action of the force for bolts arranged in a staggered order	u + 1,5d

Note 1. ***d*** – diameter of the bolt hole; ***t*** is the thickness of the thinnest external element; ***u*** is

the distance between the rows of holes across the line of action of the force.

Note 2. The diameter of the holes is accepted: $\boldsymbol{d = d_b}$ for bolts of accuracy class A; $\boldsymbol{d = d_b}$ + 1mm for bolts of accuracy classes B and C in the structures of supports of overhead power lines (PL), open distribution devices (VRP) and contact networks of transport (KMT), in other cases $\boldsymbol{d = d_b}$ + (1; 2 or 3 mm).

Note 3. In single-bolt connections of lattice elements (braces, risers and struts, except for those that are constantly working in tension), with an element thickness of up 6 mm to 390 N/mm^2 and drilled holes, the distance $\boldsymbol{l_1}$ from the edge of the element to the center of the hole along the line of action of the force is allowed to be from 1.5 $\boldsymbol{d}$ to 1.35 $\boldsymbol{d}$ without tolerance in the direction of reduction during the manufacture of elements, which must be noted in the project.

Note 4. When placing bolts in staggered order at distances not less than specified in item[3], the cross-sectional area of the net element $\boldsymbol{A_n}$ should be determined taking into account its weakening by holes located in the calculated cross-section of the element, which is oriented across the line of action of the force. In connections in which the bolts work mainly in tension, as a rule, bolts of accuracy classes B and C or high strength should be used.

The requirements for bolts under different conditions of their use are given in the table 3.7.

Table 3.7

Requirements for bolts under different conditions of their use [5, table D.3]

Bolt strength class and requirements for them according to GOST 1759.4 in structures that			
are not calculated on endurance		are calculated on endurance	
when working bolts on			
stretch or shear	cut	stretch or shear	cut
5,6	5,6	5,6	5,6
–	5.8	–	–
8.8	8.8	8.8	8.8
10.9	10.9	10.9	10.9
–	12.9	–	12.9
Note 1. High-strength bolts according to GOST 22356 made of 40X "select" steel are used in the same structures as bolts of strength classes 10.9. **Note 2.** In connections that are not designed for durability, it is allowed to use bolts with a subhead of accuracy class B and C according to DSTU GOST 7795, DSTU GOST 15590.			

The calculated force that can be absorbed by one bolt should be determined depending on the type of stress state according to the formulas:

- at the cut (conditional) of the bolt according to the formula (16.12) [5] :

$$N_{bs} = R_{bs} A_b n_s \gamma_b \gamma_c \gamma_n; \tag{3.16}$$

- when the metal of the connecting elements is crushed in the hole according to the formula (16.13) [5] :

$$N_{bp} = R_{bp} d_b \Sigma t_{min} \gamma_b \gamma_c \gamma_n; \tag{3.17}$$

- when stretching a bolt according to the formula (16.1 4) [5] :

$$N_{bt} = R_{bt} A_{bn} \gamma_c \gamma_n, \tag{3.1 8 }$$

where R_{bs}, R_{bp}, R_{bt} – calculated resistances of one-bolt connections to shear, bending, and tension, respectively (see Tables 3.8, 3.9);

R_{bun} is the characteristic resistance of the steel of the bolts, which is taken to be equal to the temporary resistance σ_{in} accordance with state standards and technical conditions for bolts, see table 3.9;

$R_{b\,in\,n}$ is the characteristic resistance of the steel of the bolts, which is taken to be equal to the yield strength σ_t according to state standards and technical conditions for bolts, see table 3.9;

n_s is the number of calculated cut planes of one bolt, pcs.;

γ_c is coefficient of working conditions, which is determined according to Appendix 10;

γ_b – coefficient of working conditions of the bolted connection from table 3.10;

A_b is the cross-sectional area of the bolt rod, determined by
by the formula $A_b = \pi d_b^2/4$ or from the table 3.11;

d_b – outer diameter of the bolt rod;

Σt_{min} – the smallest total thickness of the elements in the connection, which crumple in one direction;

A_{bn} is the net cross-sectional area of a threaded bolt, which is accepted according to the table 3.11.

Table 3.8

Formulas for determining calculated resistances bolted joints [5, tab. 7.4]

A tense state	Conven-tional sign	Design resistance of a single-bolt joint in shear, tension, and bending for bolts					
		strength class					high-strength steel grades 40X «select»
		5.6	5.8	8.8	10.9	12.9	
Cut	R_{bs} [1)]	$0{,}42R_{bun}$	$0{,}42R_{bun}$	$0{,}40R_{bun}$	$0{,}40R_{bun}$	$0{,}35R_{bun}$	$0{,}37R_{bun}$
Stretching	R_{bt} [1), 2)]	$0{,}75R_{bun}$	$0{,}75R_{bun}$	$0{,}68R_{bun}$	$0{,}60R_{bun}$	$0{,}5R_{bun}$	$0{,}50R_{bun}$
Crimping: bolts of accuracy class A	R_{bp} [3)]	$1{,}60R_u$					
bolts of accuracy class B and C		$1{,}35R_u$					

[1)] Value R_{bs} , R_{bt} for bolts of strength classes 8.8, 10.9 and made of steel 40X «select» given for bolts without coating (for example, without galvanizing, aluminizing).

[2)] Value R_{bt} specified for bolts with additional subsequent tempering at temperatures 650°C.

[3)] Value R_{bp} indicated for connecting elements made of steel with a yield strength of up to 440 N/mm^2 and at $R_{bun} > R_{un}$

Table 3.9

Characteristic supports of steel bolts and calculated supports of one-bolt shear and tension joints, N/mm^2 [5, tab. D.4]

Strength class of bolts	R_{bun}	R_{byn}	R_{bs}	R_{bt}
5,6	500	300	210	225
5.8	500	400	210	-
8.8	800	640	320	435
10.9	1000	900	400	540
12.9	1200	1080	420	-
40X "select"	1100	990	405	550

Table 3.10

Coefficients of the working conditions of the bolted connection [5, table 16.4]

Characteristics		Characteristic rolling resistance of steel of connecting elements, R_{yn}, N/mm^2	Value a/d, s/d	The value of the coefficient γ_b
bolt connection	tense state			
One-bolt, precision class bolts A, B and C or high-strength	Cut	-	-	1.0
	Crumpling	R_{yn} <290	1.5≤ a/d ≤2	0.4 a/d + 0.2
			1.35≤ a/d <1.5	a/d – 0.7
		290≤ R_{yn} ≤390	1.5≤ a/d ≤2	0.5 a/d
			1.35≤ a/d <1.5	0.67a/d – 0.25
		R_{yn} >390	a/d ≥2.5	1.0
multi-bolt, precision class bolts A, B* and C* or high-strength	Cut	-	-	1.0
	Crumpling	R_{yn} <290	1.5≤ a/d ≤2	0.4 a/d + 0.2
			2≤ s/d <2.5	0.4 s/d
		290≤ R_{yn} ≤390	1.5≤ a/d ≤2	0.5 a/d
			2≤ s/d ≤2.5	0.5 s/d - 0.25
		R_{yn} >390	a/d ≥3	1.0
			s/d >3	1.0

Note 1. a is the distance from the edge of the element to the center of the nearest hole along the line of action of the force; d – diameter of the bolt hole; s is the distance between the centers of the holes along the line of action of the force.

