MINISTRY OF HIGHER EDUCATION, SCIENCE AND INNOVATION OF THE REPUBLIC OF UZBEKISTAN

NAMANGAN ENGINEERING CONSTRUCTION INSTITUTE

FACULTY OF CONSTRUCTION

" CIVIL ENGINEERING " DEPARTMENT

WOOD STRUCTURES

STUDY METHODOLOGY COMPLEX

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Faculty of CONSTRUCTION

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Study methodology complex was created based on the requirements of the sample program on the subject of “Wood constructions”.

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It was discussed at the ___ meeting of the "Architecture " department of the SMC "__"______ 2024 and was recommended for discussion at the faculty council.

Head of the department: ________ dots. I.Axmedov

It was discussed and recommended for use at the Council of the Faculty of Construction of SMC (___ ____ 2024).

Chairman of the faculty council: ________ prof. M.Mansurov

Agreed:

Head of the educational methodological department________ _____________

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I. INTERACTIVE EDUCATIONAL METHODS USED IN TEACHING THE MODULE

I.1. “Insert” method

The purpose of the method: This method is used to facilitate students' acceptance of the new information system and the assimilation of knowledge, and this method also serves as a memory exercise for students.

Procedure for implementing the method:

-Before the lesson, the teacher prepares the input text in the form of a handout or presentation, which covers the main concepts of the subject;

- a text explaining the essence of a new topic is distributed to learners or shown in the form of a presentation;

- Learners get to know the text individually and express their personal views through special symbols. When working with the text, students are advised to use the following special characters:

Table 1

“Insert” method

Signs	Information	Acquaintance information given izo h
"V" is familiar information		
? - I did not understand this information, I need an explanation.		
"+" this information is new to me.		
"-" against this opinion or this information?		

At the end of the specified time, the information that is unfamiliar and incomprehensible to the students will be analyzed and explained by the teacher, and their essence will be fully explained. The training ends after answering the questions.

I.2. Problem teaching technology

Problem-based learning technology is a developmental learning technology that stimulates the process of active learning and forms a logical sequential style of thinking. This technology is one of the most effective technologies in high school. The essence of problem-based teaching is the teacher's management of new knowledge acquisition by organizing problem situations in students' learning and solving educational (better, life) issues, problems and questions.

The problem situation in its nature and content is similar to the logical sequence in the teaching of subjects. The principle of sequence consists in the fact that the knowledge acquired in the previous stages, that is, education, is taken into account when determining the content of education at each stage of higher level of education. Similarly, if new knowledge in a situation is linked to previous knowledge, it is considered a problematic situation.

The activity of knowing, searching consists of the following stages:

1. Problem situation.

2. Learning problem.

3. Search for a solution to the problem.

4. Solve the problem and check it.*The problem solve schedule*

Formulation of the problem:	Final summary:	
Forming subproblems	Content of solutions	Conclusions
1	*2*	*3*
1.		
2.		
3.		

Procedures and regulations

1. To work in a group to solve a problem and write a presentation sheet - 20 min.
2. Presentation of the problem solution - 8 min. up to
3. Team discussion, forming conclusions - 10 min. up to
4. Mutual evaluation - 1 min.

Evaluation indicators and criteria

Each group evaluates the presentation of other groups, sums up the points according to the criteria.

Evaluation indicators and criteria	Maximum score	1 - group	2 - group	3 - group

Solutions:	**1.2**			
- correct formulation of the problem and sub-problem;	0.4			
- matching the solution to the form of the problem and sub-problem;	0.4			
- logicality, accuracy, brevity of conclusions.	0.4			
Presentation:	**1.4**			
- accuracy and comprehensibility of answers;	1.0			
- the activity of each group participant (questions, additions).	0.4			
Regulations	**0.4**			
Total score	**3.0**			

Summative assessment of group work

Group	1	2	3	General score	Grade (total score divided by 2) 2.2 - 3 points - "excellent" 1.2 - 2 points - "good" 0.5 - 1.1 points - "satisfactory"
1	-				
2		-			
3			-		

I.3. "Case-study" method

"Case-study" is an English word ("case" - specific situation, event, "stadi" - to study, analyze) aimed at carrying out teaching based on the study and analysis of specific situations is a method. This method was first used in Harvard University in 1921 in order to use practical situations in the study of economic management sciences. In a case, open information or a specific event can be used as a situation for analysis. Case actions include: Who, When, Where, Why, How, What.

The work stages	**Activity shape and content**
Step 1: Introduction to the case and its information supply	– individual audio-visual work; – familiarization with the case (in text, audio or media form); – generalization of information; – information analysis; – identifying problems

Step 2: Clarifying the case and setting the educational task	– individual and group work; – determining the priority hierarchy of problems; – determination of the main problem situation
Step 3: Searching for a solution to the educational task by analyzing the main problem in the case, developing ways to solve it	– individual and group work; – development of alternative solutions; – analysis of opportunities and obstacles of each solution; – choosing alternative solutions
Step 4: Formulation and justification of the case solution, presentation.	– individual and group work; – substantiating the possibilities of implementing alternative options; – preparation of creative project presentation; – clarification of practical aspects of the final conclusion and solution of the situation

I.4. "Severe attack of thoughts" method

The essence of the "Severe attack of ideas" method is as follows:

- helping to realize the personal potential of each student performing certain tasks among the team;
- is to create in students the ability to put forward an idea against the opinion expressed by a certain community (group).

The above-mentioned "Severe Attack of Thoughts" method can be successfully applied in the course of training organized in social, humanitarian and natural sciences.

During the application of the method, the following situations occur:

1) to achieve thorough assimilation of certain theoretical knowledge by students;
2) saving time;
3) encourage each student to be active;
4) forming their ability to think freely. Training based on the use of this method is organized in several stages. They are:

Stages of using the "Severe Attack of Thoughts" method

Step 1: Forming small groups that include students who are close to each other in spirit and are equal in number	Step 2. Identifying the objectives arising from the nature of the task or assignments assigned to the groups
Step 3: Development of certain ideas by groups (solution of tasks)	Step 4: Discuss the solutions to the tasks, classify them into categories according to their correct solutions.
Step 5: Re-categorization of the solutions of tasks, that is, their evaluation based on criteria such as correctness, time spent to find a solution, clear and clear statement of solutions	Step 6: Discussing certain critical comments made in the initial steps regarding the solutions of tasks and coming to a unified conclusion about them

I.5. Discussion exercise

Controversial practical exercise management levers

moderator takes on all the tasks - managing the stages of the discussion, confirming the validity and correctness of the answers, defining the terms and concepts used, using the relationships correctly , etc. Correctly manage the distribution of presentations.

one who determines and fully evaluates the reports of the parties : relevance, scientific aspect, **logicality** and clear presentation of the issues, clear presentation of the conclusions.

Competitors form a competitive process between accepted research. He can not only criticize the main position of the speaker, but also find weaknesses or mistakes in his ideas and offer his own decisive points.

Expert - evaluates the productivity of all discussions, including the opinions expressed by the participants of the discussion, conclusions made, proposals and hypotheses.

Debate regulation transfer order

1. Starter lecture topic and of speakers presentations announcement does _
2. The lecture is 5 minutes continue is enough
3. Reviewer - 2 minutes .
4. The opponent is a lecture topic according to thoughts 1-3 minutes present is enough

5. Collective discussion - 5-10 minutes .

Evaluation indicators and criteria

Evaluation indicators and criteria (in points)	**Debate participants**			
	Speakers			
	1	2	3	4
Content of the lecture (2.5):				
- compatibility with the topic (1.5);				
- rationality, accuracy (0.5);				
- brevity of conclusions (0.5);				
Use of infrastructure technologies (visibility) - (0.9).				
Regulations (0.6)				
Total (4.0)				
	Reviewers			
The lecture description (3.0)				
- of the lecture strong sides identify (1,2)				
- of the lecture weak sides identify (1,2)				
Regulations (0.6)				
Total (3.0)				
	Opponents, participants			
Questions:				
- for each (0.3)				
In addition				
- for each (0.3)				
- by essence (0.3)				
- Total (3.0)				

Debate to the participants note

1. Discussion is not a sum of relations, but a method of problem solving.
2. Don't talk too much and let others do the talking.
3. In order to achieve the goal, restrain your emotions and speak in detail.
4. Study the situation of your opponents and treat them with respect.
5. Be critical and considerate of the opinions expressed by your opponents.
6. Do not deviate from the subject of the discussion and speak with the right approach.

I.6. "Brainstorming"

"Brain Storming" method has universal application. This method was first used by Osborne (USA) in 1963. The task of "Brainstorming" is to create new ideas with the help of a microgroup, or the strength of the microgroup as a whole is greater than the sum of the strengths of its individual members. "Brainstorming" encourages problem solvers to generate more ideas, including incredible and even fantastic ones. The more ideas there are, the more likely at least one of them is the same term. This is the principle behind brainstorming. "Brainstorming" is used to create a bank of ideas for the most appropriate solution to a problem or task.

"Brainstorming" is conducted according to the following rules:

- the opinion should be expressed as loudly as possible without any restrictions;
- any opinion can be said, it is accepted.
- ideas are not explained, they are said directly related to the task;
- Criticism or discussion of said ideas is not allowed until suggestions are stopped;
- an expert group or tape recorder will record all the said proposals.

After the "brainstorming" is stopped, the group of experts discusses all the ideas (opinions) mentioned and chooses the most suitable one. "Brainstorming" can be conducted individually or in pairs (triads) in lectures, in practical and seminar sessions, in microgroups of 4-6 people, as well as in groups and individually. "Brainstorming" creates conditions for increasing students' activity in training and for everyone to search for the most optimal solution to the topic.

In mastering the topic, it can be organized by clarifying words and phrases that are used in the topic in the form of ideas for "Brainstorming", but that the student has certain ideas about the subjects he has previously mastered.

I.7. Bloom's taxonomy

Bloom's questions. Observations and analysis of pedagogical literature show that an important factor in the development of students' thinking ability is the questions the teacher asks them and students to each other. It is noted that 80-85 percent of the questions that teachers ask students require only evidential knowledge, and they limit themselves to repeating (doing) what they remember in their answers. It is probably because of this, that the knowledge acquired by students is often bookish, and it is not a secret that they face serious difficulties in their practical application.

So, what question can be added to the list of questions that develop thinking skills? In our opinion, only a question, the correct answer of which is not clearly stated in the educational literature or not told by the teacher, makes the student think. An example of such questions can be the questions known as "Bloom's questions" in world pedagogy and corresponding to the six levels of mastery: knowledge, understanding, application, analysis, synthesis and evaluation. For example, questions such as: "Why?", "Compare?", "Divide into components?", "What are the most important features?", "How would you solve this?", "What is your attitude to this?" encourages thinking at the level of higher intellectual operations (analysis, synthesis, evaluation). Or, after reading a passage from the text, it is appropriate to ask the following questions that encourage students to think: "How can you title this passage?", "Five points from the passage that fully convey its content find a word?", "What question would you ask the author?".

When thinking about the question that the teacher asks the students, it should be clear, concise, understandable and concise, and only one learning element (concept, law, rule, etc.) should be asked per question. it should be emphasized that it is necessary. It is also important to use key words and phrases related to the topic or text in the content of the given questions. Categories of Bloom's Taxonomy

I.8. Cluster formation

Cluster - an English word that means a cluster. Sorting information into clusters is an interactive pedagogical strategy that develops multivariate thinking, the skills of making connections between the studied concepts (events, events), helps students to think freely and openly about a topic. will give. Clustering can be used to stimulate thinking during the invitation, comprehension, and reflection phases of a lesson. Basically, it encourages new ideas and new thinking on a particular topic. The sequence of creating a cluster is as follows:

• write the main word or sentence on a large sheet of paper in the middle of the blackboard;

• write words or sentences that you think are related to this topic ("brainstorming");

- make connections between concepts and ideas;
- write down all the options you remember.

II. THEORETICAL MATERIALS

Lecture 1. Introduction. The history of the development of wood structures.

Wood is the oldest building material, as old as the construction art itself. The dialectical law of the development of technology in a spiral, in which each new turn opens up more and more possibilities of this material. The history of the use of wooden structures extends back many centuries and is lost in the depths of centuries. In primitive society, wood played a particularly important role in construction, which is explained by the abundance of forest resources, the relatively easy processing of wood and its high strength. Primitive people built small primitive dwellings on the ground and piles, small fences and bridges out of wooden trunks with stone axes. The simplest hut-like structures were made of timber, which was worked with imperfect stone implements. Apparently, this period should be called the beginning of construction.

The first constructions of the truss-beam system were primitive. They used natural forks of trunks to support the beams, and vines, bows, leather straps, etc. were used to secure them. Improvements in the technique of rolling roofing allowed for the construction of single- and double-pitch roofs. Sectional houses of the Trypilian culture (4th millennium B.C.) are an example of that. However, the flowering of log architecture, born with a stone axe, began much later. Among the earliest written sources containing information about the wooden construction is the work of the ancient Greek historian Herodotus (5th century BC). According to this source in central and eastern Europe were built wooden cities with high fortress walls and temples.

The metal axe and log-cutting, as a new reliable way of connecting logs, were the main engines of progress in wooden architecture. The log cabin allowed wooden structures to grow upward, to expand and taper with a tent, and it adopted the struts of the flat roof structures. With a broad iron carpenter's axe, they learned how to hew "four edged" timbers. The first floorboards and planks were pounded with metal wedges, and to obtain "planking" (flat board) the humpback was hewn with an axe.

A particularly high level of building art was characteristic of ancient Rome, where rational stone and wooden structures were created. Ancient Roman builders built wooden houses, temples and bridges over large rivers. For example, Caesar's legions built a large bridge over the Rhine River. Roman builders erected multi-span arched and girder bridges, used wooden constructions of ceilings of public buildings. Outstanding wooden temples built in the Middle Ages in China and Japan with the use of bamboo wood survived to this day.

In the countries of northern Europe wood frame construction was widely developed, and wooden roof rafters were widely used there as well. In Norway up to the present time wooden churches dating back to XII-XIII centuries have been preserved.

Wooden constructions were especially widespread in our forest-rich country. In the Middle Ages practically all dwelling houses, palaces, the majority of temples and fortresses were built of wood with the walls of round logs. In the X century in Novgorod an oak 13-domed church of St. Sophia was erected. The first towers of the Moscow Kremlin and the walls connecting them were erected of oak timber in the XIII century. The small Church of the Resurrection of Lazarus in Murom, built in the XIV century, has survived to this day.

A true revolution in woodworking was the use of the saw. Sawn timber and board became the main building materials. Sharply reduced waste in the processing of logs allowed the use of wood where there was little of it.

In the XVI-XVIII centuries, mainly in the north and east of the country continued intensive construction of dwelling houses and churches with log walls made of round logs. In 1551 the first large-scale construction of a whole wooden fortress of Sviyazhsk on the bank of the Volga River at the mouth of the Sviyaga River was carried out. Fortress walls 3 versts long with 18 towers, 370 houses, benches and towers were made near Uglich. After the test assembly they were dismantled again, floated on the Volga to the mouth of the Sviyaga, and there was a new fortress, which served as a base for the troops of Ivan the Terrible during the siege and capture of Kazan.

At the end of the XVII century near Moscow, in the village of Kolomenskoye, a magnificent palace of Tsar Alexei Mikhailovich was built. A model of this palace is in the museum of the Kolomensky Reserve. In 1714 was built and survived a remarkable 22-domed wooden Transfiguration church in the village of Kizhi. In 1738 was built a wooden spire of 72 m high tower of Adimiralty in Petersburg. In the XVIII century began the widespread construction of wooden core structures of beams, logs and planks.

The further process of mechanization of wood processing as a building material is obliged to the invention of a circular saw in 1799, which allowed to increase sawing speed considerably. Around the same time the first planing machines appeared, replacing the planers that had long been used in carpentry.

At the end of the XVIII century. P. Kulibin developed an original design for a huge wooden bridge over the Neva River in St. Petersburg with a span of 300 m. The bridge had a combined system and consisted of a number of flexible arches and stiff arched trusses. A 0.1 m model was made, its tests showed the rightness and sufficient stability of the construction. It was impossible to build a life-size model due to the lack of construction methods for such large structures. Kulibin's idea of combined constructions was used later on in the solutions of large building structures.

At the beginning of the XIX century in Russia, during the construction of the Moscow Manege (arch. A. Betancourt) large-span wooden planked triangular trusses with a span of 50 m were developed and first used in the roof. In the middle of the same century, D. I. Zhuravsky designed and directed the construction of a number of wooden bridges of the Moscow and St. Petersburg Railway. The largest of them - a bridge across the Mstu River had seven spans of 61 m each. This bridge had wooden beam cross braces and steel tie-rods supports. He also created a method of calculation of wooden elements for shear under bending and a method of determining allowable wood stresses by experiment.

At the beginning of the twentieth century. V. G. Shukhov developed the first wooden spatial structures. Under his guidance the first wooden vault of the span of 21 m consisting of three layers of boards, connected with nails, was constructed in Nizhny Novgorod. In Orsk, he built a 36-metre high cooling tower with a grid structure of rods, located crosswise on the surface of a hyperboloid of rotation and connected with bolts at the intersections.

The richest forest resources of our country have always conditioned the wide use of wood as the main building material. Wooden construction in Russia, Finland and Sweden was based on a peculiar log construction system. The improvement of woodworking techniques contributed to the emergence of a variety of structures and architectural forms. These were various residential, commercial, fortress and religious log structures. They differed by their originality, original beauty and had no analogues in other countries [1].

Having undergone a long and complex path of development from semi-dugout, huts and pile structures, Russian wooden housing has received a characteristic treatment, preserved until the XX century. Of particular interest were the peasant dwellings of the Russian North, characterized by their distinctive design and layout, the richness of decor in the form of carving and painting. Wood was ubiquitously used for the construction of religious buildings. The use of the tent in the crowning pillars of temples led to the creation of magnificent works of architecture.

Lecture 2. Types of wooden constructions and areas of application. Physical and mechanical properties.

Wood reserves in our forests are about 80 billion m3. Annually about 280 million m3. of business wood, i.e. suitable for the manufacture of structures and products, is harvested. However, this amount is far from the natural annual growth of timber in the remote areas of Siberia and the Far East.

The harvested timber in the form of standard-length logs is delivered by road, rail, and water transport, or by rafting on rivers and lakes, to the timber-processing factories. There they are used to produce sawn material, plywood, wood-based panels, structures and building components. When logging and wood processing a large amount of waste is generated, the effective use of which is of great national economic importance. The production of insulating fiberboard and particle board from waste wood, which are widely used in construction, allows saving a large amount of industrial wood.

Coniferous wood is used for making the basic elements of wooden constructions and building parts. Straight high trunks of coniferous trees with a small number of knots allow to get straight lumber with a limited number of defects. Coniferous timber contains resin, making it more resistant to moisture and rotting than deciduous timber.

Most hardwoods are less straight, have more knots, and are more susceptible to rot than softwoods. It is almost never used for the basic elements of wooden building structures.

Oak wood stands out among hardwoods for its increased strength and resistance to rotting. However, due to scarcity and high cost, it is only used for small connecting parts.

Birch wood also belongs to the hard hard hardwoods. It is used mainly for plywood construction. It needs protection against rotting.

Wood structure

As a result of plant origin, wood has a tubular layered-fiber structure. The main mass of wood consists of wood fibers located along the trunk. They consist of elongated hollow shells of dead cells (tracheids, about 3 mm long) of organic substances (cellulose and lignin).

Wood fibers are arranged in concentric layers around the trunk axis, which are called annual layers because each layer grows during the year. They are clearly visible in the form of a number of rings on cross-sections of the trunk, especially in conifers. You can tell the age of a tree by their number.

Each annual layer consists of two parts. The inner layer (wider and lighter) consists of soft early wood, formed in spring when the tree is growing fast. Early wood cells have thinner walls and wider cavities. Late wood cells have thicker walls and narrower cavities. The strength and density of the wood depends on the relative content of the late wood.

The middle part of softwood trunks is darker in color, contains more resin, and is called the core. Then comes the sapwood and finally the bark.

In addition, the wood has horizontal core rays, a soft core, resin passages, and knots.

Sort, blemishes and quality of wood

Construction timber is divided into round and sawn timber.

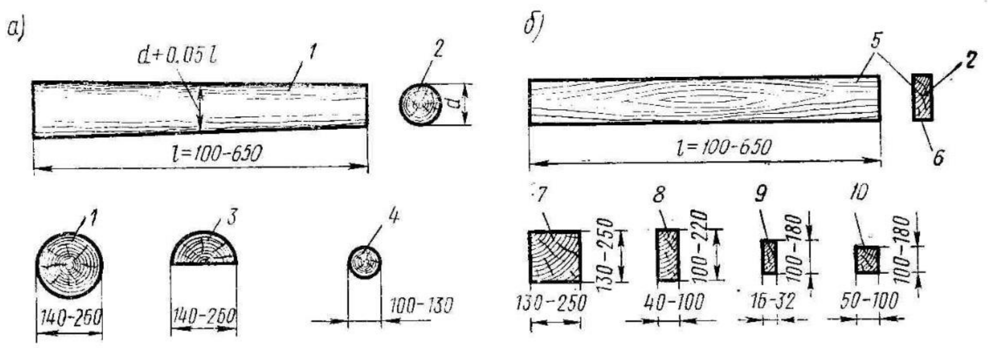


Figure 1. Wood materials: a) sawn; b)- circular; 1. the wide surface side of the board; side at the 2nd end; 3-thick side; 4-edged wood; 5-thick board; 6-thin board; 7th rail; 8-wooden beam; 9- a straight beam on one side; 10-cantilever beam.

Round logs, also called logs, are parts of tree trunks with smoothly sawn ends. Logs have a natural truncated-conical shape. The decrease in their thickness along the length is called runaway. Average runoff is 0.8 cm per 1 m of length (for larch 1 cm per 1 m of length) log. Average logs have a thickness of 14 to 24 cm large - up to 26 cm. Logs 13 cm and less thick are used for temporary constructions.

Sawn timber is produced by sawing logs longitudinally in frame saws or circular saws. They have rectangular or square cross-sections. The wider sides of a sawn timber are called planks and the narrower sides are called selvedges. Sawn timber is of standard length between 1m and 6.5 m, graded every 0.25 m. The width of sawn timber varies from 75 mm to 275 mm, the thickness varies from 16 mm to 250 mm.

The quality of a sawn timber is mainly determined by the degree of homogeneity of the timber's structure, which affects its strength. The degree of the homogeneity of wood is determined by the size and the number of areas where the homogeneity of the wood structure is broken and the strength is reduced. Such areas are called blemishes.

The main unacceptable defects in wood are: rotting, worm holes, and cracks in the cleavage areas in the joints.

The most common and unavoidable blemishes of wood are knots - overgrown remnants of former branches of the tree. Knots are allowable with limitations.

The slant of the fibers (obliquity) in relation to the axis of the element is also allowed with the limitation of a vice. It is formed as a result of the natural helical arrangement of the fibers in the trunk, as well as in the sawing of logs as a result of their runaway.

Cracks arising from the drying of the wood are also among the limited allowable defects.

The defects also include soft heartwood, knots falling out, and other less widespread violations of the homogeneity of the structure of wood.

The quality of timber timber is determined by its grade (selection, I, II, III, IV), which is established depending on the type, size, location, and quantity of defects. Timber for load-bearing elements of wooden structures must meet the requirements of I, II and III grades.

Grade I wood is used in the most important stressed tension elements. These are separate tensioned rods and boards of tensioned zones of glued beams with a section height of more than 50 cm.

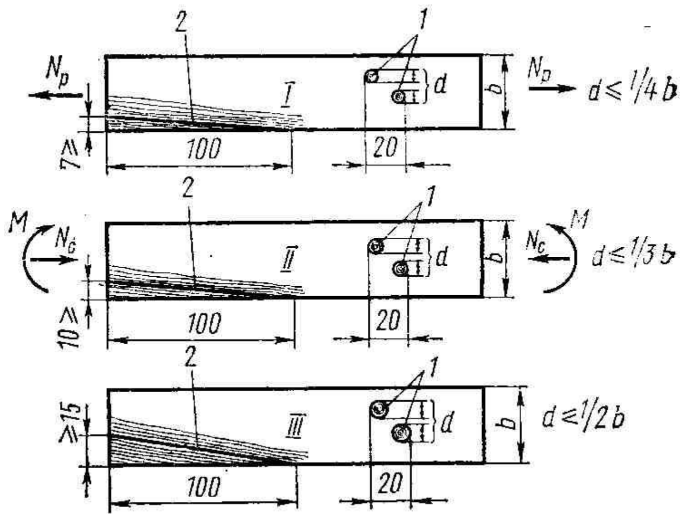


Figure 2. Categories of wood materials by quality:
a, b, v -1, 2 and 3 - categories: 1 - fiber slope; 2 - branches.

Cross-cut ≤ 7%.
The total diameter of knots on a length of 20 cm d ≤ 1/4b.

Grade II wood is used in compressed and bendable elements. These are separate compressed rods, boards of the extreme zones of glued beams with a height of less than 50 cm; boards of the extreme compressed zone and the stretched zone located above the boards of the 1st grade in glued beams with a height of more than

50 cm, boards of the extreme zones of the working glued compressed, bent and compressed-bent rods.*Косослой ≤10%.*

The total diameter of knots on a length of 20 cm d ≤ 1/3b.

Grade III wood is used in less stressed medium glued compressed, bent and compressed-bent elements, as well as in low-critical elements of decking and battens.

Slant ≤12%.

The total diameter of knots on a length of 20 cm d ≤ 1/2b.

Wood properties

Physical properties

Density. Wood belongs to the class of light structural materials. Its density depends on the relative volume of the pores and their moisture content. The standard density of wood should be determined at a moisture content of 12%. Freshly chopped wood has a density of 850 kg/m3. The calculated density of coniferous wood in the composition of structures in rooms with a standard air humidity of 12% is taken equal to 500 kg / m3., In a room with an air humidity of more than 75% and in the open air - 600 kg / m3.

temperature expansion. Linear expansion during heating, characterized by the coefficient of linear expansion, in wood is different along and at angles to the fibers. The coefficient of linear expansion α along the fibers is $(3 \div 5) \cdot 10^{-6}$, which makes it possible to build wooden buildings without expansion joints. Across the wood fibers, this coefficient is 7-10 times less.

The thermal conductivity of wood due to its tubular structure is very low, especially across the fibers. Thermal conductivity coefficient of dry wood across the fibers $\lambda \approx 0.14$ W/m·ºC. A beam 15 cm thick is equivalent in thermal conductivity to a brick wall 2.5 bricks (51 cm) thick.

The heat capacity of wood is significant, the heat capacity coefficient of dry wood is C = 1.6KJ/kg·ºC.

Another valuable property of wood is its resistance to many chemical and biological aggressive environments. It is chemically more resistant material than metal and reinforced concrete. At ordinary temperatures, hydrofluoric, phosphoric and hydrochloric (low concentration) acids do not destroy wood. Most organic acids do not weaken wood at ordinary temperatures, so it is often used for structures in chemically aggressive environments.

Mechanical properties of wood

Strength. Wood is a medium-strength material, but its relative strength, given its low density, makes it comparable to steel.

Wood is an anisotropic material, so its strength depends on the direction of the forces in relation to the fibers. When forces are applied along the fibers, the cell shells work under the most favorable conditions and the wood shows the greatest strength.

The average strength limit of pine wood without blemishes along the fibers is:

In tension - 100 MPa.

In bending - 80 MPa.

In compression - 44 MPa.

When stretching, compression and shear across the fibers, this value does not exceed 6.5 MPa. The presence of defects significantly (~30%) reduces the strength of wood in compression and bending, and especially (~70%) in tension. The duration of the load significantly affects the strength of wood. Under unrestricted long-term loading, its strength is characterized by the ultimate strength of long-term resistance, which is only 0.5 of the ultimate strength under standard loading. Wood shows its highest strength, 1.5 times the short-term strength, under the shortest impact and blast loads. Vibratory loads, which cause stresses of variable sign, reduce its strength.

The stiffness of wood (its degree of deformability under load) depends significantly on the direction of the loads in relation to the fibers, their duration and the humidity of the wood. The stiffness is determined by the modulus of elasticity E.

For softwood along the fibers E = 15000 MPa.

In SNIP II-25-80 modulus of elasticity for any wood species Eo = 10000 MPa. E90 = 400 MPa.

At increased humidity, temperature, as well as under the joint action of permanent and temporary loads, the value of E is reduced by coefficients of working conditions mv, mt, md < 1.

Effect of humidity. A change in moisture content between 0% and 30% reduces the strength of the wood by 30% of the maximum. Further changes in moisture content do not lead to a decrease in the strength of the wood.

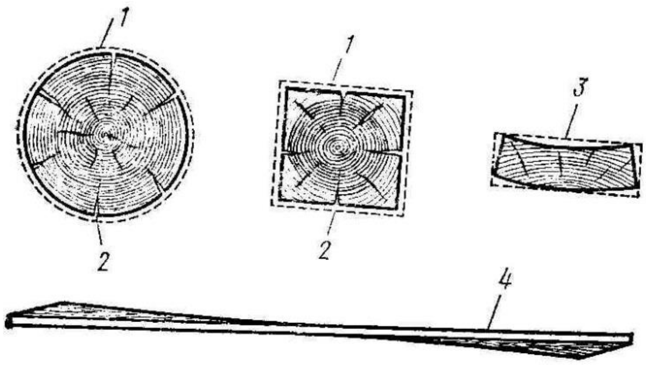


Figure 3. Deformations during drying of wood materials: 1. reduction in the dimensions of the section; 2 - cracking; 3 and 4 - crosswise and longitudinally.

Transverse changes in moisture content (shrinkage and swelling) lead to warping of wood. The greatest shrinkage occurs across the fibers, perpendicular to the annual layers. Deformation of shrinkage develops unevenly from the surface to the center. With shrinkage, not only warpage but also shrinkage cracks appear.

In order to compare the strength and stiffness of the wood, a standard moisture content of 12% was set

$$B_{12}=B_W[1+\alpha(W-12)],$$

где α – a correction factor, in compression and bending α = 0,04.

Influence of temperature. As the temperature rises, the tensile strength and modulus of elasticity decrease and the brittleness of the wood increases. The ultimate strength of wood G_t at temperature t in the range from 10 до 30 ºC be determined from its initial strength- G_{20} at a temperature of20 ºC c with the correction factor β = 3,5 МПа.

$$G_t = G_{20} – \beta(t\text{-}20).$$

Construction plywood

Construction plywood is a prefabricated sheet wood material. It usually consists of an odd number of thin layers of veneers. The fibers of adjacent veneers are arranged in mutually perpendicular directions.

SNIP II-25-80 on the design of wooden structures recommends the following types of waterproof plywood as a building material:

1. FSF plywood glued with phenol-formaldehyde adhesives. This plywood is produced:

- from birch wood (5 and 7-layer, thickness of 5 - 8 mm or more).
- from larch wood (7-layer, 8 mm thick and thicker).

