Saimaa University of Applied Sciences
Technology, Lappeenranta
Double Degree Programme in Civil and Construction Engineering

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CALCULATION OF KERTO-S ROOF BEAMS ACCORDING TO RUSSIAN NORMS

Bachelor’s Thesis 2010

ABSTRACT

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Technology, Double Degree Programme in Civil and Construction Engineering
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The purpose of the research was to design and calculate Kerto-S beams for roof structures with different parameters according to Russian norms and make a comparison between these results and Eurocode calculations. In addition, the task was to make a report with the diagrams of the final results. The study was commissioned by the Finnforest company.

In the theoretical part of the study the main issues were the calculations of a simple rafter beam and a main beam on two supports with different parameters according to Russian norms and regulations, using Mathcad software; drawing up diagrams with the relation between calibre of cross-sections, span lengths between supports, steps and maximum characteristic loads; making a comparison between the results from two methods of designing. The information was gathered from literature, norms, regulations, reference books, hand books, Internet and lectures.

In the empirical part of the study the main concerns were finding out the possibilities of applying the characteristic values of physical-mechanical properties of LVL material to calculations according to Russian norms and determining the values of factors for such material. The information was gathered with the help of teachers from certificates of LVL products, norms and regulations.

The final results of this thesis were the diagrams of relations between the maximum span length of the Kerto-S rafter beam and cross-section for appropriate steps; the diagrams of relation between maximum characteristic loads of the Kerto-S main beam and span lengths for appropriate cross-sections. The results can be applied to customers’ estimation of the maximum loads and span lengths of the LVL beams.

Keywords: LVL (Laminated Veneer Lumber) Kerto-S, Rafter beam, Main beam, Roof structure, Span length, Cross-section, Characteristic load

CONTENT

1 INTRODUCTION

The further development of timber structures is impossible without engineering operation, planning and implementation of new technologies for manufacturing products based on wood, with the creation and usage of modern materials in accepted schemes of structural mechanics. The restriction of sizes and the physical-mechanical properties of normal timber do not permit to create different structures (e.g. large-span wooden elements: beams, arches, trusses) that are capable of competing with steel and reinforced concrete ones in strength properties, cost and speed of installation. Thus, there is a need for the development and application of modern materials with different properties, in harmony with traditional timber for creating a product with balanced index - price / quality / reliability. Nowadays it is necessary to find technically advanced and ecological solutions of various tasks on the domestic market in Russia.

To sum up the facts above it can be claimed that the task of developing and applying laminated veneer lumber (LVL), as a structural material for roof structures, is actual and current for Russia. The lack of domestic experience of designing structures from such a material and high specific properties of it, require comprehensive theoretical and experimental control when designing individual elements, fragments, joints and the whole structure. Also fire-resistance, climate influence and technical-economic research are needed to be taken into account in order to systematize and analyze the results. It is also necessary to create and introduce authorities who control the designing of the structures made of LVL with appropriate coefficients and factors. Thereby, there is a need to perform the characteristic values of physical-mechanical properties obtained from domestic experimental research. (Givotov, 2009)

The main purpose of this thesis is to find out algorithm and specifics in designing and calculating certain structures made of LVL Kerto-S according to Russian norms, standards and regulations and further analyzing the differences between results from these calculations and calculations according to Eurocode.

The objectives of the study are:

- To identify the possibility of applying LVL Kerto-S products to Russian markets according to standards and certificates
- To calculate of a simple rafter beam on two supports with different parameters according to Russian norms and regulations, using the Mathcad software
- To calculate of a simple main beam on two supports with different parameters according to Russian norms and regulations, using the Mathcad software
- To draw up diagrams with the relation between calibre of cross-sections, steps, span of supports and maximum characteristic loads
- To calculate the same structures according to Eurocode5, using the Finnwood software
- To compare the diagrams of appropriate calculations and explain differences between them
- To make conclusions about the usage of LVL Kerto-S products for structures in the Russian market

These aims were solved for standard sizes of Kerto-S products in the thesis within the borders of structural mechanics, strength of materials, timber structures and with the help of computer programs.

2 GENERAL INFORMATION ABOUT LVL KERTO-S

Basic data about history, application, main properties, advantages of LVL Kerto-S are presented in this chapter together with the standard sizes and the technological process of production.

2.1 Common definitions and properties

Laminated veneer lumber (LVL) is a material produced by pressing spruce veneers with the parallel arrangement of fibres, with a preliminary treatment of a synthetic binder on them. LVL Kerto is produced from 3mm thick, rotary-peeled softwood veneers that are glued together to form a continuous billet. The billet is cut to length and sawn into beam, plank or panel sizes according to the customer's order. (Finnforest's brochure 2009, p. 3)

LVL was first used to make airplane propellers and other high-strength aircraft parts during World War II. But in civil construction this product was invented by the Wayerhauser company (USA-Canada) in 1960 in order to get the beams of infinite length with high and stable strength characteristics. It was an unusual new product called "truss joists". Every year its consumption has been growing worldwide not only in construction but also in the production of furniture, stairs, windows and doors, prefabricated panels for frame houses and so on. (Neuvonen et al., 1998)

Kerto is a laminated veneer lumber (LVL) product used in all construction works, from new buildings to renovation and repair. It is used in a variety of applications including beams, joists, trusses, frames, components of roof, floor and wall elements, components of the door and vehicle industry, concrete formwork and scaffold planking. Kerto is a strong and dimensionally stable product, which does not wrap or twist. It derives its high strength from the homogeneous bonded structure which also keeps the effects of any defective single veneers down to a minimum. A notable feature of Kerto-S is that the grains run longitudinally throughout its veneer layers. The finished panel is cross-cut and rip-sawn to order. Kerto-S is normally supplied in the form of straight beams but it may also be cut to required shapes.

Kerto-S unites excellent technical performance with the ease of use. Strength, dimensional precision and stability are the essential qualities of Kerto-S. In fact, as beams it is a perfect choice whenever the requirements include long spans and minimal deflection. They are suitable with all roof shapes, also performing well as joists and lintels, truss constructions and frames. Kerto-S is also a wide-used material in the manufacture of prefabricated components. Kerto's light weight is of great advantage especially during building repairs. The erection work can be carried out by fitters, without any heavy hoisting machinery, even in narrow spaces. Kerto-S can be coated to blend in with the rest of the architecture thus making a harmonious whole. (Finnforest's brochure 2009, p.4)

2.2 Standard sizes of Kerto-S beams

The large variety of standard sizes gives a customer a wide range of possibilities in the designing of cost-saving structures without the overrun of materials. The availability of sizes is also very important for quicker construction when time is restricted. Furthermore, it is possible to order your own desired size for unique structures.

Typical cross-sections of Kerto-S beams are given in table 2.1 below:

Table 2.1 Standard sizes of Kerto-S beams (Finnforest's brochure 2009, p. 4)

Kerto-S: Standard sizes									
Thickness (mm)	Height (mm)								
	200	225	260	300	360	400	450	500	600
27	•	•							
33	•	•	•						
39	•	•	•	•					
45	•	•	•	•	•				
51	•	•	•	•	•	•			
57	•	•	•	•	•	•	•		
63	•	•	•	•	•	•	•	•	
75	•	•	•	•	•	•	•	•	•

2.3 Technological process of LVL manufacturing

The way of production from an untreated log to a final structural element is shown below. Advanced control methods are very important in every stage of the production line in order to get superior quality. The manufacturing process provides LVL with a homogenous structure.

This process is briefly shown in figure 2.1

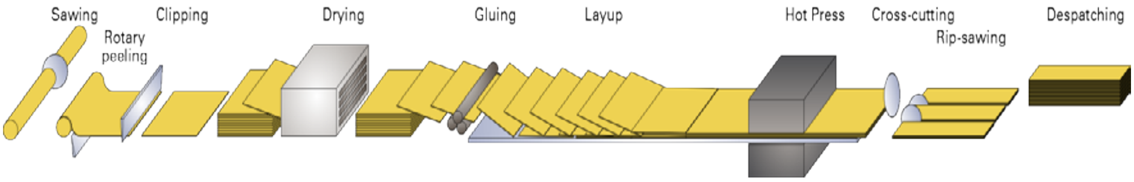


Figure 2.1 LVL production (Finnforest's website 2009)

The information about LVL manufacturing is demonstrated with an example of a factory in the Ugra region (Khanty-Mansiysk Autonomous region) in Russia. The LVL production technology can be divided into the stages that are described in chapters 2.3.1- 2.3.8. (Production technology 2005)

2.3.1 Log handling and hydrothermal treatment

The source material for LVL manufacturing is softwood logs of 4-6 meters. The logs are soaked in water (+ 45 ° C) for 24 hours. After that, the soaked logs are debarked and cut into 1980 ± 10 mm pieces, which are automatically transported to the peeling line (figure 2.3).

Figure 2.2 The process of hydrothermal treatment and peeling

2.3.2 Peeling, clipping and stacking line

The logs are peeled into veneers with the thickness of 3.2mm. After that, veneers are cut into the sheets of specified sizes. The core part of the logs, bark, sawdust and pieces of veneer are waste products. The waste of material in manufacturing is minimal for LVL because almost the whole log is used for veneers production, and just a small number of veneers are rejected due to a defect removal (figure 2.4).

Figure 2.3 Peeling, clipping and stacking line

2.3.3 Drying and grading line

The drying of veneers is proceeded in a large heating room that is heated by thermo oil. The raw veneers of several sheets are loaded into the dryer and transported between the rollers through sixteen sections of the hot part of the dryer. In some cases there is a dryer that goes along the billet itself. After passing through the hot part of the dryer veneers go through three sections of the cooling part, where veneers are cooled down and then transported to the grading area. After that, veneers are sorted by their moisture content with the use of a hygrometer. Wet veneers go to a separate tray (figure 2.5).

Figure 2.4 Drying and grading line

2.3.4 Patching and splicing line

The splicing machine trims the veneer, cuts off defects and faults, joins veneer strips with adhesive threads impregnated with a liquefied glue, and by means of spot gluing with a thermoplastic adhesive, trims and automatically assembles veneers into stacks. Veneers are set into the machine manually, perpendicular to the grain (figure 2.6).

Figure 2.5 Patching and splicing line

2.3.5 Scarfing line

The scarf joining machine makes 28 ± 1mm wide scarf edges on the both sides of the veneer. Then, it joins veneer pieces into desired veneer sheets. After that, veneer sheets pass through a temposonic grading device “Metriguard”, which defines dry veneer density with the help of ultrasonic wave measurements. The denser the veneer, the faster the waves go through it (figure 2.7).

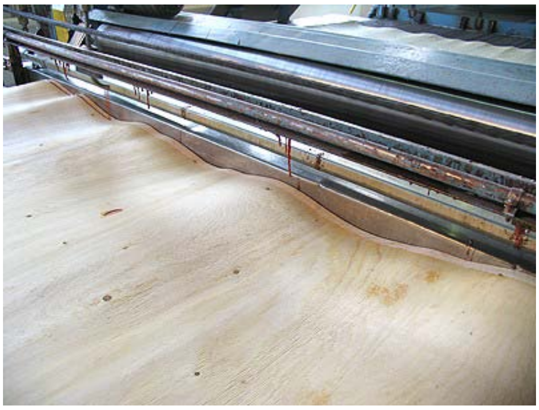


Figure 2.6 Scarfing line

2.3.6 Lay-up station and pre-pressing line

Laying up and pre-pressing are performed at an automated line with the use of a phenol-formaldehyde glue. Using the special type of a curtain coater, the glue is spread on the upper side of each sheet, except on the upper faces. The sheets are laid up to a continuous mat, so that veneer scarf-joints are more than 10 cm apart from each other (figure 2.8).

The lay-up and pre-pressing processes include the following operations:

- Submission of stacks to the lay-up station and transportation of them to a corresponding tray
- Submission of veneer sheets to the gluing kitchen
- Spread of glue and assembly of packages
- Pre-press of packages

- Submission of packages to the edge trimming station
- Sawing into the specified length of billets

Figure 2.7 Lay-up station and pre-pressing line

2.3.7 Hot pressing line

The width of the billet coming from the lay-up station and pre-pressing line is 1.85 m and the thickness is 27-75 mm. This billet is cut into a desired length (maximum 18 m) and laid under a hot press at the temperature of about +140 ° C. The duration of the final pressing depends on the thickness of the billet and the type of the adhesive. The average time is about 37 minutes for LVL 45mm (figure 2.9).

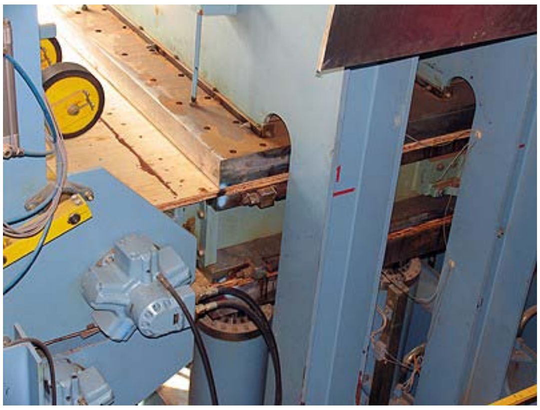


Figure 2.8 Fragment of hot pressing line

2.3.8 Final sizing and grading line

LVL-billets are cut accurately to the customer's requirements. In the ripping saw billets are cut either to the longitudinal direction of beams (Kerto-S and Kerto-T) or to boards (Kerto-Q). In the Finnforest factory there is a special saw for oblique sawing, so billets can be sawed straight or obliquely by the customer's request (figure 2.10).

The process of sizing and grading consists of the following operations:

- Submission of unimproved material to the trimming saw
- Trimming, preparing and getting the samples for tests
- Moving the element due to a roller conveyor of the multi-blade machine
- Trimming the edges of elements and sawing them to a desired width

Finally, a pack of elements is set on the conveyor for packaging and labeling.

Figure 2.9 Final sizing and grading line

2.4 Advantages of the LVL Kerto-S

The most important reasons for LVL's success are the quality of the product itself and its properties. LVL is a high-quality structural material with uniform engineering properties and dimensional flexibility, which makes it superior to sawn timber and glued laminated timber, particularly for large-span structures. One of the main ideas of LVL is to disperse or remove strength-reducing characteristics. And LVL is an engineered highly predictable, uniform lumber product, because natural defects such as knots, slope of grains and splits are dispersed throughout the material or removed altogether within the veneer assembly. In spite of this, the natural aesthetic beauty of sawn timber includes the appearance of knots, wane, resin pockets, splits, slope of grain, and a few other less significant defects. Besides all these, Kerto-S has quite many other advantages that are represented below (Finnforest's brochure 2009, p. 3).