* To calculate a multi-bolt connection for shear and crumpling when using bolts of classes B, C, high-strength bolts with unregulated tension at all values of the characteristic resistance R_{yn} of the rolling of the connected elements, the value of the coefficient γ_b should be multiplied by 0.9.

Note 2. For the calculation of the multi-bolt joint for buckling, the value of the coefficient γ_b is taken as less than the one calculated with the accepted values of a, d, s.

Table 3.11

Cross-sectional areas of bolts, cm^2, according to GOST 1759.4 [5, table D.8]

d_b, mm	16	(18)	20	(22)	24	(27)	30	36	42	48
A_b	2.01	2.54	3.14	3.80	4.52	5.72	7.06	10.17	13.85	18.09
A_{bn}	1.57	1.92	2.45	3.03	3.53	4.59	5.61	8,16	11.20	14.72

Note 1. Cross-sectional areas of bolts with a diameter exceeding 48 mm are accepted in accordance with GOST 24379.0.

Note 2. The dimensions given in parentheses are not recommended for use in structures other than overhead power line (PL) supports, open switchgear (OSG) and contact networks of transport (CNT).

When acting on a bolted connection of a longitudinal force ***N*** passing through the center of gravity of the connection, it should be assumed that this force is distributed evenly between the bolts [5, p. 16.2.10]. Therefore, the number of bolts is determined by the formula (16.15) [5]:

$$n \geq \frac{N\gamma_n}{N_{b,min}}, \tag{3.19}$$

where $N_{b,min}$ is the smallest of the values N_{bs} , N_{bp} , N_{bt} , calculated according to (3.16),
(3.17), (3.18).

In bolted connections of one element with another through gaskets or other intermediate elements, as well as in elements with a one-sided overlay, the number of bolts calculated by formula (3.19) must be increased by 10% [5, p. 16.2.14]. In the bolted connections of protruding angle shelves or channels with the help of spacers, the number of bolts attaching the spacer to this shelf must be increased by 50% compared to the calculation.

Example 3.5. Design a bolted connection of two sheets of size 200×10 mm from steel **C245** . An axial tensile force of 200 kN acts in the connection. Reliability coefficient according to the responsibility of the building $\boldsymbol{\gamma_n = 1.0}$.

Solution:

1. To make the connection, it is advisable to use bolts of accuracy class B. In the bolted connection, shearing and wrinkling will occur. Since the two sheets are directly connected to each other without overlays, only one bolt shear plane will occur between the connecting sheets. That is, $\boldsymbol{n_s = 1}$.
2. Taking into account the data of tables 3.8 and 3.9, we choose the minimum class of bolts: **5.6** .
3. From the table 3.11 we choose bolts of the smallest diameter $\boldsymbol{d_b = 16}$ mm, for which $\boldsymbol{A_b = 2.01}$ cm^2 , $\boldsymbol{A_{bn} = 1.57}$ cm^2 . Under the bolt with a diameter 16 mm of accuracy class B, there should be a hole with a diameter of 18 mm.
4. According to Appendix 10, note 5, the coefficient of the working conditions of steel structures $\boldsymbol{\gamma_c = 1.0}$.
5. According to Appendix 9, for rolled sheet steel **C245 ,** we determine $\boldsymbol{R_{yn}}$ = 245 N/mm^2 ; $\boldsymbol{R_y}$ = 240 N/mm^2 .
6. According to the table 3.10 for a multi-bolt connection in shear, the coefficient is $\boldsymbol{\gamma_b = 1.0}$, which should be multiplied by $\boldsymbol{0.9}$, taking note 1 into account.
7. According to the table 3.9 for bolts of class **5.6 ,** the calculated

shear resistance is $\boldsymbol{R_{bs} = 210}$ N/mm^2.

8. According to the formula (3.16), the calculated force that can be absorbed by one bolt during shear (conditional) is:

$$\boldsymbol{N_{bs} = R_{bs} \times A_b \times n_s \times \gamma_b \times \gamma_c \times \gamma_n = 210 \times 201 \times 1 \times 0.9 \times 1.0 \times 1.0 = 37989} \text{ N}.$$

9. According to the formula (3.19), the number of bolts in the connection can be pre-set:

$$n \geq \frac{N\gamma_n}{N_{b,min}} = \frac{200 \cdot 10^3 \cdot 1{,}0}{37989} = 5.2.$$

We accept **6** bolts, which we place in two rows of 3 pcs.

10. According to the table 3.6 the location of the bolts should be as follows: the distance between the centers of the holes for the bolts along the line of action of the force ≥2d (36 mm),≤ 8 d (144 mm),≤ 12 t (120 mm), we accept 40 mm;

    the distance between the centers of the bolt holes across the line of force action ≥2d (36 mm),≤ 8 d (144 mm),≤ 12 t (120 mm), we accept 50 mm;

    the distance from the center of the bolt hole to the edge of the element along the line of action of the force ≥is 1.5d (27 mm), ≤4d (72 mm), ≤8t (80 mm), we accept 30 mm;

    the distance from the center of the bolt hole to the edge of the element across the line of action of the force ≥is 1.5d (27 mm), ≤4d (72 mm), ≤8t (80 mm), we take 50 mm.

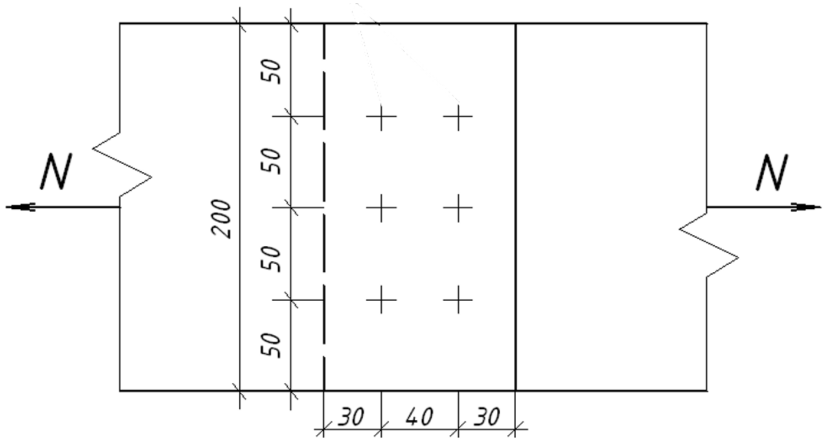


Fig. 3.2. Before designing a bolted connection

Therefore, the slope area is 100 mm.