Sheets of plywood with a thickness of more than 15 mm are called plywood boards. Shear strength of plywood in the plane perpendicular to the sheet is about 3 times higher than the strength of wood when sheared along the fibers, which is its important advantage.

The modulus of elasticity of birch plywood along the fibers is 90%, and across - 60% of the modulus of elasticity of wood along the fibers. The modulus of elasticity of larch plywood is 70% and 50% of the wood's Eo, respectively.

Banelized plywood (BPS) differs from FSF plywood in that its outer layers are impregnated with water-resistant alcohol-soluble resins. It is 7 to 18 m thick. Its strength along the fibers is 2.5 times, and across is 2 times higher than the strength of softwood along the fibers. It is used in particularly adverse moisture conditions.

Rotting and protection of wooden structures against rotting

Rotting is the destruction of wood by simple plant organisms - wood-destroying fungi. Some fungi affect trees in the forest that are still growing and drying out. Warehouse fungi destroy timber while it is stored in warehouses. House fungi - (merilius, poria, etc.) destroy wood of building structures during operation.

The fungi develop from spore cells that are easily transported by air movement. As they grow, the spores form a fruiting body and a fungus, the source of new spores.

Protection against rot

1. Sterilization of wood in the process of high-temperature drying. Heating of wood at t > 80oC, which leads to the death of fungal spores, fungus and fruiting bodies of the fungus.

2. Structural protection assumes operation mode, when the humidity of wood W<20% (the lowest humidity at which fungi can grow).

2.1 Protection of wood from atmospheric moisture - waterproofing of coatings, the necessary slope of the roof.

2.2 Protection against condensation moisture - vapour barrier, ventilation of constructions (drying vents).

2.3 Protection against moisture from capillary moisture (from the ground) - arrangement of waterproofing. Wooden structures should be supported on the foundation (with bitumen or Ruberoid insulation) at least 15 cm above the ground or floor level.

3 Chemical rot protection is necessary when wood moisture is unavoidable. Chemical protection consists of impregnation with substances that are toxic to fungi, i.e. antiseptics.

Water-soluble antiseptics (sodium fluoride, sodium silica) are substances that are colorless and odorless, harmless to humans. They are used in closed rooms.

Oily antiseptics are mineral oils (coal tar, anthrosene, slate, wood creosote, etc.). They are insoluble in water, but harmful to humans, so they are used for structures in the open air, in the ground, over water.

Impregnation is performed in autoclaves under high pressure (up to 14 MPa).

Protection against sharps beetles is heating to t>80°C or fumigation with poisonous gases like hexachlorane.

Combustion and protection of wooden structures from fire

Characterized by fire resistance limit (about 40 minutes for the bar 17 x 17 cm, loaded to a stress of 10 MPa).

Protection

1. Constructive. Elimination of conditions favorable to fire.
2. Chemical (fire impregnation or staining). Impregnated with substances called flame retardants (e.g. ammonium salt, phosphoric and sulfuric acid). Impregnation is carried out in autoclaves at the same time as antisepsis. When heated, the flame retardants melt and form a fire-retardant film. Protective painting is performed with compositions based on liquid glass, superfluor, etc.

Lecture 3. Loads affecting wooden structures. Calculation of wooden elements according to limit states.

In accordance with current standards, wooden structures must be calculated using the method of limiting states.

The limit states are those states of the structures at which they cease to meet the requirements of operation. The external cause that leads to the limit state is a force impact (external loads, reactive forces). Limit states can come under the influence of the working conditions of wooden structures, as well as the quality, sizes and properties of materials. There are two groups of limit states:

1 - on carrying capacity (strength, stability).

2 - by deformations (deflections, displacements).

The first group of limit states is characterized by the loss of bearing capacity and complete unsuitability for further operation. It is the most critical one. The following limit states of the first group can occur in wooden structures: destruction, loss of stability, overturning, inadmissible creep. These limit states do not occur if the conditions are met:

$$\sigma \leq R,$$

$$\tau \leq R_{ск} \ (or\ R_{ср}),$$

i.e. when the normal stresses (σ) and tangential stresses (τ) do not exceed some limiting value R, called the design resistance.

The second group of limit states is characterized by such signs, at which the operation of structures or constructions, although difficult, however, is not completely excluded, i.e. the structure becomes unsuitable only for normal operation. Suitability of the structure for normal operation is usually determined by the deflections

$$f \leq [f], \ or$$

$$f/l \leq [f/l].$$

This means that the bending elements or structures are suitable for normal operation when the greatest value of the ratio of deflection to span is less than the maximum allowable relative deflection [f/l] (according to SNIP II-25-80).

The purpose of the design of structures is to prevent the occurrence of any of the possible limit states, both during transportation and installation, as well as during the operation of structures. The calculation for the first limit state is carried out according to the calculated values of the loads, and for the second - according to the normative values. Normative values of external loads are given in SNiP "Loads and Effects". The design values are obtained taking into account the load safety factor γn. Structures are calculated for the unfavorable combination of loads (own weight, snow, wind) the probability of which is taken into account by the combination factors (according to SNIP "Loads and Effects").

The main characteristic of materials, by which their ability to resist the forces is estimated, is the standard resistance R_n. The normative resistance of wood is calculated from the results of numerous tests of small samples of pure (without the inclusion of defects) wood of the same species, humidity 12%:

$$R^{н} = R^{\text{ср}}_{\text{Вр}}(1 - t \cdot V),$$

where:

$R^{\text{ср}}_{\text{Вр}}$ – the arithmetic mean value of the tensile strength,

V – variation coefficient,

t – reliability index.

Regulatory resistance $R^{н}$ is the minimum probabilistic limit of strength of pure wood, obtained by static processing of the results of tests of standard specimens of small size for short-term loading.

The design resistance R is the maximum stress the material in the structure can withstand without collapsing, taking into account all the adverse factors in operating conditions that reduce its strength.

When passing from the normative resistance $R^{н}$ to the calculated R, it is necessary to take into account the effect on the strength of wood of the long-term action of the load, vices (knots, slanting, etc.), the transition from small standard samples to elements of construction sizes. The joint effect of all these factors is taken into account by the material safety factor (k). The design resistance is obtained by dividing $R^{н}$ by the material safety factor:

$$R = R^{н}/к,$$

$$к = \frac{1}{к_{дл} \cdot к_{одн}},$$

$к_{дл}$=0,67 – duration factor for the combined action of permanent and temporary loads;

$к_{одн}$=0,27÷0,67 – the homogeneity coefficient, which depends on the type of stress state, taking into account the influence of vices on the strength of wood.

The minimum value of the code is taken in tension, when the effect of vices is particularly great. Design resistance to are given in Table 3 of SNiP II-25-80 (for softwood). R of wood of other species are obtained by using the transition coefficients also given in SNIP.

The preservation and strength of wood and wooden structures depend on temperature and humidity conditions. Moisture contributes to rotting of wood, and increased temperature (beyond a known limit) reduces its strength. Consideration of these factors requires the introduction of coefficients of working conditions: $m_{в} \leq 1$, $m_{T} \leq 1$.

In addition, SNIP suggests taking into account the coefficient of ply for laminated elements: $m_{сл}$=0,95÷1,1;

girder coefficient for high beams, more than 50 sm: $m_{б} \leq 1$;

preservative coefficient: $m_{a} \leq 0,9$;

bending coefficient for bent elements: $m_{гн} \leq 1$ и др.

The modulus of elasticity of wood, regardless of species, is assumed to be:

$$E=10000 \text{ МПа};$$

$$E_{90}=400 \text{ МПа}.$$

Calculation characteristics of structural plywood are also given in SNIP, and, when checking the stresses in the elements made of plywood, as well as for wood, enter the coefficients of working conditions m. In addition, for the design resistance of wood and plywood, coefficient m_{dl}=0.8 is introduced if the total design force from permanent and temporary loads exceeds 80% of the total design force. This factor is introduced in addition to the reduction that is included in the material safety factor.

Lecture 4. Connections of wooden structural elements.

Timber dimensions (length and cross-sections) are limited, so they can be used separately only in the form of struts and beams of low load-bearing capacity. To create most building structures, timber elements must be firmly and reliably connected to each other. With the help of joints a number of elements are connected along the length - spliced, the width - spliced, connected at an angle by nodes and attached to the supports - anchored.

Joints are the most critical parts of wooden structures. When making many joints, holes and incisions are made in the structural elements, weakening their sections and increasing their deformability. Destruction of wooden structures begins in most cases in the joints. The deformability of joints explains the increased deflections of wooden structures. Thus, the strength and deformability of the structure as a whole depend on the correct design, calculation and fabrication of joints.

Anisotropy of structure, low strength of wood at shear, stretching across fibers and buckling are the reasons of great complexity and variety of connections of wooden structures.

The most simple and reliable solutions are the constructions of compressed wooden elements, in which efforts are transferred directly from the element, to the element and special working connections are not required. The connections of bendable elements, in which working connections are required to transfer the forces, are more difficult to solve.

Tensile element connections are the most difficult to solve. There is a risk of brittle fracture of wood in the weakened sections, as well as the risk of shearing and stretching across the fibers. The use of compliantly working ties in tensile element connections reduces the risk of brittle fracture. The complexity of connections of tensile wooden elements leads them in a number of structures to replace them with metal ones.

According to the nature of work, all the main connections of wooden structures can be divided into the following groups:

a) joints without special connections that require calculation - stops and joints;

b) connections with connections working in compression - dowels and pins;

c) connections with bending connections - dowels, pins, nails, screws, wooden plates and pins;

d) connections with tensile bonds - with bolts, nails, screws and clamps;

e) joints with bonds working in shear - with adhesive joints.

Due to the fact that the same ties are included in different groups, it is convenient to study the connections of wooden structures in the following order: connections without special ties, with wooden ties, with metal ties, with adhesive ties.

Glue joints, the most advanced and technologically advanced, are the main connections of the elements in the factory fabrication of wooden structures. Connections that do not require special connections (stops and joints) are mainly used in the construction of wooden structures. Metal joints are universal and are widely used in both major methods of making wooden structures. Connections with wooden ties are obsolete types of connections that require considerable expenditure of manual labor. They are rarely used and only in the construction of wooden structures.

All joints of wooden structures are malleable, with the exception of glue joints. Deformations in them are formed as a result of looseness arising during fabrication, from shrinkage and buckling of wood, especially across the fibers and bending of connections. The magnitude of these deformations under prolonged action of design loads in the joints, where the wood works across the fibers, is taken to be 3 mm, and in all other cases - 1.5-2 mm.

In most connections of wooden structures, except for glued connections, as a result of compressive forces or initial crimping, e.g. when bolts are placed, friction forces arise between the connected elements, which reduce the forces in the connections. However, these forces may be reduced to zero as a result of the possible reversal of forces, shrinkage of the wood and weakening of the initial tension in the bonds, which is why they are not taken into account in the calculation. They are taken into account only during the short-term action of compression with friction coefficients of plate to plate of 0,2, end to plate of 0,3 and when they cause additional stresses with friction coefficient of 0,6.

Calculation of the strength of the connections of wooden structures is made on the basis of the methodology outlined

§2 Connections without special bonds

Joints in which minor forces are applied or forces are transferred directly from one element to another do not require special connections to be calculated. These connections include structural joints, elbow stops and elbow joints.

Structural joints are joints in which the forces occur much less than their load-bearing capacity and they do not need to be calculated. In wooden structures, structural joints in the quarter, tongue and groove, half-tree and oblique butt joints are most commonly used.

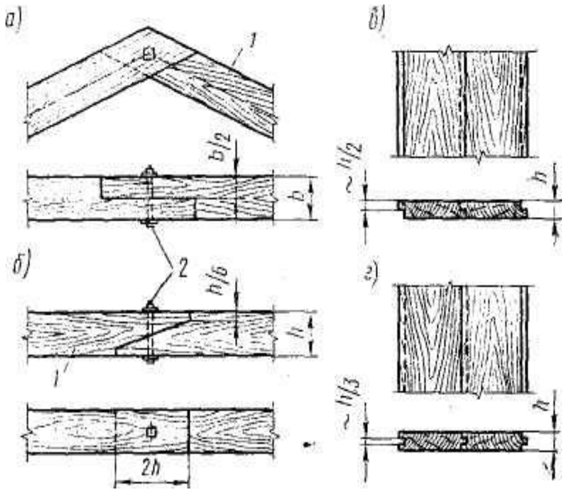


Figure 4 - Structural cutouts:

a - half-timbering; b - oblique scuttle; c - connections in the quarter; d - tongue and groove; - connected elements; 2 - tie bolts

Joint in a quarter is a fusion of boards with the edges of the width, for which they are cut unilateral slots deep, a little more than half the thickness, which include the resulting protrusions edges of adjacent boards. Wall sheathing made of boards connected in a quarter, prevent the walls from blowing and penetration of atmospheric precipitation. Concentrated loads in such cladding is distributed on the two adjacent boards.

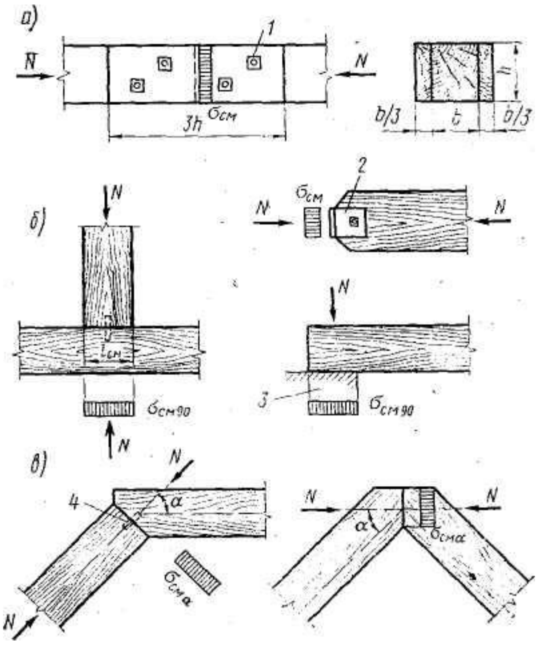


Figure 5 - Frontal stops:

a - longitudinal; b - transverse; c - inclined;

1 - tie bolt; 2 - knot mount; 3 - support; 4 - pin

Tongue-and-groove joint is a fusion of boards or beams, one of which has double-sided grooves, the other - one middle groove (tongue), equal to about Uz thickness, which includes the resulting protrusion (ridge) of the neighboring board. Planks of boards connected in tongue and groove, prevent spillage of backfill, and the concentrated loads on them are distributed to a number of neighboring boards.

Half-timbering is the connection of the ends of beams or logs with cuts up to half the thickness at an angle in the same plane, tightened by a structural bolt. This is how, for example, the ends of the rafters in the ridge of the roofs are connected.

Oblique scaling is a longitudinal jointing of beams or log ends in which made one-sided oblique inserts of length equal to twice the height of the section, with ends equal to 0.15 of the section height. The oblique nails are bolted together by structural bolts and are used to connect purlins and beams along their length.

Frontal stops (Fig. 2) are the simplest and most reliable connections used in most types of wooden structures for fastening compressed rods. They work and are rated for the buckling that occurs in them from the action of compressive forces. They cannot work in tension. Frontal stops can be longitudinal, transverse and inclined.

A longitudinal forehead stop is a connection of the right-angle cut end of a compressed rod with a support, a node diaphragm or the end of another rod of the same type in a compressed junction. In the joint, the stop is overlapped structurally by double-sided overlays at least 1/z of the rod thickness and at least three section heights on the bolts. In the longitudinal frontal stop, the wood works for buckling along the fibers and has the highest design resistance. In most cases, the buckling stresses reach a significant value and require checking by formula (5.15) only in supports where only part of the face area works on buckling.

A transverse forehead stop is the connection of two rods at right angles when the end of the compressed rod rests against the plate of the other and is secured with structural bolt-on linings. This is how, for example, the posts are connected to the upper and lower framing members. In this connection, the wood of the end works to buckle along the fibers, and the wood of the plate works across the fibers. The joint is calculated only on the smaller wood strength for local buckling across the fibers by formulas (5.13) and (5.15) in order.

An inclined forehead stop is the connection of two compressed rods at an angle less than straight. In this case, the end of one of them is formed at a right angle. This is, for example, how push-pull props are connected to ledgers in substructures. In this connection, the area where the buckling occurs at an angle to the wood fibers has less buckling resistance and must be tested for overall buckling strength at an angle using formulas (5.14) and (5.15). Formula (5.14) can be simplified by substituting the values of the calculated buckling resistance along and across the fibers:

$$R_{см\alpha} = \frac{13}{1 + 6{,}22 \cdot \sin^3 \alpha} \qquad (1)$$

The one-tooth fore-end socket is an easy-to-manufacture connection of two rods at an angle. It is mainly used to connect the rods of low-span sod and strut systems at nodes during their construction, whereby one of the rods to be cut must be compressed. An example of the frontal joint is the support node of a triangular beam low-span truss (Fig. 3).

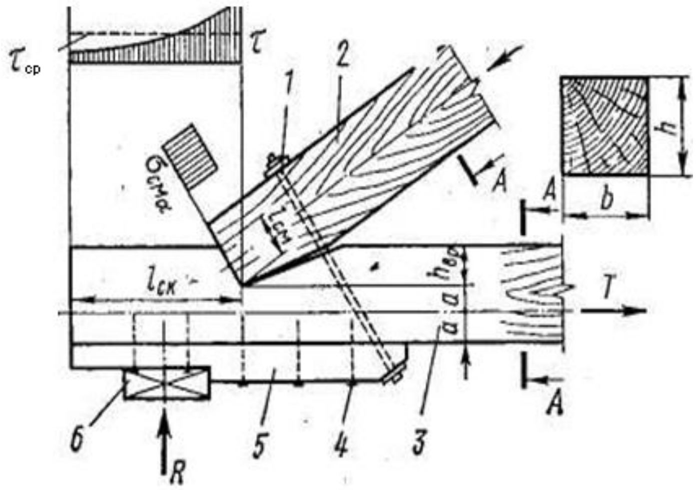


Figure 6 - Frontal gouge:

1 - emergency bolt; 2 - cutting element; 3 - supporting element; 4 - nails; 5 - subgirder; 6 - supporting pad

The cut-in rod of the upper chord of the truss with a part of the "tooth" end cut at right angles and cut off from the bottom is inserted into the cut-in rod of the lower chord and rests against its working surface. The narrow

The narrow wedge-shaped gap prevents undesirable compression of the non-working surfaces of the joint. The depth of the joint hvr shall be not more than 1/z, and the distance from its top to the end of the bottom chord lk - not less than 1.5 of its cross-section height h to obtain sufficient tensile and shear areas. The grommet shall be centered along the axes of the support, the upper girder and the lower girder section weakened by the grommet, so that this section does not experience not only tension but also bending caused by the eccentricity of the tensile force. The bend is additionally tightened by an inclined bolt perpendicular to the upper chord, which is called an emergency bolt. It prevents the rods from diverging during installation of the truss in case of tensile stress in the upper chord. In the event of a shear failure of the girder, the emergency bolt is engaged and prevents the risk of sudden collapse of the truss. The nailed support beam protects the bottom chord from local buckling on the support and the necessity of a weakening slot for the emergency bolt washer in it.

The front gusset operates and is calculated for buckling from the action of the compressive force in the cut rod N and shear from the action of the horizontal projection of this force T equal to the tensile force in the lower chord of the truss.

Wood buckling. From the action of the compressive force N over the area of the end stop of the compressed rod in the working surface of the tensile tie-in, there arise uniform buckling stresses σ. The area of the buckling F is determined depending on the depth of the cutting $h_{вр}$, the angle of inclination of the compressed rod a and the width of the cutting b, which in beams is equal to the width of the section, and in logs with a diameter d are found from the expression $b = 2\sqrt{h_{вр}\left(\alpha - h_{вр}\right)}$. Accordingly, the area of buckling is equal in the log joints $F = \frac{bh_{вр}}{\cos\alpha}$; in log cuts $F = \frac{0,71bh_{вр}}{\cos\alpha}$.

The calculation is made according to the strength of the working area of the mortise at the local buckling at an angle to the fibers of the stretched rod. The design resistance to local buckling at an angle to the fibers, due to the small length of the buckling area and the significant supporting action of neighboring wood sections, is determined by the formula (5.14), taking into account the increased coefficient of working conditions $m_{см2} = 1,65$:

$$R_{см90} = 1,65 \cdot 1,8 = 3МПП \quad и \quad R_{см\alpha} = \frac{13}{1 + 3,22 \cdot \sin^3\alpha} \tag{2}$$

The strength of the front joint at local buckling is checked according to the formula (5.15). The same formula, rewritten in relation to the compressive force N, is used to determine the buckling load capacity of a timber gouge.

Shearing of wood. From the action of shear forces T along the fibers of wood along the shear area F equal to "the product of the width of the felling b by the length of shear $l_{ск}$ there are shear stresses τ. The length of the shear area $l_{ск}$ is equal to the distance from the lower point of the joint to the end of the stretched rod, but it is considered not longer than the length equal to 10 depths of the joint $h_{вр}$ (see Ch. 5).

The shear stresses τ are distributed along the length of the shear area particularly unevenly, since the shear forces act on one side of the shear area and reach their maximum near the joint. The shear stresses are somewhat reduced here as a result of the pressure created by the vertical component of the compression force.

The calculation is carried out by the shear strength according to the average values of shear stresses. The design average shear resistance $R_{ск}^{ср}$ is determined by the formula (5.17), where the coefficient $\beta = 2{,}25$ is taken, and the shoulder of the pair of shear forces $e = 0{,}5h$.

When considering the length of the shear area equal to no more than double the section height of the stretched rod, it is allowed to take the calculated average resistance to shear equal to $R_{ск}^{ср} = 1{,}2MPa$. The front joint is tested for shear strength according to the formula (5.18). The same formula, but in relation to shear forces T, can be used to determine the shear carrying capacity of the gusset.

The double-tooth front fist bend is characterized by the fact that the compressed rod is cut into the other by two teeth, resulting in two squeezing and shearing areas in the fist bend. This type of joint is more complicated, labor-intensive and requires higher accuracy of workmanship to ensure joint operation of all work areas. This type of joint is used in some cases to join rods at an angle of 45° or more.

Connections with wooden ties are time-consuming and outdated building connections. Here small wooden inserts serve as connections. They are tightly inserted into the corresponding holes in the elements to be connected - logs or beams - and ensure that they work together in bending, taking the shear forces. Connections can be doweled, tongued or doweled.

Keyed connections are made with bars - dowels or pads, which work on crushing and shear and create a transverse expansion of the elements, perceived by the bolts. Plate connections are made with oak plates (plate dowels), which work for bending and buckling of wood and do not create a transverse expansion. Pin joints are made with oak pins (oak dowels), which also work for bending and buckling without transverse expansion.

These connections are used in some temporary wooden structures and hydraulic engineering.

§ 3 Connections with steel bonds

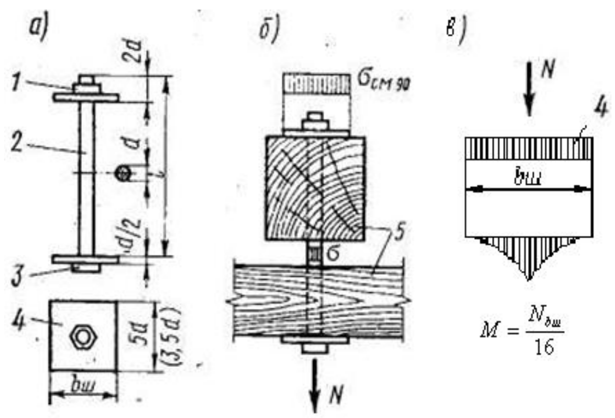


Figure 7 - Stretch bolts: a - general view; b - bolt and wood scheme; c - washer scheme; 1 - nut; 2 - rod; 3 - head; 4 - washer; 5 - elements to be connected

Connections in which forces are absent or tensile, compressive or shear forces act are successfully solved with steel bonds. These connections include bolts, rods, nails, screws, claw washers, clamps and some other connections. Steel ties, depending on the nature of their work, can be part of tie, tensile or bending - nail connections. They are the most universal and are used both in the factory, and postrekuetschego the manufacture of wooden structures. The most common steel connections are bolts and nails.

Bolt connections. Bolts (Fig. 4) are standardized products made of construction steel grade C38/23. The bolts used in most wooden structures are called black bolts and are manufactured without precision machining. They are of considerable length, corresponding to the large cross-sections of wooden elements, and are provided with large square washers, necessary to distribute the force in the bolt over a sufficient area of wood. Bolt section sizes are given in appendix V. Bolts with diameters of 12, 16 and 20 mm are the most common.

In order to put the bolts in the elements being joined, drill holes of the same diameter as the bolt. To ensure reliable alignment of the holes when assembling structures, drill holes with a single pass of the drill through the connected elements or in individual elements according to the templates. Bolt connections are made with tensioned, tensioned and bent bolts.

Tie-bolt connections are used for the tight connection of individual elements when they are cross-linked and in certain structural components. They can generate only minor forces and do not need to be calculated. The cross-sections of the tie bolts are set for design considerations. The diameter of the bolts shall not be less than 12 mm and less than 1/20 of the total thickness of the elements being joined.

Tie bolt washers should be at least 3.5 cm wide and at least 0.25 cm thick. Tie bolts often loosen during the first years of operation and need to be tightened.

Tensioned bolt connections are used when anchoring wooden structures to supports, when suspending to overlapping structures and equipment and in knot connections. They support the tensile forces N acting in the connections.

Bolt works and is calculated on the tensile area of cross-sectional area weakened by threading F. Design resistance of steel is taken reduced by 20% taking into account the concentration of tensile stresses and in the area of the thread. The calculation is carried out according to the formula

$$\sigma = N/F \leq 0{,}8R. \qquad (3)$$

According to the same formula, rewritten with respect to the required cross-sectional area of the bolt $F_{тр}$, with the help of the table data it is possible to choose the cross-section of the bolt.

The wood under the bolt washers works and is calculated for local buckling. The design resistance to buckling under washers at buckling angles from 90 to 60° is taken into account the small area of buckling and significant supporting action of the surrounding wood sections, with an increased coefficient of working conditions $m_{см3} = 2{,}2$ and is the design resistance $R_{ск90} = 18 \cdot 2{,}2 = 4\,MPa$ to buckling under washers at an angle a to fibers is determined by the formula (5.14), which after substitution of numerical values of design resistances is as follows

$$R_{см\alpha} = \frac{13}{1 - 2{,}25 \cdot \sin^3 \alpha} \tag{4}$$

Calculation for buckling under the washers is done by formula (5.15).

Bolt washers work and are calculated for bending from the reactive pressure of crumpled wood as square plates of width b, resting in the center on the bolt nut. The greatest bending moment M in the middle section of the washer, loosened by a hole of diameter d, and the required thickness of the washer $\delta_{тр}$ can be approximated from the expressions

$$M = Nb/16; \quad \delta_{тр} = \sqrt{\frac{6M}{(b-d)R}}.$$

The tensile rods of circular cross-sections with washers and nuts at the ends are calculated similarly. Their maximum flexibility must not exceed 400. If a number of bolts are used in the connection, the calculated resistance is reduced by 0.85, taking into account the possible unevenness of its distribution between the bolts.

Joints with bendable bolts belong to the class of nail joints, in which bonds, in this case bolts, work mainly for bending without expansion. These connections are widely used in the joints and nodes of wooden structures, preventing mutual shifts of the connected elements, and the forces in them can be alternating. The washers of these bolts do not absorb design forces and have the same dimensions as the tie bolts. From the longitudinal forces acting in such a connection, along the contact area of the bolt with the hole in the wood of the elements being joined, unequal along the perimeter and length of the crushing and shearing and stretching across the fibers between the holes arise. Reactive wood pressure results in bending and shearing forces in the bolt.

Bolt placement in the joint is made according to the rules that eliminate the risk of premature failure of wood elements from shear and stretching across the fibers. The distance between the axes of the bolts along the fibers and to the ends of the elements shall not be less than 7d, and across the fibers between the axes - 3.5 d and to the edges - 3 d.

Bolted joints can be symmetrical, when longitudinal forces act along one axis, relative to which the elements are symmetrically arranged, and asymmetrical, when the axes of the elements do not coincide and there is no symmetry of the connection. The elements to be connected can be located along one axis along the fibers or at an angle to each other.

Lecture 5. Calculation of wooden construction elements for elongation and compression.

The elements of wooden structures are boards, bars, planks and logs of solid section with the dimensions given in the sawn and circular material classifications. They can be independent structures, such as beams or struts, as well as the bars of more complex structures. The forces in the elements are determined by general methods of structural mechanics. Checking the strength and deflections of the element consists in determining the stresses in the sections, which must not exceed the design resistance of wood, as well as its deflections, which should not exceed the limits set by the design standards.

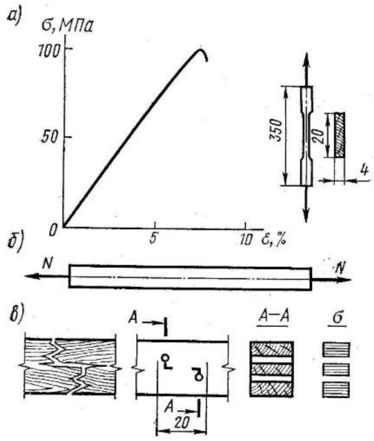


Figure 8. Stretchable element: a-deformation graph and sample; b-working circuits and voltage circuits.

Tensile stresses are applied to the bottom chords and individual struts of trusses, arches and other through structures. The tensile force N acts along the axis of the element and tensile stresses σ arise at all points of its cross-section, which are accurately assumed to be equal in magnitude.

Tensile stresses in wood are almost elastic and show high strength. Fracture occurs brittle in the form of an almost instantaneous rupture. Standard specimens in tensile tests have a "figure of eight" appearance.

As can be seen from the tensile diagram of wood without vices, the dependence of strain on stress is close to linear, and the strength reaches 100 MPa.

However, the tensile strength of real wood, given its considerable fluctuations, the great influence of vices and the duration of loading is much lower: for unglued wood of I grade R_p=10 MPa, for glued wood the influence of vices decreases, so R_p=12 MPa. The strength of the stretched elements in those places where there is a weakening is reduced as a result of stress concentration at their edges, i.e. entered the coefficient of operating conditions m_0=0.8. Then design resistance R_p=8 MPa is obtained. The verification calculation of the tensile elements is carried out by the formula:

$$\sigma = \frac{N}{F_{\text{нт}}} \le R_p \text{ , где}$$

$F_{\text{нт}}$ – the area of the cross-section under consideration, and the weaknesses located in the section of 20 cm. length are considered to be combined in one cross-section. The same formula is used to select the sections, but in relation to the desired (required) area $F_{\text{тр}}$.