Advantages of the LVL Kerto-S:

- High characteristic strength values; more than two times higher that of normal timber
- Large variety of available sizes; thickness 27-75mm, width 200-900mm, length 4-12m (for roof structures)

- Precise in dimensions, does not warp or twist; completely homogeneous material with constant properties along the entire length and constant physical properties, which do not depend on seasonal factors, unlike other timber does
- Beautiful, sound-insulating, fire-safe, high thermal properties
- Light weight and excellent workability; excluding the use of lifting equipment and mechanisms that will allow faster and cheaper construction
- Environmentally friendly product and aesthetics; wood procured according to the Pan-European Forest Certification (PEFC) standard

3 GENERAL INFORMATION ABOUT WOODEN ROOF STRUCTURES

The main idea of this chapter is to provide brief information about the advantages of wooden roof structures compared with steel and concrete structures, installation of roof structures, main components of the roof. Types of rafters and main beams, and principles of calculating them are also discussed. Practically all this information below concerns roof structures made of any wooden material particularly from LVL.

3.1 Common principles of erecting roof structures made of Kerto-S

Basically, the load bearing system of a roof structure for residential houses is made of wood. Such systems are easy to manufacture and install. Wooden roof components can be adjusted simply on the building site; they can be shortened, cut, added or changed in configuration. Sometimes it is necessary to do it on-site because of errors in the construction of walls. Performing these steps with metal elements is rather difficult, and with reinforced concrete structures it is almost impossible. That is why concrete and steel – in spite of the fact that they are more durable and stronger than wood - are used only during the construction of considerable stone, brick or concrete houses with heavy and complex roof structures. And timber is more lightweight than other traditional structural materials. So, there is usually no need to use heavy machineries and lifting devices. Elements can be adjusted only by labour force. Also the covering of the roof with reinforced concrete or metal is harder than with wood. (Installation of roof system 2007)

The installation of a roof system begins with the leveling of the top surface of bearing walls by mortar. Waterproofing layer with the further installation of plate and sill is laid on the mortar. Next, the roof structure, which consists of the stands, rafters, ridge beams, purlins, coatings and other components is erected. First, end rafters are installed and then intermediate ones. The distance between rafters is determined by the designed structural system and bearing capacity of the whole structure. For increasing the durability of the structure proper ventilation is provided on the loft. (Installation of roof system 2007)

3.2 Wooden roof components

Generally, wooden roof structures involve three main parts: structural part, covering part and drainage part. The structural part is a load-bearing part of the roof that carries loads from above structures and weather conditions, including self-weight, weight of the coverings, heat insulation, wind load, snow load and service load. This part consists of main beams, rafters, stands, studs, bracings and other structural elements. The covering part is intended to protect the inside area of the loft from the influence of weather and it consists of boards or sheet material, shingles, tiles, slate or metal coverings and substrate material. Drainage is required for the diversion of rain and melt water. The drainage part consists of flashings and a drainage system. (Roof system components 2009)

Components of roof structure are given below in figure 3.1

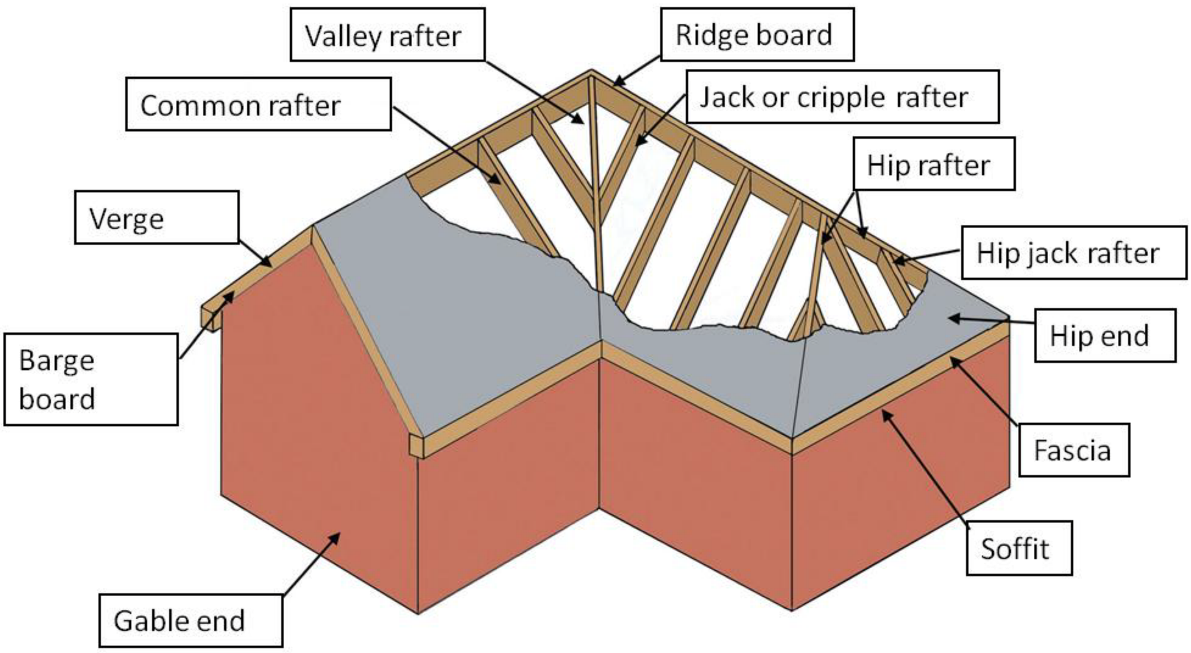


Figure 3.1 Components of roof (Roof components 2008)

Definitions for figure 3.1 are explained below:

- Rafter is one of a series of sloped structural members
- Ridge is a top edge of two intersecting sloping roof surfaces
- Valley is an angle formed at the intersection of two sloping roof surfaces
- Fascia is a flat board, band or face located at a cornice's outer edge
- Soffit describes the underside of any construction element
- Bargeboard is a board fastened to the projecting gables of a roof to give them strength and to mask, hide and protect the otherwise exposed end of the horizontal timbers and purlins of the roof to which they were attached
- Gable is generally a triangular portion of a wall between the edges of a sloping roof

3.3 Basic information about rafters

Rafter is a beam or sloped structural member, which extends from the ridge or hip to the downslope perimeter or eave, forming part of the internal framework of a roof. It is designed to support the roof deck and its associated loads. (Dictionary 2004) In civil construction rafters are typically made of wood. Rafters are shown on figures 3.2 and 3.3.

The stability and strength of the roof are completely dependent on their supporting structure - the roof system. Rafters are the main load-bearing part of the roof structure. They are designed to withstand not only the weight of the roof, but the pressure of snow and wind. Therefore a rafter's system calculation is based on the type of roofing material, as well as the normal, for appropriate area, wind force and weight of the snow cover.

There are two types of rafters:

- With intermediate support (between the walls)
- Without intermediate support

Figure 3.2 Different kinds of rafters (Picasa 2010)

Figure 3.3 Common rafters (FreeTorg 2007)

Generally, a rafter is calculated as a beam on two supports with the effect of the bending moment and compressive force for normal strength, shear strength, stability, stiffness and local compressive deformation near supports. In some cases, a rafter can be supported by a crossbar in the middle part and so it is calculated as a continuous beam on three supports. The sequence of calculations and main principals are described in chapter 4. (Shishkin, 1974, p. 41)

3.4 Basic information about main beams

A main beam for a roof in civil engineering is a main horizontal structural element of the roof structure, which is needed in order to support rafter beams and carry loads from the self-weight of the roof, snow, wind and services (Great Encyclopedic Dictionary). The top intermediate main beam is called a ridge beam in construction. The picture of such a beam is shown on figure 3.4.

Figure 3.4 Ridge beam (Building materials 2007)

A main beam is a simple beam on two supports, which takes a bending moment stress and shear stress. It is laid on stands or columns with the distance from 2m to 8m (depending on the maximum load) between them. A ridge beam is calculated for normal strength, shear strength, common stability, stiffness and local compressive deformation near supports. In some cases, the main beam can be compiled from two various beams, which are linked together, in order to carry huge loads and make a needed support area for rafters. The sequence of calculations and main principals are described in chapter 4.

4 STRUCTURAL ANALYSIS OF WOODEN ROOF BEAMS BASED ON RUSSIAN NORMS

Formulated general principals about designing and calculating of Kerto-S roof beams are presented in this chapter. All information complies with the theory of timber structures analysis according to Russian norms. Methods, which are given below, cover two structural elements of the roof system: the main beam and rafter beam. Besides structural investigations it is necessary to study the protection methods of timber structures against moisture, fire, biological destruction, corrosion according to SNiP “Timber Structures”. This should be made properly in order to find out / analyse standard working conditions of the structure and its durability. They can be both construction methods and treatment procedures.

4.1 Designing of roof beams

The designing and calculating of Kerto-S beams for roof structures were implemented in accordance with the following Russian norms, rules and regulations: SNiP “Loads and Influences”, SNiP “Timber Structures”, STO “36554501-002-2006”, STO “36554501-015-2008”, timber structures designing handbook, textbooks, information guides and other reference books. The suggestions and assistance were given by teachers from the Saimaa University of Applied Sciences and the Saint-Petersburg State University of Architecture and Civil Engineering.

Calculations were done by the Mathcad program and the Finnwood software. Formulas, factors and values of different parameters were taken from the above mentioned normative documents. Source information for calculations is taken from the brochure of the Finnforest company and characteristic ultimate (1^{st}- in Russian norms) and serviceability (2^{nd}) limit states are taken from the Certificate of Conformity (see Appendix 3). An example of the calculations of a rafter beam and main beam for one cross-section (51x200mm) is given in Appendix 1.

The reliability of the results ensures the correct using of scientific positions in the field of building mechanics, timber structures and strength of materials. In spite of this, some parts of the calculations require further investigations and analyses, by reason of the absence of some information about the LVL material in Russian norms.

4.2 Rafter beam calculation

There is a calculation of the Kerto-S rafter beam according to the sequence in 4.2.1 in this part. The calculation of rafter beams should be implemented according to a bearing capacity (1^{st} limit state) and the deflections (2^{nd} limit state) of the elements. It was happened so that the maximum span length is taken from a strength calculation (normal stress) for every cross-section. All other calculations are satisfied with this length.

4.2.1 Initial data for calculating rafters

Rafters, in this case, are calculated in order to find out the maximum span length, which depends on the cross-section and step between rafters. It is needed for making diagrams with relation between the span length and cross-section. And as it was mentioned before the source data is taken from the company's brochure to compare the results.

The calculation sequence is as follows:

- Compilation of loads
- Estimation of span length from strength calculation
- Verification of span length on the assumption of stiffness calculation
- Verification of span length on the assumption of shear strength calculation
- Verification of span length on the assumption of stability calculation
- Verification of span length on the assumption of calculation of the local plastic compression on the lower support

- Verification of span length on the assumption of calculation of the local plastic compression on the upper support

The source data for calculations is as follows:
Width of support >120mm. Roof slop i=1:3. Self weight g_k=0.9kN/m^2. Snow load on the ground S_k=2.5kN/m^2. Distance between upper supports for providing stability <400mm. Final deflection depends on span length, according to norms. Span between rafters k_1=900mm, k_2=1200mm. (Finnforest's brochure 2009)

The principal scheme of the rafter beam is shown in figure 4.1 below:

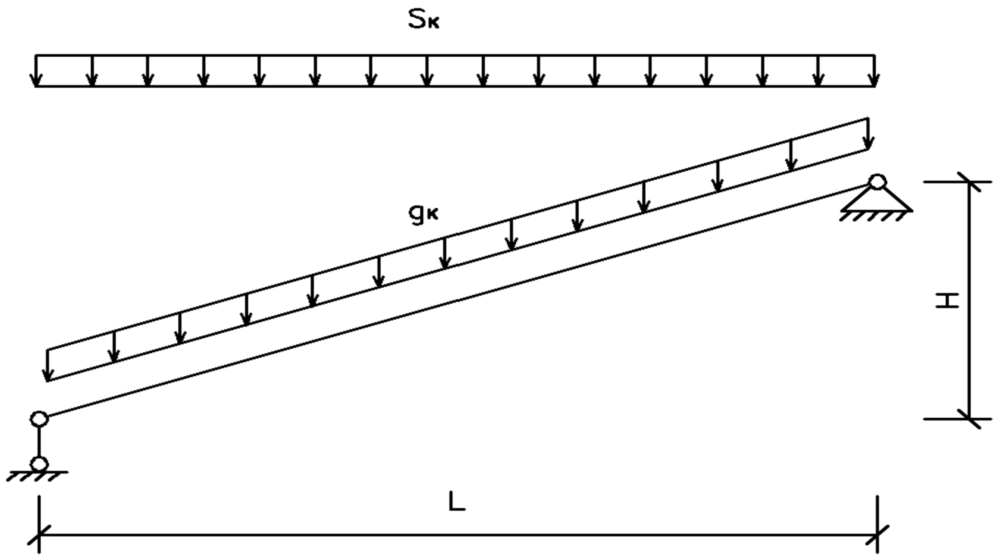


Figure 4.1 Rafter beam design scheme

4.2.2 Load compilation

Strength characteristic values are taken from the Certificate (Appendix 3) and listed below.