11. In the designed bolted connection, crumpling of the connecting sheets may occur. To calculate according to the formula (3.17), you need to determine the coefficient γ_b . For table 3.10, let's set the values: a = 30 mm, d = 18 mm, s = 40 mm, a / d = 30/18 = 1.7; s / d = 40/18 = 2.2.

$$\gamma_b = 0.4 \text{ a / d} + 0.2 = 0.867;$$

$$\gamma_b = 0.4 \text{ s / d} = 0.889.$$

    Finally , $\gamma_b = 0.867 \times 0.9 = 0.78$ (according to Table 3.10, taking into account Notes 1, 2).

12. According to v. 3.8 $\boldsymbol{R_{bp} = 1.35\ R_u = 1,35 \times 360 = 486\ N/mm^2}$, where $\boldsymbol{R_u = 360\ N/mm^2}$ according to Appendix 9 for C245.
13. Calculation of crumpling is carried out according to the formula (3.17):

    $\boldsymbol{N_{bp} = R_{bp}\, d_b\, \Sigma t_{min}}\, \gamma_b\, \gamma_c\, \gamma_n =$

$= \boldsymbol{486} \times \boldsymbol{16} \times \boldsymbol{10} \times \boldsymbol{0.78} \times \boldsymbol{1.0} \times \boldsymbol{1.0} = \boldsymbol{60652.8}$ N = $\boldsymbol{60.6}$ kN.

14. From the condition of crumpling strength, the number of bolts in the connection:

$$n \geq \frac{N\gamma_n}{N_{b,min}} = \frac{200 \cdot 1{,}0}{60{,}6} = 3.3.$$

So, we leave **6** bolts from the condition of strength per section.

15. check the tensile strength of the sheets in the section weakened by the bolt holes:

$$\sigma = \frac{N}{(b - kd) \cdot t} = \frac{200 \cdot 10^3}{(200 - 3 \times 18)10} = 136{,}98 \text{ N/}mm^2 < R_b \gamma_c = 240 \text{ N/}mm^2.$$

So, strength is ensured.

TASKS FOR INDEPENDENT WORK:

To design a bolted connection of two sheets of steel of a certain class of given thickness and width. An axial tensile force acts in the joint. Reliability coefficient for the responsibility of the building to accept $\gamma_n = 1.0$.

Second last digit cipher	Sheet width, mm	Strength class of rolled steel	The last digit cipher	Presence of connecting plates	Sheet thickness, mm	Axial tensile force, kN
0	210	C255	0	+	16	210
1	220	C245	1	–	14	220
2	230	C235	2	–	12	230
3	240	C255	3	–	10	240
4	250	C245	4	–	8	250
5	260	C235	5	+	6	260
6	270	C255	6	–	8	270
7	280	C245	7	–	10	280
8	290	C235	8	–	12	290
9	300	C255	9	+	14	300

APPENDIXES

Appendix 1

STRENGTH CLASSES OF WOOD

[2, appendix B, table B.1, p. 81]

Characteristic values of strength, stiffness and density for coniferous wood

No	Strength classes	C14	C16	C18	C20	C22	C24	C27	C30	C35	C40	C45	C50
1	2	3	4	5	6	7	8	9	10	11	12	13	14
Strength value, N/mm^2													
1	Bend, $\boldsymbol{f}_{m,k}$ i	14	16	18	20	22	24	27	30	35	40	45	50
2	Stretch along, $\boldsymbol{f}_{t,0,k}$ a	8	10	11	12	13	14	16	18	21	24	27	30
3	Stretch across, $\boldsymbol{f}_{t,90,k}$	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
4	Compression along, $\boldsymbol{f}_{c,0,k}$ a	16	17	18	19	20	21	22	23	25	26	27	29
5	Compression across, $\boldsymbol{f}_{c,90,k}$	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 twisting, $\boldsymbol{f}_{v,k}$ c	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0

No	Strength classes	C14	C16	C18	C20	C22	C24	C27	C30	C35	C40	C45	C50
Stiffness value, N/mm^2													
7	The modulus of elasticity along $E_{0,\ mean}$ [a,b]	7000	8000	9000	9500	10000	11000	11500	12000	13000	14000	15000	16000
8	Transverse modulus of elasticity, $E_{90,\ mean}$ [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 value, kg/m^3													
10	density, ρ_k	290	310	320	330	340	350	370	380	400	420	440	460

Note. The values of the characteristic tensile strength across the direction of the fibers, the characteristic strength under the action of chipping and torsion differ from the calculated values according to DSTU EN338, but only the values presented here should be used in the calculation.

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

[b] Characteristic value of shear modulus $G_{R,k}$ 1.0 N/mm^2 can be accepted for all strength classes during calculations. For chipping stresses, it is necessary to take the value of the shear modulus, which is equal to $G_{R,mean} = 0.10\ G_{mean}$.

[with] For the characteristic value of stiffness $E_{0.05}$, $E_{90.05}$ and $G_{0.05}$ calculated values are:

$E_{0.05} = 2/3\ E_{0,mean}$, $E_{90.05} = 2/3\ E_{90,mean}$ and $G_{0.05} = 2/3\ G_{mean}$.

Appendix 2

PROPERTIES OF MATERIALS

[2, appendix A, table A.1, p. 79]

The value of the conversion factor, which takes into account the duration of the load and the temperature and humidity conditions of operation $\boldsymbol{k_{mod}}$

Building material	Load class by duration of action	Operating classes		
		1	2	3
Solid wood, glulam, beam glulam, plywood, orthogonal glulam	permanent	0.60	0.60	0.50
	lasting	0.70	0.70	0.55
	of medium duration	0.80	0.80	0.65
	short term	0.90	0.90	0.70
	instant	1.10	1.10	0.90
Chipboard, cement chipboard (CSP), fiberboard (Type HB.NLA2)	permanent	0.30	0.20	–
	lasting	0.45	0.30	–
	of medium duration	0.65	0.45	–
	short term	0.85	0.60	–
	instant	1.10	0.80	–
OSB plates (OSB/2, OSB/3 and OSB/4)	permanent	0.40	0.30	–
	lasting	0.50	0.40	–
	of medium duration	0.70	0.55	–
	short term	0.90	0.70	–
	instant	1.10	0.90	–
fiberboard (Type MBH.LA2)	permanent	0.20	0.15	–
	lasting	0.40	0.30	–
	of medium duration	0.60	0.45	–
	short term	0.80	0.60	–
	instant	1.10	0.80	–

Appendix 3

Examples of assigning a load class by duration of action

[2, tab. 5.2, p. 10]