Compressed elements

Struts, struts, upper chords and individual truss rods work in compression. In cross-sections of the element from the compressive force N acting along its axis, there are almost equal in magnitude compressive stresses σ (rectangular epiure).

Standard specimens in compression test have the form of a rectangular prism with the dimensions.

The wood performs reliably in compression, but it is not completely elastic. Up to about half of the tensile strength, the deformation growth follows a near-linear law, and the wood works almost elastically. As the load increases, the increase in strain increasingly outpaces the increase in stress, indicating the elastic-plastic nature of the wood.

Failure of specimens without blemishes occurs at stresses as high as 44 MPa, plastically, as a result of the loss of stability of a number of fibers, as evidenced by the characteristic crease. Vices reduce the strength of wood less than in tension, so the calculated resistance of real wood in compression is higher and is for grade 1 wood Rc = 14 ÷ 16 MPa, and for the 2 and 3 grades this value is slightly lower.

Calculation of the strength of compressed elements is made by the formula:

$$\sigma = \frac{N}{F_{\text{нт}}} \le R_c \text{ , где}$$

R_c – design resistance to compression.

The elements that are crumpled over the entire surface are calculated in the same way. Compressed rods of great length and not fixed in transverse direction should be, in addition to strength calculation, calculated for longitudinal bending. The phenomenon of longitudinal bending lies in the fact that a flexible centrally-compressed straight rod loses its straight shape (loses stability) and begins to bulge at stresses much lower than the ultimate strength. The test of a compressed element with regard to its stability is carried out by the formula:

$$\sigma = \frac{N}{\cdot F_{расч}} \le R_c ,$$

where:

$F_{расч}$ – calculated cross-sectional area,

φ – coefficient of longitudinal bending.

$F_{расч}$ принимается равной:

Lecture 6. Calculation of bending of wooden structural elements.

In bendable elements from loads acting across the longitudinal axis, there are bending moments M and transverse forces Q, determined by the methods of structural mechanics. For example, in a single-span beam of span l from a uniformly distributed load q arise bending moments $M_{\max} = \frac{ql^2}{8}$ and shear forces $Q_{\max} = \frac{ql}{2}$.

From the bending moment in the sections of the element there are deformations and bending stresses σ, which consist of compression in one part of the section and tension in the other, as a result the element bends.

The diagram, as for compression, is linear up to about halfway, then curves, showing an accelerated growth of deflections.

$R_u^{вр}$ =80 MPa – ultimate bending strength of pure wood in short-term tests. Fracture of the specimen begins with the appearance of folds in the outermost compressed fibers and ends with the rupture of the outermost stretched fibers. Rated bending resistance by SNIP II-25-80 is recommended to take the same as in compression, ie for grade 1 R_u = 14 MPa - for elements of rectangular cross-section height to 50 cm. Beams with a cross-sectional dimension of 11 - 13 cm at the height of 11 - 50 cm have less cut fibers when sawing than boards, so their strength increases to R_u = 15 MPa. Logs over 13 cm in width and 13 - 50 cm section height do not have cut fibers at all, so R_u =16 MPa.

Расчетное сопротивление изгибу по СНиП II-25-80 рекомендуется принимать таким же, как и при сжатии, т.е. для 1 сорта R_u=14 МПа – для элементов прямоугольного сечения высотой до 50 см. Брусья с размерами сечения 11 – 13 см. при высоте сечения 11 – 50 см. имеют меньше перерезанных волокон при распиловке, чем доски, поэтому их прочность повышается до $R_и$=15 МПа. Бревна шириной свыше13 см. при высоте сечения 13 – 50 см. совсем не имеют перерезанных волокон, поэтому $R_и$=16 МПа.

1. Calculation of bending elements for strength

Produced by the formula:

$$\sigma = \frac{M}{W_{\text{cal}}} \leq R_и \ ,$$

where:

M – maximum bending moment,

W_{cal} – calculated moment of resistance of the cross-section.

For the most common rectangular cross section

$$W = \frac{I}{h/2} = \frac{bh^2}{6} \ ; \ \ I = \frac{bh^3}{12} \ .$$

The selection of the section of bending elements is made by the same formula, determining $W_{\text{тр}} = \dfrac{M}{R_и}$, then, by setting one of the section dimensions (b or h), find the other dimension.

2. Calculation of the stability of the form of deflection of elements of rectangular constant section

Produced by the formula:

$$\sigma = \frac{M}{{}_{м}W_{\text{бр}}} \leq R_и \ ,$$

where:

M – максимальный изгибающий момент на рассматриваемом участке l_p,

$W_{бр}$ – максимальный момент сопротивления брутто на рассматриваемом участке l_p,

$\varphi_м$ – coefficient of stability.

3. Testing of bendable elements by deflections

Determine the relative deflection, whose value should not exceed the limit value regulated by SNIP:

$$\frac{f}{l} \leq \left[\frac{f}{l}\right]$$

The maximum deflection f of articulated and cantilevered bendable elements of constant and variable cross-section should be determined by the formula:

$$f = \frac{f_0}{k}\left[1 + c\left(\frac{h}{l}\right)^2\right],$$

where

f_0 – the deflection of the constant-section beam without taking into account the shear deformations (e.g., for a single-span beam $f_0 = \frac{5}{384}\frac{q^н l^4}{EI}$;

h – the greatest height of the section;

k – coefficient, taking into account the variability of section height, for a constant-section beam k=1;

c – the coefficient taking into account the shear deformation from the shear force.

The values of the coefficients k and c are given in SNiP.

Glued curvilinear elements bent by the moment M, which reduces their curvature, should be tested additionally for radial tensile stresses according to the formula:

$$\sigma_r = \frac{(\sigma_0 + \sigma_i)h_i}{2r_i} \le R_{p90},$$

where:

σ_0 – normal stresses in the outermost fiber of the tensile zone.

σ_i – normal stresses in the intermediate fiber of the section for which the radial tensile stresses are determined;

h_i – the distance between the outermost and the considered fibers;

r_i – is the radius of curvature of the line passing through the center of the normal tensile stress epure, enclosed between the outermost and the considered fibers.

The oblique bend

It occurs in the elements, the axes of sections of which are inclined to the direction of the loads, such as in the timber beams of pitched roofs.

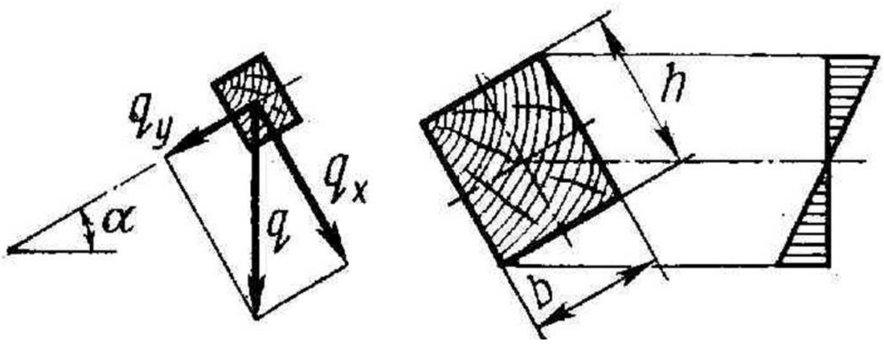


Figure 9. The oblique bend

$q_x = q\sin\alpha;$

$q_y = q\cos\alpha;$

$M_x = M\sin\alpha;$

$M_y = M\cos\alpha.$

The vertical load q and the bending moments M at oblique bending at an angle α are decomposed into the normal (q_y) and rolling (q_x) components.

The strength test for oblique bending is carried out according to the formula:

$$\sigma = \frac{M_x}{W_x} + \frac{M_y}{W_y} \le R_u \ .$$

The selection of cross-sections of oblique bending elements is made by the method of attempts. The deflections are calculated taking into account the geometric sum of the deflections relative to each of the section axes:

$$\frac{f}{l} = \frac{\sqrt{f_x^2 + f_y^2}}{l} \le \left[\frac{f}{l}\right].$$

Lecture 7. Calculation of cracking of wooden structural elements.

Cracking in wood can occur in longitudinal planes along the fibers. Cracking stress - T causes cracking and tensile stress- τ in wood. The strength of the wood in the crack is also very small due to the fact that the wood is fibrous. The fibers in the wood are weakly bonded, so the wood easily cracks $\tau = 6{,}8 МПа$ under moderate stresses.

In bending, the maximum shearing force *Q (MN)* on bending elements is calculated using the following formula:

$$\tau = \frac{Q \cdot S}{I \cdot b} \le R_{yor},$$

where: S is the static moment of the cracking surface relative to the neutral axis ($S = \frac{b \cdot h^2}{8}$); Q - maximum shear force; J is the moment of inertia of the total surface ($J = \frac{b \cdot h^3}{12}$); R_{yor} - calculated resistance to cracking ($R_{ёр} = 1{,}6 МПа$); b - the width of the section.

The following formula is used to calculate joints for cracking:

$$\tau = \frac{T}{A} \le R_{cr},$$

where: τ - test voltage; T - tensile stress; A - crack surface; Rortyor is the calculated average resistance to cracking.

bu yerda: τ - urinma kuchlanish; T - yorilishdagi zo'riqish; A - yorilish yuzasi; R_{cr} - average calculated resistance to cracking.

$$R_{cr}^{avr} = \frac{R_{cr}}{1 + \frac{\beta \cdot l_{cr}}{e}},$$

here: - $R_{cr} = 2{,}1MPa$ calculated maximum wood cracking resistance; l_{cr}- the length of the crack area; e - cracking stress eccentricity; $\beta = 0{,}25$ - coefficients when the stress in cracking is one-sided and $\beta = 0{,}125$ - two-sided.

Lecture 8. Calculation and design of rafters.

Inclined wooden rafters are widely used constructions in construction practice. They are widely used in covering the roofs of one- and multi-story dwellings, public buildings, and agricultural buildings.

The rafters of sloping roofs have a simple design, are easy to prepare, and due to constant exposure to the wind, wooden elements do not rot and serve for a long time.

The main structural elements of the pitched rafter roof covering system include: waterproofing layer of the roof (asbestos-cement sheets, tiles, tin, plastic sheets, wrapping materials, etc.), floor or bars, rafter legs and rafter substructures.

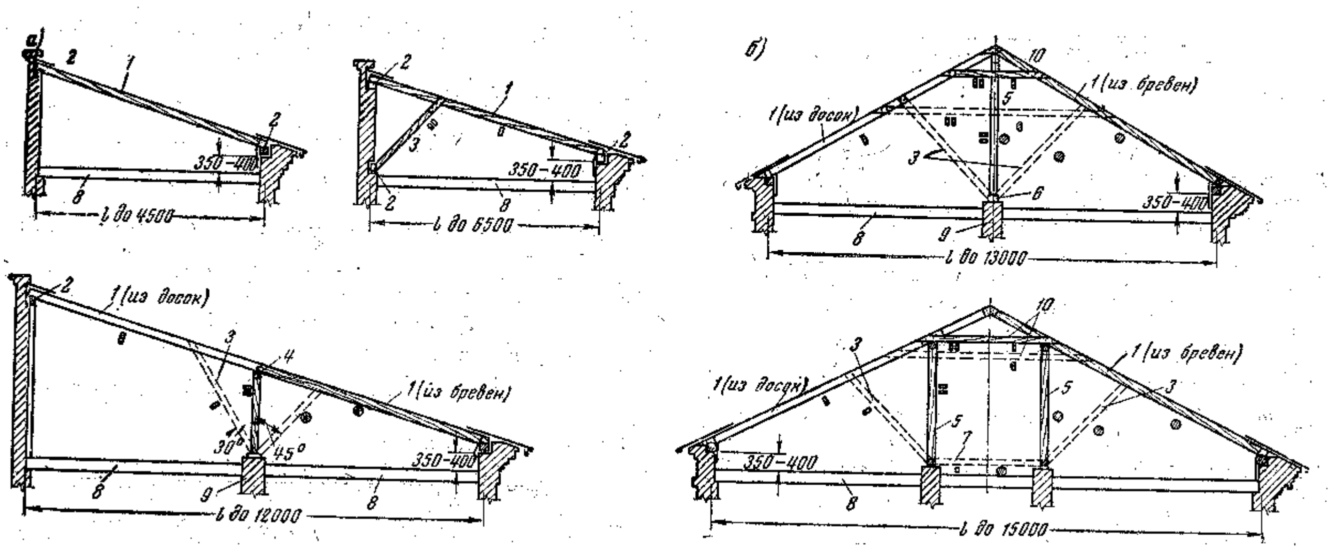


Figure 10. Inclined rafters:
a-one slope; b-double slope; 1-stropila leg; 2nd mauerlat; 3rd cup; 4-let it be tempting; 5th column; 6-tag sarrov; 7th pillar; 8th medium; 9-longitudinal walls or columns

Flooring and railings. Floors and bars projected under the covers are calculated according to two variants of the sum of loads:

1. specific weight and snow load (according to stability and coolness);
2. specific weight and accumulated load of 1000N;

in which the accumulated load is multiplied by the loading coefficient γ_f=1,2 (calculation is performed only for strength).

The calculated bending strength of the wood is multiplied by a factor of 1.15, taking into account the working conditions. In addition, when calculating the accumulated load, the calculated resistance is multiplied by a factor of 1.2 (assembly load).

The floor and the fence are calculated taking into account the continuity of their two spans. The calculated interval is the distance between the axes of the legs of the sling.

The maximum bending moment at the middle support when a two-span beam is subjected to a uniformly distributed load from its own weight and snow:

$$M' = 0{,}125 \cdot q \cdot \ell^2 \tag{3.1}$$

relative coolness between:

$$\frac{f}{\ell} = \frac{2{,}13 \cdot q^{н} \cdot \ell^3}{384 \cdot E \cdot J} \tag{3.2}$$

The maximum bending moment in the span when the two-span beam is loaded with specific weight (q) and accumulated load (R):

$$M'' = 0{,}07 \cdot q \cdot \ell^2 + 0{,}207 \cdot P \cdot \ell \tag{3.3}$$

The accumulated load on a double-layered bed (protective and working) or a single-layered bed with underlaying distribution bars is assumed to be distributed over a width of 0.5 m of the working bed. When calculating lattice beams, the accumulated load R is assumed to be placed on one beam.

In cases where the angle of the roof slope is $\alpha \geq 10^o$, the specific weight of the grid and covering is taken into account on the roof surface (slope), and the snow load is equally distributed on its horizontal projection. Therefore, the total load acting on a length of 1 m of the beam is:

$$q = g \cdot S + P_{сн} \cdot S \cdot \cos\alpha \,,$$

where: g – constant load affecting 1 m^2 surface of the roof slope;

$R_{СН}$ – snow load affecting 1 m^2 horizontal projection of the roof;

S – the distance between the axes of the beams according to the roof slope.

The strength of the lattice bars is checked according to the following formula, taking into account the oblique bending:

$$\sigma = \frac{M_x}{W_x} + \frac{M_y}{W_y} \leq R_u , \quad (3.4)$$

where: M_x va M_u – to the main axes X and U of the calculated bending moment
relative founders;

W_x va W_y – X and U axes of the rough cross section moments of resistance for

The complete coolness of Brusok is determined by the following formula, taking into account the skewness:

$$f = \sqrt{f_x^2 + f_y^2} , \quad (3.5)$$

where: f_x and fy are the *X*- and *Y*-axes of the block.

Stropila legs

For the legs of the stropila, boards, beams (chorkirra), plates or beams are used. Beams made of boards and logs are the main solution for modern prefabricated industrial construction.

In areas where wood is considered a local construction material, rafters are mainly made at the construction site; in such conditions, the restoration of rafters from circular cross-section logs, which have certain advantages, can be successfully used in construction practice.

In our country, it is widely used to restore stropila from poplar wood.

Stropilas are usually made of small logs (diameter 12-24 cm). Poles cost up to 2 times cheaper than sawn wood materials. The calculated bending resistance of beams (R_u=1600 n/cm^2) is slightly higher than that of boards (R_u=1300 n/cm^2); the fire resistance limit of beams is higher than that of boards. It can be seen that it is advisable to use the beams in the rafters.

Properly designed and repaired sloped rafters are considered a non-sloping structure. In order to prevent the formation of raspor force in the stropila, the support surfaces of the carved columns in the places where the stropila legs rest on the mauerlats and progons are made horizontal. must be turned off.

If there is an angle of the roof slope$\alpha \leq 10^o$, the rafter legs are considered as beams with a horizontal axis, if there is -$\alpha > 10^o$ as beams with an oblique axis. In the second case, the value of the load falling on the surface of 1m^2 of the roof surface (slope) is divided by cosα, and it is brought to the load affecting the surface of 1m2 of the covering plan. The load acting on the rafter leg is collected by the load area, the width of which is equal to the pitch of the rafters.

The maximum bending moment generated in the leg of a two-support free-standing truss is determined by the following formula:

$$M = \frac{q \cdot \ell^2}{8}$$

where: q is the full (permanent and snow loads) load acting on the length of 1 m of the horizontal projection of the rafter leg;

ℓ - the distance of the rafter leg in horizontal projection.

The uniformity of the rafter legs is checked according to the following formula, taking into account the inclination of the axis of the element:

$$\frac{f}{\ell_1} = \frac{5q^n \cdot \ell^3}{384 \cdot E \cdot J \cdot \cos\alpha} \le \frac{1}{200}. \tag{3.6}$$

If the rafter leg has an additional support in the form of a progon (Fig. 3.4) or a brace (Fig. 3.5), it is considered as a two-span continuous (connected) beam.

The bending moment generated in the section at the middle support is determined by the following formula:

$$M = \frac{q \cdot \left(\ell_1^3 + \ell_2^3\right)}{8 \cdot \left(\ell_1 + \ell_2\right)}, \tag{3.7}$$

where: ℓ_1 and ℓ_2- horizontal distances from the end supports of the rafter leg to the middle support.

Lecture 9. Calculation and design of Vassa beams

Wooden slats are load-bearing elements in wooden gable roofs. A large amount of wood is used for their preparation. The correct design of wooden floors determines the economic efficiency of the construction. Beds serve as a basis for warm roofing layers. They provide priority to the main load-bearing structures and resist vertical and wind loads. The construction of the floor also depends on the properties of the roof and the roof covering heat savers.

Wooden floors are mainly divided into wood and glue types. Wooden floors are the most common and used floors. Second and third grade wood materials are used for their preparation. Therefore, the beds are relatively cheap. Their main disadvantage is high labor cost for preparation and low load carrying capacity. Wooden beds can be made up to 3 meters long and in solid and grid-like forms. In lattice-shaped floors, wooden planks are placed at intervals of at least 2 cm.

Single-layer single-layer and two-layer single-layer mats are made. The gaps of the first layer of cross-beam boards are left open at least 2 cm, and a protective wooden layer is nailed on top at an angle of 45□60 degrees. In this case, the first layer board is the main working layer. The thickness of the board in the protective layer is at least 16 mm, and the width is 100 mm.

Wooden floors are calculated according to the normative and computationally distributed and accumulated loads for bending (Fig. 32). The specific weight of the floor is determined by the thickness and density of the heat insulator, roof elements, and these loads are evenly distributed over the surface of the floor.

Beds on sloping roofs with a slope at an angle of α are considered loads. The amount of snow load is determined taking into account the construction area. As accumulated individual loads, 1 kN is assumed due to the presence of a person on the floor during the installation process. When determining the values of calculated loads, γ =1.1, γ =1.3 for heat storage and roof, and γ =1.6 for snow load are accepted. Cross-sectional dimensions can be determined using the following formulas:

$$\boldsymbol{W_т = \frac{M}{R}} \;; \qquad \boldsymbol{b_т = \frac{6 \cdot W_т}{h^2}} \;; \qquad W = \frac{b \cdot h^2}{6}$$

b_t - the width of the board required.

Beds are calculated according to the first and second limit states. In the calculations, permanent and snow loads are taken into account as the main total load.

Using the following formula in the first limiting case:

$$\boldsymbol{\sigma = \frac{M}{W} \leq R_{эг}} \;,$$

The second group is calculated using the following formula for the limit state:

$$\boldsymbol{II \rightarrow \frac{f}{l} = \kappa \cdot \frac{q^{м} \cdot l^3}{E \cdot I} \leq \left[\frac{f}{l}\right] = \frac{1}{150}}$$

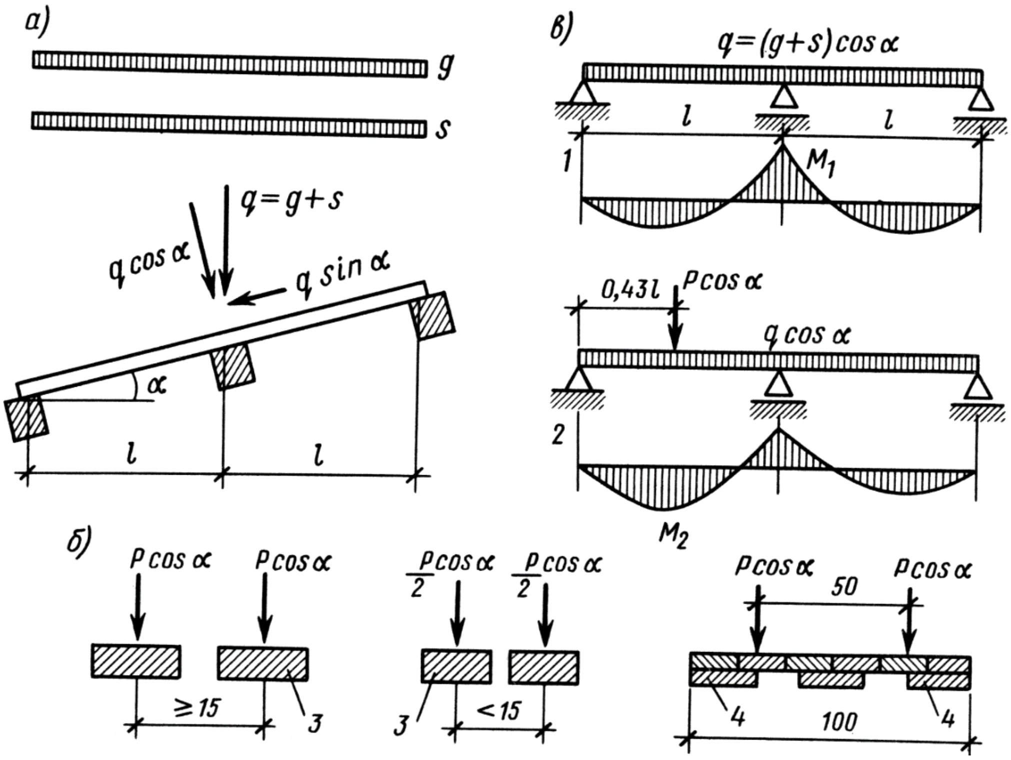


Figure 11. Schemes of calculation of floors: a- scheme of flat loads; b- the same, affecting the point; v- stress scheme; 1- first collective loads; 2- second collective loads; 3- open bed shelves; 4- working floorboards.

In addition to permanent and snow loads, it is also calculated based on the sum of loads during the installation process. In this case, evenly distributed loads from specific gravity are equal to *-q=g* and the accumulated load from installers -R is calculated at a distance of *0,43·l* of the span. Then, the maximum bending moment

$$M = 0{,}07 \cdot q \cdot l^2 + 0.21 \cdot P \cdot l, \text{ ga teng bo'ladi.}$$

If the value of the above bending moment is smaller than the value of the previous moment, there is no need to continue the calculation, otherwise the strength is recalculated using the following formula for the first group limit state:

$$\sigma = \frac{M}{W} \le R_{Эд}$$

where: $R_{EG} = m_{i.sh} \cdot R = 1{,}2 \cdot 13 = 15{,}6$ MPa, va $R = 13$ MPa equal to; m_N- a coefficient that takes into account the operating conditions in temporary loads.

Lattice battens work on pitched roofs in the inclined bending position and are also calculated using the following formulas:

$$\sigma = \frac{M_x}{W_x} + \frac{M_y}{W_y} \le R_{ЭГ} \quad \text{va} \quad f = \sqrt{f_x^{\,2} + f_y^{\,2}}$$

1 meter or the width of one board is taken as the calculation width. If the board step is greater than 15 cm, each board is subjected to a load of $R=1,2\ kN$, if the step is less than 15 cm, then a load of $R \cdot cos\alpha/2$ is applied.

When calculating two-layer floors, only the working floor is considered to be bent according to the standard load, because the protective boards of the floor receive the part of the load distributed along the slope. The calculated width is taken as b=1 m. The accumulated load is considered to be spread over 0.5 meters, and since the calculated width is 0.5 m, the value $R=1,2\ kN/0,5=2,4\ kN$ is obtained.

Laminate floors are large sheets made in factory conditions. Their length is $l=3 \div 6\ m$ and width $b=1 \div 1,5\ m$. The plate consists of a wooden frame and glued plywood covers. Glued mats perform the functions of a mattress and a blanket. The frame consists of longitudinal and transverse plank ribs, and their thickness is not less than 25 mm. Longitudinal ribs are placed in steps of no more than 0.5 m, and transverse ribs are placed at a distance of no more than 1.5 meters. Plywood with a thickness of not less than 8 mm is used as a coating.

Glued floor slabs must rest on the main load-bearing structures of at least 55 mm. Laminated floor slabs can be made with upper and lower plywood cover and boxed appearance. the geometric characteristics of the calculated cross-section of the laminated floor slabs can be determined in the form of a symmetrical joint or an asymmetrical shape.

The neutral axis is at $Z = h/2$ when the height of the cross section is h, and when the cross section is $Z = S/A$,

where: S - static moment of wall and belt; A- section surface.

The compressive and bending strength of the upper coating is calculated using the following formula:

$$\sigma=M/(\varphi_f \cdot W) \le R_{f.s.},$$

where: $R_{f.s.}=12\ MPa$, φ_f - the coefficient of superiority of plywood, it is determined using the following formula and inequalities:

$$\frac{a}{\delta} \ge 50 \text{ in, } \quad \varphi = \frac{1250}{(a/\delta)^2};$$

$$\frac{a}{\delta} < 50 \text{ da , } \quad \varphi_\phi = 1 - \frac{(a/\delta)^2}{5000};$$

where: a- distance between longitudinal ribs, δ- the thickness of the plywood.

The bending elongation of the subfloor is calculated using the following formula:

$$\sigma = M/(W \cdot m_\phi) \le \quad R_{\phi.ч} \qquad (4.11)$$

where : $R_{\phi.ч}$ - calculated tensile strength ($R_{\phi.ч}=13\ MPa$); m_ϕ q0,6- coefficient that takes into account the weakness in the joints of the plywood.

Lecture 10. Design and calculation of cover plates.

In the case of roll roofing in the uninsulated coatings used solid board decking. In insulated coatings on top of these decks laid solid board insulation, directly on which or the levelling layer glued roll carpet. A variant is possible, when the insulation is laid between purlins with plasterboard ceiling lining.

When scaly roofing of asbestos-cement or fiberglass sheets in the uninsulated coatings used rarefied board decks (crates).

Flake roof is impermeable due to loose joints, so rarefied flooring provides ventilation of the cavities beneath it and the drying of the wood in operation. Discharged planking can also serve as a base for tile roofs and steel sheet roofs.

Planking is made of boards on nails and laid on purlins or main bearing structures of the roofing at a distance of not more than 3 m between them. The working boards of the decking should have a length long enough to support them on at least three supports to increase their bending stiffness compared with a single-span support.

Discharged planking

Spread decking, also called purlins, is a non-continuous row of boards laid with a pitch determined by the type of roofing and calculation. The gaps between the edges of the boards for better ventilation should be at least 2 mm.

To accelerate the assembly of this flooring is advisable to collect from prefabricated boards, connected to the bottom of the crossbar and struts.

Continuous decks

Of the solid decking, the most common is the double cross decking, which consists of two layers - the lower working and the upper protective.

The working deck is a discharged or continuous row of thicker boards and bears all the loads acting on the pavement.

The protective planking is a continuous row of boards with a minimum thickness of 16 mm. It is laid on the working planking at an angle of 45^o-60^o and nailed to it.

Double cross decking has considerable rigidity in its plane and serves as a reliable link between the runs and basic load-bearing structures of the covering. It is advisable to assemble this planking also from prefabricated large boards.

The decks are also used from solid single-layer boards connected at the bottom with struts and cross-bars, which have less rigidity than the double ones.

Calculation of board decks is made on the strength and bending deflections for the action of design and normative loads:

- of constants from the own mass of the pavement g, kN/m^2

- постоянных от собственной массы покрытия *g*, кН/м^2

1. temporary from the snow mass p, kN/m^2
2. from the weight of a person with a load P, kN

Loads are determined by taking into account the shape of the pavement and overload coefficients.

The concentrated load from the weight of a person with a load has values:

$P^н$=1 кН (100 кg.), and taking into account the overload factor: *P=1,2 кN (120 кg).*

Calculation of decks and purlins, working, as a rule, on the transverse bending, made by the scheme of the two-span beam at two combinations of loads:

1) the load from own weight of the roof and snow load (g+p)

- for strength:

$$\sigma = \frac{M_{\max}}{W_{расч}} \le R_u ,$$

where:

$$M_{\max} = \frac{q_x l^2}{8} ;$$

<u>- on deflections:</u>

$$\frac{f}{l} = \frac{2{,}13}{384} \frac{q_x^н l^3}{EI} \le \left[\frac{f}{l}\right] \text{, где}$$

$$\left[\frac{f}{l}\right] = \frac{1}{150}$$

<u>2) load from own weight of the pavement and concentrated load in one span from the weight of a person with a load P - only on the strength</u>

<u>The maximum moment is under the concentrated load:</u>

$$M_{\max} = 0{,}07 g_x l^2 + 0{,}21 P_x l .$$

The strength calculation in this case is made by the same formula as in the previous one

It is convenient to calculate by taking the width of the flooring b = 100 cm.

In case of continuous decking or purlins, if the distance between the axes of boards or bars does not exceed 15 cm, assume that the concentrated load is transferred to two boards or bars, and at a distance of more than 15 cm - one board or bar.

In case of two decks (working and protective, directed at an angle to the working one) or in case of one-layer decking with a distribution bar, filed at the bottom in the middle of the span, as well as in case of laying over the decking of a board insulation, the concentrated load R_n = 1 kN is taken to be distributed over the width of 0.5 m of the working deck.

Board planking of flooring, wall lining and cladding

Flooring

These are solid rows of planks that serve as the base of the finished floor or the finished floor itself. They are laid on the intermediate joists - joists or directly on the beams and nailed to them. Decking boards for clean flooring connect the edges of the tongue and groove. Floor decks work and calculated for the bending of the action of loads of its own mass, useful loads equal to 1.5 kN/m^2. in residential, and at least 2 kN/m^2 (200 kg/m^2) in industrial buildings and concentrated loads equal to 1.5 kN (150 kg). The maximum deflection of the flooring must not exceed 1/250 of the span. Additionally check the shakiness of the deck. The checks consist in the fact that its deflection from the concentrated load of 0.6 kN should not exceed 0.1 mm.