$f_{c.0.k} = 35N/mm^2$ - characteristic value of the compressive strength parallel to the grain

$f_{c.90.edge.k} = 6N/mm^2$ - characteristic value of the compressive strength perpendicular to the grain, edgewise

$f_{v.0.edge.k} = 4.1N/mm^2$ - characteristic value of the shear strength edgewise

$f_{m.0.edge.k} = 44N/mm^2$ - characteristic value of the bending strength edgewise

$E_{0.mean} = 13800N/mm^2$ - mean value of the elastic modulus

$i = 1/3$ - slope of the roof; $\alpha = arctg(i) = 18.435\deg$ - angle of the slope

$g_k = 0.9kN/m^2$ - characteristic value of the dead load (self-weight of the roof structure)

$S_{k\,f} = S_k \cdot c_e \cdot c_t \cdot \mu$ - characteristic value of the live load (snow weight) on the horizontal projection of the roof surface (STO 36554501-015-2008- "Loads and Influences" 2008, p. 10)

S_k - weight of the snow mass per $1m^2$ on the horizontal ground surface

c_e - factor for demolition of the snow mass from roof surface by reason of wind load and other agents

c_t - thermal factor that takes into account snow melting by reason of heat losses through the upper ceiling in the loft

μ - transfer factor that converts value of the snow weight on the ground to weight on the horizontal projection of the roof surface

$$S_{k\,f} = S_k \cdot c_e \cdot c_t \cdot \mu = 2.5 \cdot 1 \cdot 1 \cdot 1 = 2.5kN/m^2$$

$c_e = 1.0$ - for roof structures with slop i >20%, single- or multi-span buildings without bay windows, designed in regions with wind velocity V>4m/s (STO 36554501-015-2008- "Loads and Influences" 2008, p. 11)

V>4m/s - average wind speed for three the most coldest months of the year (taken from map of wind regions)

$c_t = 1.0$ - for roofs (ceilings) with thermal insulation (low heat losses) (STO 36554501-015-2008- "Loads and Influences" 2008, p. 11)

$\mu = 1.0$ - for roofs with slope angle $\alpha < 25°$ (SNiP 2.01.07-85*- "Loads and Influences" 1987, p.23)

$q_k = (g_k \cdot \cos\alpha + S_{k.f} \cdot \cos^2\alpha) \cdot k$ - normal component of the characteristic value of the whole load, acting on the rafter beam (Filippov, Konstantinov 1965, p. 148)

k - step of the rafters (m)

$q_k = (g_k \cdot \cos\alpha + S_{k.f} \cdot \cos^2\alpha) \cdot k = (0.9 \cdot 0.949 + 2.5 \cdot 0.949^2) \cdot k = 3.104 \cdot k(kN/m)$

$g_d = g_k \cdot \gamma_f$ - design value of the dead load

γ_f - load safety factor for self weights of building structures (Appendix 2)

$\gamma_f = 1.1$ - for wooden structures

$\gamma_f = 1.2$ - for concrete structures with mean density lower than 1600 kg/m^3, insulations, facing layers

For this case, a load safety factor was taken as 1.15 (average between these values above), because the self weights of wooden components and concrete or facing material components (tiles) are approximately the same.

$g_d = 0.9 \cdot 1.15 = 1.035 kN/m^2$

$S_d = S_{k.f} / 0.7$ - design value of the live load (SNiP 2.01.07-85*- "Loads and Influences" 1987, p.10)

$S_d = S_{k.f} / 0.7 = 2.5/0.7 = 3.571 kN/m^2$ - this value corresponds to 5-6 snow region in Russian Federation (Appendix 2)

0.7 - transfer factor that converts characteristic value of the live load to design value of the live load.

$q_d = (g_d \cdot \cos\alpha + S_d \cdot \cos^2\alpha) \cdot k$ - normal component of the design value of the whole load, acting on the rafter beam (Filippov, Konstantinov 1965, p. 148)

$q_d = (g_d \cdot \cos\alpha + S_d \cdot \cos^2\alpha) \cdot k = (1.035 \cdot 0.949 + 3.571 \cdot 0.949^2) \cdot k = 4.196 \cdot k(kN/m)$

4.2.3 Strength calculation

A strength calculation is made due to the normal stress of the element. Firstly, the formulated general theory of this calculation and then the estimation of the maximum span length are presented here. A material factor is taken for traditional wood, not for LVL material. A design length factor is taken for hinge-supported compressed elements, in spite of tensile parts in the real scheme.

4.2.3.1 Sequence of calculation

The strength calculation of structures with the effect of the bending moment and compressive force should be made according to the following formula (SNiP II-25-80*- "Timber structures" 1983, p.12):

$$\frac{N}{F_d} + \frac{M_{\max}}{\zeta \cdot W_d} \leq f_{c.0.d} \qquad (4.1)$$

$M_{\max}$ - maximum bending moment in the intermediate cross-section from transversal component of the load

N - compressive force

F_d - design value of the cross-section area

W_d - design value of the section modulus

$f_{c.0.d}$ - design value of the compressive strength along the grain

ζ - additional bending moment factor (from deflection by reason of longitudinal force).

$$N = q_d \cdot \frac{l_d}{2} \cdot tg\alpha \qquad (4.2)$$

l_d - span length of the beam between supports.

$$W_d = \frac{b \cdot h^2}{6}\ ; \qquad I = \frac{b \cdot h^3}{12}$$

I - inertia moment of the cross-section.

$$M_{\max} = q_d \cdot \frac{l_d^{\,2}}{8} \qquad (4.3)$$

For hinge-supported elements with the symmetrical curve of the bending moment of sinusoidal, parabolical, polygonal and familiar contours and also for cantilever elements an additional bending moment factor should be defined from the following expression (SNiP II-25-80*- “Timber structures” 1983, p.12):

$$\zeta = 1 - \frac{N}{\varphi \cdot f_{c.0.d} \cdot F_d} \qquad (4.4)$$

φ - buckling factor (from longitudinal component of the load)

$\varphi = \frac{A}{\lambda^2}$, (SNiP II-25-80*- “Timber structures” 1983, p.9)

$\lambda = \frac{l_d \cdot \mu_0}{r}$, (SNiP II-25-80*- “Timber structures” 1983, p.9)

$r = \sqrt{\frac{I}{F_d}}$

λ - flexibility of an element

A - factor for LVL, the value of which is “2500” (STO 36554501-002-2006- “Glued and solid wood structures” 2006, p. 16)

μ_0 - design length factor, the value of which is “1” (SNiP II-25-80*- “Timber structures” 1983, p.13)

r - radius of inertia

$$f_{c.0.d} = f_{c.0.k} \cdot \frac{m_{dl} \cdot m_b}{\gamma_m \cdot \gamma_n} \qquad (4.5)$$

m_{dl} - load duration factor (safety factor)

m_b - operation factor, that depends on environmental conditions

γ_m - material factor (safety factor)

γ_n - function factor that depends on purpose of the building (safety factor)

Values of these factors are taken from “Timber structures designing handbook for II-25-80*” 1983, (Appendix 2):

$m_{dl} = 0.66$ - for the action of permanent load and short-term load

$m_b = 0.9$ - for A3(C2) environmental class

$\gamma_m = 1.13$ - for compression of glued elements

$\gamma_n = 0.95$ - for II function group of the building (residential construction)

The material factor is estimated according to the following formula (“Timber structures designing handbook for II-25-80*” 1983, p. 18):

$$\gamma_m = \frac{1-\eta_n \cdot \nu}{1-\eta \cdot \nu},$$

η_n - factor which depends on the adopted level of safety (confidence level) and type of density function for characteristic values of strength

η - factor which depends on the adopted level of safety (confidence level) and type of density function for design values of strength

ν - variation factor for strength, which depends on properties of the material

$\eta_n = 1.65$ - for regular density function and level of safety P=0.95 (“Timber structures designing handbook for II-25-80*” 1983, p. 16)

$\eta = 2.33$ - for regular density function and level of safety P=0.99 (“Timber structures designing handbook for II-25-80*” 1983, p. 16)

$\nu = 0.13$ - for compressive strength along the grain. (GOST 164830-89- “General requirements for physical-mechanical experiments” 1989, p. 3)

$$\gamma_m = \frac{1-\eta_n \cdot \nu}{1-\eta \cdot \nu} = \frac{1-1.65 \cdot 0.13}{1-2.33 \cdot 0.13} = 1.13$$

4.2.3.2 Estimation of the maximum span length of Kerto-S roof beam

The calculation of the span length value is made from strength valuation by the system of formulas: 4.1, 4.2, 4.3, 4.4, buckling factor determination and flexibility determination.

The maximum span length has been defined from the following system:

$$\left\{\begin{array}{c} \dfrac{N}{F_d} + \dfrac{M_{\max}}{\zeta \cdot W_d} = f_{c.0.d} \\ N = q_d \cdot \dfrac{l_d}{2} \cdot tg\alpha \\ M_{\max} = q_d \cdot \dfrac{l_d^{\,2}}{8} \\ \lambda = \dfrac{l_d \cdot \mu_0}{r} \\ \varphi = \dfrac{A}{\lambda^2} \\ \zeta = 1 - \dfrac{N}{\varphi \cdot f_{c.0.d} \cdot F_d} \end{array}\right\} \rightarrow solve, N, M_{\max}, \lambda, \zeta, \varphi, l_d \rightarrow l_d \qquad (4.6)$$

This result can also be defined from the expression below:

$$l_d^{\,2} \cdot \frac{q_d}{8 \cdot W_d \cdot \left(1 - \dfrac{q_d \cdot tg(\alpha) \cdot l_d^{\,3} \cdot \mu_0^{\,2}}{2 \cdot F_d \cdot f_{c.0.d} \cdot A \cdot r^2}\right)} + q_d \cdot \frac{l_d \cdot tg(\alpha)}{2 \cdot F_d} = f_{c.0.d} \qquad (4.7)$$

Then, using the value of the span length, which is taken from strength calculation above (formula 4.6 or 4.7), it is necessary to check the stiffness of the structure, its stability, shear strength and the probability of a local collapse close to supports.

4.2.4 Stiffness calculation

The deflections of elements should be lower than are allowed in SNiP “Loads and Influences”. The maximum deflection of the hinge-supported and cantilever bending elements of permanent and variable cross-sections should be defined with the following formula (SNiP II-25-80*- “Timber structures” 1983, p.16):

$$f_{\max} = \frac{f_0}{k} \cdot \left(1 + c \cdot \left(\frac{h}{l_d}\right)^2\right) \qquad (4.8)$$

f_0 - deflection of a permanent section beam with height “h” without accounting of shear deflection

k - variability of the height of the section, the value of which is “1” for permanent section

c - shear deflection factor

l_d - span length of an element from formula 4.7

h - height of an element

The deflection of hinge-supported elements with the effect of symmetrical bending and compressive stresses should be defined with the following formula (SNiP II-25-80*- “Timber structures” 1983, p.16):

$$f_N = \frac{f_{\max}}{\zeta}, \quad \text{where}$$

ζ - defined from formula 4.4

$$f_0 = \frac{5}{384} \cdot \frac{q_k \cdot l_d^{\ 4}}{m_b \cdot E_{0.mean} \cdot I} \cdot \gamma_n \qquad (4.9)$$

According to paragraphs 3.2.a, 3.5 in SNiP II-25-80*- “Timber structures”, the mean value of the elastic modulus should be multiplied by an operation factor; and the deflection should be multiplied by a function factor.

Finally, formula 4.8 transforms into:

$$f_N = \left(\frac{5}{384} \cdot \frac{q_k \cdot l_d^{\ 4}}{m_b \cdot E_{0.mean} \cdot I} \right) \cdot \left[1 + c \cdot \left(\frac{h}{l_d} \right)^2 \right] \cdot \frac{\gamma_n}{\left(1 - \frac{q_d \cdot l_d^{\ 3} \cdot tg(\alpha)}{A \cdot r^2 \cdot f_{c.0.d} \cdot F_d \cdot 2} \right)} \leq f_u \qquad (4.10)$$

$c = 19.2$ - accounting of the shear deflection. (Appendix 2)

The limit values of deflections (f_u) according to STO 36554501-015-2008- “Loads and Influences” are evaluated through the following expressions:

- for span length $l \leq 1$ the final deflection should be lower than $l/120$
- for span length $l = 3$ the final deflection should be lower than $l/150$
- for span length $l = 6$ the final deflection should be lower than $l/200$
- for span length $l = 12$ the final deflection should be lower than $l/250$
- for span length $l = 24$ the final deflection should be lower than $l/300$

4.2.5 Shear strength calculation

A shear strength calculation is presented for the normal cross-section of the element in this part. The designing of bending elements for shear strength should be defined using the formula below (SNiP II-25-80*- “Timber structures” 1983, p. 11):

$$\frac{V \cdot S_g}{I_g \cdot b_d} \leq f_{v.0.edge.d} \qquad (4.11)$$

V	-	design value of the shear force
S_g	-	gross static moment of the sheared part of the cross-section related to a neutral axle
$f_{v.0.edge.d}$	-	design value of the shear strength from bending
I_g	-	gross inertia moment of the cross-section

b_d - design value of the width of the element

$$S_g = \frac{F_d}{2} \cdot \frac{h}{4}; \qquad V = q_d \cdot \frac{l_d}{2}$$

l_d – span length of an element from formula 4.7.