Load class by duration of action	Load examples
Permanent	Own weight of structures; weight of various types of backfill; weight of permanent partitions, stationary equipment, communications; structures of suspended ceilings; soil pressure
Long-lasting	Loads during cargo storage; load from water in tanks
Medium duration	Snowy; evenly distributed useful loads on ceilings and balconies; temporary loads in parking garages and traffic zones; effects related to changes in humidity; weight of non-stationary equipment; weight of temporary partitions
Short-term	Temporary load on stairs; temporary concentrated loads; horizontal loads on partitions and parapets; temporary loads for maintenance of roofs and people's stay; transport loads; impacts from vehicles and mechanisms; cargo storage; wind load
instant	Incidental influences

Appendix 4

Reliability coefficients according to the material of γ_M

[2, tab. 6.1, p. 12]

Basic combinations	γ_M
Solid wood	1.3
Glued wood	1.25
Glued veneer (), plywood,	1.2
Chipboard, fiberboard, MDF	1.3
connection	1.3
Metal toothed plates	1.25
Random combinations	1.0

Appendix 5

The ratio of calculated and geometric lengths for risers

[2, tab. 9.1, p. 21]

Element type	Type of load	l_{ef}/l
Hinged end support	The axial force is concentrated at one end of the riser	1.0
	Evenly distributed load along the length of the element	0.73
Hard pinching of one end	The axial force is concentrated at one end of the riser	2,2
	Evenly distributed load along the length of the element	1.2
Hard clamping of one end and hinged resistance of the other	The axial force is concentrated at one end of the riser	0.8
The ratio between the calculated length l_{ef} and the geometric length l are given for elements with the specified type of torsion-limiting bearing and loaded at the center of gravity.		

Appendix 6

The ratio of calculated and geometric lengths for beams

[2, tab. 9.2, p. 24]

Element type	Type of load	l_{ef}/l
Hinged end support	Permanent moment	1.0
	Evenly distributed load	0.9
	Concentrated force inside the span	0.8
Hard pinching of one end	Evenly distributed load	0.5
	Concentrated force at the free end	0.8
The ratio between the calculated length l_{ef} and the span l corresponds to elements with the specified type of torsion-limiting bearing and loaded at the center of gravity. If the load is applied near the compressed edge of the element, then l_{ef} should be increased by ***2h*** or reduced by ***0.5h*** for the load of the stretched zone of the beam.		

Appendix 7

Value of limit deflections of wooden structures

[2, tab. 10.2, p. 29]

Element	w_{inst}	w_{fin}	$w_{net,fin}$
A beam on two supports	l / 300 to l / 500	l / 150 to l / 300	l / 150 to l / 350
Cantilever beam	l / 150 to l / 250	l / 75 to l / 150	l / 125 to l / 175
Covering or overlapping elements with ceilings with plaster or plasterboard			
A beam on two supports			l / 250
Cantilever beam			l / 125
Covering or ceiling elements without a ceiling with plaster or plasterboard			
A beam on two supports			l / 150
Cantilever beam			l / 75
l is the span of the beam or the length of the console			

Appendix 8

Recommended formulas for determining the deflection of hinged beams of rectangular cross-section from bending and increasing coefficients that take into account the effect of shear

[2, tab. 10.3, p. 29]

Load case	Deflection from bending, mm	Increasing coefficient during shear
A uniformly distributed load along the length of a hinged beam is equal to the full load q_d	In the middle of the span $\frac{5ql_{ef}^4}{32E_{o,mean}bh^3}$	$\left[1+0,96\frac{E_{o,mean}}{G_{o,mean}}\left(\frac{h}{l_{ef}}\right)^2\right]$
…	…	…

Appendix 9

Characteristic and calculated resistances in tension, compression and bending for sheet, wide-band universal and shaped rolled products in accordance with the strength classes of rolled steel

[5, table D.2, p. 131]

Strength class of rolled steel	Roll thickness [1)], mm	Characteristic resistance [2)], N/mm², rolled				Design resistance [3)], N/mm², rolled			
		sheet, broadband universal		shaped		sheet, broadband universal		shaped	
		R_{yn}	R_{un}	R_{yn}	R_{un}	R_y	R_u	R_y	R_u
1	2	3	4	5	6	7	8	9	10
C235	from² to 20	235	360	235	360	230	350	230	350
	over 20 to 40	225	360	225	360	220	350	220	350
	over 40 to 100	215	360	-	-	210	350	-	-
	more than 100	195	360	-	-	190	350	-	-
C245	from² to 20	245	370	245	370	240	360	240	360
	over 20 to 30	-	-	235	370	-	-	230	360
C255	from² to 3.9	255	380	-	-	250	370	-	-
	from 4 to 10	245	370	255	380	240	360	250	370
	over 10 to 20	245	370	245	370	240	360	240	360
	over 20 to 40	235	370	235	370	230	360	230	360
C275	from² to 10	275	380	275	390	270	370	270	380
	over 10 to 20	265	370	275	380	260	360	270	370
C285	from² to 3.9	285	390	-	-	280	380	-	-
	from 4 to 10	275	390	285	400	270	380	280	390
	over 10 to 20	265	380	275	390	260	370	270	380
C295	up to 100	295	430	295	430	285	420	285	420
C325	over 10 to 20	325	470	325	470	315	460	315	460
	over 20 to 40	305	460	305	460	300	450	300	450
	over 40 to 60	285	450	-	-	280	440	-	-
	over 60 to 80	275	440	-	-	270	430	-	-
	over 80 to 100	265	430	-	-	260	420	-	-
C345	from² to 10	345	490	345	490	335	480	335	480
	over 10 to 20	325	470	325	470	315	460	315	460
	over 20 to 40	305	460	305	460	300	450	300	450
C345K	from 4 to 10	345	470	345	470	335	460	335	460
C355	from 8 to 50	355	450	-	-	350	440	-	-
C375	from² to 10	375	510	375	510	365	500	365	500
	over 10 to 20	355	490	355	490	345	480	345	480
	over 20 to 40	335	480	335	480	325	470	325	470

End of Appendix 9

1	2	3	4	5	6	7	8	9	10
C390	from 4 to 50	390	540	-	-	380	530	-	-
C390K	from 4 to 30	390	540	-	-	380	530	-	-
C420	from 4 to 4 p.m	420	540	-	-	410	530	-	-
	from 16 to 40	400	530	-	-	390	515	-	-
	from 40 to 63	390	530	-	-	380	515	-	-
	from 63 to 80	370	520	-	-	360	505	-	-
C440	from 4 to 30	440	590	-	-	430	575	-	-
	over 30 to 50	410	570	-	-	400	555	-	-
C460	from 4 to 4 p.m	460	570	-	-	445	555	-	-
	from 16 to 40	440	560	-	-	430	545	-	-
	from 40 to 63	430	560	-	-	420	545	-	-
	from 63 to 80	410	540	-	-	400	530	-	-
C490	from 8 to 50	490	590	-	-	475	575	-	-
C500	from[3] to 50	500	590-770	-	-	485	575-750	-	-
	from 50 to 100	480		-	-	465		-	-
C590	from 10 to 36	590	685	-	-	540	617	-	-
C590K	from 10 to 40	590	685	-	-	540	617	-	-
C620	from[3] to 50	620	700-890	-	-	600	680-865	-	-
	from 50 to 100	580		-	-	565		-	-

1) The thickness of the shelf is taken as the thickness of the shaped roll.
2) The guaranteed values of the yield strength and
temporary resistance.
3) Values of calculated resistances are obtained by dividing the characteristic resistances
resistances
on reliability coefficients by material γ_t with rounding up to 5 N/mm^2.
For strength classes of rolled steel C235-C500; C620 is taken into $\boldsymbol{\gamma_{account} = 1.025}$,
and for classes C590; C590K is taken into $\boldsymbol{\gamma_{account} = 1.1}$.