Ceiling linings

These are continuous rows of thin boards nailed to the beams below. In the absence of plaster, the boards are connected by tongue and groove edges to eliminate through gaps. Liners work on bending, and nails - to pull, as a rule, with an excessive margin of safety under the load of its own mass.

Wall cladding

Represents a continuous vertical rows of thin boards arranged horizontally and connected by the edges in the quarter or tongue and groove. Wall sheathing works to bend from pressure and wind suction, usually with an excessive margin of safety.

Clayface decks

Glued plywood decks are the most effective and promising type of building envelopes. The boards consist of a board frame and plywood cladding connected by glue They have length l=3-6 m, width b = *1*- ,5 m, corresponding to the size of the plywood sheet.

The frame of the panels consists of longitudinal and transverse ribbed boards, which may be solid or glued. Longitudinal ribs, solid along the length, are set at a distance of not more than 50 cm from each other. Transverse ribs are set at a distance of not more than 1.5 m, as a rule, at the joints of plywood, and interrupted at the intersections with longitudinal ribs. Panel cladding consists of FSF plywood sheets of high water-resistance, at least 8 mm thick, butted together on the length "by ear". Plywood panels are glued to the frame in such a way that the direction of plywood outer fibers coincides with the direction of longitudinal ribs to make the plywood work in the direction of its greater strength and stiffness. Plywood panels act as decking, purlins, water and vapor barriers. They are characterized by low weight with considerable carrying capacity, have great rigidity in their plane. Surfaces of the panels facing the interior of the room are coated with fire retardant compositions to increase their degree of fire resistance.

According to the shape of the cross-sectional section, the glued panels can be of the following types:

1) box-shaped;

2) ribbed, with the cladding upwards;

3) ribbed, cladding downwards

Korobtsovuyu gluefanernaya panel is used in insulated coatings with a roll roof and a smooth ceiling It has a double-sided sheathing, which together with the ribs form a series of cavities in which a layer of vapor barrier stacked insulation. All the cavities of the flooring panels are connected with holes in a single ventilated layer (dehumidifying vent), communicating with the outside air, which provides a drying mode of operation of the flooring.

Ribbed adhesive panel with paneling upwards is used in cold and insulated coatings with a roll roof without a smooth ceiling. It has only one top sheathing, on top of which the insulation and roll carpet are laid.

Ribbed adhesive veneer panel with sheathing facing down is used in insulated and cold roofings with a roof of corrugated asbestos cement sheets. It has only one bottom sheathing. Roofing sheets are laid along the longitudinal ribs, and the insulation is placed along the sheathing between the ribs.

The most common are box-shaped adhesive panels, which are used not only as cladding roof structures, but also as wall panels.

Glued panels are supported on the main load-bearing structures with the width of the support platforms at least 5.5 cm They are attached to the supports with screws or nails.

To ensure the joint deflections of the entire flooring panels are connected to each other along the edges. Connect can be blind dowels, which are placed every 1.5 - 2 m or nails, punching through the connecting strip at 50 cm.

Lecture 11. Design and calculation of wooden columns.

The loads absorbed by the flat load-bearing roof structures (beams, roof arches, trusses) are transferred to the foundation through the studs or columns.

In buildings with wooden load-bearing roof structures, it is advisable to use wooden studs, although sometimes it is necessary to install reinforced concrete or metal columns.

Wooden posts are compressed or compression-bent load-bearing structures supported by foundations. They are used in the form of vertical rods supporting a covering or slab, in the form of struts of strut systems, in the form of rigidly terminated struts of single-span or multi-span frames.

By design they can be subdivided into glued posts and posts made of solid elements.

Glued columns

Board-glued and glue-phaner posts are factory-made elements.

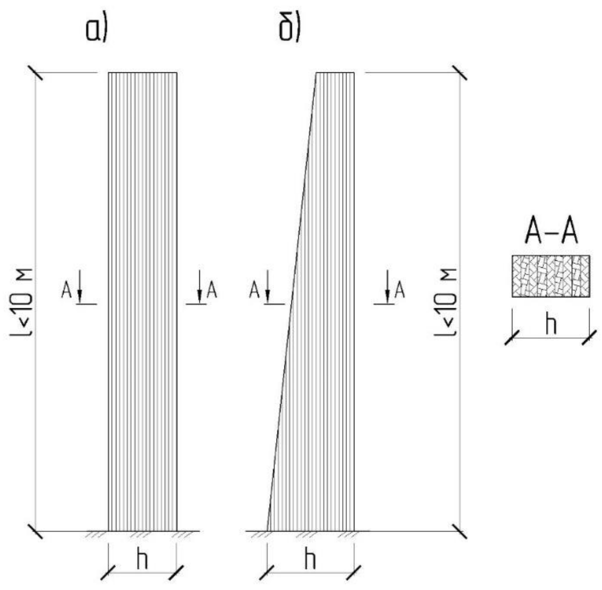


Figure 12 - Board-glued uprights
a) constant rectangular and square cross section;
b) variable rectangular section

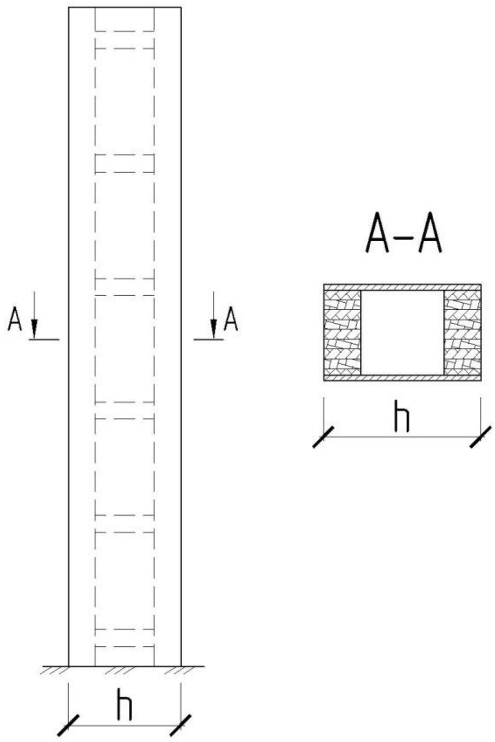


Figure 13 - Glue stakes

Glued stands can have a larger cross section and a height of up to 8-10 m. For their manufacture, wood of 2 and 3 grades is used. The advantages of such props are their industrial nature, simplicity of transportation and installation.

Solid element racks

They are subdivided into the following types:

1) in the form of a single beam or log

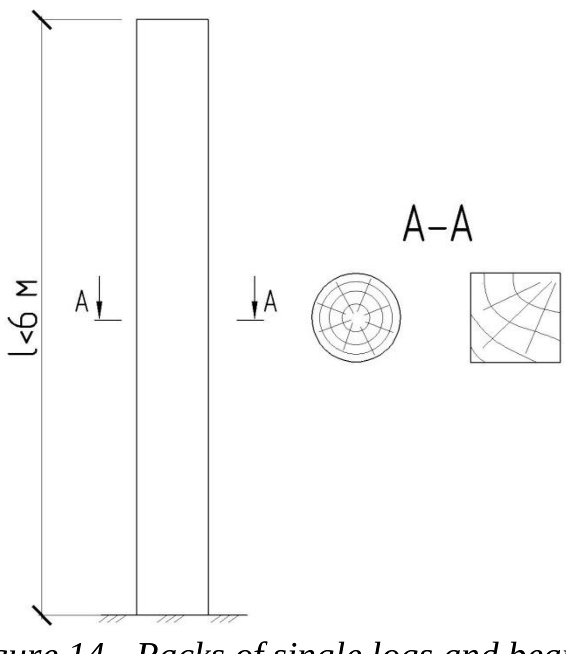


Figure 14 - Racks of single logs and beams

Such props have a relatively low load-bearing capacity. Their height and cross-section size are limited by the timber assortment.
These props are usually hinged to the foundation.
1) Stands in the form of composite section elements assembled of two or more beams, planks or logs connected by bolts or other pliable connections.

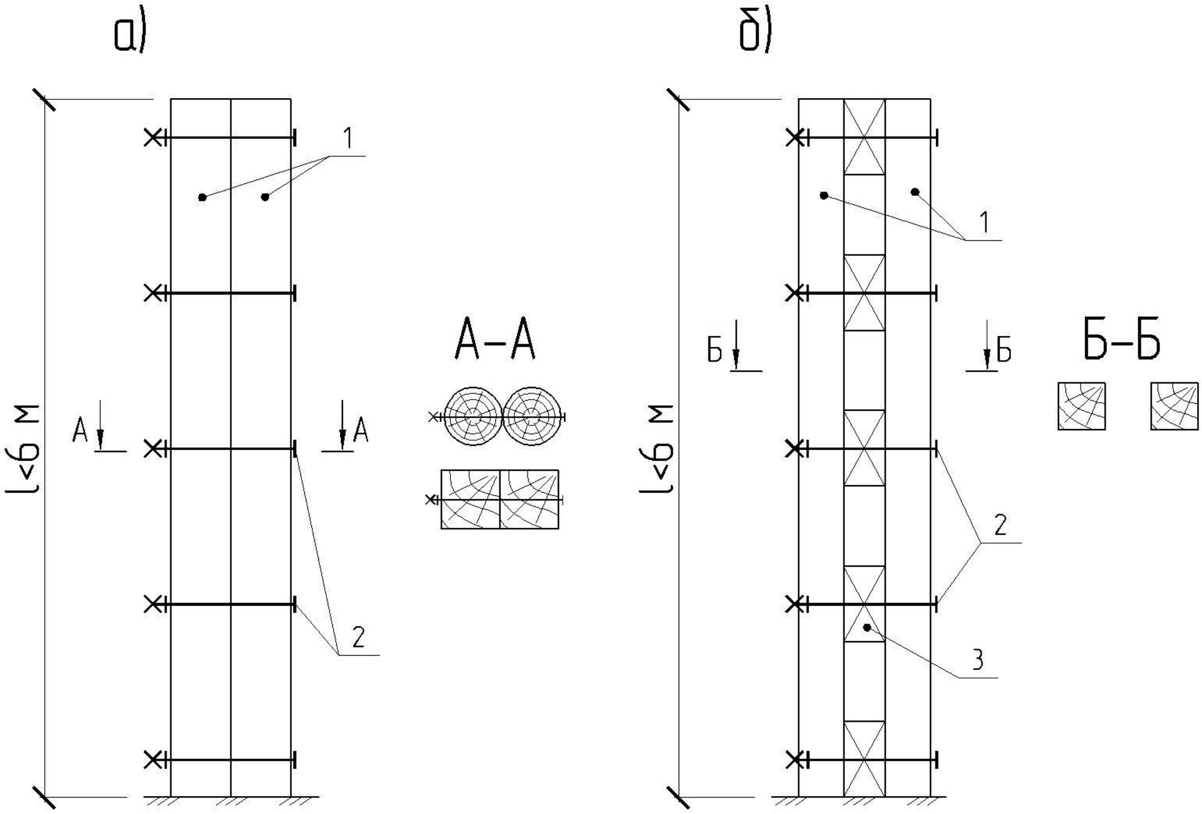


Figure 15 - Composite Bar Struts
a) solid; b) through with spacers; 1 - bars; 2 - bolts; 3 – spacers

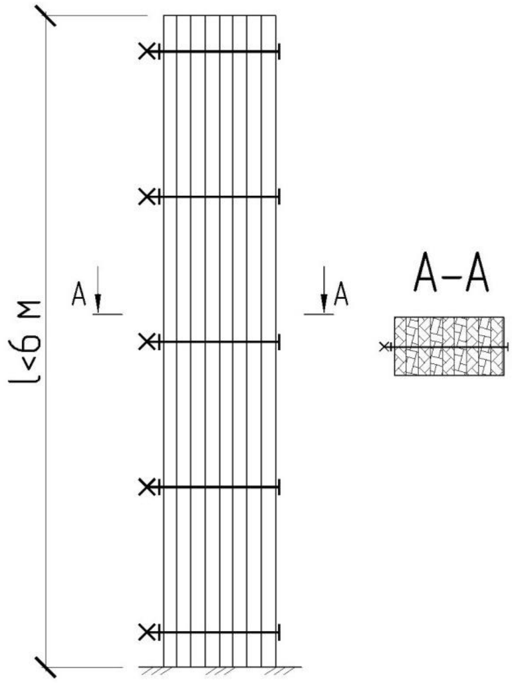


Figure 16 - Composite plank column

The composite section props also have a limited height, but their load-bearing capacity can be significantly higher than that of the single section props.

The connections used to join these props (bolts, nails, dowels) are malleable, which increases the flexibility of the props and must be taken into account in the calculation.

Lattice props.

Most commonly used as compression-curved frame struts. They can be with parallel chords or with one sloping chord. A variation of the latter are triangular struts.

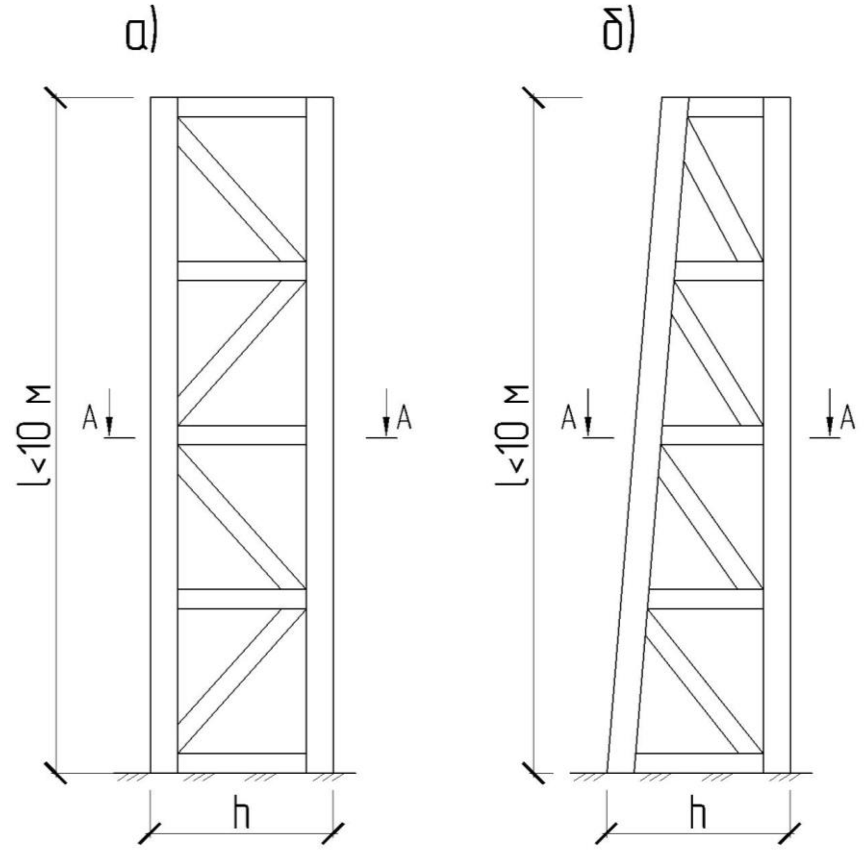


Figure 17 - Lattice props

a) rectangular; b) triangular

b) Elements of lattice props are connected in knots on bolts.

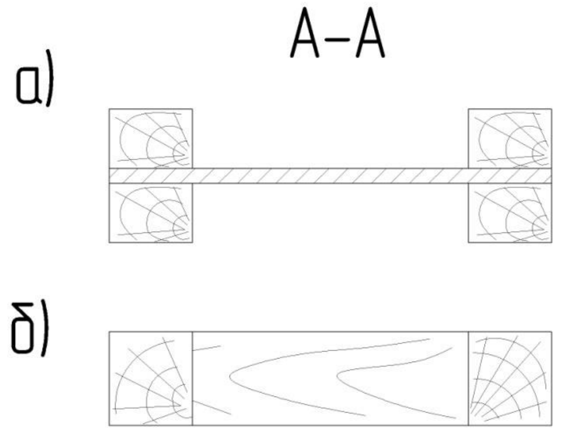


Figure 18 - Sectional view of a lattice post

a) Belts of two branches, lattice of one branch; b) Belts and lattice of one branch

If the lattice is made of one branch and the belts of two (Fig. 7a), the lattice is passed between the branches of the belts and fastened directly to the latter. If the ribs and the lattice are made as one branch (Fig. 7b), then the connection between the lattice elements and the ribs is made buttwise, and the joints are bolted together with steel overlays.

The parallel girders can be staggered. In this case, the higher outer chord is supported by the bearing structures of the covering and the inner chord by the crane girders.

Calculation of columns

Calculation of forces in the struts is made taking into account the loads applied to the strut.

Mid-columns

The middle columns of the building frame work and are calculated as centrally compressed elements for the greatest compressive force N from the own weight of all the roof structures (G) and the snow load and snow load (P_{cn}).

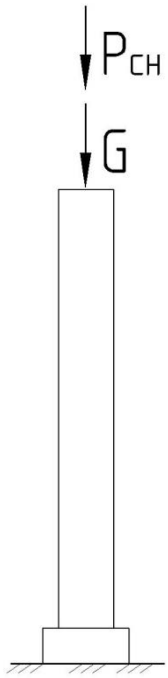


Figure 19 - Loads on the middle column

Calculation of centrally compressed middle props is carried out:

a) for strength

$$\frac{N}{F_{\text{НТ}}} \leq R_{\text{c}} \,,$$

where R_c is the design resistance of wood to compression along the fibers;

F_{nt} - net cross-sectional area of the element;

b) for stability

$$\frac{N}{\varphi \cdot F_{\text{расч}}} \leq R_c,$$

where φ is the coefficient of longitudinal bending;

F_{calc} - design cross-sectional area of the element;

Loads are collected from the coverage area of the plan, falling on one middle strut ($S_{\text{ср}}$).

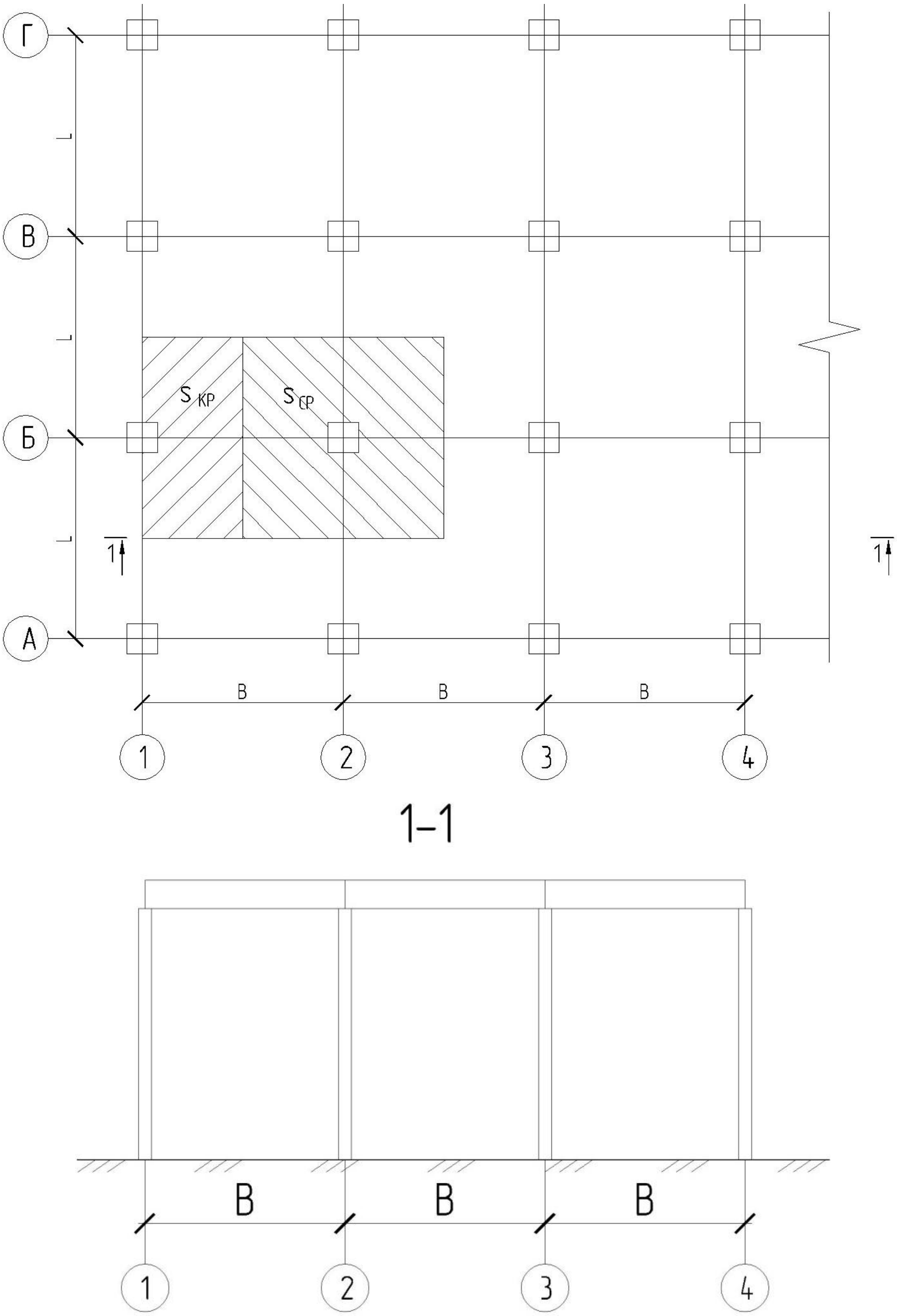


Figure 20 - Load areas of the middle and outermost columns

The outermost post is under the action of longitudinal loads (G and P_{cn}), which are collected from the area S_{cf} and transverse P_1, P_2 and *X*. In addition, the longitudinal force P_B arises from the action of the wind.

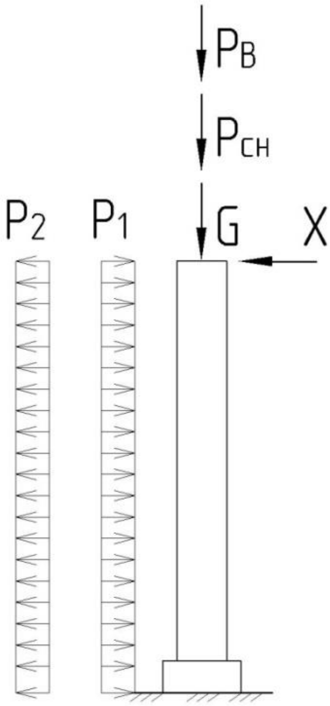


Figure 21 - End column loads

G – load from the own weight of the coating structures;

P_{CH} – load from the weight of the snow cover;

P_B - vertical wind load;

P_1- wind load from the wind on the left (wind pressure);

P_2 – wind load (suction) with the wind on the right;

X – wind load (suction) with the wind on the right;

In the case of rigid embedding of props for a single-span frame:

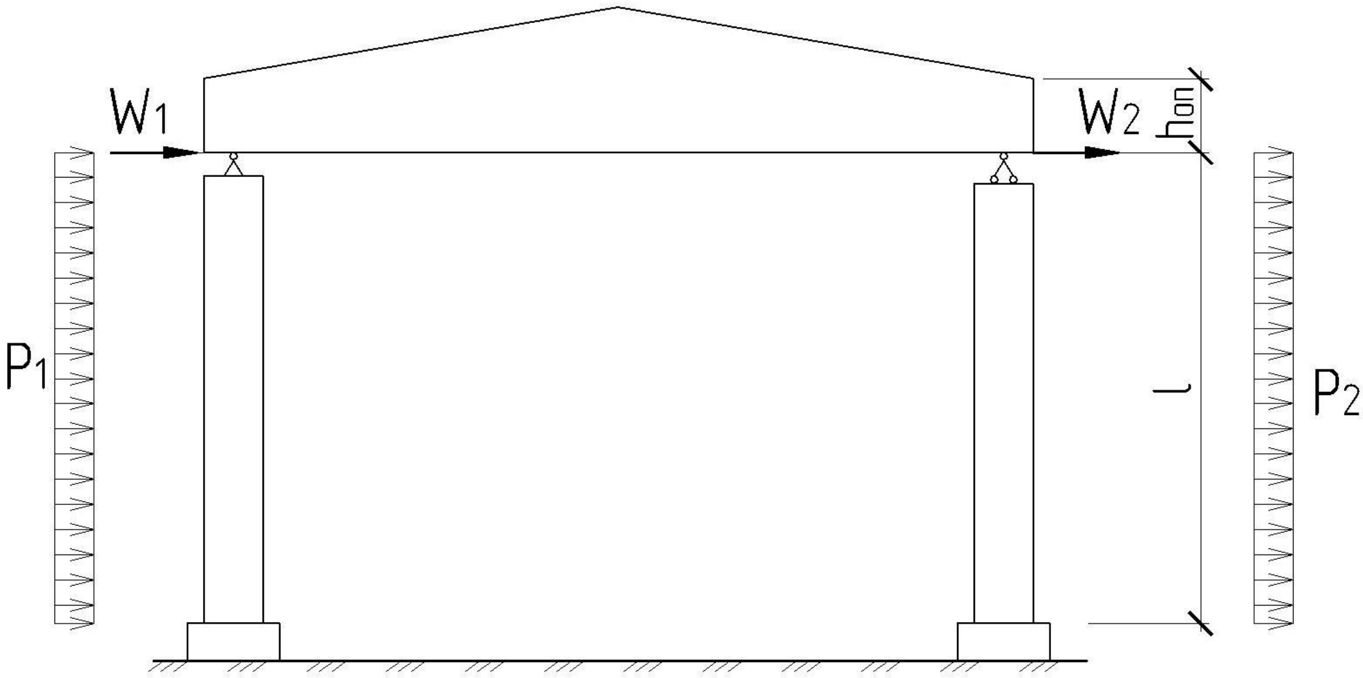


Figure 22 - Scheme of loads when the uprights are rigidly jammed in the foundation

$$X = \pm[0.188 \cdot l \cdot (p_1 - p_2) + 0.5 \cdot (W_1 - W_2)]$$

where: W_1, W_2 - are horizontal wind loads from the wind on the left and right, respectively, applied to the post at the junction point of the transom.

$$W_1 = p_1 \cdot h_{\text{оп}} \qquad W_2 = p_2 \cdot h_{\text{оп}}$$

where $h_{\text{оп}}$ - the height of the support section of the ledger or beam.

The influence of the forces W_1 and W_2 will be significant if the transom on the support has a significant height.

In the case of articulated support of the strut on the foundation for a single-span frame:

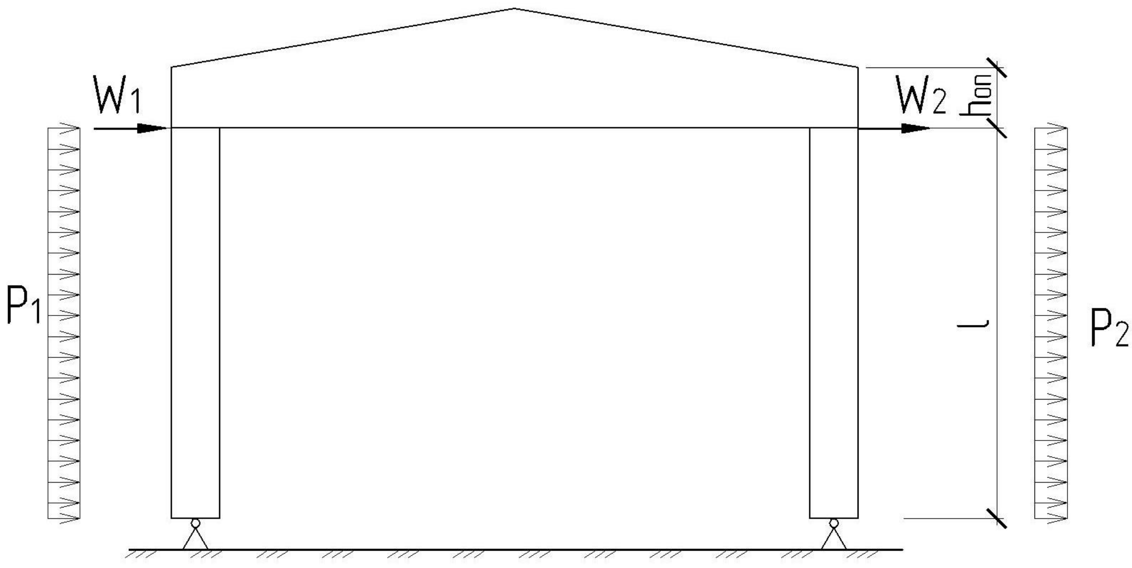


Figure 23 - Scheme of loads at the articulated support of the columns on the foundation

$$x = [0.313l \cdot (p_1 - p_2) + 0.5(w_1 - w_2)]$$

For multi-span frame structures, p_2 and w_2 will be zero when the wind is on the left and p_1 and w_2 will be zero when the wind is on the right.

The edge struts are calculated as compression-bending elements. The values of the longitudinal force N and the bending moment M are taken for the combination of loads at which the greatest compressive stresses occur.

The eccentricity is equal to:

$$e = \frac{h - h_{\text{В}}}{2}$$

It is recommended to determine as max for the following load combinations:

1) 0.9(G + P_c + wind on the left)
2) 0.9(G + P_c + wind on the right)
3) G + P_c

For the strut included in the frame, the maximum bending moment is taken as max from the calculated M_l on the left and M_{pr} on the right for the wind case:

$$M_{л} = 0.5p_1 l^2 - Xl - N\text{e}$$

$$M_{\text{пр}} = -0.5p_2 l^2 - Xl - N\text{e},$$

where: e is the eccentricity of application of longitudinal force N, which includes the most unfavorable combination of loads G, P_c, P_b - each with its own sign.

The eccentricity for struts with constant section height is zero (e=0), and for struts with variable section height is taken as the difference between the geometric axis of the support section and the axis of application of the longitudinal force.

Calculation of compression-curved extreme struts is performed:

(a) For strength:

$$\sigma_{\text{с}} = \frac{N}{F_{\text{расч}}} + \frac{M_{\text{д}}}{W_{\text{расч}}} \leq R_c$$

б) on the stability of the flat bend form in the absence of anchoring or with the calculated length between the anchoring points $l_p > 70b^2/n$ according to the formula:

$$\sigma_{\text{с}} = \frac{N}{\varphi R_c F_{\text{бр}}} + \left(\frac{M_{\text{д}}}{\varphi_{\text{м}} R_u W_{\text{бр}}}\right)^n \leq 1$$

The geometric characteristics included in the formulas are calculated in the support section. From the plane of the frame the struts are calculated as a centrally compressed element.