Formula 4.11 can be simplified as follows:

$$\frac{3 \cdot V}{2 \cdot F_d} \leq f_{v.0.edge.d} \qquad (4.12)$$

$$f_{v.0.edge.d} = f_{v.0.edge.k} \cdot \frac{m_{dl}}{\gamma_m \cdot \gamma_n} \cdot m_b \qquad (4.13),$$

In expression 4.13 all factors have the same values as mentioned above for the compressive strength, except the material factor, the value of which is "1.25" for shear strength according to "Timber structures designing handbook for II-25-80*" and GOST 164830-89- "General requirements for physical-mechanical experiments". (Appendix 2)

$\nu = 0.2$ - for shear strength edgewise (Paragraph 1.7 in GOST 164830-89)

$$\gamma_m = \frac{1-\eta_n \cdot \nu}{1-\eta \cdot \nu} = \frac{1-1.65 \cdot 0.2}{1-2.33 \cdot 0.2} = 1.25$$

4.2.6 Stability calculation

The stability calculation of structures with the effect of the bending moment and compressive force should be evaluated with the following expression (STO 36554501-002-2006- "Glued and solid wood structures" 2006, p. 23):

$$\frac{N}{\varphi \cdot f_{c.0.d} \cdot F_d} + \left(\frac{M_{\max}}{\zeta \cdot \varphi_M \cdot f_{m.0.edge.d} \cdot W_d} \right)^n \leq 1 \qquad (4.14)$$

n - for elements without fastening of the tensile side out of bending plane the value equal to “2” (Paragraph 4.18 in STO 36554501-002-2006)

ζ - defined from formula 4.4

φ - buckling factor for the flexibility of an element with design length l_1 out of bending plane (STO 36554501-002-2006- “Glued and solid wood structures” 2006, p. 16)

φ_M - factor for elements with the effect of bending moment (STO 36554501-002-2006- “Glued and solid wood structures” 2006, p. 20)

$$\varphi = \frac{A}{\lambda^2}; \qquad \lambda = \frac{l_1 \cdot \mu_0}{r}; \qquad l_1 = 400mm$$

For elements with the effect of the bending moment of rectangular cross-section, hinge-supported out of the bending plane and fastened from rotation around the longitudinal axle, the φ_M factor should be defined from the following expression (formula 23 in the SNiP II-25-80*- “Timber structures”):

$$\varphi_M = 140 \cdot \frac{b^2}{l_p \cdot h} \cdot k_\phi \qquad (4.15)$$

l_p - distance between the support sections of an element or distance between the supported points of fastenings when the compression side is fixed. In this case, it is l_1

k_ϕ - factor that depends on the shape of the bending moment curve

k_ϕ - for parabolic shape value is “1.13” (Appendix 2)

$$f_{m.0.edge.d} = f_{m.0.edge.k} \cdot \frac{m_{dl} \cdot m_b}{\gamma_m \cdot \gamma_n}$$

In expression 4.14 all factors have the same values as mentioned earlier for the compressive strength, except the material factor, the value of which is “1.16” for the bending strength according to “Timber structures designing handbook for II-

25-80*" and GOST 164830-89- "General requirements for physical-mechanical experiments". (Appendix 2)

$\nu = 0.15$ - for bending strength (Paragraph 1.7 in GOST 164830-89)

$$\gamma_m = \frac{1-\eta_n \cdot \nu}{1-\eta \cdot \nu} = \frac{1-1.65 \cdot 0.15}{1-2.33 \cdot 0.15} = 1.16$$

4.2.7 Calculation of the local plastic compression on the lower support

The value of mutilation force Q should be lower than the design bearing capacity of the joint $T_{\max}\,(T_{\max} > Q)$, calculated from the following expression (SNiP II-25-80*- "Timber structures" 1983, p. 17):

$$T_{\max} = f_{c.\alpha.d} \cdot F_w \qquad (4.16)$$

$F_w = b \cdot S_1$ - mutilation area of the joint

$S_1 = 120mm$ - width of support

$f_{c.\alpha.d}$ - mutilation bearing capacity of timber at angle to the grains. (SNiP II-25-80*- "Timber structures" 1983, p. 5)

$$f_{c.\alpha.d} = \frac{f_{c.0.d}}{1+\left(\frac{f_{c.0.d}}{f_{l.c.90.edge.d}}-1\right)\cdot \sin(90\deg-\alpha)^3} \qquad (4.17)$$

$f_{c.0.d}$ - mutilation bearing capacity of timber along the grain

$f_{l.c.90.edge.d}$ - mutilation bearing capacity of timber perpendicular to the grain

$$f_{l.c.90.edge.k} = f_{c.90.edge.k} \cdot \left(1+\frac{8}{S_1/10+1.2}\right)$$

$$f_{c.0.d} = f_{c.0.k} \cdot \frac{m_{dl} \cdot m_b}{\gamma_m \cdot \gamma_n} \qquad (4.18)$$

$$f_{l.c.90.edge.d} = f_{l.c.90.edge.k} \cdot \frac{m_{dl} \cdot m_b}{\gamma_m \cdot \gamma_n} \qquad (4.19)$$

In expressions 4.18 and 4.19 all factors have the same values as given earlier for the compressive strength, except the material factor, the value of which is "1.25" for mutilation strength according to "Timber structures designing handbook for II-25-80*" and GOST 164830-89- "General requirements for physical-mechanical experiments". (Appendix 2)

$\nu = 0.2$ - for local compressive strength (Paragraph 1.7 in GOST 164830-89)

$$\gamma_m = \frac{1-\eta_n \cdot \nu}{1-\eta \cdot \nu} = \frac{1-1.65 \cdot 0.2}{1-2.33 \cdot 0.2} = 1.25$$

$Q = \dfrac{q_d \cdot l_d}{2 \cdot \cos(\alpha)}$ - mutilation force (4.20)

l_d - span length of an element from formula 4.7

4.2.8 Calculation of the local plastic compression on the upper support

The value of mutilation force V should be lower than the design bearing capacity of the joint $T_{\max}$ ($T_{\max} > V$), calculated from the following expression (SNiP II-25-80*- "Timber structures" 1983, p. 17):

$$T_{\max} = f_{l.c.90.edge.d} \cdot F_w \qquad (4.21)$$

$F_w = b \cdot S_1$	-	mutilation area of the joint
$S_1 = 126mm$	-	width of support
$f_{l.c.90.edge.d}$	-	mutilation bearing capacity of timber perpendicular to the grain

$$f_{l.c.90.edge.k} = f_{c.90.edge.k} \cdot \left(1 + \frac{8}{S_1/10 + 1.2}\right)$$

$$f_{l.c.90.edge.d} = f_{l.c.90.edge.k} \cdot \frac{m_{dl} \cdot m_b}{\gamma_m \cdot \gamma_n} \qquad (4.22)$$

In expression 4.22 all factors have the same values as given earlier for the compressive strength, except the material factor, the value of which is "1.25" for mutilation strength according to "Timber structures designing handbook for II-25-80*" and GOST 164830-89- "General requirements for physical-mechanical experiments". (Appendix 2)

$\nu = 0.2$ - for local compressive strength (Paragraph 1.7 in GOST 164830-89)

$$\gamma_m = \frac{1-\eta_n \cdot \nu}{1-\eta \cdot \nu} = \frac{1-1.65 \cdot 0.2}{1-2.33 \cdot 0.2} = 1.25$$

$V = \frac{q_d \cdot l_d}{2}$ - mutilation force

l_d - span length of an element from formula 4.7

4.3 Main beam calculation

There is a calculation of the Kerto-S main beam according to the sequence in 4.3.1. The calculation of main beams should be implemented according to the bearing capacity (1st limit state) and deflections (2nd limit state) of the elements. The maximum characteristic load for different cross-sections and spans is taken from various calculations.

4.3.1 Initial data for calculating main beams

Main beams, in this case, are calculated in order to find out the maximum characteristic load. It depends on the cross-section and distance between supports. It is needed for making diagrams with relations between the characteristic load and span length. And as it was mentioned before the source data is taken from the company's brochure for comparing the results.

The calculation sequence is shown below:

- Determination of the maximum characteristic load from strength calculation
- Determination of the maximum characteristic load from shear strength calculation
- Determination of the maximum characteristic load from stiffness calculation
- Determination of the maximum characteristic load from stability calculation
- Determination of the maximum characteristic load from local compression calculation
- Selection of the minimum value of characteristic load from calculations above

The source data for calculations is as follows:
Width of supports >120mm. Percentage of the self-weight 20%. Service classes 1-2. Wind loads are not taken into account in calculations. Final deflection depends on span length, according to norms.

The principal scheme of the main beam is presented below:

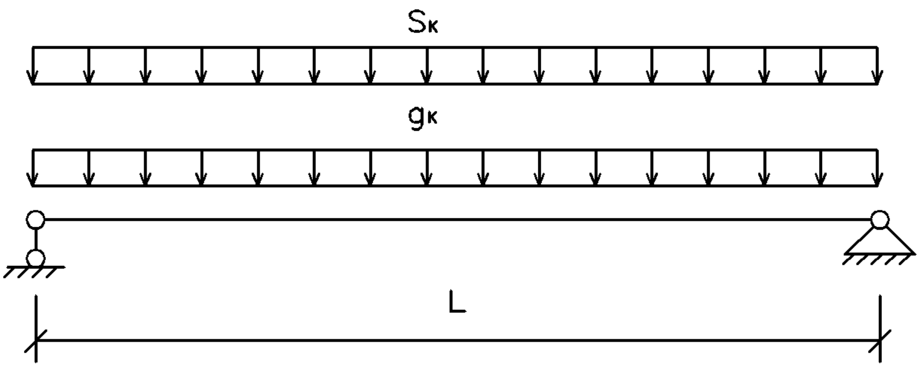


Figure 4.2 Main beam design scheme

4.3.2 Strength calculation

The designing of the bending elements for the strength of the normal stresses should be defined from the following expression (SNiP II-25-80*- “Timber structures” 1983, p. 11):

$$\frac{M_{\max}}{W_d} \leq f_{m.0.edge.d} \qquad (4.23)$$

$M_{\max}$	-	maximum bending moment in the intermediate cross-section from the whole load
W_d	-	design value of the section modulus
$f_{m.0.edge.d}$	-	design value of the bending strength edgewise

$$W_d = \frac{b \cdot h^2}{6}; \quad (4.24) \qquad M_{\max} = q_d \cdot \frac{l_d^{\,2}}{8} \qquad (4.25)$$

$$f_{m.0.edge.d} = f_{m.0.edge.k} \cdot \frac{m_{dl} \cdot m_b}{\gamma_m \cdot \gamma_n},$$

The values of the factors below are taken from “Timber structures designing handbook for II-25-80*” (Appendix 2):

$m_{dl} = 0.66$	-	for the action of permanent load and short-term load
$m_b = 0.9$	-	for A3(C2) environmental class
$\gamma_m = 1.16$	-	for bending glued elements
$\gamma_n = 0.95$	-	for II function group of the building (residential construction)

$\nu = 0.15$ - for bending strength (Paragraph 1.7 in GOST 164830-89)

$$\gamma_m = \frac{1 - \eta_n \cdot \nu}{1 - \eta \cdot \nu} = \frac{1 - 1.65 \cdot 0.15}{1 - 2.33 \cdot 0.15} = 1.16$$

From expressions 4.23, 4.24 and 4.25 the value of the maximum design load can be extracted:

$$q_d = f_{m.0.edge.d} \cdot \frac{4 \cdot b \cdot h^2}{3 \cdot l_d^{\ 2}} \qquad (4.26)$$

Then, the characteristic value of the maximum load could be found from the following sequence:

$q_k = g_k + S_k$; $\qquad g_k = 0.2 \cdot q_k$

$q_d = g_d + S_d$; $\qquad q_d = g_k \cdot 1.15 + 4 \cdot g_k \cdot \frac{1}{0.7}$; $\qquad g_k = \dfrac{q_d}{\left(1.15 + \frac{4}{0.7}\right)}$

Finally:

$$q_k = 5 \cdot g_k = \frac{5 \cdot q_d}{\left(1.15 + \frac{4}{0.7}\right)} \qquad (4.27)$$

4.3.3 Shear strength calculation

The designing of bending elements for shear strength should be defined using the following formula (SNiP II-25-80*- “Timber structures” 1983, p. 11):

$$\frac{V \cdot S_g}{I_g \cdot b_d} \leq f_{v.0.edge.d} \qquad (4.28)$$

V - design value of the shear force

S_g - gross static moment of the sheared part of the cross-section related to the neutral axle

$f_{v.0.edge.d}$ - design value of the shear strength from bending

I_g - gross inertia moment of the cross-section related to the neutral axle

b_d - design value of the width of the element

$$S_g = \frac{F_d}{2} \cdot \frac{h}{4}; \quad (4.29) \qquad V = q_d \cdot \frac{l_d}{2} \quad (4.30)$$

Using expressions 4.29 and 4.30 formula 4.28 can be simplified as follows:

$$\frac{3 \cdot V}{2 \cdot F_d} \leq f_{v.0.edge.d} \qquad (4.31)$$

$$f_{v.0.edge.d} = f_{v.0.edge.k} \cdot \frac{m_{dl}}{\gamma_m \cdot \gamma_n} \cdot m_b \qquad (4.32)$$

In expression 4.32 all factors have the same values as presented earlier for the bending strength, except the material factor, the value of which is “1.25” for shear strength according to “Timber structures designing handbook for II-25-80*” and GOST 164830-89- “General requirements for physical-mechanical experiments”. (Appendix 2)

$\nu = 0.2$ - for shear strength edgewise (Paragraph 1.7 in GOST 164830-89)

$$\gamma_m = \frac{1 - \eta_n \cdot \nu}{1 - \eta \cdot \nu} = \frac{1 - 1.65 \cdot 0.2}{1 - 2.33 \cdot 0.2} = 1.25$$

From expressions 4.30 and 4.31 the value of the maximum design load can be extracted:

$$q_d = \frac{f_{v.0.edge.d} \cdot F_d \cdot 4}{3 \cdot l_d} \qquad (4.33)$$

Then, the characteristic value of the maximum load could be found from the formula 4.27:

$$q_k = \frac{5 \cdot q_d}{\left(1.15 + \frac{4}{0.7}\right)}$$

4.3.4 Stiffness calculation

The deflections of elements should be lower than are allowed in SNiP "Loads and Influences". The maximum deflection of the hinge-supported and cantilever bending elements of permanent and variable cross-sections should be defined with the following formula (SNiP II-25-80*- "Timber structures" 1983, p. 16):

$$f_{\max} = \frac{f_0}{k} \cdot \left(1 + c \cdot \left(\frac{h}{l_d}\right)^2\right) \qquad (4.34)$$

f_0 - deflection of permanent section beam with height "h" without accounting of shear deflection

k - variability of the height of the section, the value of which is "1" for permanent section

c - shear deflection factor

l_d - span length of an element

h - height of an element

$$f_0 = \frac{5}{384} \cdot \frac{q_k \cdot l_d^{\,4}}{m_b \cdot E_{0.mean} \cdot I} \cdot \gamma_n \qquad (4.35)$$

According to paragraphs 3.2.a, 3.5 in SNiP II-25-80*- "Timber structures", the mean value of the elastic modulus should be multiplied by an operation factor; and the deflection should be multiplied by a function factor.

Finally formula 4.34 transforms into:

$$f_{\max} = \left(\frac{5}{384} \cdot \frac{q_k \cdot l_d^{\,4}}{m_b \cdot E_{0.mean} \cdot I}\right) \cdot \left[1 + c \cdot \left(\frac{h}{l_d}\right)^2\right] \cdot \gamma_n \qquad (4.36)$$

$c = 19.2$ - accounting of the shear deflection. (Appendix 2)

The limit values of deflections according to STO 36554501-015-2008- “Loads and Influences” are evaluated through the following expressions:

- for span length $l \leq 1$ the final deflection should be lower than $l/120$
- for span length $l = 3$ the final deflection should be lower than $l/150$
- for span length $l = 6$ the final deflection should be lower than $l/200$
- for span length $l = 12$ the final deflection should be lower than $l/250$
- for span length $l = 24$ the final deflection should be lower than $l/300$

For the span length between [1; 3] the value of the relative deflection limit could be found from:

$$f_u / l = -\frac{1}{1200} \cdot l_d + \frac{11}{1200}$$

For the span length between [3; 6] the value of the relative deflection limit could be found from:

$$f_u / l = -\frac{1}{1800} \cdot l_d + \frac{1}{120}$$

For the span length between [6; 12] the value of the relative deflection limit could be found from:

$$f_u / l = -\frac{1}{6000} \cdot l_d + \frac{3}{500}$$

$$f_{\max} \leq f_u \qquad (4.37)$$

From expression 4.37, the characteristic value of the maximum load could be easily expressed.