Appendix 10

Coefficients γ_c of operating conditions of steel structures

[5, table 5.1, p. 17]

	Structural elements	Coefficient of working conditions γ_c
1	2	3
1	Beams of solid section and compressed elements of floor trusses under the halls of theaters, clubs, cinemas, under stands, under premises of shops, book stores and archives, etc. with a temporary load that does not exceed the weight of the floor	0.90
2	Columns of public buildings and supports of water towers	0.95
3	Columns of one-story industrial buildings with overhead cranes	1.05
4	Compressed main elements (except for supporting) grids of a folded T-shaped section from two angles in welded trusses of coatings and overlaps when calculating the stability of the specified elements with flexibility $\lambda \geq 60$	0.80
5	Tensions, pulls, braces, suspensions when calculating the strength in the cross-section without relaxation	0.90
6	Cross-sections of structural elements made of steel with a yield strength of up to 440 N/mm^2 , carrying a static load, when calculating the strength in the cross-section weakened by bolt holes (except for friction joints): - solid beams and columns; - rod structures of coatings and ceilings	 1.10 1.05
7	Compressed lattice elements of spatial lattice structures, made of single equal-shelf angles according to Figure 13.3 [5, p. 63], which are attached with one shelf (for unequal-shelf angles - with a larger shelf): a) directly to the belts with the help of welds or two or more bolts, which are installed along the angle: - braces (Figure 13.3, a) [5, p.63];	0.90
	- struts (Figure 13.3, b, c, e) [5, p. 63];	0.90
	- braces (Figure 13.3, c, d, d, e) [5, p. 63];	0.80
	b) directly to the belts with the help of one bolt or through a socket, regardless of the type of connection	0.75

1	2	3
8	Elements of flat trusses from single angles, compressed elements made of single angles, which are attached with one shelf (for uneven shelf angles - with a smaller shelf), with the exception of the elements listed in item 7 of this table	0.75
9	Support plates made of steel with a yield strength of up to 390 N/mm^2, bearing a static load, thickness, mm: a) up to 40, inclusive;	1.20
	b) more than 40 to 60 inclusive;	1.15
	c) more than 60 to 80 inclusive	1.10

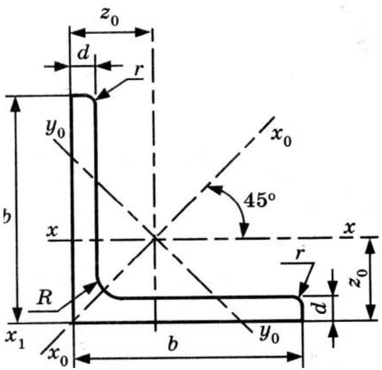
Note 1. Coefficients $\gamma_c < 1.0$ should not be taken into account in the calculation, except for the calculations indicated in notes 2, 3.
Note 2. When calculating the strength in a section weakened by bolt holes, the coefficients given in positions 6 and 1, 6 and 2, 6 and 5 should be considered together.
Note 3. When calculating base plates, the coefficients given in positions 9 and 2, 9 and 3 should be taken into account together.

Note 4. When calculating the connections, the coefficients γ_c for the elements listed in positions 1 and 2 should be taken into account together with the coefficient of the operating conditions of the connection γ_b.

Note 5. In cases not determined by these Norms, γ_c = 1.0 is accepted in the calculation formulas.

Appendix 11

Equal angle according to GOST 8509-93



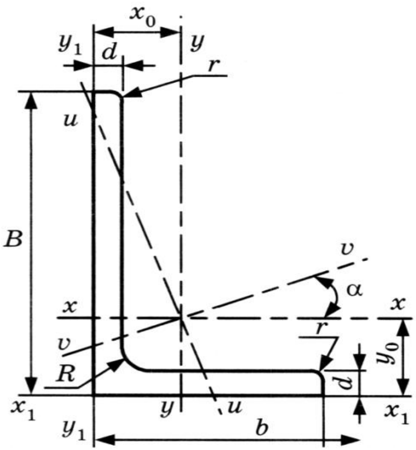
b is the width of the shelf
d - shelf thickness
R is the radius of the internal rounding
r is the radius of rounding of the shelf
I is the moment of inertia
i is the radius of inertia
z_o is the distance from the center of gravity to the outer edge of the wall

No profile	Dimensions, mm				Cross-sectional area, cm²	Additional values for axes								Mass of 1 run.m, kg
						x is x		$x_o - x_o$		y_o - y_o		$x_1 - x_1$	z_o, see	
	b	***d***	***R***	***r***		I_x, cm ^{4}	i_x, see	I_{xo}, cm ^{4}	i_x, see	I_{yo}, see ^{4}	i_y, see	I_{x1}, cm ^{4}		
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
5,6	56	3.5	6	2	3.86	11.6	1.73	18.4	2.18	4.80	1.12	20.3	1.50	3.03
		4			4.38	13.1	1.73	20.8	2.18	5.41	1.11	23.2	1.52	3.44
		5			5.41	16.0	1.72	25.4	2.17	6.59	1.10	29.3	1.57	4.25
6.3	63	4	7	2,3	4.96	18.9	1.95	29.9	2.46	7.81	1.25	33.1	1.69	3.89
		5			6.13	23.1	1.94	36.6	2.44	9.52	1.25	41.7	1.74	4.81
		6			7.28	27.1	1.93	42.9	2.43	11.2	1.24	50.2	1.78	5.71