The calculation of compressed and compression-curved composite sections is done according to the above formulas, but when calculating the coefficients φ and ξ in these formulas the increased flexibility of the strut is taken into account due to the suppleness of the links connecting the branches. This increased flexibility is called the reduced flexibility λ_n.

The calculation of lattice struts can be reduced to the calculation of trusses. In this case, the wind uniformly distributed load is reduced to concentrated weights at the nodes of the truss. It is assumed that the vertical forces G, P_c, P_b are taken up only by the chords of the strut.

Lecture 12. Design and calculation of wooden beams.

There are the following main types of continuous girder structures:

- beams and purlins of solid section;
- composite beams on pliable joints;
- laminated beams.

Beams and beams in one piece

The main functional purpose of beams and purlins is that they serve as load-bearing structures for roofs. Beams and purlins of full cross-section are made of ribbed boards, planks and logs, often edged on both sides. Due to the limited section sizes and timber length, such beams are used for spans of up to 6 m and relatively low loads.

Beams and purlins of cover

These structures support decks and are laid on walls, columns and main supporting structures with a spacing of 1 to 3 m. They can be:

a) single-span, freely supported;

b) multispan, continuous and cantilever-beam.

Beams and purlins are calculated for bending from uniformly distributed load q, which consists of own mass of pavement g and snow p.

The maximum relative deflection of beams and purlins of coverings should not exceed *1/200l.*

Single-span beams.

These are usually installed in relatively small roof coverings sloping along the roof slopes and supported by longitudinal walls and ridge purlins. Such structures are calculated for bending as freely supported beams. In addition to bending moments, longitudinal forces (tensile and compressive) arise in the beams due to the action of the gable component, however, due to the fact that the slopes of the beams usually do not exceed 1:2, they have little effect on the bearing capacity of the beams and therefore are not considered in the calculation.

Single span purlins

These are longitudinal rows of freely supported beams installed on the main load-bearing structures and transverse walls of the roof.

The neutral axes of the purlin sections have the same slope to the horizon as the covering (angle α).

$$q_x = q \cdot cos\alpha; \; q_y = q \cdot sin\alpha$$

The purlins are held from sliding down the slope by sections of thick boards - bosses, nailed to the supports, or by metal corners. Boards (bosses) from below at the ends of the purlins protect the main load-bearing structures from leaving their plane, i.e. these bosses play the role of connections. The outriggers are connected along the length on the supports by means of oblique chamfering or planking. The purlins are calculated for bending from the normal component of the load (q_x) only, if the gable component is taken up by the planking (as, for example, in a double cross planking). If there is no such flooring, the purlin works and is calculated for oblique bending from the normal (q_x) and gable (q_y) components of the load using the formulas for oblique bending:

$$\sigma = \frac{M_x}{W_x} + \frac{M_y}{W_y} \le R_u \,, \; \frac{f}{l} = \frac{\sqrt{f_x^2 + f_y^2}}{l} \le \frac{1}{200} .$$

The nails - the bosses' fasteners - work and are calculated for the gable component of the support reaction from the middle runs $R_y = q_{yl}$, as an asymmetrical single-cut connection with bendable nails. To reduce the calculated span of the beams, they are sometimes reinforced with sub-bars on the supports and bolted to the beams.

Twin multi-span beams

They are located across the roof slopes and are supported by the main load-bearing roof structures and transverse walls, to which they are attached in the same way as single-span purlins. Paired purlin consists of two rows of boards on the rib, connected with nails. Between the joints, the boards are nailed together with structural nails every 0.5 m. Such purlins are recommended only in combination with decking that supports the gable components.

Calculation of the coupled beam is performed according to the scheme of the multi-span continuous beam for the normal component of the load.

The maximum bending moment will be over the supports:

above the second $M_1 = \frac{ql^2}{10}$, and over the intermediate ones $M = \frac{ql^2}{12}$. Stresses are checked and sections are selected according to the moment on the intermediate supports M:

$$\sigma = \frac{M}{W_p} \leq R_u$$

The section on the second support, reinforced with a third board, usually works with a margin of safety.

Nail joints work for the transverse forces $Q_{гв}$ acting in them in the joints. Nails are calculated for bending. The deflections from the normal components of the standard load are calculated for the first span of the purlin, where the relative deflection has the greatest value:

$$\frac{f}{l} = \frac{2{,}5}{384}\frac{q_x^н l^3}{EI}.$$

In some cases it is possible to reduce the length of the first bays to 0.8*l*. In this case the bending moments on all intermediate supports and deflections of all spans can be considered equal, and there is no need to reinforce the purlin in the first spans.

Cantilever beam purlins

These are longitudinal rows of beams or logs with butt joints outside the supports.

Longer beams form two brackets in the intermediate spans and one in the outer spans, on which shorter beams are supported by an oblique deflection, tightened by a bolt. Such purlins are used in roofs when the pitch of the main supporting structures does not exceed 4.5 m, allowing the use of timber of standard length. Calculation of cantilever beam purlins is performed according to the scheme of a multispan statically determined beam with spans *l* for normal components of loads. The purlins, depending on the location of the joints, are equipotential and equipotentially deflected.

In the equimental purlins the joints are spaced at 0.15*l* and the outermost spans are reduced to 0.85*l*. The bending moments at the supports and in the spans are equal to $M = \frac{ql^2}{16}$, and the maximum relative deflections are equal:

$$\frac{f}{l} = \frac{2}{384}\frac{q^{н}l^3}{EI}.$$

In equal deflection purlins, the joints are spaced at 0.2*l*, and the outermost spans are reduced to 0.8*l*. In this case the maximum bending moments equal to $M = \frac{ql^2}{12}$, the relative deflections in all spans are equal:

$$\frac{f}{l} = \frac{1}{384}\frac{q^{н}l^3}{EI}.$$

Covering beams

Floor beams are the supports of the floor decks of intermediate floors, attic ceilings and work platforms. In most cases, they are single-span beams freely supported by walls, columns and partitions of the building. These beams work on bending from own weight of the slab and temporary useful load. They are calculated in terms of strength and bending deflections. Limit deflection $\frac{f}{l} = \frac{1}{250}$. Additionally, the interfloor beams must be tested for unsteadiness from the action of a concentrated load P=0.6 kN (60 kg) by the formula:

$$f = \frac{Pl^3}{48EI} \le 0.05\text{см}.$$

Undercuts are often made in such beams at the supports. The depth of undercutting must not exceed ¼ of the section height, the length must not exceed the section height. In this case, a check is made for shearing in the dangerous section from the action of the support reaction R according to the formula:

$$\tau = \frac{R}{bh} \le 0{,}4\,\text{MPa}.$$

Elements of composite wooden structures on pliable joints

Composite beams on pliable joints

Many wooden structures (beams, frames, arches) are made composite. The need for such structures is caused by limitations in the size of the timber in terms of length and cross-sectional area. In composite wooden structures, individual beams and boards are connected by connections that can be rigid (adhesive, providing a monolithic connection) and pliable. Composite wooden construction elements with pliable connections consist of boards connected by nails or logs and beams connected by height by bolts or wooden inserts. Compliant bracing is the ability of braces to allow the connected timbers or boards to move relative to each other when the structure is deformed. The suppleness of the bonds worsens the performance of the composite compared to the same element in a single section. Composite element with pliable connections has a lower bearing capacity and a higher deformability. Therefore, when calculating and designing composite elements, it is necessary to take into account the suppleness of ties.

Fundamentals of the bond pliability account

The issues of taking into account the suppleness of bonds in the calculation of composite rods were first developed in our country.

In this problem, the provision on the elastic work of the material of the elements and connections is accepted. In SNiP II-25-80 there are calculation formulas, which give approximate solutions, obtained from the exact solutions by a number of simplifications.

Calculation of transverse bending

In order to understand the nature of the work of elements on compliant bonds for transverse bending, let us take three beams with the same loads, spans and cross-sections. The first beam has a one-piece cross-section (C), the second has two beams without any bonds (O), and the third has two beams with pliable bonds (P).

When bending, the deformations of a composite beam on composite links will be greater than those of a solid section beam, but less than those of a beam without links:

$f_Ц < f_П < f_О$. Consequently, the composite beam on compliant links occupies an intermediate position between the beam of solid section and the composite beam without links, so we can write that during the deformation under load in the composite beam on compliant links, unlike the beam of solid section, besides the rotation of the supporting section, the shift δP of the upper chord relative to the lower.

$$W_Ц > W_П > W_О$$

$$I_Ц > I_П > I_О$$

It follows from these inequalities that the geometric characteristics of a composite beam on pliable links ($I_Ц$, $W_Ц$) can be expressed through the geometric characteristics of a solid section beam by multiplying by the coefficients kw and kzh, less than 1, which take into account the pliability of links, then:

$$W_П = k_w W_Ц\ ,\ k_W < 1;$$

$$I_П = k_{ж} I_Ц\ ,\ k_{ж} < 1.$$

The deflection of the beam on compliant links increases accordingly to the reduction of the moment of inertia:

$$f_П = \frac{f_Ц}{k_{ж}}.$$

The values of coefficients k_w and $k_{ж}$ are given in SNiP, depending on the span and the number of layers in the element. Calculation of a composite beam on composite links is thus reduced to the calculation of the beam of a solid section with the introduction of coefficients taking into account the compression of links:

1) normal stresses are determined by the formula:

$$\sigma_и = \frac{M}{W_Ц k_w} \le R_и\ , \text{где}$$

$W_ц$ – the moment of resistance of the composite beam as a single piece;

k_W<1 – is a coefficient that takes into account the pliability of the ties.

In the same way, the suppleness of the bonds is taken into account when calculating the stability of the flat bending form.

2) the deflection of the composite beam on composite links in the general case:

$$f_П = k\,\frac{p_н l^3}{EI_y k_{ж}} \le [f]пр,$$

where: I_y – the moment of resistance of the beam as a single piece;

$k_{ж}$<1 – is a coefficient that takes into account the shear caused by bonding compliance.

Composite beams on pliable bonds

Such beams are labor-intensive structures of construction production, require consumption of beams and boards of large cross-sections and are allowed for use only in temporary buildings and structures. Composite beams are also formed by reinforcing beams with insufficient load-bearing capacity with side sheathing. Composite beams on composite links include cross-side nailed beams and timber beams on oak planked dowels. These beams work for transverse bending and are calculated with regard to the pliable bonds according to the general principles discussed earlier.

Board-girder nail beams with a crosswall can have a span of up to 12 m. and a height in the middle not less than 1/7 of the span, and on supports not less than 0.4 height in the middle. These beams have an I-beam cross-section, constant in length in single girders and variable in double girders. The girders consist of double ribbed boards connected along the length with bolts.

The walls are formed from two crossed layers of boards with a thickness not less than the thickness of the boards of the belts, inclined at an angle of 30^o - 45^o to the horizon. Belts are connected to the wall with nails on both sides. The wall is connected by short structural nails. The transverse wall of these beams cannot absorb the normal stresses, but works and is calculated to absorb the transverse force. The boards of the upper girder are calculated for compression and stability. The lower girder is calculated for tensile strength over the section loosened by the joint bolts. Nails are calculated for bending from the action of shear force Q. The number of nails is reduced in steps from the supports to the middle of the span in accordance with the epigraph Q.

Beams on plate dowels (Derevjagin's beams) are formed by joining two or three beams, joined together by oak plate dowels inserted in special sockets.

During the manufacturing process, these beams are given a construction elevation, which ensures that the plates are tightly clamped in their sockets. These beams work and are calculated for bending as composite beams on compliant links, and the number of plates is determined by their bending and buckling load-bearing capacity.

The depth of dowel insertion does not exceed 1/5 of the beam height. Calculation of composite beams in terms of strength is carried out taking into account the coefficient $kw<1$, and in terms of deflections taking into account the coefficient kzh. Relative deflection of composite beams must not exceed 1/300 of the span.

Lecture 13. Design and calculation of wooden arches.

Arches as well as frame arches are spanned structures, i.e. they are characterized by the horizontal component of the support reaction (spreading).

Arches are used as the main load-bearing structures of buildings for various purposes. They are used in covers of industrial, agricultural and public buildings with spans of 12 to 70 m. In foreign construction, arches with spans of up to 100 m and more are successfully used.

According to the static scheme, the arches are divided into three hinged and two hinged without a key hinge:

They are divided according to the scheme of their support into the arches with the bolts supporting the expansion and into the arches without the bolts, the expansion of which is transferred to the supports.

In most cases, the piers are made of rebar or profiled steel. It is possible to use wooden laminated pendentives in chemically aggressive environments where metal will corrode.

According to the shape of the axis of the arches are divided into:

- triangular of straight half arches

- pentagonal

- segmented, the axes of half-arcs are arranged on a common circle

- lancet, consisting of half-arcs, the axes of which are located on two circles, closing in a key at an angle.

By design, arches are divided into:

1) one-piece (triangular shaped only);

2) arches of trusses

3) girder arches on planked dowels (Derevjagina)

4) circular arches that consist of two or more rows of posts connected by dowels

5) cross boarded arches on nails

6) glued arches (boarded and glued panel arches).

Of the above-mentioned types of arches, prefabricated glued arches are the most widely used. The spacing and load-bearing capacity of such arches can meet the requirements of the construction of different purpose coatings, including unique in its size.

The arches of the other types are of the construction type and are now almost never used. Board-glued wooden arches are a package of bent boards glued along the seam.

Board-glued arches can have any of the above types of axis, i.e. they can be triangular (without puffs - at height 1/2*l* and with puffs - at height 1/6 ... 1/8*l* in covers up to 24 m), pentagonal with bent sections in places of axle fractures, flat segmental two- or three-hinged with a lift of at least 1/6*l* (in rare cases 1/7 ... 1/8*l*) and high three-hinged pointed from elements of circular outline with a lift of 1/3 ... 2/3*l*. The last two types of laminated arches (segmental and lancet) are recommended as the main ones.

The cross-section of glued arches is recommended to be rectangular and constant along the entire length. The height of the cross section is appointed from 1/30...1/50 of the span. The thickness of layers for making arches with a radius of curvature up to 15 m is taken no more than 4 cm.

Glulam arches have prospects for lightweight pavement applications. They are usually triangular in shape and consist of box-shaped glue-veneer half arches. Such arches have a low mass and allow significant wood savings. However, they require the consumption of water-resistant plywood, are more time-consuming to produce than board-glued ones and have a lower fire-resistance limit.

Самым распространенным и перспективным видом арок являются дощатоклееные арки.

Calculation of arches

Calculation of arches is carried out according to the rules of structural mechanics, and the strut of flat double-hinged arches at the boom of lifting not more than 1/4 of the span is allowed to determine on the assumption of the hinge in the key.

Calculation of the arches after the collection of loads is carried out in the following order:

1) geometric calculation of the arch;

2) static calculation;

3) selection of sections and check of stresses;

4) calculation of the nodes of the arch.

The loads acting on the arch can be distributed and concentrated. The constant uniform load g from the mass of the covering and the arch itself is defined taking into account the pitch of the arches. It is usually conditionally considered in the strength reserve, evenly distributed along the span length, for which purpose its actual value is multiplied by the ratio of the arch length to its span *S/l*.

The mass of the arch can be set in advance using the coefficients of $k_{св}$=2…4, and determine it depending on the mass of the pavement gn, snow p and other loads from the expression

$$g_a = \frac{g_n + p + \cdots}{\frac{1000}{k_{св} l} - 1}$$

The snow load p is determined by the standards of loads and impacts, conditionally distributed evenly along the length of the span of pavement.

When calculating segmental arches at f/l≥1/8 it is also necessary to consider the distribution of snow load on triangular epiures at the value of the transition factor in the key 0, near the supports - from 1.6 to 2.2 on the one hand and from 0.8 to 1.1 - on the other.

Arched arches can be conventionally considered triangular when determining snow loads.

The wind load q is determined by the standards of loads and impacts, taking into account the pitch of the arches and is considered to be applied normally to the surface of the pavement. In this case, to simplify the calculation, the curvilinear diagrams of this load can be replaced by rectilinear normal to the chords of the half arches.

In case of lancet arches, they can be conventionally considered as triangular, and the load will be distributed normally to the chords of the half arches.

The concentrated, temporary loads *P* include the mass of suspended equipment and temporary loads on it.

The geometric calculation of the arch consists in determining all dimensions, angles and their trigonometric functions of the half-arch required for further calculations. The input data are the span *l*, the height f and, in case of lancet arches, also the radius of the half-arch r or its height f.

Based on these data, the length S/2 and the angle α of the half-arch are determined for triangular arches. In segmental arches the radius

$$r = (l^2 + 4f)/8,$$

center angle φ from the condition $tg\varphi = \frac{l}{2(r-f)}$ and the arc length $\frac{S}{2} = \pi r \frac{\varphi}{180}$ half-arch and find the equation of the arc in coordinates centered at the left leg

$$y = \sqrt{r^2 - (l/2 - x)^2} - r + f$$

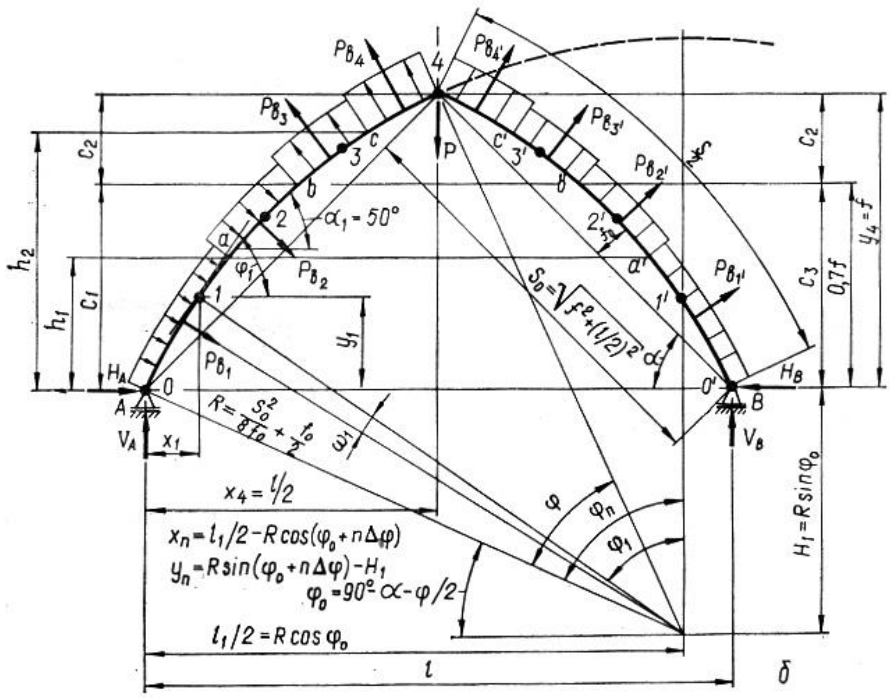


Figure 24 - Geometric and calculation diagram of the arch

In lancet arches the angle of inclination α and the length l of the chord, the central angle φ and the length S/2 of the half-arch, the coordinates of the center a and b, the angle of inclination of the reference radius φ_0 and the arc equation of the left half-arch are determined $y = \sqrt{r^2 - (a - x)^2} - b$. Then half the span of the arch is divided into an even number, but not less than six equal parts, and in these sections the coordinates x and y, the angles of inclination of the tangents α and their trigonometric functions are determined.

<u>Static calculation</u>

The support reactions of a three-hinged arch consist of vertical and horizontal components. Vertical reactions R_a and R_b are determined as in a single-span freely supported beam from the condition that the moments in the support hinges are equal to zero. Horizontal reactions (thrust) H_a and H_b are determined from the condition that the moments in the ridge hinge are equal to zero.

It is convenient to determine reactions and forces in sections of only one left semi-arch in the following order:

first, the forces from a single load on the right and left, then from left-handed, right-handed snow, wind from the left, wind from the right and the mass of the equipment.

Bending moments should be determined in all sections and illustrated with diagrams.

Longitudinal and transverse forces can only be determined in sections at the hinges, where they reach their maximum values and are necessary for calculating nodes. It is also necessary to determine the longitudinal force at the site of the maximum bending moment for the same combination of loads.

Forces from bilateral snow and dead weight are determined by summing up the forces from unilateral loads.

The results obtained are summarized in a table of efforts, which is then used to determine the maximum design forces for the main most unfavorable combinations of loads.

These sections should include:

1) dead weight and snow;

2) own weight, snow and equipment weight;

3) all acting loads, including wind loads with a factor of 0.9, introduced into the forces from live loads.

For glued arches, the "Manual" to SNiP II -25-80 recommends performing strength calculations with the following combinations of loads.

a) in gentle arches ($f < 1/3l$)

- calculated permanent and temporary (snow) load on the entire span and temporary load from suspended equipment;

- calculated permanent load on the entire span, one-sided temporary (snow) load on half of the span and temporary load from suspended equipment;

- calculated permanent load on the entire span, one-sided temporary (snow) load distributed over a triangle by $l/2$, and temporary load from suspended equipment;

b) pointed arches ($f \geq 1/3l$)

- calculated permanent and temporary (snow) loads on the entire span and temporary load from suspended equipment;

- calculated permanent load on the entire span, temporary (snow) load on S/2 or part of the span in accordance with SNiP "Loads and Impacts" and live load from suspended equipment;

- wind load with constant and other temporary (taking into account the combination factor of 0.9).

The maximum bending moments usually occur in sections near a quarter of the span of the arch under the action of one-sided live loads. In triangular arches, the moments from vertical loads are reduced due to the reverse moments M from the eccentricity e of the longitudinal forces N

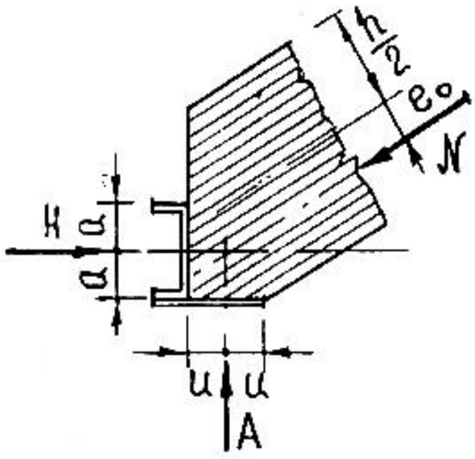


Figure 25 - Force impacts in the supporting node of the arch

The largest longitudinal forces occur in sections near the supports, and the largest transverse forces occur in sections near the hinges.

The forces in the puff suspensions arise from the loads suspended from them and from the own mass of the puffs.

The selection of sections and the verification of stresses are carried out according to the maximum values of the design forces. In this case, wind loads are taken into account only in cases where the wind increases the design forces by more than 20%.

Arches work and are calculated for compression with bending for strength and stability in and out of the plane of the arch.

The selection of sections is carried out by the method of attempts by the magnitude of the bending moment with a conditionally reduced, for example, up to 0.8 R_{u}, the design resistance of wood to bending.

When calculating arches, the following checks are performed

1. Strength test for normal stresses:

$$\delta = \frac{N}{F_{расч}} + \frac{M_g}{W_{расч}} \leq R_c$$

2. Calculation for the stability of a flat form of deformation (from the plane of the arch)

$$\frac{N}{\varphi R_c F_{бр}} + \left(\frac{M_g}{\varphi_м R_и W_{бр}}\right)^n \leq 1$$

3. Checking the stability in the plane of the arch is carried out according to the formula

$$\frac{N}{\varphi F_{расч}} \leq R_c$$

where φ= f (λ) – buckling coefficient, $\lambda = \frac{l_0}{r}$.

The estimated length of the element l_0 should be taken according to SNiP II - 25-80, depending on the design scheme and the loading scheme of the arch.

When calculating the arch for strength and stability of a flat shape, deformations N and M_g should be taken in a section with a maximum moment (M_{max}), and the calculation for stability in the plane of curvature and determining the coefficient ξ to the moment M_g should be determined by substituting the values of the compressive force N_0 in the key section of the arch, because in this section, the force is of greatest importance.

Puffs and suspension arches work and are calculated for tension.

Arch knots

The main nodal connections of three-hinged arches are support and ridge hinges.

The supporting nodes of the arches without puffs are usually made in the form of frontal stops in combination with metal shoes by welding a sheet structure, which serve to fasten them to the supports.

The shoe consists of a base plate with holes for anchor bolts and two vertical gussets with holes for semi-arch mounting bolts.

Lecture 14. Design and calculation of wooden truss

Through-bearing wooden structures are those in which the chords are connected to each other not by a continuous wall of boards or plywood (as in flat continuous structures), but by a grid consisting of individual rods - struts and supports.

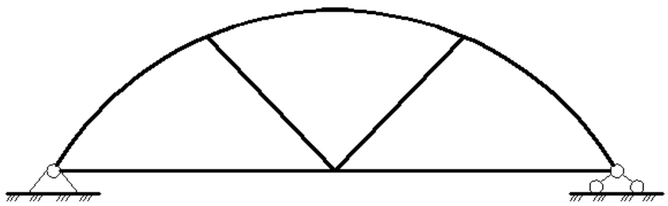
The use of a grid instead of a solid wall reduces the material consumption of the structure, especially for large spans. At the same time, the continuous structures have a large number of knots at the joints between the lattice and the girders, which makes the production of such structures much more complicated. Therefore, the choice of the type of structures - continuous or through - is made on the basis of technical and economic data, taking into account the purpose of the room.

<u>Through structures can be:</u>

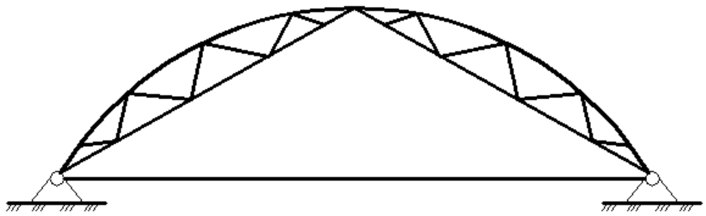
1) beams (trusses);

2) struts (arches and frames);

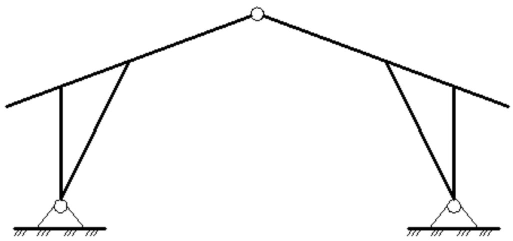
3) lattice columns.



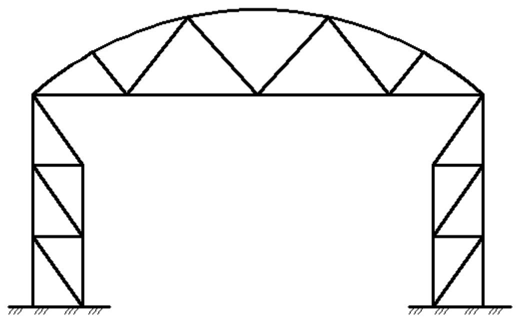
farm



through arch



braced frame



trellis rack

Today we are going to study the main type of through structures - trusses. As a rule, trusses are used in statically determined schemes with regard to both the support fixtures and the lattice solution.

Depending on the design features associated with the method of manufacture, trusses are subdivided into factory-made (of glued elements) and post-production (of solid elements).

Factory-made trusses are the most common in construction. They include metal-wood trusses, the upper chord and compressed bars of which are made of laminated timber, and the lower chord and stretched bars of the lattice are made of steel.

The advantages of laminated timber make it possible to use not only the steel bottom chord but also the wooden bottom chord if necessary, for example in corrosive environments.

According to the outline of the trusses are divided into:

1. triangular;

2. trapezoidal;

Polygonal (often pentagonal);

Segmental.

In order to reduce the magnitude of the bending moment, the transfer of the compressive force in the nodes of the upper girder from the straight elements is carried out with eccentricity, as in the arches. The first panel of the bottom chord, where there are no forces, can be made of wood, and the supporting downward strut, which absorbs a large tensile force, can be made of steel, as well as the middle panel of the bottom chord. The trapezoidal single girder has a similar design solution.

Parallel chord trusses can also be used.

Triangular laminated trusses can have an upper chord made of two laminated panels of different lengths, the longest and strongest being the first panel from the support. Two struts are also made of glulam. The bottom chord and the tensile member are made of steel. The upper chord panels are eccentric at the joints.

The segmental laminated trusses are assembled so that the arc of the upper chord is made of curved elements of equal length. All nodes, including the top chord nodes, are centered on the element axes. The upper chord of such a truss can be either split or non-split. Due to the curvilinear outline of the upper chord, a reverse bend is created in relation to the chord bending axis under the action of the external load, so this truss has a lightly loaded grid, which simplifies the design of its elements and nodes.

The trusses of post construction are those with elements made of solid logs, beams or boards with node connections on dowels (bolts, nails) or on fore-end joints. The stretched elements of the lattice and the lower chord of the truss are often made of steel.

Structural trusses can be triangular or polygonal in shape.

The trusses of the central elements with a steel bottom chord with a triangular outline allows simply organize a flat pitched roof. In these trusses, the upper chord and struts are made of beams and the central tensile member is made of round steel.

With a polygonal outline approaching the outline of the moment diagram in a simple beam, the forces in the panels of the upper chord change little from the diagram to the middle of the span and small forces occur in the lattice elements. This makes it possible to create both the upper chord and the lattice elements of wood and only the lower tensile chord is made of profiled steel.

The disadvantage of such a truss is the small number of nodes.

The lattice trusses have a triangular or pentagonal outline.

The lattice layout in these trusses is such that the wooden struts are compressed and the metal struts are stretched. This allows the compressed struts to be fixed to the chords by means of fore-end notches which only accept compressive forces, while the tensile struts are made of round steel. Ties are provided at one end with a thread and a nut, which makes it possible to seal the nodes during assembly.

In pentagonal trusses, the struts can receive tensile forces near the middle of the span under one-sided snow load and turn off from operation.

In order to maintain geometric stability, the truss lattice is provided with additional compensating downward struts.

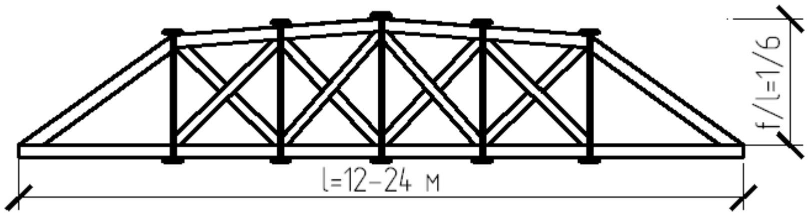


Figure 26 - Pentagonal truss made of beams or logs on front joints.

Calculation of trusses.

The procedure for calculating trusses is the same as the calculation procedure for flat load-bearing wooden structures:

1. static calculation;
2. selection of the cross-section of the truss elements;
3. calculation of nodes.