4.3.5 Stability calculation

The stability calculation of structures with the effect of the bending moment should be evaluated with the following expression (STO 36554501-002-2006- “Glued and solid wood structures” 2006, p. 20):

$$\frac{M_{\max}}{\varphi_M \cdot W_d} \leq f_{m.0.edge.d} \qquad (4.38)$$

φ_M - factor for elements with the effect of the bending moment (STO 36554501-002-2006- “Glued and solid wood structures” 2006, p. 21)

For elements with the effect of the bending moment of a rectangular cross-section, hinge-supported out of the bending plane and fastened from rotation around the longitudinal axle, the φ_M factor should be defined from the following expression (STO 36554501-002-2006- “Glued and solid wood structures” 2006, p. 21):

$$\varphi_M = 140 \cdot \frac{b^2}{l_p \cdot h} \cdot k_\phi \qquad (4.39)$$

l_p - distance between the support-sections of an element or distance between the supported points of fastenings when the compression side is fixed. In this case, it is $l_1 = 400mm$

k_ϕ - factor that depends on the shape of the bending moment curve

k_ϕ - for parabolic shape value “1.13” (Appendix 2)

$$W_d = \frac{b \cdot h^2}{6}; \qquad M_{\max} = q_d \cdot \frac{l_d^{\,2}}{8} \qquad (4.40)$$

$$f_{m.0.edge.d} = f_{m.0.edge.k} \cdot \frac{m_{dl} \cdot m_b}{\gamma_m \cdot \gamma_n}, \qquad (4.41)$$

In expression 4.41 all factors have the same values as mentioned above for the bending strength. (Appendix 2)

From expressions 4.38 and 4.40 the value of maximum design load can be extracted:

$$q_d = \frac{8 \cdot f_{m.0.edge.d} \cdot \varphi_M \cdot W_d}{{l_d}^2} \quad (4.42)$$

Then, the characteristic value of the maximum load could be found from the formula 4.27:

$$q_k = \frac{5 \cdot q_d}{\left(1.15 + \frac{4}{0.7}\right)}$$

4.3.6 Local compression calculation

The value of mutilation force V should be lower than the design bearing capacity of the joint $T_{\max}$ ($T_{\max} > V$), calculated from the following expression (SNiP II-25-80*- "Timber structures" 1983, p. 17):

$$T_{\max} = f_{l.c.90.edge.d} \cdot F_w \quad (4.43)$$

$F_w = b \cdot S_1$	-	mutilation area of the joint
$S_1 = 120mm$	-	width of support
$f_{l.c.90.edge.d}$	-	mutilation bearing capacity of timber perpendicular to the grain

$$f_{l.c.90.edge.k} = f_{c.90.edge.k} \cdot \left(1 + \frac{8}{S_1/10 + 1.2}\right)$$

$$f_{l.c.90.edge.d} = f_{l.c.90.edge.k} \cdot \frac{m_{dl} \cdot m_b}{\gamma_m \cdot \gamma_n} \qquad (4.44)$$

In expression 4.44 all factors have the same values as mentioned above for the bending strength, except the material factor, the value of which is 1.25 for mutilation strength according to “Timber structures designing handbook for II-25-80*” and GOST 164830-89- “General requirements for physical-mechanical experiments”. (Appendix 2)

$\nu = 0.2$ - for local compressive strength (Paragraph 1.7 in GOST 164830-89)

$$\gamma_m = \frac{1-\eta_n \cdot \nu}{1-\eta \cdot \nu} = \frac{1-1.65 \cdot 0.2}{1-2.33 \cdot 0.2} = 1.25$$

$$V = \frac{q_d \cdot l_d}{2}$$ - mutilation force (4.45)

From expressions 4.45 and ($T_{\max} > V$) the value of maximum design load can be extracted:

$$q_d = \frac{T_{\max} \cdot 2}{l_d} \qquad (4.46)$$

Then, the characteristic value of the maximum load could be found from the formula (4.27):

$$q_k = \frac{5 \cdot q_d}{\left(1.15 + \frac{4}{0.7}\right)}$$

4.3.7 Selection of the minimum value of a characteristic load

The results of the calculation of the maximum characteristic load for each kind of estimation can be introduced in a table for different cross-sections and span lengths (percentage of dead load: 20%). An example of such table for a shear strength calculation is presented below:

Table 4.1 Maximum characteristic loads from shear strength calculation (kN/m)

	Cross-section (mm)							
Span (m)	51x200	45x260	45x300	51x300	45x360	51x400	57x450	75x500
2	10.16	11.65	13.44	15.24	16.13	20.32	25.55	37.35
2.5	8.13	9.32	10.76	12.19	12.91	16.25	20.44	29.88
3	6.77	7.77	8.96	10.16	10.76	13.54	17.03	24.9
3.5	5.8	6.66	7.68	8.71	9.22	11.61	14.6	21.34
4	5.08	5.83	6.72	7.62	8.07	10.16	12.77	18.67
4.5	4.51	5.18	5.97	6.77	7.17	9.03	11.35	16.6
5	4.06	4.66	5.38	6.09	6.45	8.13	10.22	14.94
5.5	3.69	4.24	4.9	5.54	5.87	7.39	9.29	13.58
6	3.39	3.88	4.48	5.08	5.38	6.77	8.52	12.45
6.5	-	-	-	-	-	-	7.86	11.49
7	-	-	-	-	-	-	7.3	10.67
7.5	-	-	-	-	-	-	6.81	9.96
8	-	-	-	-	-	-	6.39	9.34

Then, the minimum values of characteristic loads could be chosen from the tables for strength calculation, shear strength calculation (table 4.1), stiffness calculation, stability calculation, and local compression calculation, which have been done above.

4.4 Summary of the calculation part

Maximum span lengths for different cross-sections and steps of rafter beams are estimated from strength calculations. The results of other calculations fulfil the requirements of normative documents with span lengths taken from strength calculations. These values are given in table 4.2 below:

Table 4.2 Maximum span length (m)

Section (mm)	Step (m)	
	0.9	1.2
51x200	3.49	3.03
45x260	4.26	3.69
45x300	4.92	4.26
51x300	5.24	4.54
45x360	5.9	5.11
51x400	6.98	6.05
57x450	8.3	7.19
75x500	10.57	9.16

It is obvious that for a bigger step of the same cross-section value of the maximum span length is lower, because of the loading area. And there are no exceptions in the expansion of the maximum span lengths when cross-sections are growing. A bigger cross-section provides a larger span.

The maximum values of characteristic loads for main beams from different calculations are given in table 4.3 below:

Table 4.3 Final values of the maximum characteristic loads (kN/m)

Span (m)	Cross-section (mm)							
	51x200	45x260	45x300	51x300	45x360	51x400	57x450	75x500
2	10.16	11.65	13.44	15.24	16.13	20.32	24.03	31.61
2.5	7.52	9.32	10.76	12.19	12.91	16.25	19.22	25.29
3	5.22	7.77	8.96	10.16	10.76	13.54	16.02	21.07
3.5	3.84	5.72	6.78	8.63	9.22	11.61	13.73	18.06
4	2.94	4.38	5.2	6.61	8.07	10.07	12.01	15.81
4.5	2.32	3.46	4.1	5.22	6.63	7.96	10.68	14.05
5	1.88	2.8	3.32	4.23	5.37	6.45	9.61	12.65
5.5	1.39	2.32	2.74	3.49	4.44	5.33	8.37	11.5
6	1.01	1.95	2.31	2.94	3.73	4.48	7.03	10.54
6.5	-	-	-	-	-	-	5.99	9.73
7	-	-	-	-	-	-	5.16	8.81
7.5	-	-	-	-	-	-	4.5	7.68
8	-	-	-	-	-	-	3.95	6.75

The trend is that for small spans (2÷3.5m) the values of maximum characteristic loads are taken from the shear strength calculation for this table. And values are taken mostly from the strength calculation for last three spans. The only problem was noticed in the final results that for the 45x 360 cross-section values are lower than for the previous cross-section. It is so because they were taken from the stability calculation and the 51x300 cross-section is thicker than 45x360. In the final table 4.3 these values are changed for the next bigger results from the strength calculation in order to draw a fine diagram.

Diagrams for the maximum span length of the Kerto-S rafter beam and maximum characteristic load for the Kerto-S main beam are shown below in charts 4.1.1, 4.1.2 and accordingly 4.2.1, 4.2.2 (in the names of the charts RN means Russian norms and EN means European norms):

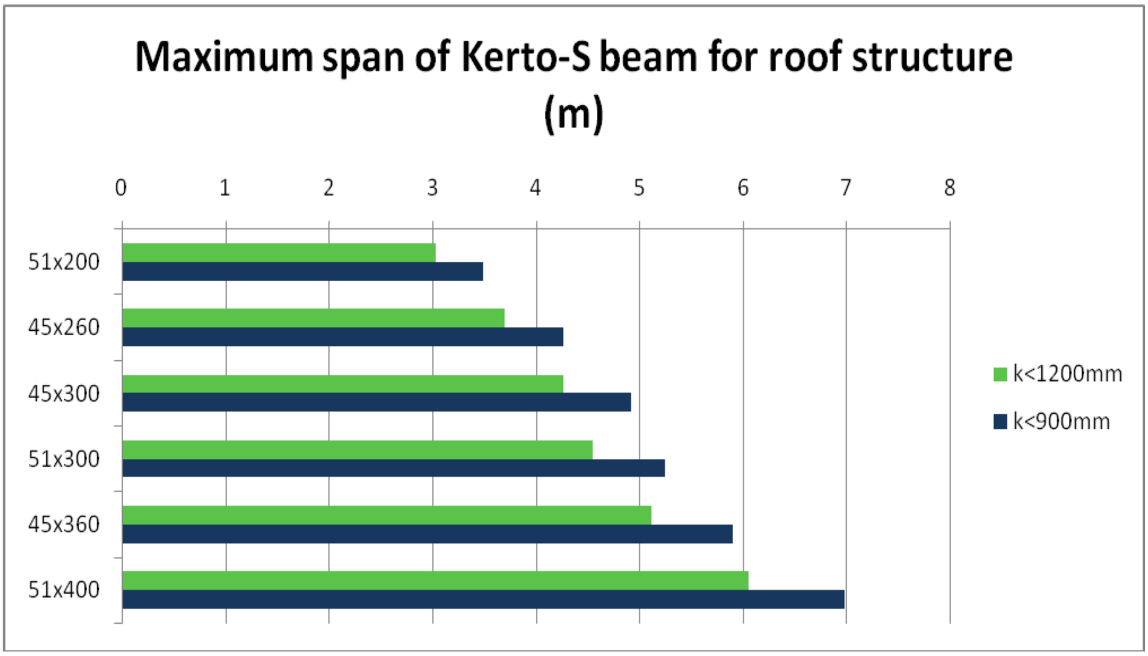


Chart 4.1.1 Maximum span lengths of the rafter beam (common sections), RN

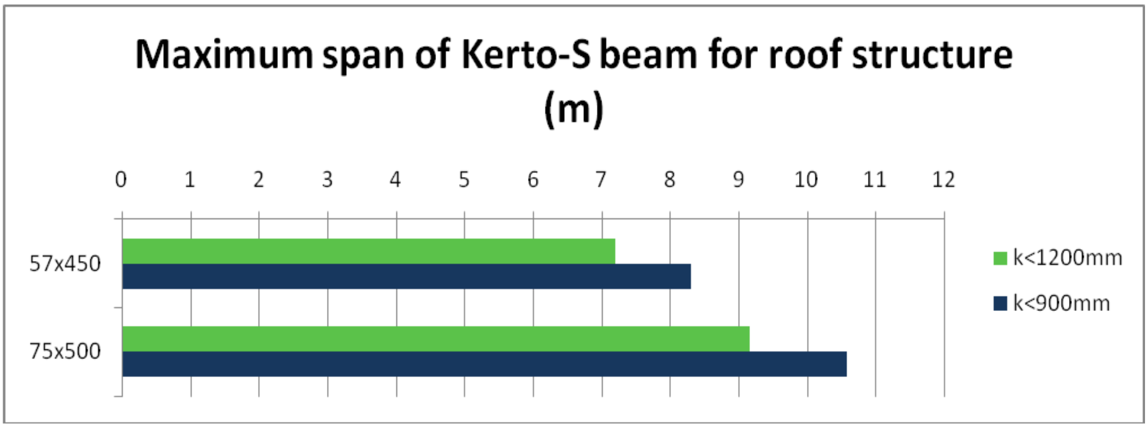


Chart 4.1.2 Maximum span lengths of the rafter beam (additional sections), RN

These two diagrams above are illustrations of table 4.2 of the maximum span length of the Kerto-S rafter beams. The values for “x” axle are taken directly from that table.