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
7	70	4.5	8	2.7	6.20	29.0	2.16	46.0	2.72	12.0	1.39	50.9	1.88	4.87
		5			6.86	31.9	2.16	50.7	2.72	13.2	1.39	56.7	1.90	5.39
		6			8.15	37.6	2.15	59.6	2.70	15.5	1.38	68.3	1.94	6.40
		7			9.42	43.0	2.14	68.2	2.69	17.8	1.37	80.3	1.99	7.39
		8			10.7	48.2	2.12	76.4	2.67	20.0	1.37	91.9	2.02	8.40
7.5	75	5	9	3	7.39	39.5	2.31	62.8	2.91	16.4	1.49	69.7	2.02	5.80
		6			8.78	46.6	2.30	73.9	2.90	19.3	1.48	83.9	2.06	6.89
		7			10.1	53.3	2.30	84.6	2.89	22.1	1.48	97.8	2.10	7.93
		8			11.5	59.8	2.28	94.9	2.87	24.8	1.47	113	2.15	9.03
		9			12.8	66.1	2.27	105	2.86	27.5	1.47	127	2.18	10.0
8	80	5.5	9	3	8.63	52.7	2.47	83.6	3.11	21.8	1.59	93.3	2.17	6.77
		6			9.38	57.0	2.47	90.4	3.10	23.5	1.58	102	2.19	7.36
		7			10.8	65.3	2.46	104	3.10	27.0	1.58	119	2.23	8.48
		8			12.3	73.4	2.44	116	3.07	30.3	1.57	137	2.27	9.66
9	90	6	10	3.3	10.6	82.1	2.78	130	3.50	34.0	1.79	145	2.43	8.32
		7			12.3	94.3	2.77	150	3.49	38.9	1.79	169	2.47	9.66
		8			13.9	106	2.76	168	3.49	43.8	1.78	194	2.51	10.9
		9			15.6	118	2.75	186	3.45	48.6	1.77	219	2.55	12.2
10	100	6.5	12	4	12.8	122	3.09	193	3.88	50.7	1.99	214	2.68	10.0
		7			13.8	131	3.08	207	3.87	54.2	1.98	232	2.71	10.8
		8			15.6	147	3.07	233	3.86	60.9	1.98	265	2.75	12.2
		10			19.2	179	3.05	284	3.85	74.1	1.96	333	2.83	15.1
		12			22.8	209	3.03	331	3.81	86.9	1.96	402	2.91	17.9

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
10	100	14	12	4	26.3	237	3.00	375	3.78	99.3	1.94	472	2.99	20.6
		16			29.7	264	2.98	416	3.74	112	1.94	542	3.06	23.3
11	110	7	12	4	15.2	176	3.40	279	4.28	72.7	2.19	309	2.96	11.9
		8			17.2	198	3.39	315	4.28	81.8	2.18	353	3.00	13.5
12.5	125	8	14	4.6	19.7	294	3.86	467	4.87	122	2.49	515	3.36	15.5
		9			22.0	327	3.86	520	4.86	135	2.48	581	3.40	17.3
		10			24.3	360	3.85	571	4.85	149	2.48	649	3.45	19.1
		12			28.9	422	3.82	670	4.81	174	2.45	782	3.53	22.7
		14			33.4	482	3.80	764	4.78	200	2.45	917	3.61	26.2
		16			37.8	539	3.78	853	4.75	224	2.43	1061	3.68	29.7
14	140	9	14	4.6	24.7	466	4.34	739	5.47	192	2.79	819	3.78	19.4
		10			27.3	512	4.33	814	5.46	211	2.78	910	3.82	21.4
		12			32.5	602	4.30	957	5.43	248	2.78	1096	3.90	25.5
16	160	10	16	5.3	31.4	774	4.96	1229	6.26	319	3.19	1355	4.30	24.6
		11			34.4	844	4.95	1341	6.24	348	3.18	1495	4.35	27.0
		12			37.4	913	4.94	1450	6.23	376	3.17	1634	4.39	29.4
		14			43.3	1046	4.91	1662	6.20	431`	3.15	1911	4.47	34.0
		16			49.1	1175	4.89	1866	6.16	485	3.14	2191	4.55	38.5
		18			54.8	1299	4.87	2061	6.13	537	3.13	2472	4.63	43.0
		20			60.4	1419	4.85	2248	6,10	589	3.12	2756	4.70	47.4

Equal angle according to GOST 8510-86

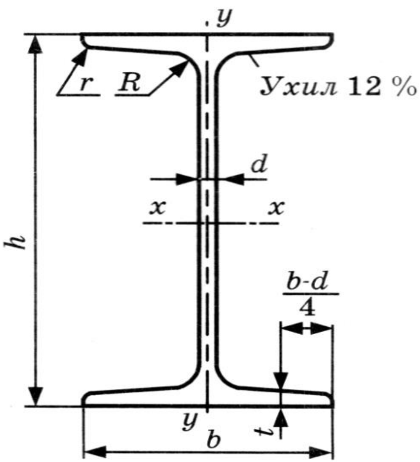


B is the width of the larger shelf
b is the width of the smaller shelf
d **-** shelf thickness
R is the radius of the internal rounding
r is the radius of rounding of the shelf
I is the moment of inertia
i is the radius of inertia
x_0, y_0 is the distance from the center of gravity to the outer edge of the wall

No profile	Dimensions, mm					Cross-sectional area, cm^2	Additional values for axes										Angle of inclination of the axis , tgα	Mass 1 m, kg
							x is x		***y is y***		$x_1 - x_1$	***in*** $_1$ ***- in*** $_1$	***and - and***	***v - v***	Center of gravity			
	B	***b***	***d***	***R***	***r***		I_x, cm^4	i_x, see	I_y, cm^4	i_y, see	I_{x1}, cm^4	I_{y1}, cm^4	I_u, cm^4	I_v, cm^4	x_0, see	y_0, see		
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
6.3 / 4	63	40	4	7.0	2	4.04	16.3	2.01	5.16	1.13	32.9	8.51	3.08	18.4	0.91	2.03	0.397	3.17
			5			4.98	19.9	2.00	6.26	1.12	41.4	10.8	3.72	22.4	0.95	2.08	0.396	3.91
			6			5.90	23.3	1.99	7.28	1.11	49.8	13.1	4.35	26.2	0.99	2.12	0.393	4.63
			8			7.68	29.6	1.96	9.15	1.09	66.8	17.9	5.57	33.2	1.07	2.20	0.386	6.03
7 / 4.5	70	45	5	8	3	5.07	25.3	2.23	8.25	1.28	51.0	13.6	4.86	28.7	1.03	2.25	0.407	3.98
			5			5.59	27.8	2.23	9.05	1.27	56.9	15.2	5.35	31.5	1.05	2.28	0.406	4.39