The calculation of trusses is preceded by the collection of loads. The loads acting on a truss consist of permanent (from the truss's own mass and the enclosing structures of the roof) and temporary (most often only from snow).

The static calculation of a truss is reduced to the determination of forces from external loads in the elements of the truss. For all rods, the longitudinal force N is determined, and for the upper chord, the bending moment M is also determined.

Determination of forces in the rods can be done graphically or analytically. In this case, the curvilinear axes of the top chord panels in the areas between adjacent nodes are replaced by chords which constrain these arcs.

<u>The forces are determined separately:</u>

1) for the case of loading with snow load evenly distributed on half of the span;

2) for the case of loading with snow load on the whole span;

3) for the case of loading by constant load (own weight of the truss and the weight of the enclosing structures of the roof) over the whole span of the truss.

It is advisable to first determine the force from a single load and then multiply by the values of the actual loads to obtain the true value of the forces in the rods.

When calculating the forces in the middle struts, two cases are taken into account: when the strut is compressed and when it is stretched.

<u>The design forces in the rods are determined for the following two combinations of loads:</u>

1) Evenly distributed permanent load on the whole span, temporary (snow) load on half of the truss span.

2) Uniformly distributed permanent and temporary loads over the whole span of the truss.

Selection of truss element sections.

The width of the cross-section of the truss elements is determined by the limiting value of flexibility. The following limiting values of flexibility (λ_{pr}) are set for truss elements:

- for the upper belt $\lambda_{пр}$=120 ;

- for lattice elements $\lambda_{пр}$=150 ;

- for the steel bottom chord $\lambda_{пр}$=400.

The width of the cross-section of the top chord and grid elements is advisable to assign the value of the radius of inertia.

$r = \frac{l}{\lambda_{ПР}}$, where: l – calculated length of a truss rod

$$b_{min} = \frac{r}{0{,}29}$$

The section height of the upper chord is determined using an approximate formula for the moment of resistance:

$$W = \frac{M_{MAX}}{0{,}8R_{и}}$$

The moment of resistance on the other side is equal:

$$W = \frac{b \times h^2}{6}$$

From here we find h using the known b and W.

After selecting the cross-sections of the elements of the truss, check their strength.

Compressed truss elements are tested for stability using the formula:

$$\sigma = \frac{N}{\varphi \times F_{РАСЧ}} \leq R_C$$

φ – the coefficient of longitudinal bending, taken according to SNIP;

R_C – design resistance of wood to compression.

<u>Stretched wooden elements are tested for strength according to the formula:</u>

$$\sigma = \frac{N}{F_{НТ}} \leq R_P$$

<u>steel according to the formula:</u>

$$\sigma = \frac{N}{F \times m} \leq R$$

где m – the working condition factor (if the belt consists of two elements, then m=0,85).

In the case where the upper chord is loaded with an inter-nodal load, it is tested as a compressed-curved element for strength according to the formula:

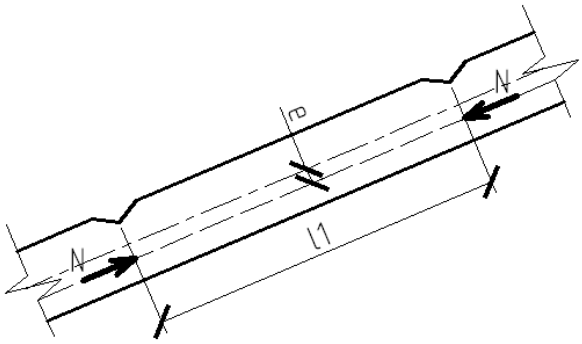
$$\sigma = \frac{N}{F_{РАСЧ}} + \frac{M_Д}{W_{РАСЧ}} \leq R_C$$

$$M_д = \frac{\xi}{M}$$

The bending moment M caused by the presence of an inter-nodal uniformly distributed load is determined by the girder formulas:

$$M = \frac{q \times l^2}{8}$$

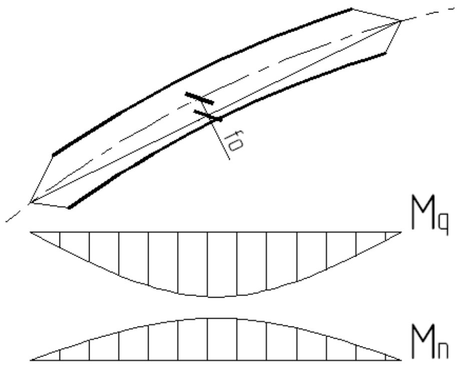
The value of the bending moment M_q can be reduced by the unloading moment M_n created by eccentrically applying a longitudinal compressive force N.



The total bending moment in the middle of the span *l*, in this case is calculated by the formula

$M=M_q-M_n$, где $M_n=N\cdot e$

In segmental trusses, the eccentricity of the force N is obtained by the curvature of the top chord panel axis.



The longitudinal force directed along the arc chord creates the unloading bending moment

$$M_n=N\cdot f_0$$

The value of f_0 can be calculated by the formula:

$$f_0 = \frac{l_0^2}{8r_0}$$

l_0 - chord length;

r_0 – the radius of the arc along which the upper belt is outlined.

For the continuous upper chord, the bending moments at the far end of the panel support will be equal:

- midspan $M^{/}_{\text{ПР.}} = \frac{q \times l^2}{14} - 0{,}64 \times N \times f$

- in support $M^{/}_{\text{ОП.}} = -\frac{q \times l^2}{10} + 0{,}72 \times N \times f$

The deflections of the trusses are not checked if the requirements for lift and span length (*f\l*) are met, since these ratios provide the required stiffness of the trusses.

To prevent undesirable effects caused by displacements of knots and deflections of the bottom chord that still occur during operation, trusses are designed with a construction elevation **(**$\sim \frac{1}{200 \times l}$ **)**. The construction lift is not taken into account when calculating the forces.

Lecture 15. Design and calculation of wooden frames.

Frame is one of the main load-bearing wooden structures. Their form is suitable for many industrial and public buildings. Frame columns and beams serve as the basis for roof and wall structures. But the frame requires a lot of wood materials and they are used at intervals of *12÷24* meters. In foreign countries, wooden frames are also used at intervals of up to 60 meters.

According to static schemes, frames are divided into static definite and static uncertain types (Fig. 45). Their advantage is that the stresses in the frame sections do not depend on the settlement of the foundation, and their nodal solutions are solved more simply. The disadvantage is that there is a large tension in the nodes.

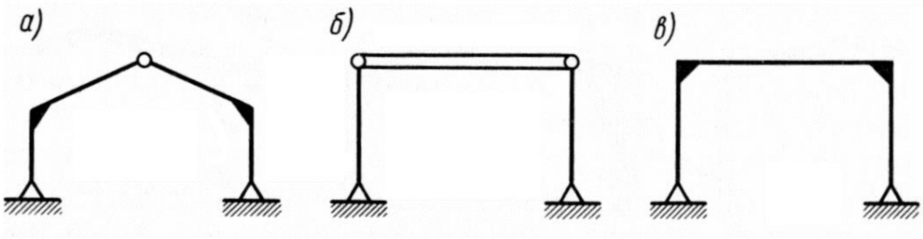


Figure 27. Schemes of wooden frames: a-three-hinged; b-double-hinged, bikr fixed; в-two-hinged hinged hinges.

A two-hinged single-base nodal scheme is statically indeterminate once. The advantage of this scheme is that the value of the bending moment at the junction of the frame beam with the column is zero. The disadvantage is the presence of single base nodes in the ram. Bikr buttresses are more complex than hinged buttresses. Two-hinged, hinged-node frames are also statically indeterminate once. Three-hinged glued wooden frames are the most common frames. They are mortared and the number of mortars can be from two to four.

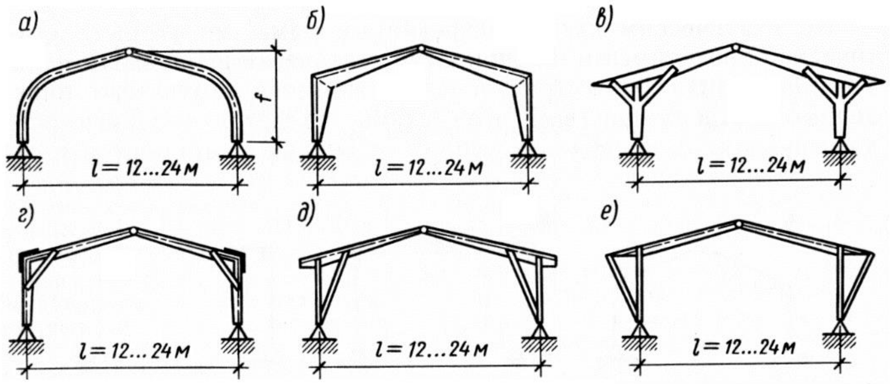


Figure 28. glued wooden three-hinged frames. a-curve is glued; b- fracture glued; v - four-stroke; г-two-tone; d- internal support with mortar; e- external support mortar.

The width of the sections of these glued wooden frames is constant, and the height of the section is variable.

Bending glued three-hinged frames, consisting of two G-shaped pentagonal half-frames. The width of the cross-section of the frame is constant, and the height of the cross-section is variable. The advantage of these rams: easy assembly of rams consisting of large semi-rams and short assembly time; variability of cutting height; where there is a maximum bending moment, it is possible to prepare a large section, and a small section if the bending moment is small (which, in turn, leads to saving wood).

Disadvantage: low level of transportability in transport (due to the large frame); the value of the compressive force in the bent part compared to the value of the straight ram.

A straight frame consists of pillars and pillars. One G-frame consists of two half-frames. The largest bending moment in the frame is created at the extension node of the frame. Since the roof of the frame is flat with a straight line, it is easy to install beams and beams, as well as make a slope on the roof. The maximum bending moment is generated in the section of the toothed seam of the plant.

Glued three-hinged four-span frame consists of two columns, two semi-serials with variable cross-section and four spans with constant cross-section. Mortars act as an additional support for the beams and, therefore, partially reduce the value of the bending moment in the beams.

Glued three-hinged double-hinged frames - consist of two columns, two semi-serials with variable cross-section and two cross-sections with fixed cross-section. The main disadvantage of this frame is the large amount of tensile stress in the extension part.

The glued wooden base consists of a three-hinged frame with internal mortar - two half-frames, two mortars and two columns. three-hinged frame with glued wooden base with external hinges - similar to internal hinged frames, only the hinges are external in these frames. Two-hinged glued wooden frames (Fig. 47) consist of three structural elements: two vertical columns and horizontal beams. These frames are easy to make compared to other frames, and since they are made up of individual parts, they are highly transportable. Fastening a horizontal roof to a pole is much easier.

Double-hinged glued wooden frames are designed with single support and hinged support. There are three main nodes in frames: base, extension, triple node. Frames are also made of whole wood. Such frames are cheaper than glued wooden frames, but they are used only in small spans (mainly up to 15 m). Mortar frames are also made from whole-cut wood. Their distance can be up to 9 m.

III- PRACTICAL TRAINING MATERIALS

In practical training, students learn the basics of calculating the load-bearing capacity of wooden and plastic structures and the cross-sectional area of the element.

1-practical training. Determination of the calculation scheme of the element in the calculation of wooden structures. Determination of forces in wooden structural elements. Construction of the Maxwell-Cremon diagram. Knot cutting method

Designing is the process of developing technical documents, which consists of technical and economic bases, calculations, drawings, estimates and other documents necessary for construction. The design institutes carry out the design based on the regulatory documents used for construction. CR (SNiP) is the main document for construction standards and regulations and State Standards (DAST).

Design and construction of cities and settlements, all types of buildings and structures, selection and design of engineering equipment and constructions, construction price determination are defined in CR. CR consists of 5 parts: 1-organization, management and economy; 2-design standards; 3-organization of production and acceptance of work; 4-estimate norms; 5. Material and labor consumption standards. QMQs are issued with a unique four-digit code. They are part number, group number, document number and year of approval. For example, CR 2.03.01-96 "Concrete and reinforced concrete structures" reads as follows Part 2, Group 3, Number 1, year of approval - 1996.

CR 2.03.08-98 Wooden structures

CR 1.02.07-97 Engineering research in construction

CR 1.01.04-98 Architectural-construction terms

ShNQ 1.03.01-03 Composition of project documents for the construction of enterprise buildings and facilities, development approval and approval

CR 2.08.01-94 Residential buildings

ShNQ 2.07.01-03 Development and construction planning of urban and rural settlements.

CR 2.04.08-96 Gas supply. Design standards

CR 3.07.03-97 Irrigation systems. Organization of production and acceptance of work

CR 3.03.04-98 Production of prefabricated reinforced concrete structures and products

ShNQ 4.02.00-04 General Regulation on the production and application of elemental resource estimate norms for construction works

The main normative document in the design of construction structures is the second part of the CR, the design norms, in its group 03, the design of individual products and structures is envisaged.

DAST defines standards, rules and requirements for technical documents (drawings, calculation standards), building materials (brick, concrete, glass), structures and items (beams, columns, window and door blocks). In addition, DASTs provide methods for testing, construction, and quality control of materials.

Buildings and structures should have the necessary strength, priority and capital. The priority and stability of the building is provided on the basis of the appropriate calculation of construction structures and the correct acceptance of the constructive solution. The capital of the building is characterized by the service life of the structures, which depends on their durability and fire resistance.

Buildings and structures are divided into 4 groups by capital. Group 1 includes buildings with high requirements for durability and fire resistance (theaters, palaces of culture, museums), as well as important objects in the national economy (hydro and power plants, metro stations). Group IV includes buildings with the least requirements (warehouses, sheds).

Long service life is the service life of the structure, during which it does not lose its operational quality, durability and superiority. According to longevity, constructions are divided into 3 categories: the first - service life of at least 100 years, the second - 50 years, the third - 20 years. The durability category of the structure corresponds to the capital group of the building.

Fire resistance is determined by the fire resistance limit and flammability group of the main load-bearing structures of buildings and structures. The fire resistance limit of the structure is determined by the time the structure can resist the effects of fire and high temperature and is calculated in hours.

Buildings and structures are divided into five categories according to fire resistance. For example, buildings with reinforced concrete and stone structures belong to category I...III (depending on the fire resistance characteristics of walls, cladding and partitions); Category IV includes wooden buildings whose construction is fireproof (CR 2.01.02-96 "Fire Resistance Standards").

2-practical training. Loads affecting wooden structures. Calculation of wooden construction elements according to limit states.

The loads affecting the structure are:

1. Permanent loads are loads resulting from the specific weights of all elements of the structure.
2. Temporary loads - loads caused by the effects of snow and wind.

Special loads are loads created as a result of earthquake, explosion, inertia force and various dynamic effects.

In the calculation of the first and second limit states, it is necessary to determine the standard and computational loads. They are determined on the basis of permanent, temporary and special loads necessary for calculations.

Permanent normative loads are determined using the volumetric weight and dimensions of the elements.

Temporarily standard snow and wind loads are determined with the help of construction standards and regulations (QMQ) maps depending on the climatic conditions of the construction site.

An example. Determine the snow and wind loads for the city of Tashkent?

From QMQ to Tashkent city, I-district in terms of snow and the load is equal to 0.5 kN|m2.

According to the influence of the wind, it is the III-region and the pressure is equal to 0.38 kN|m2.

In the calculations, the effect of loads from people and equipment included in the above loads is also taken into account. For example, during the installation of floors, workers climb on the floors and install them, and during the installation process, the additional temporary load from the weight of the person affects the structural elements. Some structures have overhead cranes and they are adapted for lifting loads. The weight of these devices is also included in the calculations.

In calculations, the specific weight of the structure is calculated using the following formula:

$$q^{м} = \frac{g^{м} + s^{м}}{\frac{1000}{K_{x.o} \times l} - 1}$$

where: q^M is the standard specific weight of the structure; g^M - standard value of external permanent loads falling on the structure; S^M - temporary standard snow load; $K_{x.o}$-coefficient of specific weight of construction (coefficient depending on the type of construction); l-interval.

Task. Determine the standard and calculated values of the snow load on the roof of a two-slope building in Namangan region. Roof slope $\alpha = 14^{\circ}$ and the value of the permanent nominal load acting on the roof covering $g^{м} = 0{,}8кН/м^2$.

Solution:

The building is located in the 1st snow district of the Tashkent region according to the map of the Ministry of Internal Affairs and Communications $S^{м} = 0{,}5кН/м^2$ is equal to The slope of the roof $\alpha = 14^{\circ}$ in 25° is equal to ($\mu = 1$ is the coefficient that takes into account the shape of the roof).

The ratio of the permanent standard load to the temporary standard snow load is calculated:

$$\frac{g^{м}}{S^{м}} = \frac{0{,}8}{0{,}5} = 1{,}6 \quad \text{ga teng.}$$

So, since it is 1,6≥1, the reliability coefficient for snow load is equal to $\gamma = 1{,}4$.

Then the value of the calculated snow load per 1 m^2:

$$S = S^{м} \cdot \gamma = 0{,}5 \cdot 1{,}4 = 0{,}7кН/м^2.$$

A limit state is a state in which it is impossible to use constructions resulting from the influence of external and internal stresses.

Wooden and plastic structures are calculated according to two groups of limit states: load-bearing capacity and deformation.

The first limit state is the most dangerous. In the first limit state, the structure loses its load-bearing capacity as a result of failure or loss of priority. This situation does not occur if the maximum values of normal and test stresses do not exceed the minimum calculated resistance value of the materials. This condition is expressed in the form of the following formula:

$$\sigma \;\; \textit{or} \;\; \tau \le R$$

where: σ - normal stress; τ - test voltage; R is the calculated resistance.

The second limit state is relatively safer. In this case, the structure is considered unusable under normal conditions. This situation does not occur if the maximum relative deviation does not exceed the permissible limit value. This condition is expressed using the following formula:

$$f/l \le [f/l]$$

where: f and $[f]$ - actual and allowable deflections.

The main goal of performing calculations is to avoid the first and second limit states. In the calculation of wooden structures according to the first limit state, the calculation load is used, and in the calculation according to the second limit state, the standard load is used. Professor A. S. Streletsky developed the basic system of discretionary engineering calculation. In this case, the condition of not breaking and not breaking must be met. Based on this system, the limit load should be smaller than the minimum load carrying capacity of the structure. When calculating according to the second limit state, the modulus of elasticity of wood is equal to E=10000 MPa along the fibers, and E_{90}=400 MPa in the direction transverse to the fibers. The shear modulus is equal to 500 MPa in the directions along the wood fibers and transverse to the fibers.

3-practical training. Calculation of elements acting on central compression.

Central compression. In compression members, knots and oblique layers of wood have less influence on its strength than in tension, while local weaknesses such as grooves and holes significantly reduce the load-bearing capacity in exchange for the plastic properties of wood in compression. does not cause a decrease.

According to the quality requirements for wood, compressible elements belong to II class elements.

The strength of central compressive elements is calculated according to the following formula:

$$\sigma_c = \frac{N}{A_{нт}} \leq R_c \, , \tag{2.2}$$

where: σ_c - compressive stresses, MPa;

N – calculated compressive force, kN;

A_{nt} – net cross-sectional area of the compressible element, sm^2;

R_s – calculated resistance of wood to compression along the fibers, MPa.

Durability calculation is mainly performed for short rods of conventional length. Long rods that are not fixed with ties in the transverse direction are considered longitudinal bends; they lose their linear position under the influence of force, which is called loss of priority. When this priority is lost, the axis of the stern is bent at stresses lower than the limit of the compressive strength of the wood. The priority of the stern is determined by the critical load calculated according to Euler's formula:

$$N_{кр} = \frac{\pi^2 E \cdot J}{l_o^2} \, , \tag{2.3}$$

where: E- modulus of elasticity;

J - minimum moment of inertia of the stern;

ℓ_o- is the calculated length of the stern, determined by the following formula: $\ell_o = \mu_o \cdot \ell$;

ℓ - free length of the stern;

μ_o - the value of the coefficient is taken as follows:

1) loaded with longitudinal forces on the ends of the boom: if the edges are hinged, and if the element is hinged at intermediate points μ_o=1; if one end is hinged and the other is fastened with a bikr (hip) mahkamlansa μ_o =0,8; if one end is clamped and the other - free end is loaded μ_o=2,2; if both edges are clamped μ_o =0,65;

2) Longitudinal load is evenly distributed along the length of the element: if both edges are hinged μ_o=0,73; if one end is clamped and the other is free μ_o=1,2.

The critical stress is found by the following formula:

$$\sigma_{кр} = \frac{\pi^2 \cdot E}{\lambda^2}, \qquad (2.4)$$

where: $\lambda = \frac{\ell_o}{r}$ - inclination (flexibility) of the stern;

$r = \sqrt{\frac{J}{A}}$ - radius of inertia of the stern.

Longitudinal bending is calculated according to the following formula:

$$\sigma_c = \frac{N}{\varphi \cdot A_{хис}} \leq R_c \qquad (2.5)$$

where: $\varphi = \frac{\overline{A}}{\underline{\lambda^2}}$ - longitudinal bending coefficient; for wood Ā=3000, for plywood Ā=2500, for polyester fiberglass Ā =1097, for organic glass Ā=580. Limit inclination of structural elements ($\lambda_{\max}$) It should not exceed the values given in table 2.1;

A_{his} - The computational surface of the cross-section of the element is equal to the following:

1) in weakening that did not reach the edge of the element (Fig. 2.3): a) if their surface If $A_{zaif} \leq 0{,}25\ A_{br}$, $A_{his}=A_{br}$; b) if their surface $A_{zaif}>0{,}25\ A_{br}$, $A_{хис} = \frac{4}{3} A_{нт}$;

2) in the case of symmetric weakening of the edge of the element (see Fig. 2.3): $A_{his}=A_{nt}$;

where: A_{br} – gross cross-sectional area, A_{nt} - netto cross-sectional area.

The value of Coefficient of longitudinal bending φ can also be taken according to the graph presented in Figure 2.4.

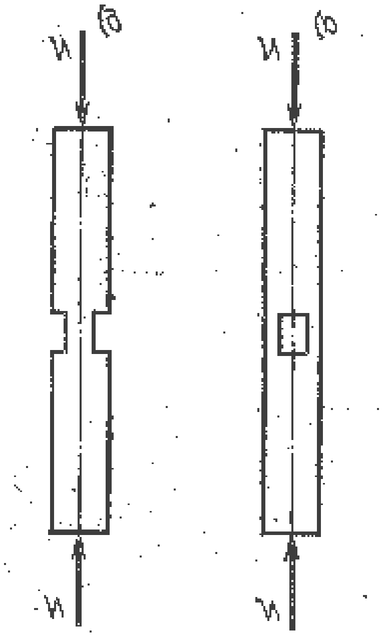


Figure 3.1. Weakenings in compression elements:
a - does not go to the edge;
b - that goes out to the edge

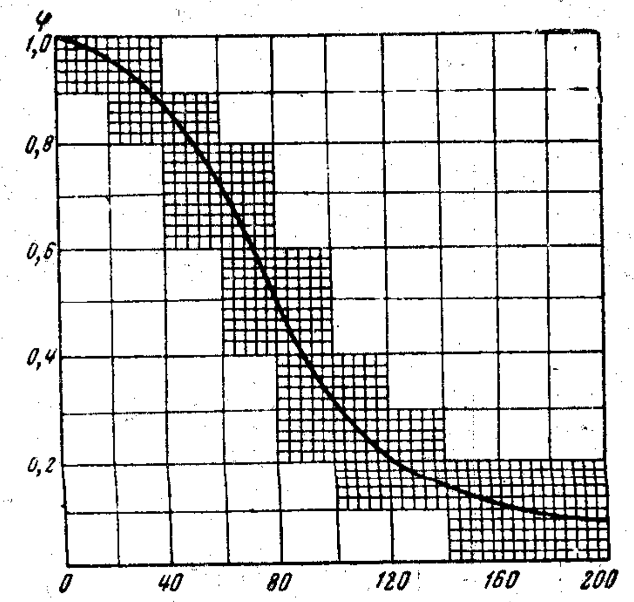


Figure 3.2. Longitudinal bending coefficient at different inclinations

Limit values of construction elements

Table 3.1

Elements of constructions	Boundary inclinations, λ_{max}
Compression belts of farms, support beams and support columns; columns	120
Remaining compressive elements of trusses and other lattice structures	150
Compressible elements of connections	200
Tensile belts of farms in the vertical plane	150
Remaining tensile elements of trusses and other lattice structures	200

4-practical training. Calculation of elements working on the central stretch.

Central stretch. Tensile stresses are dangerous for the elements of wooden constructions, because in such a state of stress, wood defects, especially the presence of branches and oblique layers (marking), can lead to brittle failure.

Branches not only weaken the cross-section of the element, but due to their symmetrical location, they cause an uneven distribution of stresses in the weakened cross-section, as a result of which the resistance of the tensile element decreases sharply.

In the places where there are grooves and holes in the elastic elements, there is a concentration of stresses, as a result of which the strength of the weakened element decreases.

Tensile elements of wooden structures are type 1 elements, therefore, high-quality wooden materials with the least defects should be used in their preparation. The strength of wooden elements operating in central tension is calculated according to the following formula:

$$\sigma_{чуз} = \frac{N}{A_{нт}} \le R_p \cdot m_o ,$$

where: σ_p - tensile stresses;

N – calculated stretching force, kN;

A_{nt} – symmetrical weakened cross-sectional surface of the element, sm^2;

R_R – calculated resistance of wood to stretching along the fibers, MPa, is accepted according to table 3;

m_o = 0,8 the coefficient takes into account the stress concentration in weakened areas.

When determining the surface of the weakened section Fnt, all weaknesses located in a section up to 200 mm in length are considered to be located in one section

Task1 Check the strength of the elastic element, weakened by the grooves and holes opened for the installation of cross-section bolts (Fig. 2.2). Calculated stretching force N=75 kN; dimensions of the section of the element w_{xh}=15x20 cm; depth of grooves on both sides h_{vr} = 3.5 cm; holes diameter d=1.6 cm. The element is made of type 1 pine wood.

Kesimi boltlar o‘rnatilishi uchun ochilgan o‘yiqlar va teshiklar bilan zaiflashtirilgan cho‘ziluvchi elementning mustahkamligi tekshirilsin (2.2-rasm). Hisobiy cho‘zuvchi kuch N=75 kN; element kesimining o‘lchamlari bxh=15x20 sm; har ikkala tomondan o‘yiqlarning chuqurligi h_{vr} = 3,5 sm; teshiklar diametri d=1,6 sm. Element 1-navli qarag‘ay yog‘ochidan tayyorlanadi.

Solving:

Stergen gross cross-section surface:

$$A_{br}=bxh=15\cdot 20=300\ sm^2.$$

Cut surface weakened by grooves:

$$A'_{weak}=2\cdot 3,5\cdot 20=140\ sm^2.$$

The distance between the holes opened for bolts is S=10<20 cm, so we assume that the weakening of all three holes is located in one section.

Weakened surface due to bolt holes:

$$A''_{weak} = 3\cdot 1,6\cdot(15-2\cdot 3,5) = 38,4\ см^2 .$$

Stergen net section surface: $A_{\text{weak}} = A_{бр} - (A'_{\text{weak}} + A''_{\text{weak}}) = 300 - (140 + 38{,}4) = 121{,}6\ см^2$.

Calculated tensile stress:

$$\sigma_p = \frac{N}{A_{нт}} = \frac{75 \cdot (10)}{121{,}6} = 6{,}17\ MPa < R_p \cdot m_o = 10 \cdot 0{,}8 = 8\ MPa\ ,$$

therefore, the strength of the element is ensured.

5-practical training. Calculation of elements working in bending.

Bending elements - beams, floor boards and coverings, roofs, panels, rafters are the most common wooden constructions. Bending moment-*M* and shearing force-*Q* appear under the influence of the transverse force acting on the bending elements and they are determined using methods of construction mechanics.

Due to bending, normal stress -σ is formed in the cross sections of the bending element. The normal stress is unevenly distributed along the height of the cross section of the bending element. Figure 12 shows the standard specimen for the bending test and the diagrams and plots of the bending strain, bending moment and stresses.

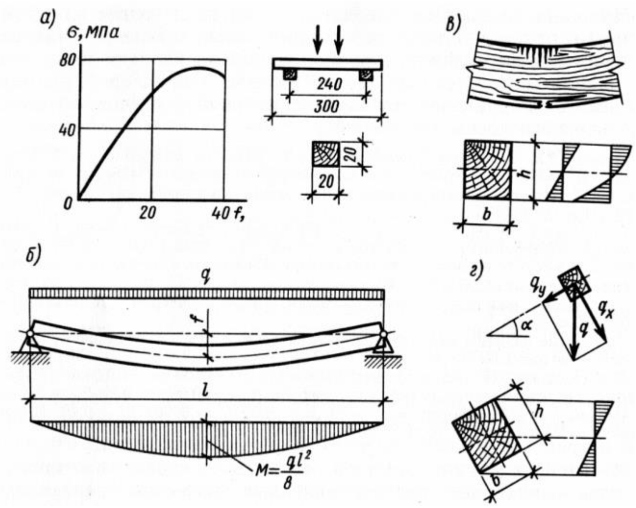


Figure 4.1. Bending element: a-bending graph and example; b-operation diagram and bending moment diagram; v-disruption diagram and normal stress diagrams; Performance diagram and stress curve in g-bend bending.

The strength of bending elements is calculated using the following formula:

$$\sigma = \frac{M}{W} \le R_{эг}$$

where: W is the resistance moment of the cross section; M - bending moment; R_{eg} - resistance in calculated bending; σ - normal stress.

It is recommended to make bending elements from average second grade wood. Then R_{eg}=13 MPa is obtained in calculations.

When the cross-sectional dimensions are 13 cm and larger, R_{eg}=15 MPa is obtained. R_{eg}=16 MPa is accepted in wooden structures with a circular cross section.

Low-responsibility elements can be made from third grade wood. When calculating them - R_{eg}=8,5 MPa is obtained (vassa). The value of W for the case of a rectangular cross-section is determined using the following formula:

$$W=\frac{bh^2}{6},\ \text{for a circular cross section}\qquad W=\frac{d^3}{10}\ .$$

The dimensions of the cross-section of bending wooden elements are determined using the following formulas:

$$\boldsymbol{W_m=\frac{M}{R_{эг}}};\qquad \boldsymbol{h_m=\sqrt{\frac{6\cdot W_m}{b}}};$$

$$\boldsymbol{b_m=\frac{6\cdot W_m}{b}};\qquad \boldsymbol{d_m=\sqrt[3]{10\cdot W_m}};$$

$\boldsymbol{W_m}$, $\boldsymbol{h_m}$ $\boldsymbol{b_m}$, $\boldsymbol{d_m}$ - required resistance moment, cross-sectional height and width, and cross-sectional diameter.

If the dimensions of the cross-section are known, the value of the limit calculation loads that the element can carry can be determined using the above-mentioned basic formulas.