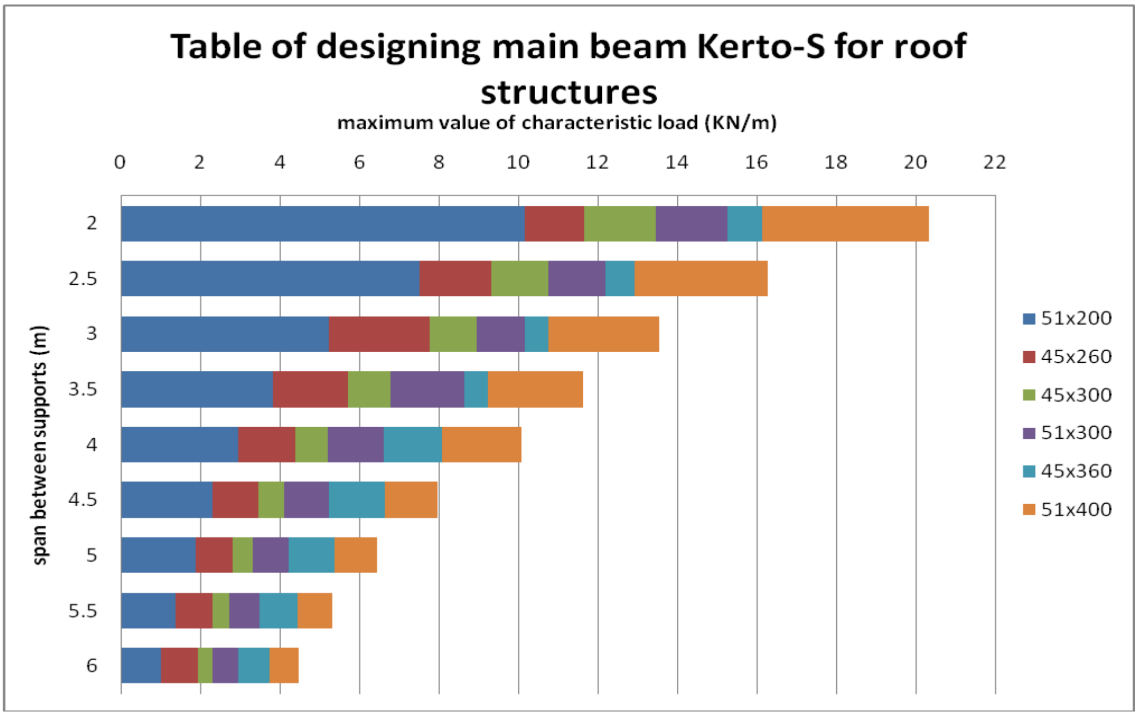


Chart 4.2.1 Maximum values of the characteristic load for the main beam (common sections), RN

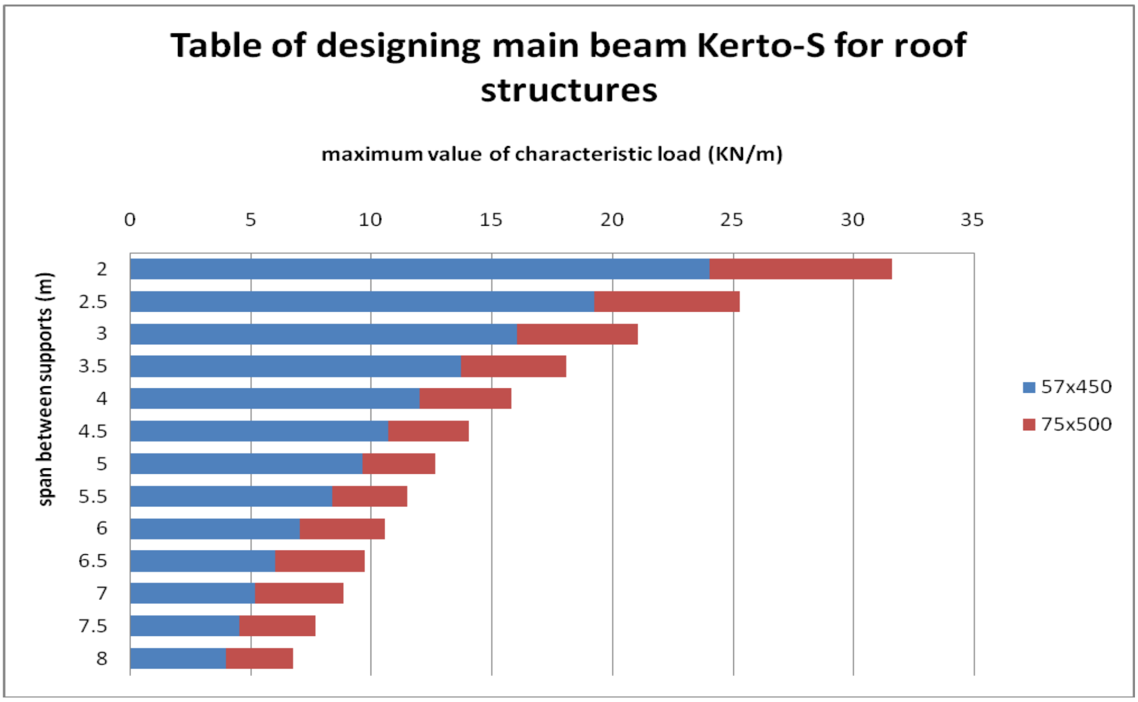


Chart 4.2.2 Maximum values of the characteristic load for the main beam (additional sections), RN

These two diagrams above are illustrations of table 4.3 of maximum characteristic loads of the Kerto-S main beams. The values for “x” axle are taken directly from that table.

5 RESULTS OF CALCULATIONS ACCORDING TO EUROCODE

The designing and calculating of Kerto-S beams for roof structures were also implemented in accordance with the Eurocode 5 and Finnwood 2.3 software. This computation was done only for the maximum span length of the rafters. Results for maximum characteristic loads were taken from Finnforest's brochure. Results of calculations are shown in tables and diagrams in this chapter.

5.1 Maximum span length calculation

An example of a structural analysis in the Finnwood 2.3 program for the rafter beam of one cross-section (51x200mm) is given below in figure 5.1:

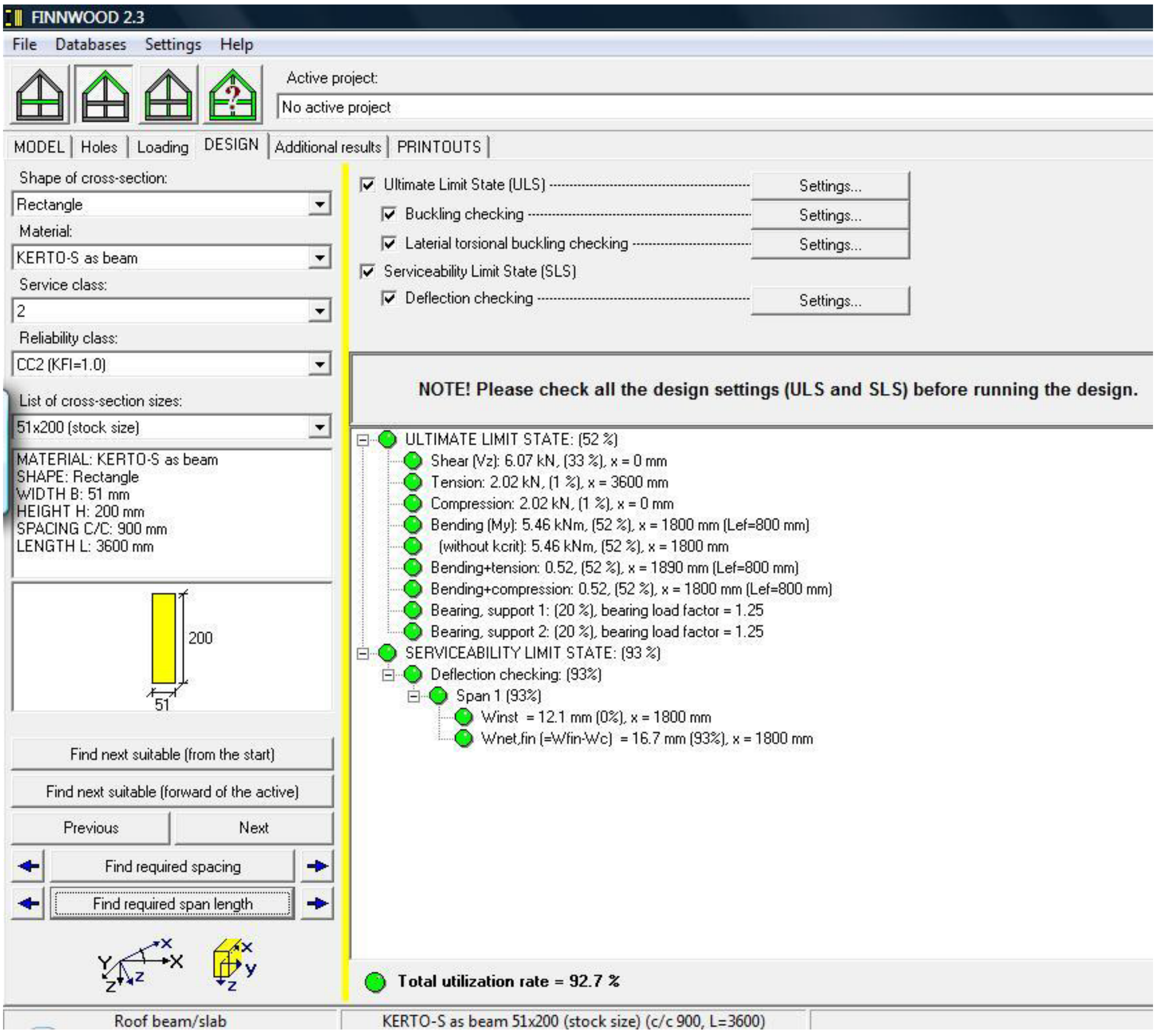


Figure 5.1 Results from Finnwood 2.3 program for 51x200 section

The results for the maximum span length of Kerto-S rafter beams were obtained from the Finnwood 2.3 program for different cross-sections and steps as shown, for example, in figure 5.1. Initial data for these calculations are as follows: width of support 122mm; roof slop i=1:3; self weight g_k=0.9kN/m^2; snow load on the ground S_k=2.5kN/m^2; distance between upper supports for providing stability <400mm; final deflection < L/200; span between rafters k_1=900mm, k_2=1200mm; service class 2.

The results are given below in table 5.1 and charts 5.1.1; 5.1.2:

Table 5.1 Maximum span length (m)

	Step (m)	
Section (mm)	0.9	1.2
51x200	3.42	3.13
45x260	4.27	3.89
45x300	4.93	4.55
51x300	5.22	4.74
45x360	5.98	5.41
51x400	6.93	6.26
57x450	8.06	7.3
75x500	9.77	8.92

It is necessary to make diagrams 5.1.1 and 5.1.2 from the values of table 5.1 in order to compare the results with the diagrams from the company's brochure.

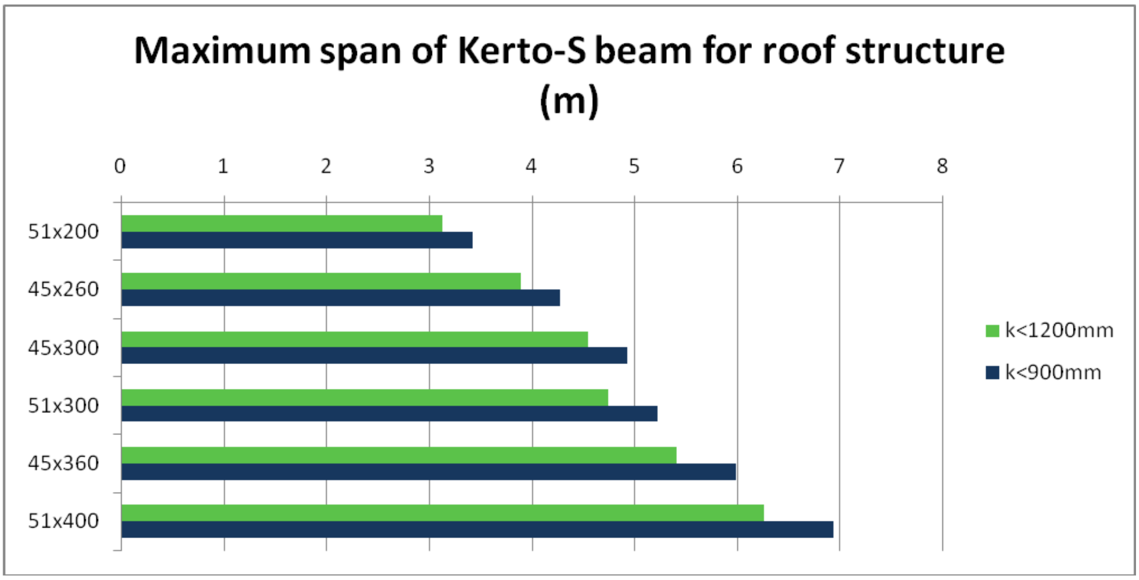


Chart 5.1.1 Maximum span lengths of the rafter beam (common sections), EN

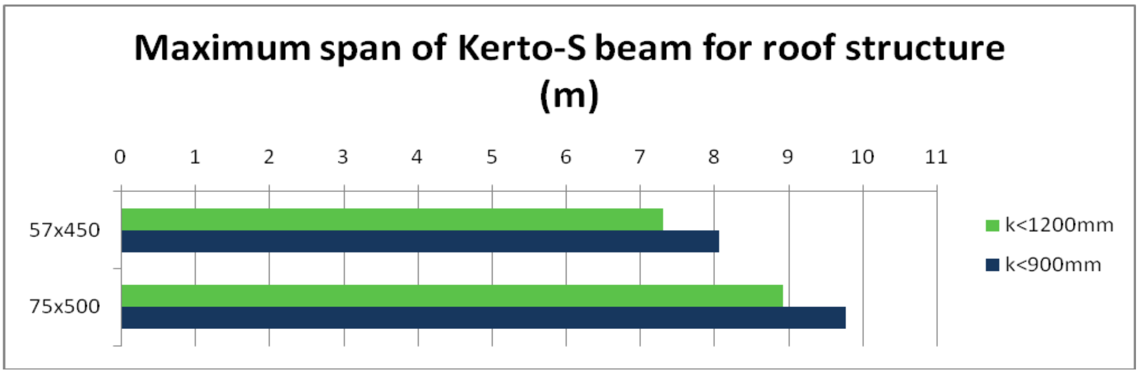


Chart 5.1.2 Maximum span lengths of the rafter beam (additional sections), EN

The values in these diagrams came out the same as in Finnforest's brochure. So, it is possible to make a conclusion that calculations have been made correctly by the Finnwood 2.3 program.

5.2 Maximum characteristic load calculation

The results for maximum characteristic loads of Kerto-S main beams were obtained from Finnforest's brochure for different cross-sections and spans. Initial data for these calculation are: width of support >120mm; percentage of self-weight 20%; service classes 1-2; wind loads are not taken into account in calculations; final deflection < L/300.