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
7.5 / 5	75	50	5	8	3	6,11	34.8	2.39	12.5	1.43	69.7	20.9	7.27	40.0	1.17	2.39	0.436	4.80
			6			7.25	40.9	2.38	14.6	1.42	84.1	25.2	8.46	47.0	1.21	2.44	0.435	5.69
			8			9.47	52.4	2.35	18.5	1.40	113	34.3	10.8	60.1	1.29	2.52	0.430	7.43
8 / 5	80	50	5	8	3	6.36	41.6	2.56	12.7	1.41	84.6	20.8	7.61	46.7	1.13	2.60	0.387	4.99
			6			7.55	49.0	2.55	14.8	1.40	102	25.1	8.81	55.0	1.17	2.65	0.386	5.93
9 / 5,6	90	56	6	9	3	7.86	65.3	2.88	19.7	1.58	132	32.2	11.8	73.2	1.26	2.92	0.384	6.17
			6			8.54	70.6	2.88	21.2	1.58	145	35.2	12.7	79.1	1.28	2.95	0.384	6,10
			8			11.2	90.9	2.85	27.1	1.56	194	47.8	16.3	102	1.36	3.04	0.380	8.79
10 / 6.3	100	63	6	10	3	9.59	98.3	3.20	30.6	1.79	198	49.9	18.2	111	1.42	3.23	0.393	7.53
			7			11.1	113	3.19	35.0	1.78	232	58.7	20.8	127	1.46	3.28	0.392	8.71
			8			12.6	127	3.17	39.2	1.76	266	67.6	23.4	143	1.50	3.32	0.391	9.89
			10			15.5	154	3.15	47.1	1.74	333	85.8	28.3	173	1.58	3.40	0.387	12.2
11 / 7	110	70	7	10	3	11.4	142	3.53	45.6	2.00	286	74.1	27.0	161	1.58	3.55	0.402	8.95
			7			12.3	152	3.52	48.7	1.99	309	80.2	28.8	172	1.60	3.57	0.402	9.66
			8			13.9	172	3.52	54.6	1.98	353	92.0	32.2	194	1.64	3.61	0.400	10.9
12.5 / 8	125	80	7	11	4	14.1	227	4.01	73.7	2.29	454	119	43.3	257	1.80	4.01	0.407	11.1
			8			16.0	256	4.00	83.0	2.28	518	137	48.9	290	1.84	4.05	0.406	12.6
			10			19.7	312	3.98	100	2.25	650	173	58.6	353	1.92	4.14	0.404	15.5
			12			23.4	365	3.95	117	2.24	782	211	69.8	412	2.00	4.22	0.400	18.4
14 / 9	140	90	8	12	4	18.0	364	4.50	120	2.58	727	194	70.4	414	2.03	4.49	0.411	14.1
			10			22.2	444	4.47	146	2.56	910	246	86.1	504	2.12	4.58	0.409	17.4
16 / 10	160	100	9	13	4	22.9	60	5.14	186	2.85	1223	300	110	682	2.23	5.19	0.391	18.0
			10			25.3	667	5.13	204	2.84	1359	336	121	750	2.28	5.23	0.390	19.9
			12			30.0	784	5.11	239	2.82	1633	406	142	881	2.36	5.32	0.388	23.6
			14			34.7	897	5.08	272	2.80	1909	477	163	1006	2.43	5.40	0.385	27.2
18 / 11	180	110	10	14	5	28.3	952	5.80	276	3.12	1930	444	165	1063	2.44	5.88	0.375	22.2
			12			33.7	1123	5.77	324	3.10	2324	538	194	1253	2.52	5.97	0.374	26.5

I-beam according to GOST 8239-89

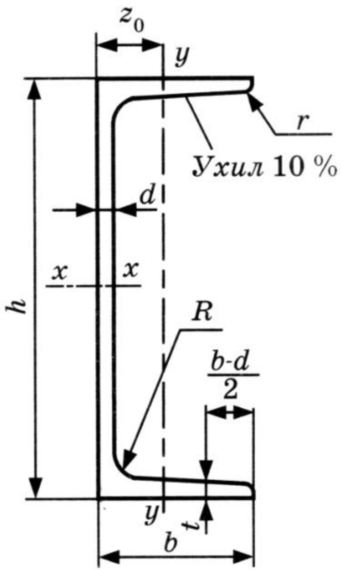


h is the height of the gable
b is the width of the shelf
d - wall thickness
t - the average thickness of the shelf
R is the radius of the internal rounding
r is the radius of rounding of the shelf
I is the moment of inertia
W is the moment of resistance
S is the static moment of the half section
i is the radius of inertia

No profile	Dimensions, mm						Cross-sectional area, cm^2	Additional values for axes							Mass of 1 run.m, kg
								x is x				***y is y***			
	h	***b***	***d***	***t***	***R***	***r***		I_x, cm^4	W_x, cm^3	i_x, see	S_x, cm^3	I_y, see ^{4}	W_{in}, cm^3	i_y, see	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
10	100	55	4.5	7.2	7.0	2.5	12.0	198	39.7	4.06	23.0	17.9	6.49	1.22	9.4
12	120	64	4.8	7.3	7.5	3.0	14.7	350	58.4	4.88	33.7	27.9	8.72	1.38	11.5
14	140	73	4.9	7.5	8.0	3.0	17.4	572	81.7	5.73	46.8	41.9	11.5	1.55	13.7
16	160	81	5.0	7,8	8.5	3.5	20.2	873	109	6.57	62.3	58.6	14.5	1.70	15.9

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
18	180	90	5.1	8.1	9.0	3.5	23.4	1290	143	7.42	81.4	82.6	18.4	1.88	18.4
18a	180	90	5.1	8.3	9.0	3.5	25.4	1430	159	7.50	89.8	114	22.8	2.12	19.9
20	200	100	5.2	8.4	9.5	4.0	26.8	1840	184	8.29	104	115	23.1	2.07	21.0
20a	200	100	5.2	8.6	9.5	4.0	28.9	2030	203	8.38	114	155	28.2	2.32	22.7
22	220	110	5.4	8.7	10.0	4.0	30.6	2550	232	9,13	131	157	28.6	2.27	24.0
22a	220	110	5.4	8,9	10.0	4.0	32.8	2790	254	9.22	143	206	34.3	2.51	25.7
24	240	120	5,6	9.5	10.5	4.0	34.8	3460	289	10.0	163	198	34.5	2.39	27.3
24a	240	115	5,6	9.8	10.5	4.0	37.5	3800	317	10.1	178	260	41.6	2.63	29.4
27	270	125	6.0	9.8	11.0	4.5	40.2	5010	371	11.2	210	260	41.5	2.54	31.6
27a	270	125	6.0	10.2	11.0	4.5	43.2	5500	407	11.3	229	337	50.0	2.79	33.9
30	300	135	6.5	10.2	12.0	5.0	46.5	7080	472	12.3	268	337	49.9	2.69	36.5
30a	300	135	6.5	10.7	12.0	5.0	49.9	7780	518	12.5	292	436	60.1	2.96	39.2
33	330	145	7.0	11.2	13.0	5.0	53.8	9840	597	13.5	339	419	59.9	2.79	42.2
36	360	140	7.5	12.3	14.0	6.0	61.9	13380	743	14.7	423	516	71.1	2.89	48.6
40	400	155	8.0	13.0	15.0	6.0	71.4	18930	947	16.3	540	666	85.9	3.05	56.0
45	450	160	8.6	14.2	16.0	7.0	83.0	27450	1220	18.2	699	807	101	3.12	65.2
50	500	170	9.5	15.2	17.0	7.0	97.8	39290	1570	20.0	905	1040	122	3.26	76.8
55	550	180	10.3	16.5	18.0	7.0	114	55150	2000	22.0	1150	1350	150	3.44	89.5
60	600	190	11.1	17.8	20.0	8.0	132	75450	2510	23.9	1450	1720	181	3.61	103.6
65	650	200	12.0	19.2	22.0	9.0	153	101400	3120	25.7	1800	2170	217	3.77	120.1
70	700	210	13.0	20.8	24.0	10.0	176	134600	3840	27.7	2230	2730	260	3.94	138.2
70a	700	210	15.0	24.0	24.0	10.0	202	152700	4360	27.5	2550	3240	309	4.00	158.6
70 b	700	210	17.5	28.2	24.0	10.0	234	175300	5010	27.4	2940	3910	373	4.09	183.7