For example, the length of a beam resting on a one-span hinge is -*l*, the dimensions of the cross section - $b\times h$, the amount of uniformly distributed load that it can carry is as follows:

$$W=\frac{bh^2}{6};\quad M=W\cdot R_{эг};\quad q=\frac{8\cdot M}{l^2}\ .$$

Bending elements are calculated according to the standard loads for the second limit state (Table 4): $\frac{f}{l}\le\left[\frac{f}{l}\right]$

For the case with uniformly distributed load:

$$\frac{f}{l}=\frac{5}{384}\cdot\frac{q\cdot l^4}{EJ}\le\left[\frac{f}{l}\right]$$

where: $\frac{f}{l}$ - true relative tilt; $E=10^4$. $\left[\frac{f}{l}\right]$- ruxsat etilgan nisbiy egilish; to'g'ri to'rtburchak kesimli yuza uchun, $J=\frac{b\cdot h^3}{12}$ ga teng.

If the relative bending of the beam is large, then the cross-section should be enlarged, and the cross-section can be determined by bending:

$$J_m = \frac{5 \cdot q \cdot l^4}{384 \cdot \left[\frac{f}{l}\right] \cdot E}; \quad (26) \quad h_m = \sqrt[3]{\frac{12 \cdot J}{b}}.$$

The tensile strength is calculated using the following formula:

$$\tau = \frac{Q \cdot S}{J \cdot b_x} \leq R_{ёр}$$

6-practical training. Calculation of crushing and cracking of wooden structures

Calculation of wood cracking. Cracking in wood can occur in longitudinal planes along the fibers. Cracking stress - T causes cracking and tensile stress-τ in wood. The strength of the wood in the crack is also very small due to the fact that the wood is fibrous. The fibers in the wood are weakly bonded , $\tau = 6{,}8.$ MPa so the wood easily cracks under moderate stresses.

In bending, the maximum shearing force Q (MN) on bending elements is calculated using the following formula:

$$\tau = \frac{Q \cdot S}{I \cdot b} \leq R_{ёр},$$

where: S is the static moment of the cracking surface relative to the neutral axis ($S = \frac{b \cdot h^2}{8}$); Q - maximum shear force; J is the moment of inertia of the total surface ($J = \frac{b \cdot h^3}{12}$); R_{yor} - calculated resistance to cracking ($R_{ёр} = 1{,}6 МПа$); b - width of section.

Fig. 19 shows the formation of tensile stresses, shear force and tensile stress in the elements.

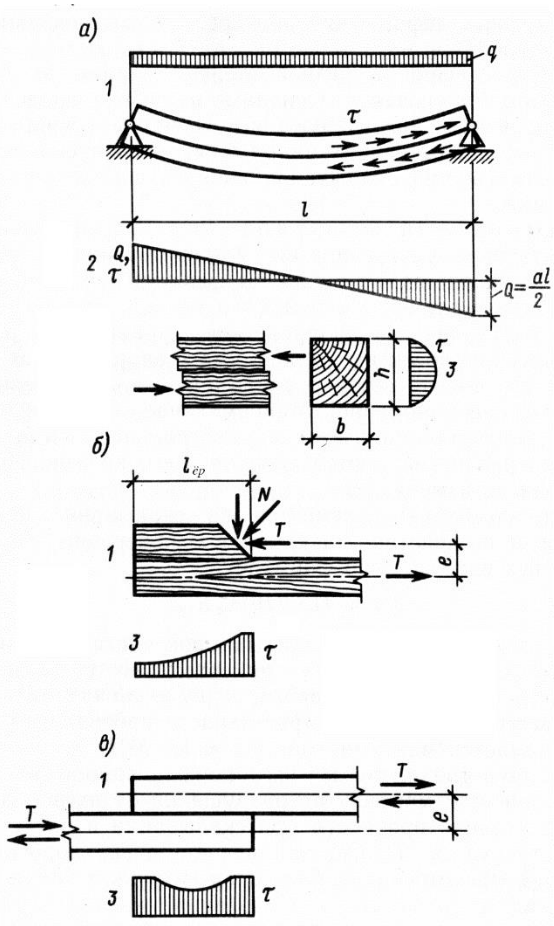


Figure 6.1. Cracking elements: a-bending crack; b-the same, from one-sided breaking forces; v-same, from the two-way breaking forces.

The following formula is used to calculate joints for cracking:

$$\tau = \frac{T}{A} \leq R_{ёр}^{урт},$$

where: τ - shear stress; T - stress at the crack; A - crack surface; R - average calculated resistance to cracking.

$$R_{ёр}^{урт} = \frac{R_{ёр}}{1 + \dfrac{\beta \cdot l_{ёр}}{e}},$$

where: $R_{ёр} = 2{,}1 МПа$ - calculated maximum wood cracking resistance; l_{yor}- the length of the crack area; e - cracking stress eccentricity; $\beta = 0{,}25$ - coefficients when the stress in cracking is one-sided and $\beta = 0{,}125$- two-sided.

Task 2. Select and check the cross-section of a single-span four-edged wooden sarrow made of second-grade wood. Sarrov is located transversely to the slope of the roof and the slope of the roof is equal to $i = 1:4$. Roofing and snow loads affect the load: standard load $g^{м} = 1 кН/м;$, calculated load, $g = 1{,}5 кН/м$; survey interval, $l = 3м.$

Solution:

The calculation scheme of Sarrov is a single-span hinged, inclined bending beam. Slope angle $-\alpha = 14^\circ$

In that case, $\sin\alpha = \sin 14^\circ = 0{,}24; \cos\alpha = \cos 14^\circ = 0{,}97; tg\alpha = tg14^\circ = 0{,}25.$

The value of the maximum bending moment,

$$M_{max} = \frac{g \cdot l^2}{8} = \frac{1{,}5 \cdot 3^2}{8} = 1{,}6875 кН \cdot м.$$

Determine the components of the bending moment along the x and u axes: $M_x = M_{max} \cdot cos\alpha = 1{,}6875 \cdot 0{,}97 = 1{,}64 кН \cdot м = 1{,}64 \cdot 10^{-3} МН \cdot м$.

$$M_y = M_{max} \cdot \sin\alpha = 1{,}6875 \cdot 0{,}24 = 0{,}405 кН \cdot м = 0{,}405 \cdot 10^{-3} МН \cdot м.$$

Sarrov cross-section is accepted in advance $b \times h = 10 \times 15 см$. In that case, the components of the cross-sectional resistance moment along the x and u axes are:

$$W_x = \frac{b \cdot h^2}{6} = \frac{10 \cdot 15^2}{6} = 375 см^3 = 375 \cdot 10^{-6} м^3;$$

$$W_y = \frac{h \cdot b^2}{6} = \frac{15 \cdot 10^2}{6} = 250 см^3 = 250 \cdot 10^{-6} м^3.$$

Calculated bending resistance of wood is equal to $R_{эг} = 13 МПа$

Stress -σ, $$\sigma = \frac{Mx}{W_x} + \frac{M_y}{Wy} = \frac{1{,}64 \cdot 10^{-3}}{375 \cdot 10^{-6}} + \frac{0{,}405 \cdot 10^{-3}}{250 \cdot 10^{-6}} = 6 МПа < R_{эг}.$$

normative load - q^m x and u the organizers of the axes.

$$q_x^м = q^м \cdot \cos\alpha = 1 \cdot 0{,}97 = 0{,}97 kH/м = 0{,}97 \cdot 10^{-3} MH/м;$$

$$q^м{}_y = q^м \cdot \sin\alpha = 1{,}0 \cdot 0{,}24 = 0{,}24 kH/м = 0{,}24 \cdot 10^{-3} MH/м;$$

Modulus of elasticity of wood, is equal to $E = 10^4 МПа$.

Moment of inertia of the section:

$$I_x = \frac{b \cdot h^3}{12} = \frac{10 \cdot 15^3}{12} = 2812{,}5 см^4 = 2812{,}5 \cdot 10^{-8} м^4;$$

$$I_y = \frac{h \cdot b^3}{12} = \frac{15 \cdot 10^3}{12} = 1250 см^3 = 1250 \cdot 10^{-8} м^4.$$

Determine the components of the slope along the x and u axes:

$$f_x = \frac{5}{384} \cdot \frac{q_x^м \cdot l^4}{E \cdot I_x} = \frac{5}{384} \cdot \frac{0{,}97 \cdot 10^{-3} \cdot 3^4}{10^4 \cdot 2812{,}5 \cdot 10^{-8}} = 0{,}0036 м;$$

$$f_y = \frac{5}{384} \cdot \frac{q_y^м \cdot l^4}{E \cdot I_y} = \frac{5}{384} \cdot \frac{0{,}24 \cdot 10^{-3} \cdot 3^4}{10^4 \cdot 1250 \cdot 10^{-8}} = 0{,}002 м$$

General relative inclination:

$$\frac{f}{l} = \frac{\sqrt{f_x^2 + f_y^2}}{l} = \frac{\sqrt{0{,}0036^2 + 0{,}002^2}}{3} = \frac{0{,}0041}{3} = \frac{1}{732} < \left[\frac{f}{l}\right] = \frac{1}{200}.$$

7-practical training. Calculation of a solid wooden column

Second variety pine of wood prepared four edged wood of the column transversely section let it be chosen. Column length $l = 4{,}5м$ and ends hinged fixed . On the column weak section no and to him $N = 300кН = 0{,}3МН$ squeezer longitudinally strength effect does

Solution:

In advance superior flexibility $\lambda = 80$ accepted as we do Priority coefficient -φ the we define:

$$\varphi = \frac{3000}{\lambda^2} = \frac{3000}{80^2} = 0{,}47 \text{ (} \lambda > 70 \text{ bo'lganligi uchun).}$$

The calculated compressive strength of wood is equal to $R_c = 15МПа$ the cross-sectional size of more than 13 cm.

The required cross-sectional area of the column

$$A_T = \frac{N}{\varphi \cdot R_c} = \frac{0{,}3}{0{,}47 \cdot 15} = 0{,}04м^2 = 400см^2.$$

If we take the cross section as a square, $b_T = h_T = \sqrt{A_T} = \sqrt{400} = 20см$.

We accept: $b = h = 20см$ ga teng

Check. Cross-sectional surface $b \times h = 20 \times 20 = 400см^2 = 0{,}04м^2$.

Radius of inertia of the section: $i = 0{,}29 \cdot 20 = 5{,}8см$.

Flexibility: $\lambda = \boldsymbol{\frac{l}{i} = \frac{450}{5{,}8} = 78 > 70}$

Ustivorlik koeffitsienti - $\varphi = \frac{3000}{\lambda^2} = \frac{3000}{78^2} = 0{,}49$

Tension: $\sigma = \frac{N}{\varphi \cdot A} = \frac{0{,}3}{0{,}49 \cdot 0{,}04} = 15{,}3 > 15МПа$,

consistency condition failed. Therefore, we increase the size of the cross section. $b \times h = 20 \times 22 = 440см^2$.

Radius of inertia on the small side of the cross section: $i = 0{,}29 \cdot 20 = 5{,}8см$ *(will be equal to , so there is no need to recalculate).*

We check the tension:

$$\sigma = \frac{N}{\varphi \cdot A} = \frac{0{,}3}{0{,}49 \cdot 0{,}044} = 13{,}91 < 15МПа$$

the consistency condition is satisfied.

8-practical training. Calculation of a composite wooden column.

Select and check the cross-section of a column with a length of $l = 3{,}5м$, the ends of which are fixed with hinges. Given: $N = 200кН = 0{,}2МН$ - longitudinal compressive force $l = 3{,}5м$ - column height $R_c = 13МПа$ - calculated resistance in compression.

Solution:

Column flexibility is assumed in advance, $\lambda = 90$, $\lambda < 120$.

Coefficient of precedence,

$$\varphi_y = \frac{3000}{\lambda^2} = \frac{3000}{90^2} = 0{,}37.$$

Let the width of the column be $b = 20см$ - Then the height of the column section,

$$h_m = \frac{l}{0{,}29 \cdot \lambda} = \frac{350}{0{,}29 \cdot 90} = 13{,}4см.$$

We consider the cross-section to consist of two four-sided timbers of the same size $b \times h = 20 \times 7см$.

$A = 2 \cdot b \cdot h = 2 \cdot 20 \cdot 7 = 280см^2$ - cross-sectional surface;

$r_x = 0{,}29 \cdot b = 0{,}29 \cdot 20 = 5{,}8$ cm - radius of inertia;

$\lambda_x = \frac{l}{r_x} = \frac{350}{5{,}8} = 60{,}3 < 70$ - flexibility;

$\varphi_x = 1 - 0{,}8\left(\frac{\lambda}{100}\right)^2 = 1 - 0{,}8\left(\frac{60{,}3}{100}\right)^2 = 0{,}71$ - coefficient of precedence,

Tension -σ=$\frac{N}{\varphi_x \cdot A} = \frac{0{,}2}{0{,}71 \cdot 0{,}028} =$ *10,06 MPa*

We check the strength and priority of the column, taking into account the inclination of the column to the u-axis.

Cross section and moment of inertia of one square log,

$$A_1 = b \cdot h_1 = 20 \cdot 7 = 140см^2;\ a = 7/2 = 3{,}5;$$

$$J = 2(b \cdot h_1^3/12 + A_1 \cdot a^2) = 2(20 \cdot 7^3/12 + 140 \cdot 3{,}5^2 = 4573см^4$$

Radius of inertia - r_u,

$$r_y = \sqrt{J/A} = \sqrt{4573/140} = \sqrt{32{,}66} = 5{,}72см.$$

The flexibility of the column in the state without taking into account the inclination in the joint,

$$\lambda_u = \frac{l}{r_y} = \frac{350}{5{,}72} = 61{,}2;$$

We attach two square pieces of wood to each other using bolts with a diameter of $d = 2см$,

$$\frac{d}{h_1} = \frac{1}{3,5} < \frac{1}{2}, \qquad K_c = \frac{1,5}{d \cdot h_1} = \frac{1,5}{2 \cdot 7} = 0,107.$$

The number of fasteners - 2 per 1 meter (2 meters),

The elasticity factor is μ_u

$$\mu_y = \sqrt{1 + K_c \cdot b \cdot h \cdot n_ч / l^2 \cdot n_б} = \sqrt{1 + 0,107 \cdot 20 \cdot 14 / 350^2 \cdot 2} = 0,00006.$$

Radius of inertia, distance between bolts and flexibility of one square timber,

$$i = 0,29 \cdot h_1 = 0,29 \cdot 7 = 2,03см; \quad l_1 = 50см;$$

$\lambda_y = l_1 / i = 50 / 2,03 = 24,6$; in this case $\lambda = 60,3$ is taken as

$$\lambda_{кел} = \sqrt{(\mu_y \cdot \lambda_y)^2 + \lambda^2} = \mu_y \cdot \lambda_y = \sqrt{(0,00006 \cdot 24,6)^2 + 60,3^2} = 60,3 < 70.$$

the coefficient of precedence - φ_u

$$\varphi_y = 1 - 0,8\left(\frac{\lambda_{кел}}{100}\right)^2 = 1 - 0,8\left(\frac{60,3}{100}\right)^2 = 0,7$$

Tension - σ,

$$\sigma = \frac{N}{\varphi_y \cdot A} = \frac{0,2}{0,7 \cdot 0,0280} = 10,2MPa < 13MPa.$$

9-practical training. Calculation of a glued wooden column.

It is necessary to calculate the column of additional spans, which are connected to the main building on both sides.

The column is made by gluing pine boards. Column height N=3.6 m, step 6 m; the roof is made of panels with asbestos-cement sheets. The place of construction is the city of Novosibirsk.

We calculate the loads acting on the column.

From roof:

$$g^n = 570 + 38 = 608\, н/м^2;$$

$$g = 690 + 38 \cdot 1,1 = 732\, н/м^2;$$

where: g^n=570 $н/м^2$, g=690 $н/м^2$ - weight of roof structures;

g^n_t=38 $н/м^2$ - load acting on the specific weight of the beam.

Snow load: $P^n_{CH} = 1500\, H/м^2$; $P_{CH} = 2100\ H/м^2$.

Forces acting on the column:

a) from the beam: $N_m = g \cdot B \cdot \ell / 2 = 732 \cdot 6 \cdot 4{,}5 / 2 = 9298\, H$;

where: ℓ =4,5 m extra spacing width.

b) from the snow: $N_{сн} = 2100 \cdot 6 \cdot 4{,}5 / 2 = 26460\, H$;

v) from the wall: $N_c = 500 \cdot 6 \cdot (3{,}6 + 0{,}9) = 16560\, H$.

We take a column section of 15x40 cm, then the specific weight:

$$N_{св} = 0{,}15 \cdot 0{,}4 \cdot 3{,}6 \cdot 5000 \cdot 1{,}1 = 1190\, H .$$

From wind load:

$$q_0 = 0{,}38\, кH / м^2 = 380\, H / м^2,$$

aerodynamic coefficients: S=+0,8 va S=–0,6 is taken.

$$P_q = q_0 \cdot B \cdot \gamma_f \cdot C = 380 \cdot 6 \cdot 1{,}2 \cdot 0{,}8 = 1750\, H / м;$$

$$P_0 = -380 \cdot 6 \cdot 1{,}2 \cdot 0{,}6 = -1313\, H / м;$$
$$W_q = 380 \cdot 6 \cdot 0{,}9 \cdot 1{,}2 \cdot 0{,}8 = 1575\, H;$$
$$W_0 = -0{,}75 \cdot 1575 = -1180\, H.$$

Since the column is a two-hinged frame member, we find the horizontal unknown force generated by the force method.

From wind load:

$$X_w = \frac{W_q - W_o}{2} = -\frac{1575 - 1180}{2} = -197\, H;$$
$$X_p = -\frac{3}{16} \cdot H \cdot (P_q - P_0) = -\frac{3}{16} \cdot 3{,}6 \cdot (1750 - 1313) = -295\, H.$$

From the wall load:

$$M_c = N_c \cdot e = 16560 \cdot 0{,}3 = 4970\, н \cdot м,$$

where: e=0,3 m.

$$X_c = \frac{9}{8} \cdot \frac{M_c}{H} = \frac{9 \cdot 4970}{8 \cdot 3{,}6} = 1550\, H .$$

Bending moment at column support:

$$M_{чап} = \left[\left(W_q + X_w + X_p \right) \cdot H + \frac{P_q H^2}{2} \right] \cdot K + X_c H - M_c = \left[\left(1575 - 197 - 295 \right) \cdot 3{,}6 + \frac{1750 \cdot 3{,}6^2}{2} \right] \cdot 0{,}9 +$$
$$+ 1550 \cdot 3{,}6 - 4970 = 12460\, Нм;$$
$$M_{\text{щнг}} = \left[\left(1180 + 197 + 295 \right) \cdot 3{,}6 + \frac{1313 \cdot 3{,}6^2}{2} \right] \cdot 0{,}9 - 1550 \cdot 3{,}6 + 4970 = 12460\, Нм.$$

Transverse forces:

$$Q_{чап} = (W_q + X_w + X_p + P_q \cdot H) \cdot K + X_c = (1575 - 197 - 295 + 1750 \cdot 3{,}6) \cdot 0{,}9 + 1550 = 8200\, H;$$

$$Q_{\text{щнг}} = (1180 + 197 + 295 + 1313 \cdot 3{,}6) \cdot 0{,}9 - 1550 = 4200\, H.$$

Bo‘ylama kuch:

$$N = N_{\text{т}} + N_{\text{св}} + N_{сн} \cdot K + N_c = 9298 + 1190 + 26460 \cdot 0{,}9 + 16560 = 50860\, H.$$

where: K=0,9 – correction factor for snow load.

Structural calculation of the column

Column cross-section *bxh=15x40* cm. We calculate the geometric characteristics of the section:

$$A = b \cdot h = 15.0 \cdot 40{,}0 = 600\, см^2;$$

$$W_x = \frac{b \cdot h^2}{6} = \frac{15 \cdot 40^2}{6} = 4000\, см^3;$$

$$r_x = 0{,}289 \cdot h = 0{,}289 \cdot 40 = 11{,}6\, см; \quad \lambda_x = \frac{l_0}{r_x} = \frac{360}{11{,}6} = 31 < \lambda_{пр} = 120;$$

$$\xi = 1 - \frac{\lambda_x^2 \cdot N}{3100 \cdot A \cdot R_c} = 1 - \frac{31^2 \cdot 50860}{3100 \cdot 600 \cdot 1300} = 0{,}97,$$

where: R_c=1300 n/cm^2 – calculated resistance of wood to compression.

We calculate the stresses:

$$\sigma = \frac{N}{A} + \frac{M}{W \cdot \xi} = \frac{50860}{600} + \frac{1432000}{4000 \cdot 0{,}97} = 454\, H/см^2 = 4{,}54\, МПа < R_c = 13 \text{ MPa}.$$

Column precedence is checked:

$$l_{oy} = \lambda_{n\,p} \cdot r_y = 120 \cdot 0{,}289 \cdot 15 = 520\, см;$$
$$l_{oy} = 520\, см > H = 360\, см,$$

therefore, priority is ensured.

We determine the experimental stresses:

$$\tau = \frac{Q \cdot S}{J \cdot b \cdot \xi} = \frac{8200 \cdot 7680}{327680 \cdot 15 \cdot 0,9} = 142\, H / см^2 < R_{ck} = 160\, H / см^2$$

where: $S = \frac{b \cdot h_н^2}{8} = \frac{15 \cdot 64^2}{8} = 7680\, см^2$; $J = \frac{b \cdot h_H^3}{12} = \frac{15 \cdot 64^3}{12} = 327680\, см^4$;

h_n=64 sm – the height of the cross-section of the column at the point of attachment to the base.

According to the results of the calculations, the strength and integrity of the structure of the column is ensured.

The column is attached to the foundation by means of four anchor bolts with a diameter of d=20 mm.

10-practical training. Calculation of lattice wooden column

Lattice columns are used as a load-bearing support structure in the roofing walls of production buildings and structures. Their height can be 10 meters or more. The height of the cross-sectional surface of a rectangular column should not be less than *(1⁄6)l*. Lattice column girders can have one or two elements. Column nodes are fixed with bolts. Lattice columns are calculated taking into account the vertical external load, horizontal wind pressure, specific weight of the column.

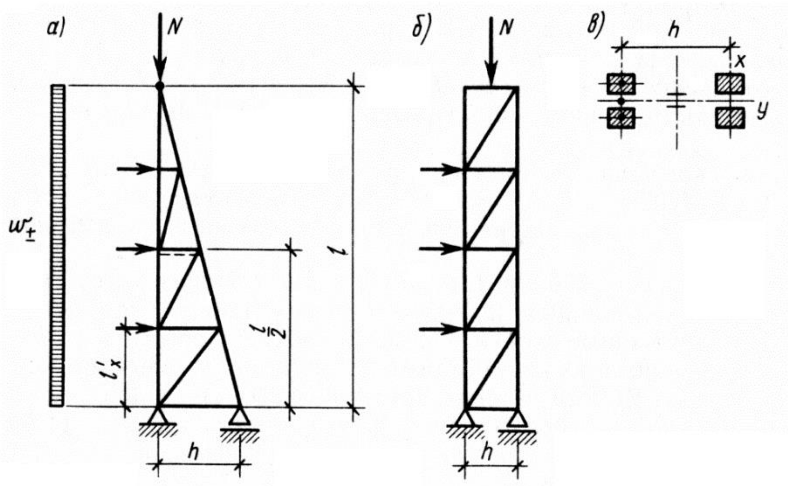
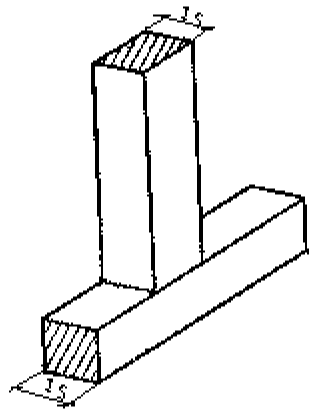


Figure 10.1. Lattice columns: a - triangular; b - rectangular; v - types of sections.

These types of columns work just like cantilever trusses. Longitudinal stresses in column girders can be determined using methods of construction mechanics or graphically - using the Maxwell-Cremont diagram. Depending on the stress values, the cross-sectional dimensions of the stern are determined.

11-practical training. Calculation of compounds.

1-task. The column and the beam are connected in a transverse way. Column section *bx h=15* cm, beam section b_1 x h_1=15x15 cm. Calculated compressive stress N=55 kN. Materials - type 1 pine wood. Check the strength of the connection.



Solution:

We determine the crushing surface:

$$F_{sm=}\ v\ x\ h=15\ x\ 15=225\ sm^2$$

The length of the crushing surface $\ell_{см}$ =15 sm.

We determine the calculated resistance of wood to local crushing in the direction transverse to the fibers:

$$R_{sm\ 90}=1{,}8\cdot\left(1+\frac{8}{l_{см}+1{,}2}\right)=1{,}8\cdot\left(1+\frac{8}{15+1{,}2}\right)=2{,}69\ МПа,$$

We calculate the crushing stresses:

$$\sigma_{см}=\frac{N}{F_{см}}=\frac{55\cdot(1000)}{225\cdot(100)}=2{,}44\ МПа<R_{см90}=2{,}69\ МПа,$$

therefore, the strength of the compound is ensured.

2-task: The cross-sectional dimensions bxh=20x20 cm are two cross-sections (brus) attached with two-sided cover-handles using bolts. Section of cover-handles bxh=8x20 cm. Materials - pine wood. Longitudinal tensile stress N=180 kN. Determine the cross-section and number of bolts.

Solution:

We assume that two rows of bolts are placed symmetrically. Then the diameter of the bolts:

$$d\le\frac{h}{9{,}5}=\frac{20}{9{,}5}=2{,}1\,см=21\text{ mm.}$$

We accept bolts with diameter d=20 mm.

The joint is double-shear n_{sr}=2, symmetrical, located along the fibers, $\alpha=0^0$, coefficient K_α=1.

The thickness of the middle border (brus) is S=15 cm, the edge cover of the handles is 8 cm.

We determine the shear load capacity of bolts according to the following conditions: according to the bending of the bolt:

$$T_c = 5 \cdot c \cdot d = 5 \cdot 20 \cdot 2 \cdot 10^{-1} = 20 \quad кН\ ;$$

according to the crushing of the edge board:

$$T_a = 8 \cdot a \cdot d = 8 \cdot 8 \cdot 2 \cdot 10^{-1} = 12{,}8 \quad кН;$$

The smallest calculated load capacity of a bolt with a diameter of 20 mm:

T_{min}=8,5 kN.

We determine the number of bolts in half of the joint:

$$n = \frac{N}{T_{\min} \cdot n_{cp}} = \frac{180}{8{,}5 \cdot 2} = 10{,}6 \text{ dona.}$$

We accept 12 bolts with a diameter d=20 mm on both sides of the joint, the total number of bolts is n=24 pieces.

In addition to the 8 bolts in the center and edges of the connection, the remaining 16 can be replaced with studs with a diameter of d=20 mm.

12-practical training. Calculation of bolted joints.

Determine the number and cross-section of the required bending bolts at the joint of two four-sided timbers of dimensions $b_1 \times h_1 = 8 \times 20 см$ with a cross-section of $b \times h = 15 \times 20 см$ double-sided timber. A longitudinal stretching force is $N = 160 кН$ applied to it.

Solution:

We derive the diameter of the bolt by placing two rows in height:

$$d \le \frac{h}{9{,}5} = \frac{20}{9{,}5} = 2{,}1 см\ .$$

We accept the diameter $d = 2 см$.

The joint is symmetrical and bisected, equal to $n_ч = 2$. The thickness of the elements in the middle is 15 cm, and those on the edge are $a = b_1 = 8 см.$. The load-carrying capacity of a single-weld bolt from the bending performance of the bolt

$$T_{эг} = 1{,}8 d^2 + 0{,}02 a^2 = 1{,}8 \cdot 2^2 + 0{,}02 \cdot 8^2 = 8{,}5 кН\ .$$

From the condition of crushing the elements,

$$T_c = 0{,}5 dc = 0{,}5 \cdot 15 \cdot 2 = 15 кН.$$

From the condition of crushing coatings,

$$T_a = 0{,}5 da = 0{,}8 \cdot 2 \cdot 8 = 12{,}8 кН.$$

Calculated minimum load capacity is equal to $T = 8{,}5кН$.

The number of required bolts is *n*,

$$n_T = \frac{N}{T \cdot n_{чок}} = \frac{160}{8{,}5 \cdot 2} = 9{,}4 \text{ ta.}$$

The total number of bolts on one side of the seam is 10 and the diameter is taken equal to $d = 20мм$.

Task-2. Determine the calculated load-carrying capacity of a nail with a diameter of $l = 8см$ and a length of $d = 0{,}5см$ driven into dry wood at a depth of $l = 10см$.

Solution:

The calculated resistance of a nail driven into dry wood is equal to $R_{см} = 0{,}3МПа$

Calculated length of the mix after removing the sharp part at the end

$$l_1 = l - 1{,}5d = 8 - 1{,}5 \cdot 0{,}005 = 7{,}25см = 0{,}0725м.$$

Calculated load carrying capacity of the nail in the seam

$$T_{с.м} = R_{с.м} \cdot \pi \cdot d \cdot l_1 = 3.14 \cdot 0{,}005 \cdot 0{,}0725 \cdot 0{,}3 = 0{,}34 \cdot 10^{-3} МН = 0{,}34кН.$$

13-practical training. Calculation of frame elements.

The frame structure is made by bending and gluing boards. Materials - spruce board with moisture φ=12% and synthetic glue. The frame span is 24 m, the height between the spans is 10 m; roof slope – 1:4 or $\alpha=14^0$; the cross-section of the frame beam is assumed to be variable in length; the distance (step) between frames B=5.4 m.

The axis passing through the center of gravity of the end section and parallel to the outer edge is taken as the reference axis of the frame. Cut height at the eaves node $h_i = \left(\frac{1}{20} - \frac{1}{40}\right) \cdot \ell$; at the tip - $h_2 \geq 0{,}3 \cdot h_1$ on the support - $h_{оп} \geq 0{,}4 \cdot h_1$. Thickness for frame $h_д \leq \frac{r_k}{250} \leq 3{,}3$ sm- boards are used.

Geometric calculation of frame

Frame construction consists of two half-frames. The half-frames are hinged to each other and to the base.

The slope of the change in the height of the element cross-section along its length: $i = \frac{1}{5} \div \frac{1}{7}$.

Radius of curvature at the bend:

$$r_k = h_д \cdot 250 = 1{,}4 \cdot 250 = 350\, см = 3{,}5\, м\ ,$$

where: $h_д$=1,4 sm - thickness of the boards used for the frame (after sawing).