The results are given below in charts 5.2.1; 5.2.2:

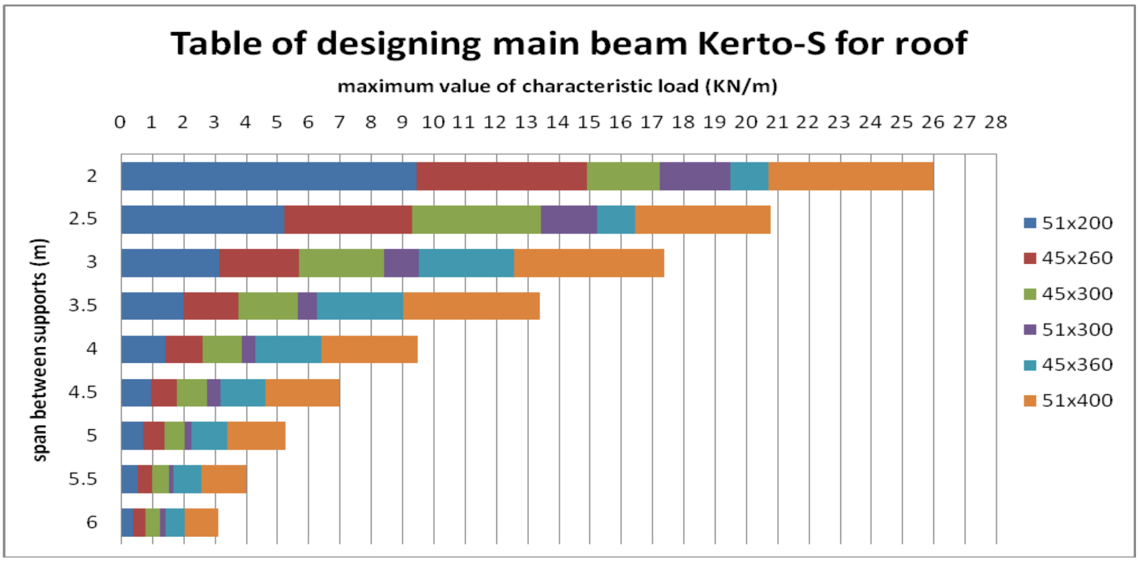


Chart 5.2.1 Maximum values of the characteristic load for the main beam (common sections), EN

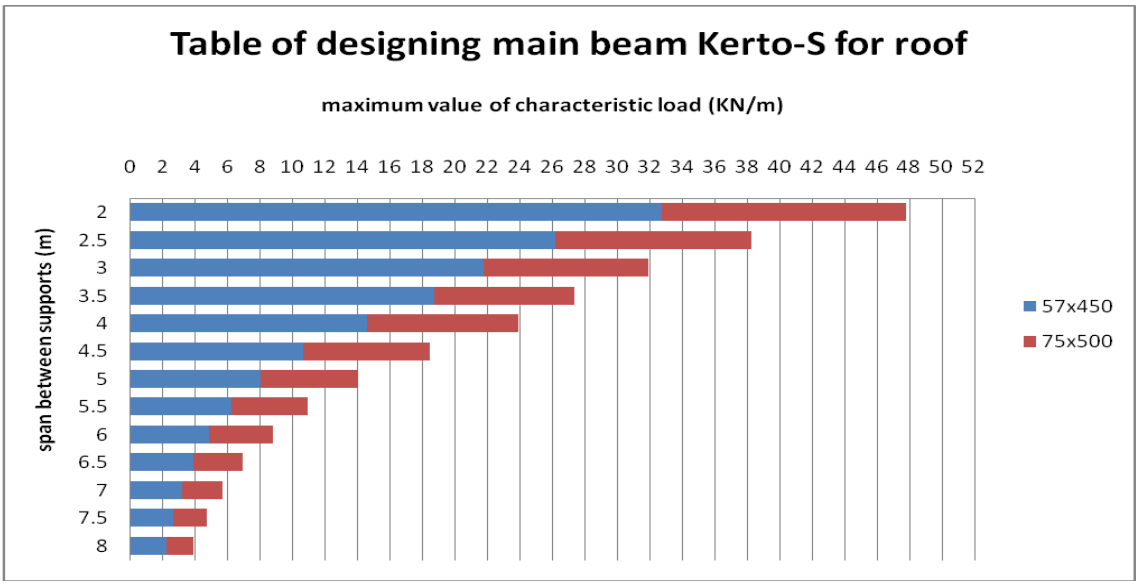


Chart 5.2.2 Maximum values of the characteristic load for the main beam (additional sections), EN

The values for these diagrams 5.2.1, 5.2.2 are evaluated from the brochure. And then the diagrams are drawn with the help of MS Excel. As a result, these diagrams are an exact copy of the ones in the brochure. They are needed in order to compare the results with the same diagrams which are made according to Russian norms.

6 CONCLUSIONS

Firstly, there are some consequences of the maximum span length of the Kerto-S rafter beam for two different calculations. The calculations for both variants were done with the same initial data, which is taken from the brochure, except the values of deflections. As a matter of fact, the differences between results of maximum span length for rafter beams made according to Russian norms and European norms are very low. And that is shown in the table 6.1 below.

Table 6.1 shows the balance between the results of maximum span lengths according to Russian norms and Eurocode:

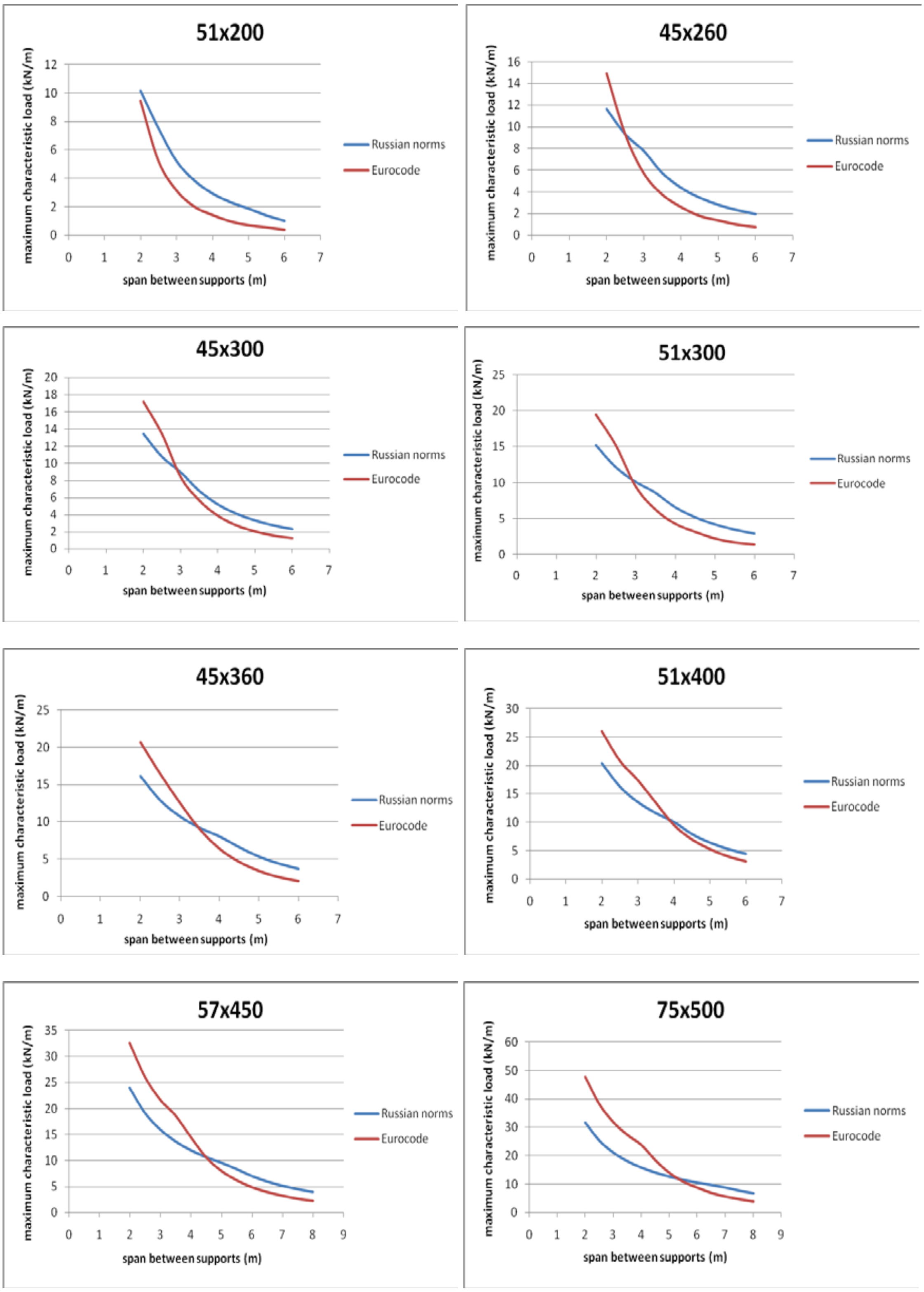
Table 6.1 Balance of results (m)

Section (mm)	Step (m)	
	0.9	1.2
51x200	-0.07	0.1
45x260	0.01	0.2
45x300	0.01	0.29
51x300	-0.02	0.2
45x360	0.08	0.3
51x400	-0.05	0.21
57x450	-0.24	0.11
75x500	-0.8	-0.24

The range of values varies from 1 cm to 30 cm. The exception is only the 75x500 section, which is 80 cm. Anyway, almost all values are similar. This fact means that all factors, coefficients, formulas for both methods finally gave the same reliability level for structures. It is interesting to notice, that in spite of differences between almost all values and ways of analyses in the two methods of calculation, they gave the equal results of the strength reserve.

General ideas about the values of the maximum characteristic loads for the Kerto-S main beams in two different calculations are shown here, too. The calculations for both variants were done with the same initial data, which is taken from the brochure, except the values of deflections.

There are diagrams below, that show the differences between maximum characteristic loads depend on the span length for both calculations for various cross-sections (charts 6.1):



Charts 6.1 Maximum characteristic loads

These diagrams were made from charts 4.4.1, 4.4.2 and 5.3.1, 5.3.2 with the help of MS Excel. In these diagrams it is shown that for small spans the values of maximum characteristic loads from Russian norms are lower than from Eurocode and for large spans they are higher. It is difficult to exlpain these differences in a physical-mechanical case, because the values of final results for the Russian variant were taken from various types of calculations.

But, it is noticed that almost all values for small (range between 2m and 3.5m) spans are taken from shear strength calculations, while the values for large spans are taken from strength and partly from stability calculations. All these values from different calculations vary quite a lot from each other. And this is another explanation of the large gaps in the diagrams.

The last observation about these differences is that a percentage ratio between permanent and live loads has been given (20% of self-weight). There are different safety factors for loads in the two methods of calculation. Probably these factors caused the differences.

To sum up, it could be said that the obtained results could be applied only to orientation and information purposes such as estimating the approximate amount of wooden product for the cost analyses of the customer's structures. And before using them, it is needed that a qualifed person checks the evaluation sequence and methods of calculations. But for real engineering it is necessary to explore and analyse these tasks properly and make investigations and experiments of different structures. There are only standards and norms for LVL Ultralam material in Russia without the characteristic values of strength properties for common LVL material. From my point of view, it would be correct to create standards and technical requirements for calculating structures using LVL Kerto material. And obviously they should meet the requirements of Russian normative documents. It would be also quicker and simplier to do such calculations with the help of a designing computer program, e.g. Finnwood program, which could calculate according to Russian norms, because it takes much time to do it by hand.

FIGURES

CHARTS

TABLES

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Mathcad calculation of the rafter beam

Common provisions about the structure

The present calculations are set for the sloped roof elements (rafter beams) of the residential building and made according to rules and regulations of the following Russian normative documents:

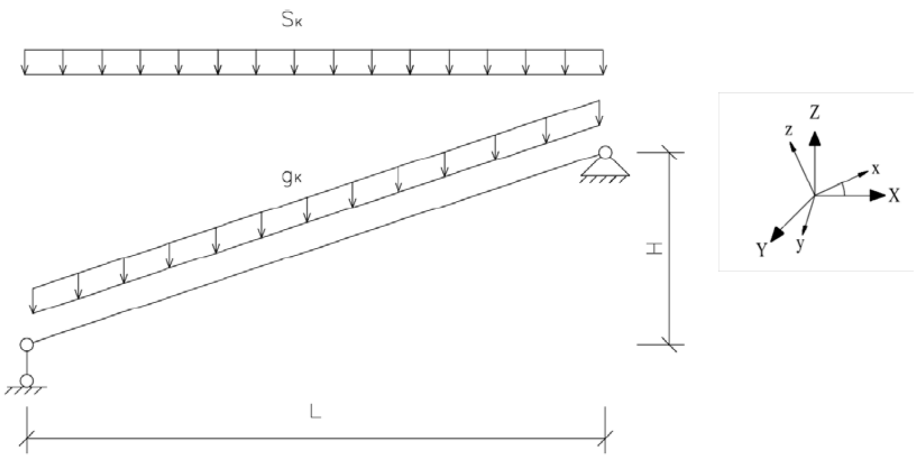
- STO 36554501-015-2008 "Loads and Influences" [1]
- STO 36554501-002-2006 "Wooden laminated and solid timber structures" [2]
- SNiP 2.01.07-85* "Loads and Influences" [3]
- SNiP II-25-80 "Timber structures" [4]

It is deemed that standard cross-sections of the Finnforest's LVL Kerto-S beams are used in the design. The minimum height of the cross-section should not be less than 500 mm for fulfilling the requirements of the heat insulation of the building. C2 service class is used.

Notes! These calculations are made for slope angle α<25 degrees. For angles α>11 degrees c_e factor should be taken equal to "1", when determining the characteristic value of the snow load. Deflection limits are set only for span lengths in the range from 3 to 6 meters.