U-bar according to GOST 8240-97 (DSTU 3436-96)



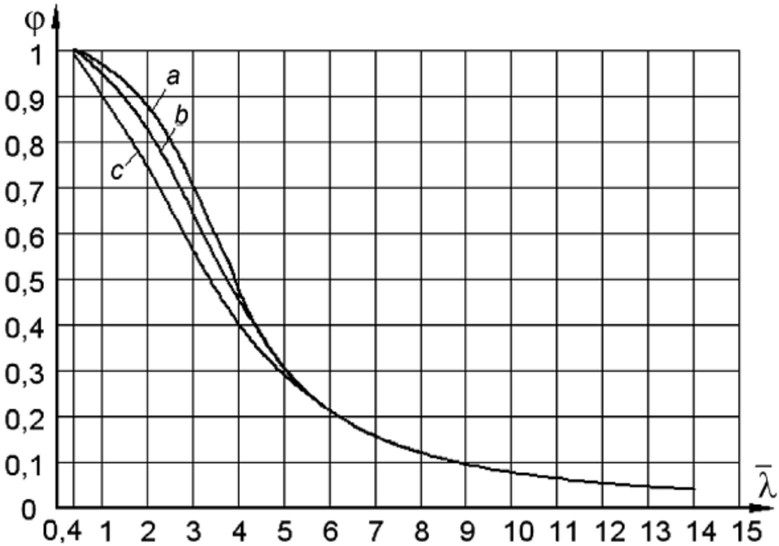
h is the height of the channel
b is the width of the shelf
d - wall thickness
t - the average thickness of the shelf
R is the radius of the internal rounding
r is the radius of rounding of the shelf
I is the moment of inertia
W is the moment of resistance
S is the static moment of the half section
i is the radius of inertia
z_o is the distance from the ***y – y axis*** to the outer face of the wall

No profile	Dimensions, mm						Cross-sectional area, cm^2	Additional values for axes								Mass of 1 run. m, kg
								x is x				***y is y***			z_o, see	
	h	***b***	***d***	***t***	***R***	***r***		I_x, cm^4	W_x, cm^3	i_x, see	S_x, cm^3	I_y, see ^{4}	W_{in}, cm^3	i_y, see		
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
5	50	32	4.4	7.0	6.0	2.5	6.16	22.8	9,12	1.92	5.59	5.61	2.75	0.954	1.16	4.84

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
6.5	65	36	4.4	7.2	6.0	2.5	7.51	48.6	15.0	2.54	9.00	8.70	3.69	1.08	1.24	5.90
8	80	40	4.5	7.4	6.5	2.5	8.98	89.4	22.4	3.16	13.3	12.8	4.76	1.19	1.31	7.05
10	100	46	4.5	7.6	7.0	3.0	10.9	174	34.8	4.00	20.4	20.4	6.46	1.37	1.44	8.56
12	120	52	4.8	7,8	7.5	3.0	13.3	304	50.7	4.78	29.6	31.2	8.52	1.53	1.54	10.4
14	140	58	4.9	8.1	8.0	3.0	15.6	491	70.1	5.61	40.8	45.4	11.0	1.71	1.67	12.2
14a	140	62	4.9	8.7	8.0	3.0	17.0	545	77.9	5.66	45.1	57.5	13.3	1.84	1.87	13.3
16	160	64	5.0	8.4	8.5	3.5	18.1	747	93.4	6.42	54.1	63.3	13.8	1.87	1.80	14.2
16a	160	68	5.0	9.0	8.5	3.5	19.5	823	103	6.50	59.4	78.8	16.4	2.01	2.00	15.3
18	180	70	5.1	8.7	9.0	3.5	20.7	1090	121	7.26	69.8	86.0	17.0	2.04	1.94	16.2
18a	180	74	5.1	9.3	9.0	3.5	22.2	1190	132	7.32	76.1	105	19.9	2.17	2.13	17.4
20	200	76	5.2	9.0	9.5	4.0	23.4	1520	152	8.06	87.8	113	20.4	2.20	2.07	18.4
20a	200	80	5.2	9.7	9.5	4.0	25.2	1670	167	8,14	95.9	139	24.3	2.35	2.28	19.8
22	220	82	5.4	9.5	10.0	4.0	26.7	2110	192	8.89	110	151	25.2	2.38	2.21	21.0
22a	220	87	5.4	10.2	10.0	4.0	28.8	2330	212	8.99	121	187	30.0	2.55	2.46	22.6
24	240	90	5,6	10.0	10.5	4.0	30.6	2900	242	9.74	139	208	31.6	2.61	2.42	24.0
24a	240	95	5,6	10.7	10.5	4.0	32.9	3180	265	9.83	151	254	37.2	2.78	2.67	25.8
27	270	95	6.0	10.5	11.0	4.5	35.2	4160	308	10.9	178	262	37.3	2.73	2.47	27.6
30	300	100	6.5	11.0	12.0	5.0	40.5	5810	387	12.0	224	327	43.7	2.84	2.52	31.8
33	330	105	7.0	11.7	13.0	5.0	46.5	7980	484	13.1	281	410	51.8	2.97	2.59	36.5
36	360	110	7.5	12.6	14.0	6.0	53.4	10820	601	14.2	350	513	61.7	3.10	2.68	41.9

Appendix 12

Stability curves for determining the stability coefficientφ [5, fig. F.1]



Appendix 13

Limit deflections f_u for the most common designs

Structural elements	f_u	Load
Beams, trusses, crossbars, purlins, slabs, decking (including transverse ribs of slabs and decking) of coverings and ceilings, open for inspection, with spans **L** , m:		Permanent and temporary long-lasting
L ≤1	L / 120	
L = 3	L / 150	
L = 6	L / 200	
L = 24 (12)	L / 250	
L = 36 (24)	L / 300	

1. For consoles, instead of **L , a** double departure is accepted.
2. For intermediate values of **L** , the deflection is determined by linear interpolation.
3. including are given in parentheses .6 m

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CONTENT

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Uzhehov Serhii Olehovych, Uzhehova Olha Anatoliivna, Rotko Svitlana Volodymyrivna, Zadorozhnikova Iryna Viktorivna, Kysliuk Dmytro Yaroslavovych, Chapiuk Oleksandr Serhiiovych

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