The radius of curvature of the outer contour:

$$r_н = r_k + h_0 = 3{,}5 + 0{,}15 = 3{,}65\, м,$$

where:

$$h_0 = \frac{h_2}{2} = \frac{0{,}3 \cdot h_1}{2} = 15\, см; \;\; h_1 = \frac{\ell}{25} \approx 100\, см.$$

$$tg14^0 = 0{,}25; \quad \cos 14^0 = 0{,}97; \quad \sin 14^0 = 0{,}242.$$

The angle of spread of the semi-frame curve: $\varphi = 90 - \alpha = 90 - 14 = 76^0$.

The length of the arc of the curved section:

$$S = \frac{\pi \cdot r_k \cdot \varphi}{180^o} = \frac{3{,}14 \cdot 3{,}5 \cdot 76^0}{180^o} = 4{,}6\, м.$$

The length of the frame beam:

$$\ell_p = \frac{0{,}5 \cdot \ell}{\cos\alpha} - r_k tg\frac{\varphi}{2} = \frac{0{,}5 \cdot 23{,}7}{0{,}97} - 2{,}73 = 9{,}48\, м.$$

Frame column height:

$$H_{ст} = H - f_0 - r_k \cdot tg\frac{\varphi}{2} = 10 - 2{,}96 - 2{,}73 = 4{,}31\, м.$$

where: $f_o = \frac{\ell \cdot tg\alpha}{2} = 0{,}25 \cdot 11{,}85 = 2{,}96\, м$.

Full length half frame:

$$l_o = H_{ст} + S + l_p = 4{,}31 + 4{,}6 + 9{,}48 = 18{,}39\, м.$$

To determine the stresses generated in the cross-sections of the frame, we divide the semi-frame axis into 8 sections. The left base is defined as the 0 point (coordinate head) and the coordinates of the sections on the circular curve of the half-frame are found from the following formulas.

$$X_n = r_k - r_k \cdot \cos\varphi_n; \quad Y_n = r_k \sin\varphi_n - H_{ст}$$

Specific gravity of the frame:

$$g^{H}_{см} = \frac{g^{H} + P^{H}}{\dfrac{1000}{K_{см} \cdot \ell} - 1} = \frac{130 + 700}{\dfrac{1000}{8 \cdot 27} - 1} = 197\, H / м^{2},$$

where: K_{sm} - 7÷9 - specific weight coefficient for three-hinged frames.

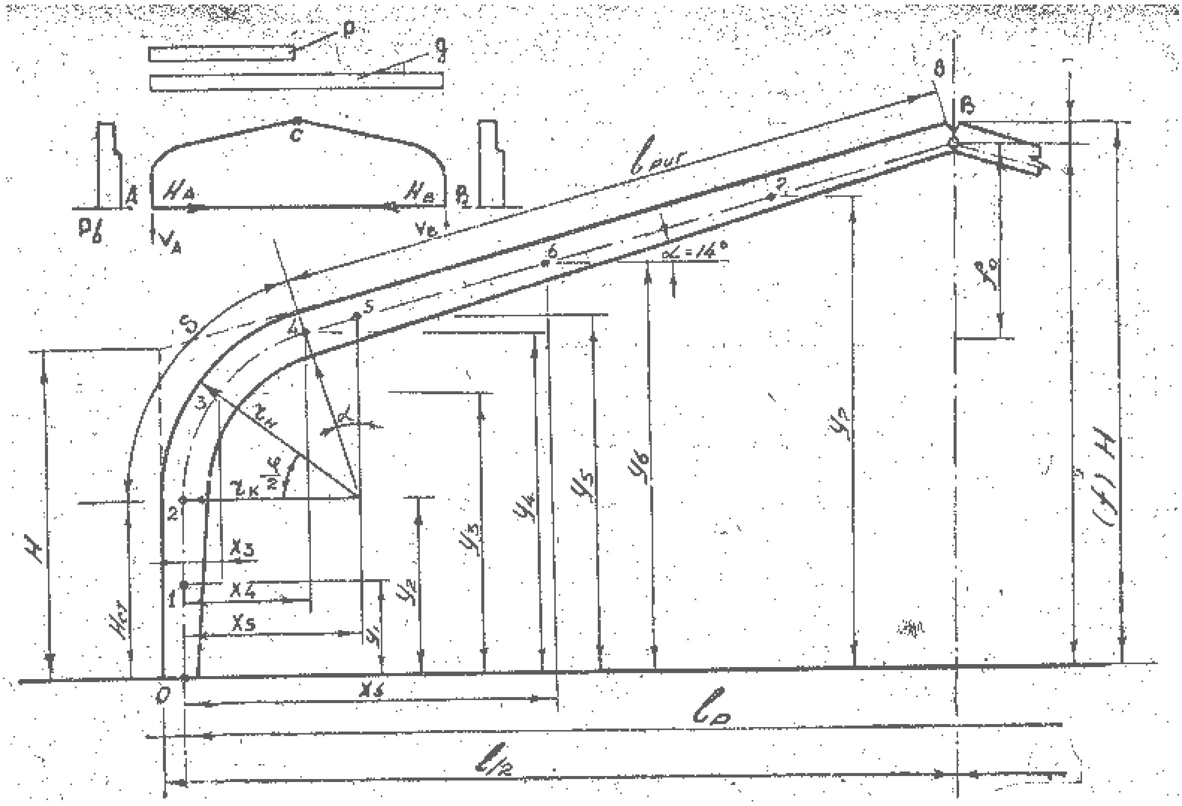


Figure 13.1. Calculation and geometric schemes of the frame made by bending and gluing from boards

Coordinates of the frame axis points

Table-2

Cross section №	0 (A)	1	2	3	4	5	6	7	8 (s)
X, m	0	0	0	0,74	2,65	3,5	6,28	9,06	11,85
U, m	0	2,155	4,31	6,45	7,70	7,91	8,60	9,30	9,999

Computational loads affecting the frame:doimiy:

$$G = 367 \cdot B = 367 \cdot 5{,}4 = 1982\, H / м^{2};$$

temporary: snow load: $P = 980 \cdot B = 980 \cdot 5{,}4 = 5292\, H / м;$

wind load: $q_в = cq_0 nB = 0,8 \cdot 300 \cdot 1,2 \cdot 5,4 = 1555\, Н/м$.

Wind load acting on the beam:

$$q_в^л = q_в \cos\alpha = 1555 \cdot 0,97 = 1508\, Н/м;$$
$$q_в^п = 0,5 q_в \cos\alpha = 0,5 \cdot 1555 \cdot 0,97 = 754\, Н/м;$$

wind load acting on columns::

$$q_{вд} = q_в = 1555\, н/м;$$
$$q_{во} = 0,75 q_в = 0,75 \cdot 1555 = 1166\, н/м;$$
$$q_{вд_1} = 0,24 q_в = 0,24 \cdot 1555 = 372\, н/м;$$
$$q_{во_1} = 0,12 q_в = 0,12 \cdot 1555 = 186\, н/м.$$

Static calculation of frame.

Base reactions from unit load q=1 N/m:

$$V_A = \frac{3q\ell}{8} = \frac{3 \cdot 1 \cdot 23,7}{8} = 8,9\, Н;$$
$$V_B = \frac{1q\ell}{8} = \frac{1 \cdot 1 \cdot 23.7}{8} = 3,0\, Н;$$
$$H_A = H_B = \frac{q\ell^2}{16H} = \frac{1 \cdot 23,7^2}{16 \cdot 10} = 3,5\, Н.$$

We determine the support reactions from the wind load:

1) $$\Sigma M_B = 0 = (1166 + 1555) \cdot \frac{7,04^2}{2} + (186 - 372) \cdot \left(7,04 + \frac{2,96}{2}\right) \cdot 2,96 + 754 \cdot \frac{11,85^2}{2} +$$
$$+ 1508 \cdot 11,85 \cdot \left(11,85 + \frac{11,85}{2}\right) - V_A \cdot 23,7.$$

$$V_A = \frac{67428 - 4690 + 52939 + 317635}{23,7} = 18283\, н;$$

2) $$\Sigma M_A = 0 = (1555 + 1166) \cdot \frac{7,04^2}{2} + (-372 + 186) \cdot 2,96 \cdot \left(7,04 + \frac{2,96}{2}\right) - 1508 \cdot \frac{11,85^2}{2} -$$
$$- 754 \cdot 11,85 \cdot \left(11,85 + \frac{11,85}{2}\right) - V_B \cdot 23,7.$$

$$V_B = \frac{67428 - 4690 - 105878 - 158817}{23,7} = 8521\, Н;$$

3) $$\Sigma M_c^{лев} = 0 = H_A \cdot 10 - 18283 \cdot 11,85 - 1555 \cdot 7,04 \cdot \left(\frac{7,04}{2} + 2,96\right) + 372 \cdot \frac{2,96^2}{2} + 1508 \cdot \frac{11,85^2}{2};$$

$$H_A = \frac{216653 + 70937 - 1629 - 105878}{10} = 18008\, Н;$$

4) $\Sigma M_c^{пр} = 0 = H_B \cdot 10 - 8521 \cdot 11{,}85 - 1166 \cdot 7{,}04 \cdot \left(\frac{7{,}04}{2} + 2{,}96\right) - 186 \cdot \frac{2{,}96^2}{2} - 754 \cdot \frac{11{,}85^2}{2}$;

$$H_A = \frac{-100973 + 53191 + 814 + 52939}{10} = 597\,H.$$

We determine the bending moments (n.m) generated at the calculated points of the half-frame from an equally distributed vertical unit load acting on the left and right half of the frame span according to the following formulas:

$$M_n^{лев} = V_A \cdot X_n - q\frac{X_n^2}{2} - H_A \cdot У_n; \quad M_n^{пр} = V_B \cdot X_n - H_B \cdot У_n; \quad M_{o(A)}^{лев} = M_{o(A)}^{пр} = 0;$$

$$M_1^{лев} = 8{,}9 \cdot 0 - 1 \cdot 0 - 3{,}5 \cdot 2{,}155 = -7{,}5; \qquad M_1^{пр} = 3 \cdot 0 - 3{,}5 \cdot 2{,}155 = -7{,}5;$$

$$M_2^{лев} = 8{,}9 \cdot 0 - 1 \cdot 0 - 3{,}5 \cdot 4{,}31 = -15{,}1; \qquad M_2^{пр} = 3 \cdot 0 - 3{,}5 \cdot 4{,}31 = -15{,}1;$$

$$M_3^{лев} = 8{,}9 \cdot 0{,}74 - 1 \cdot \frac{0{,}74^2}{2} - 3{,}5 \cdot 6{,}45 = -16{,}3; \qquad M_3^{пр} = 3 \cdot 0{,}74 - 3{,}5 \cdot 6{,}45 = -20{,}4;$$

$$M_4^{лев} = 8{,}9 \cdot 2{,}65 - 1 \cdot \frac{2{,}65^2}{2} - 3{,}5 \cdot 7{,}7 = -6{,}9; \qquad M_4^{пр} = 3 \cdot 2{,}65 - 3{,}5 \cdot 7{,}7 = -19;$$

$$M_5^{лев} = 8{,}9 \cdot 3{,}5 - 1 \cdot \frac{3{,}5^2}{2} - 3{,}5 \cdot 7{,}91 = -2{,}7; \qquad M_5^{пр} = 3 \cdot 3{,}5 - 3{,}5 \cdot 7{,}91 = -17{,}2;$$

$$M_6^{лев} = 8{,}9 \cdot 6{,}26 - 1 \cdot \frac{6{,}26^2}{2} - 3{,}5 \cdot 8{,}6 = 6; \qquad M_6^{пр} = 3 \cdot 6{,}26 - 3{,}5 \cdot 8{,}6 = -11{,}3;$$

$$M_7^{лев} = 8{,}9 \cdot 9{,}06 - 1 \cdot \frac{9{,}06^2}{2} - 3{,}5 \cdot 9{,}3 = 7; \qquad M_7^{пр} = 3 \cdot 9{,}06 - 3{,}5 \cdot 9{,}3 = -5{,}4;$$

$$M_{8(c)}^{лев} = M_{8(c)}^{пр} = 0.$$

Bending moments from wind load are found according to the following formulas:

$$\mathrm{M}_n^{л} = H_A Y_n - (q_b Y_n^2 / 2) - V_A X_n;$$
$$\mathrm{M}_n^{п} = H_B Y_n + (q_b Y_n^2 / 2) - V_B X_n$$

We determine the bending moments (n.m) generated by the wind load at the calculated points of the half-frame:

$$M_{o(A)} = 0;$$

$$M_1^{лев} = 18008 \cdot 2{,}155 - 1555 \cdot \frac{2{,}155^2}{2} = 35196; \qquad M_1^{пр} = 597 \cdot 2{,}155 + 1166 \cdot \frac{2{,}155^2}{2} = 3994;$$

$$M_2^{лев} = 18008 \cdot 4{,}31 - 1555 \cdot \frac{4{,}31^2}{2} = 63171; \qquad M_2^{пр} = 597 \cdot 4{,}31 + 1166 \cdot \frac{4{,}31^2}{2} = 13402;$$

$$M_3^{лев} = 18008 \cdot 6{,}45 - 1555 \cdot \frac{6{,}45^2}{2} - 18283 \cdot 0{,}74 = 70276; \quad M_3^{пр} = 597 \cdot 6{,}45 + 1166 \cdot \frac{6{,}45^2}{2} - 8521 \cdot 0{,}74 = 21799;$$

$$M_4^{лев} = 18008 \cdot 7{,}7 - 1555 \cdot 7{,}04 \cdot \left(\frac{7{,}04}{2} + 0{,}66\right) + 372 \cdot \frac{0{,}66^2}{2} + 1508 \cdot \frac{2{,}65^2}{2} - 18283 \cdot 2{,}65 = 49828;$$

$$M_4^{пр} = 597 \cdot 7{,}7 + 1166 \cdot 7{,}04 \cdot \left(\frac{7{,}04}{2} + 0{,}66\right) + 186 \cdot \frac{0{,}66^2}{2} + 754 \cdot \frac{2{,}65^2}{2} - 8521 \cdot 2{,}65 = 19017;$$

$$M_5^{лев} = 18008 \cdot 7{,}91 - 1555 \cdot 7{,}04 \cdot \left(\frac{7{,}04}{2} + 0{,}87\right) + 372 \cdot \frac{0{,}87^2}{2} + 1508 \cdot \frac{3{,}5^2}{2} - 18283 \cdot 3{,}5 = 39772;$$

$$M_5^{пр} = 597 \cdot 7{,}91 + 1166 \cdot 7{,}04 \cdot \left(\frac{7{,}04}{2} + 0{,}87\right) + 186 \cdot \frac{0{,}87^2}{2} + 754 \cdot \frac{3{,}5^2}{2} - 8521 \cdot 3{,}5 = 15624;$$

$$M_6^{лев} = 18008 \cdot 8{,}6 - 1555 \cdot 7{,}04 \cdot \left(\frac{7{,}04}{2} + 1{,}54\right) + 372 \cdot \frac{1{,}54^2}{2} + 1508 \cdot \frac{6{,}28^2}{2} - 18283 \cdot 6{,}28 = 14838;$$

$$M_6^{пр} = 597 \cdot 8{,}6 + 1166 \cdot 7{,}04 \cdot \left(\frac{7{,}04}{2} + 1{,}54\right) + 186 \cdot \frac{1{,}54^2}{2} + 754 \cdot \frac{6{,}28^2}{2} - 8521 \cdot 6{,}28 = 8248;$$

$$M_7^{лев} = 18008 \cdot 9{,}3 - 1555 \cdot 7{,}04 \cdot \left(\frac{7{,}04}{2} + 2{,}26\right) + 372 \cdot \frac{2{,}26^2}{2} + 1508 \cdot \frac{9{,}06^2}{2} - 18283 \cdot 9{,}06 = 1401;$$

$$M_7^{пр} = 597 \cdot 9{,}3 + 1166 \cdot 7{,}04 \cdot \left(\frac{7{,}04}{2} + 2{,}26\right) + 186 \cdot \frac{2{,}26^2}{2} + 754 \cdot \frac{9{,}06^2}{2} - 8521 \cdot 9{,}06 = 7218;$$

$$M_{8(c)} = 0.$$

14-practical training. Wood truss account.

The truss is affected by permanent and temporary loads. The third type of load is special loads, which arise from earthquakes, explosions, or other dynamic effects, and its vertical structure can create additional stresses on the truss. However, in this guide, we will limit ourselves to studying the effects of static loads.

Permanent loads - the specific weights of the roofing elements and the specific weight of the truss. Temporary roads are snow and wind loads. Permanent and temporary loads are evenly distributed over the span of the farm. Most of the time, temporary wind effects create tension in the farm masts with the opposite sign and therefore they are not taken into account in further calculations. Mainly, in the calculation of segment farms, the effects of evenly distributed permanent and temporary snow loads are taken into account. If there is suspended equipment or a ceiling, the loads falling from them on the nodes of the lower belt of the farm are also collected and calculated. Geometric and static calculations are performed on farms. Trusses of farms are especially attached depending on the types of stresses.

In the geometric calculation of the truss, the length, slope, spacing, height, and radius of curvature of the truss bars are determined.

In the static calculation of the truss, the longitudinal internal stress - N, which is formed in the truss struts, is determined from all calculated loads. The upper girder of the truss is in compression-bending, the lower girder is in extension, and the mortar and columns are in compression or extension.

Longitudinal -N forces in the trusses are determined in two ways:
1) theoretical - using classical methods of construction mechanics;
2) graph - by constructing the Maxwell-Cremon diagram.

The cross-section of the booms is determined taking into account flexibility: for high belt booms $\lambda = 120$; $\lambda = 150$ for compression bars; $\lambda = 400$ for the lower belt stretchable sterns. In this case, the distance between the nodes is taken as the length of the stern. The cross-section of the upper belt can also be determined using the following formulas using the values of internal stresses M-bending moment and N - longitudinal force:

$$\boldsymbol{A_T = \frac{0{,}7 \cdot N}{R_c}}; \qquad \boldsymbol{h_T = \frac{A_T}{b}};$$

$$\boldsymbol{W_T = \frac{M}{0{,}8 \cdot R_{эг}}}; \qquad \boldsymbol{h_T = \sqrt{\frac{6 \cdot W_T}{b}}}. \qquad (8.1)$$

where: A_T W_T, h_T - required cross-sectional area, resistance moment, cross-sectional height; M- bending moment, R_s , R_{eg} - calculated resistances in compression, bending; b - the width of the cross section.

Trusses with a length of $9 \div 36$ m are mostly used. When calculating farms, their specific weight is also taken into account and is determined using the following formula:

$$g^{\phi}{}_{x.o} = \frac{q^M + P^M}{\left(1000/K_{x.o}\, l\right) - 1}, \qquad (8.2)$$

Triangular truss. It is used in roof coverings when a large slope is required and mainly in small spans. In them, the h/l ratio is - *1/5*, if the lower arch is metal - *1/6* and if the lower arch is metal, the upper arch is glued wood - *1/7*. The slope of the roof is taken from 1:2.5 to 1:4.

15-practical training. Spatial constructions, structural forms of domes.

Construction scheme of the cover. The ribs of the dome are made by gluing boards with synthetic glue and are reinforced with steel rods placed symmetrically in the section of the element. Glued plywood sheets with a trapezoidal plan view 148 cm wide are installed on the dome ribs. Light-transmitting panels are made of light-transmitting polyester glass-plastics and rest on a frame consisting of boards, and wooden frames are installed on the wall of the dome; the width of such panels is 148 cm.

The lower base of the dome is designed from cast reinforced concrete, and the upper one is made of steel sleepers.

Each pair of dome ribs forms one three-hinged arch.

The distance between the roofs according to Ostki folk:

$$B = \frac{\pi \cdot D_H}{n} = \frac{3{,}14 \cdot 60}{32} = 5{,}89\text{м} \approx 5{,}9\text{м} < 6\text{м},$$

where: D_n=60 m; n=32 - the number of sides of a regular polygon.

The height of the rise of the spire (peak):

$$f = \frac{D_H}{6} = \frac{60}{6} = 10\text{м}.$$

Radius of curvature of the surface:

$$R = \frac{D_H^2}{8f} + \frac{f}{2} = \frac{60^2}{8 \cdot 10} + \frac{10}{2} = 50\text{м}.$$

Central angle:

$$\sin\frac{\varphi}{2} = \frac{D_H}{2R} = \frac{60}{2 \cdot 50} = 0{,}6; \qquad \varphi = 74^0.$$

The length of the arc of Ravok:

$$S = \frac{2\pi R\varphi}{360^0} = \frac{2 \cdot 3{,}14 \cdot 50 \cdot 74^0}{360^0} = 64{,}54\text{м}.$$

Static calculation of dome roof. We determine the loads that affect the condition.

Calculated load acting on the arch due to the weight of the roof structures:

$$g_0 = 650 H/\text{м}^2;$$

(for loads from all types of covering panels, their maximum value is accepted).

Snow load:

$$P_{CH}^{n} = 1000 H/м^{2};$$

$$C = \frac{D_H}{8f} = \frac{60}{8 \cdot 10} = 0{,}75.$$

From the specific gravity of the raw material:

$$g_{c.в} = \frac{650 + 1000}{\frac{1000}{2{,}5 \cdot 60} - 1} = 300 H/м^{2}$$

For calculated values of constant and snow loads, we determine the maximum ordinates of triangular load contours according to the following formulas:

$$q = (g_{c.в} + g_0 B)\frac{S}{D_H} = (300 + 650 \cdot 5{,}9)\frac{64{,}54}{60} = 4520 H/м;$$

$$P = P_{сн}^{n} \cdot \gamma_f \cdot C \cdot B = 1000 \cdot 1{,}4 \cdot 0{,}75 \cdot 5{,}9 = 6195 H/м,$$

where: $\gamma_f = 1{,}4$ - confidence factor for snow load.

At the adopted riser of the dome, the stresses in the arch due to the wind load are reduced, therefore, the wind load is not taken into account in the calculations.

We determine the estimated stresses according to the table in the appendix for the scheme of exposure to permanent loads in the full span of the arch, and temporary (snow) loads in half of the span.

The bending moment generated in the arch section at a distance of 0.2 D_n from the base:

$$M_{2\max} = 0{,}005ql^2 + 0{,}0105Pl^2 = 0{,}005 \cdot 4520 \cdot 60^2 + 0{,}0105 \cdot 6195 \cdot 60^2 = 315530 H \cdot м.$$

We determine the reaction forces generated in the support hinge of the rack:

from constant load:

$$V_A = V_B = q\frac{l}{4} = 4520\frac{60}{4} = 67800 H;$$

$$H = \frac{ql^2}{24f} = \frac{4520 \cdot 60^2}{24 \cdot 10} = 67800 H;$$

from the snow load acting on the full span:

$$V_A = V_B = 6195\frac{60}{4} = 92925 H;$$

$$H=\frac{6195\cdot 60^2}{24\cdot 10}=92925H;$$

from permanent and snow loads acting on the full span:

$$V_A=V_B=67800+92925=160725H;$$

$$H=160725H.$$

from the snow load acting on the left half of the span:

$$V_A=\frac{5P\cdot l}{24}=\frac{5\cdot 6195\cdot 60}{24}=77450H;$$

$$V_B=\frac{P\cdot l}{24}=\frac{6195\cdot 60}{24}=15490H;$$

$$H=\frac{ql^2}{48f}=\frac{6195\cdot 60^2}{48\cdot 10}=46460H;$$

From the constant snow load acting on the full span and on the left half of the span:

$$V_A=67800+77450=145250H;$$
$$V_B=67800+15490=83290H;$$
$$H=67800+46460=114260H.$$

We determine the calculated longitudinal and transverse forces resulting from full loading in the section O on the left support of the ramp at the following values of the coordinates:

x_0=0; u_0=0; φ_0=φ/2=37^0;

sinφ_0=0,6; sosφ_0=0,8.

Transverse force for beam scheme:

$$Q_{об}=H=160725H.$$

$$N_0=Q_{об}\sin\varphi_0+H\cos\varphi_0=160725\cdot 0{,}6+160725\cdot 0{,}8=225015H.$$
$$Q_0=Q_{об}\cos\varphi_0-H\sin\varphi_0=160725\cdot 0.8-160725\cdot 0{,}6=32145H.$$

We determine the calculated stress in 2 sections where the largest bending moment is generated:

x_2=0,2D_n=0,2·60=12 m; u_2=0,665f=0,665·10=6,65 m;

φ_2=21^{o}6'; $\sin\varphi_2$=0,36; cosφ_2=0,933.

Transverse force for the beam scheme when the constant load acts on the full span, and the snow load acts on the left half of the span:

$$Q_{2б} = V_A - \frac{(q+p)+(q+p)\cdot 0{,}6}{2} x_2 = 145250 - \frac{(4520+6195)+(4520+6195)\cdot 0{,}6}{2} 12 = 42390 H$$

Longitudinal tension in section 2:

$$N_2 = Q_{2б} \sin\varphi_2 + H\cos\varphi_2 = 42390\cdot 0{,}36 + 114260\cdot 0{,}933 = 15260 + 106600 = 121860 H$$

IV-INDEPENDENT EDUCATIONAL TASKS

It is a form of organizing independent education, which includes: displaying the results of construction calculations in tables, drawing projects related to science, marking them in the form of slides, displaying them in instructional tools, etc.

Compose questionnaires on the topics of independent work, answer questions in writing using recommended literature, use laws, decisions, regulatory documents, describe ways to solve problematic issues on each topic, make recommendations and others.

Suggested freelance work topics:

№	Subjects of independent education
1	Studying subjects and subjects of textbooks and manuals
2	Acquaintance with regulatory documents
3	Introduction to methods of using catalogs
4	Work on departments or subjects of special literature
5	Acquaintance with literature on specialized subjects
6	Acquaintance with new construction techniques and technologies.
7	Learning new wooden constructions.
8	In-depth study of departments and topics related to the student's educational-scientific-research activities
9	Stress-deformation conditions in wooden structures. Calculation methods
10	Determining building dimensions (length, width, height) using modules
11	Effect of loads acting on wooden structures in a complex state.
12	Calculation of wooden structures under the influence of vertical loads.

13	Calculation of wooden structures under the influence of horizontal loads.
14	Calculation of glued wooden column constructions
15	Calculation of lattice wooden column constructions

V- QUESTIONS FOR CONTROL (CC, IC, FC)

1. Qualities of wooden construction material, its types, advantages and disadvantages.
2. A brief historical overview of the use of wood as a construction material
3. Buildings and structures built using wood in developed countries.
4. Anatomical structure of wood, its micro and macro structure, properties of morning and evening wood.
5. A brief historical overview of the development of plastic as a construction material.
6. Buildings and structures built using plastics.
7. Chemical resistance of wood to aggressive environment, alkali, acid and salts.
8. Physical properties of wood, its dependence on volume weight resistance.
9. Mechanical properties of wood, its anisotropy and shear resistance, plasticity modulus.
10. Tensile, compressive and transverse bending performance of wood.
11. Influence of humidity and temperature on the strength of wood
12. Construction plywood, its composition, physical-mechanical properties, advantages and disadvantages.
13. General information about synthesis of plastics.
14. Types of resins used in plastics, their composition and properties.
15. Additives forming plastics and their effect on the physical and mechanical properties of plastic.
16. The main types of structural plastics, their composition and physical-mechanical properties.
17. Types of wood-plastic materials, their composition and physical-mechanical properties.
18. Effect of humidity and temperature on strength and deformability of plastics
19. The physical properties of plastic, volume weight, thermal expansion, and thermal conductivity depend on its composition.
20. Mechanical properties of plastic, their elasticity model and deformability.
21. Protection of wood from burning, antiprenes and measures against fire.

22. Chemical and structural measures to protect wood from burning.
23. Protecting wood from rotting, antiseptics, their composition and properties.
24. The fight against wood rotting by worm ants, insecticides, fungicides, their types and measures used.
25. Methods and measures used to protect plastics from burning.
26. Calculation of wooden structures according to the limit state.
27. Calculation of wood elements for central elongation.
28. Calculation of wood elements for central compression.
29. Calculation of the bending of wood along the elements.
30. Calculation of wood elements for oblique bending.
31. Calculation of compressive-flexible wooden elements.
32. Calculation of tensile and bending wooden elements.
33. The long-term durability of wood and plastic depends on its composition.
34. Classification of combinations of wooden structures.
35. Methods of calculation of joints of wooden structures.
36. Contact joints of wood elements and their calculation.
37. Nogelli joints of wooden elements and calculation methods.
38. Peculiarities of attaching wooden elements with nails and their calculation.
39. Connecting wooden elements using elastic fasteners and calculating them.
40. Peculiarities of calculating wood glue joints.
41. Consideration of the design of prefabricated wooden elements with a tendency to soften fasteners.
42. Classification of solid flat wooden structures.
43. Calculation and design of wooden scaffolding and obreshatka.
44. Progon calculation and design methods.
45. Types of three-layer plastic panels and their calculation and design.
46. Plate-shaped nailed beams and information on their calculation and design.
47. Adjacent beams with wooden walls, their calculation and design.
48. Types of laminated veneer lumber, their calculation and design.
49. The uniqueness of the methods of designing a beam with glue.
50. Design and calculation of glued beams.
51. Calculation and design of beams reinforced with steel rods.
52. Types of columns glued from boards, their calculation.
53. Classification of structures with a triangular shape.
54. Types of beams glued from boards, methods of their calculation.
55. Types of frames glued from wood and methods of their calculation.
56. Calculation and specificity of the design of frames with glue.
57. Classification of wooden constructions with lattice cross-section in the plane.

58. Selection of materials for lattice structures, consideration of wood defects, use of glued elements.
59. Taking into account when calculating and designing the longitudinal force in the upper belt with an eccentric.
60. Taking into account elastic, plastic, porosity when calculating the coolness of grid structures.
61. Spatial connection of roof coverings in wooden buildings, horizontal and vertical connectors, single diaphragms.
62. In the design of segmental trusses, additional torque and knots are applied

VI-GLOSSARY

Wood is a self-made, ready-made building material;

Pine, black pine, white pine, and cedar are coniferous trees;

Deciduous - white birch, oak, elm, and aspen trees;

Round building material is wood, sawed flat on both edges, cleaned of branches;

Edged wood materials - sawn wood materials are produced as a result of sawing wood lengthwise in mixed frames or rotary sawmills;

Construction plywood is a structural material made of sheet wood;

Rotting is the deterioration of wood under the influence of normal growing organisms;

A limit state is a state in which it is impossible to use constructions resulting from the influence of external and internal stresses. Wooden and plastic constructions are calculated according to two limit states: load-bearing capacity and according to deformation.

Connecting wooden constructions is a situation in which it is often necessary to extend them and increase the cross-section due to the limited size of the wooden material;

Wooden beams are load-bearing elements in wooden gable roofs;

Grooved, grooved joints - a joint of wooden elements connected without special fasteners ("quarter-groove", "slotted", "half-groove", and "slanted mortise" types of joints);

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