1. General sizes of the structural member

$L_x := 3490mm$	Horizontal dimension of the member (span length)
$\alpha := 18.4deg$	Angle of the member about the X-axle
$b := 51mm$	Width of the member
$h := 200mm$	Height of the member
$S_1 := 122mm$	Support length of the member
$k := 900mm$	c/c spacing between the rafters
$L_{k1} := 400mm$	Distance between the fastenings (battens)



Picture 1. Principal scheme of the member

2. Strength properties of the material

$f_{c.0.d} := 20.44 \frac{N}{mm^2}$	Design compressive strength along the grain
$f_{c.90.d} := 3.17 \frac{N}{mm^2}$	Design compressive strength perpendicular to the grain
$f_{m.d} := 25 \frac{N}{mm^2}$	Design bending strength
$f_{v.d} := 2.16 \frac{N}{mm^2}$	Design shear strength, edgewise
$\rho_{mean} := 5.1 \frac{kN}{m^3}$	Mean density of the Kerto-S material
$E_{0.mean} := 13800 \frac{N}{mm^2}$	Mean modulus of elasticity of the Kerto-S material

3. Collection of the loads

Characteristic value of the self-weight of the roof structure:

$$g_k := 0.9 \frac{kN}{m^2}$$

Design value of the self-weight of the roof structure:

$\gamma_f := 1.15$ - load safety factor (see table 2.1 in [1] or table 1 in [3])

$$g_d := g_k \cdot \gamma_f = 1.035 \frac{kN}{m^2}$$

Snow load:

$c_e := 1$ - factor for demolition of the snow from the roof surface (see paragraph 5.5 in [1] or in [3])

$c_t := 1$ - thermal factor (see paragraph 5.6 in [1] or in [3])

$\mu := 1$ - transfer factor (see table Б.1 of app. Б in [1] or table 1 of app. 3 in [3])

$S_d := 3.571 \frac{kN}{m^2}$ - design value of the snow load for the fourth snow region (see table 5.1 in [1] or table 4 in [3])

$S_k := 0.7 \cdot c_e \cdot c_t \cdot \mu \cdot S_d = 2.5 \cdot \frac{kN}{m^2}$ - characteristic value of the snow load

Wind load:

For the low-pitched roofs (α<20 deg.) wind load is not taken into account according to STO "Loads and Influences" or it creates negative pressure according to SNiP, that can be considered as a strength reserve.

Characteristic value of the whole line load:

$$q_k := \left(g_k \cdot \cos(\alpha) + S_k \cdot \cos(\alpha)^2\right) \cdot k = 2.794 \cdot \frac{kN}{m}$$

Design value of the whole line load:

$$q_d := \left(g_d \cdot \cos(\alpha) + S_d \cdot \cos(\alpha)^2\right) \cdot k = 3.778 \cdot \frac{kN}{m}$$

4. Checking of the member's strength with simultaneous action of the bending and compression

$$\frac{N}{F_d} + \frac{M_{max}}{\zeta \cdot W_d} \le f_{c.0.d}$$ - formula for checking of the member's strength (see formula 28 in [2] or in [4])

$$N := q_d \cdot \frac{L_x \cdot \tan(\alpha)}{2 \cdot \cos(\alpha)} = 2.311 \cdot kN$$ - longitudinal compressive force

$$M_{max} := q_d \cdot \frac{L_x^{\,2}}{8 \cdot \cos(\alpha)^2} = 6.388 \cdot kN \cdot m$$ - bending moment in the middle cross-section

$$F_d := h \cdot b = 0.01\, m^2$$ - cross-section area

$$W_d := \frac{b \cdot h^2}{6} = 3.4 \times 10^{-4} \cdot m^3$$ - section modulus about y-axle

$$I := \frac{b \cdot h^3}{12} = 3.4 \times 10^{-5}\, m^4$$ - moment of area about y-axle

$$r := \sqrt{\frac{I}{F_d}} = 0.058\, m$$ - radius of inertia

$$\mu_{0.} := 1$$ - design length factor (see paragraph 4.21 in [2] or in [4])

$$A := 2500$$ - factor for LVL material (see paragraph 4.3 in [2] or in [4])

$$\lambda := \frac{L_x}{\cos(\alpha)} \cdot \frac{\mu_{0.}}{r} = 63.705$$ - flexibility of the member (see formulas 9,10 in [2] or in [4])

$$\phi := \frac{A}{\lambda^2} = 0.616$$ - buckling factor (see formula 8 in [2] or in [4])

$$m_b := 0.9$$ - operation factor, that depends on environmental conditions (see table 5 in [2] or in [4])

$$\gamma_n := 0.95$$ - reliability factor (see table 9a in [2] or app 7 in [3])

$$\zeta := \left(1 - \frac{N}{\phi \cdot f_{c.0.d} \cdot \frac{m_b}{\gamma_n} \cdot F_d}\right) = 0.981$$ - additional bending moment factor from deflection by reason of the longitudinal force (see formula 30 in [2] or in [4])

$$\frac{\dfrac{N}{F_d}+\dfrac{M_{max}}{\zeta\cdot W_d}}{f_{c.0.d}\cdot\dfrac{m_b}{\gamma_n}} = 100.072\cdot\%$$

5. Checking of the shear strength

$$\frac{V\cdot S_b}{I_b\cdot b}\le f_{v.d}$$ - formula for checking of the shear strength (see formula 18 in [2] or in [4])

$$V := q_d\cdot\frac{L_x}{2\cdot\cos(\alpha)} = 6.947\cdot kN$$ - component of the support reaction, that is perpendicular to the member's neutral axle

$$S_b := \frac{F_d}{2}\cdot\frac{h}{4}$$ - static moment of the sliding part of the cross-section about neutral axle

formula for checking of the shear strength converts into the following:

$$\frac{3\cdot V}{2\cdot F_d\cdot\left(f_{v.d}\cdot\dfrac{m_b}{\gamma_n}\right)} = 49.925\cdot\%$$

6. Checking of the member's stability

$$\frac{N}{\phi_1\cdot f_{c.0.d}\cdot F_d}+\left(\frac{M_{max}}{\zeta\cdot\phi_M\cdot f_{m.d}\cdot W_d}\right)^n\le 1$$ - formula for checking of the member's stability with acting of bending moment and compressive force (see formula 33 in [2] or in [4])

$k_\phi := 1.13$ - factor, that depends on the shape of the bending moment curve (see table Γ.2 of app Γ in [2] or table 2 of app 4 in [4])

$$\phi_M := 140\cdot\frac{b^2}{L_{k1}\cdot h}\cdot k_\phi = 5.143$$ - factor for elements with acting of the bending moment (see formula 23 in [2] or in [4])

$$\lambda_1 := \frac{L_{k1}}{r} = 6.928$$ - flexibility of the fragment of the member

$$\phi_1 := \frac{A}{{\lambda_1}^2} = 52.083$$ - buckling factor

$n := 1$ - value of this component is "1" for elements with fastenings in the tensile side out of the bending plane (see paragraph 4.18 in [2] or in [4])

$$k_{nM} := \left(0.142\cdot\frac{L_{k1}}{h}+1.76\cdot\frac{h}{L_{k1}}\right) = 1.164$$ - factor for $\phi_{.M}$ for taking into account fastenings in the tensile side of the member (see formula 24 in [2] or in [4])

$$k_{nN} := 0.75 + 0.06\cdot\left(\frac{L_{k1}}{h}\right)^2 = 0.99$$

- factor for $\phi_{.1}$ for taking into account fastenings in the tensile side of the member (see formula 34 in [2] or in [4])

$$\boxed{\frac{N}{\phi_1\cdot k_{nN}\cdot f_{c.0.d}\cdot\frac{m_b}{\gamma_n}\cdot b\cdot h} + \left(\frac{M_{max}}{\zeta\cdot\phi_M\cdot k_{nM}\cdot f_{m.d}\cdot\frac{m_b}{\gamma_n}\cdot W_d}\right)^n = 0.135}$$

should be less than "1"

Note! The present stability calculation of the member is made with taking into account fastenings in the tensile side of the member.

7. Checking of the member's stiffness

$$f_{max} \le f_{lim.}$$

- formula for checking of the member's stiffness

$$S_{k.red} := S_k\cdot 0.7 = 1.75\cdot\frac{kN}{m^2}$$

- reduced characteristic value of the snow load (see paragraph 5.7 in [1])

$$q_{k.red} := \left(g_k\cdot\cos(\alpha) + S_{k.red}\cdot\cos(\alpha)^2\right)\cdot k = 2.186\cdot\frac{kN}{m}$$

- reduced characteristic value of the whole line load

$$l_{k.red} := \frac{L_x}{\cos(\alpha)} - 125mm = 3.553\,m$$

- distance between the internal surfaces of the walls

$$m_d := 0.8$$

- load duration factor (see paragraph 3.2.в in [2] or in [4])

$$c := 19.2$$

- shear deflection factor (see table Г.3 of app Г in [2] or table 3 of app 4 in [4])

$$f_1 := \left(\frac{5}{384}\cdot\frac{q_{k.red}\cdot l_{k.red}^{\;4}}{m_b\cdot m_d\cdot E_{0.mean}\cdot I}\right)\cdot\left[1 + c\cdot\left(\frac{h}{l_{k.red}}\right)^2\right]\cdot\frac{\gamma_n}{l_{k.red}\cdot\zeta} = 3.883\times 10^{-3}$$

- see formulas 50, 51 in [2] or in [4]

$$\begin{pmatrix} a\cdot 6 + c - \frac{1}{200} = 0 \\ a\cdot 3 + c - \frac{1}{150} = 0 \end{pmatrix} \text{solve}, a, c \rightarrow \left(-\frac{1}{1800} \quad \frac{1}{120}\right)$$

- see paragraph 2a in table Г.1 of app Г in [1] or in table 19 in [3]

$$l_{k.red.} := 3.553$$

- distance between the internal surfaces of the walls (from 6 to 12 meters)

$$f_2 := \frac{-1}{1800}\cdot l_{k.red.} + \frac{1}{120} = 6.359\times 10^{-3}$$

- formula for interpolation of the limit values of deflections

$$\boxed{f_{max} := f_1\cdot l_{k.red} = 0.014\,m}$$

should be less than:

$$\boxed{f_{lim.} := f_2\cdot l_{k.red} = 0.023\,m}$$

$$\boxed{\frac{f_{max}}{f_{lim.}} = 61.063\cdot\%}$$

8. Checking of the local compression on the support (bearing)

$V_{max} \le T_{max.1}$ - formula for checking of the bearing strength of the member (see paragraph 5.1 in [2] or in [4])

$V_{max} := \dfrac{V}{\cos(\alpha)} = 7.321 \cdot kN$ - vertical support reaction of the member

$f_{c.\alpha.d} := \dfrac{f_{c.0.d} \cdot m_b}{1 + \left(\dfrac{f_{c.0.d}}{f_{c.90.d}} - 1\right) \cdot \sin(90deg - \alpha)^3} = 3.253 \times 10^3 \cdot \dfrac{kN}{m^2}$ - design bearing capacity of the material at the angle α to the grain direction (see formula 2 in [2] or in [4])

$T_{max.1} := \dfrac{f_{c.\alpha.d}}{\gamma_n} \cdot b \cdot S_1 = 21.308 \cdot kN$ - bearing capacity of the joint on the support (see formula 56 in [2] or 52 in [4])

$\boxed{V_{max} = 7.321 \cdot kN}$ should be less than: $\boxed{T_{max.1} = 21.308 \cdot kN}$

$\boxed{\dfrac{V_{max}}{T_{max.1}} = 34.359 \cdot \%}$

Mathcad calculation of the main beam

General provisions about the structure

The present calculations are set for the horizontal roof elements (main beams) of the residential building and made according to rules and regulations of the following Russian normative documents:

- STO 36554501-015-2008 "Loads and Influences" [1]
- STO 36554501-002-2006 "Wooden laminated and solid timber structures" [2]
- SNiP 2.01.07-85* "Loads and Influences" [3]
- SNiP II-25-80 "Timber structures" [4]

It is deemed that standard cross-sections of the Finnforest's LVL Kerto-S beams are used in the design. C2 service class is used.

Notes! For roof angles α>11 degrees c_e factor should be taken equal to "1", when determining the characteristic value of the snow load. Deflection limits are set only for span lengths in the range from 1 to 3 meters. Load-bearing structures are visible from inside. If they are invisible, stiffness calculations could not be taken into account.

1. General sizes of the structural member

$L_x := 2000mm$	Horizontal dimension of the member (span length)
$\alpha := 18.4deg$	Angle of the roof
$b := 51mm$	Width of the member
$h := 200mm$	Height of the member
$S_1 := 122mm$	Support length of the member
$k := 1000mm$	Width of the loading area
$L_{k1} := 900mm$	Distance between the fastenings (rafters)

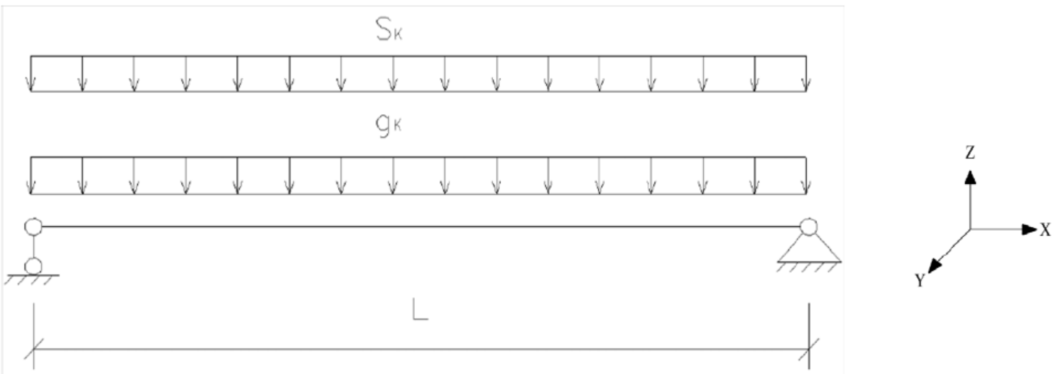


Figure 1. Principal scheme of the member

2. Strength properties of the material

$f_{m.d} := 25 \frac{N}{mm^2}$ Design bending strength

$f_{c.90.d} := 3.17 \frac{N}{mm^2}$ Design compressive strength perpendicular to the grain

$f_{v.d} := 2.16 \frac{N}{mm^2}$ Design shear strength

$\rho_{mean} := 5.1 \frac{kN}{m^3}$ Mean density of the LVL material

$E_{0.mean} := 13800 \frac{N}{mm^2}$ Mean modulus of elasticity of the LVL material

3. Collection of the loads

Characteristic value of the whole line load:

$q_k := 10.16 \frac{kN}{m}$

Design value of the whole line load:

$$q_d := \frac{q_k \cdot \left(1.15 + \frac{4}{0.7}\right)}{5} = 13.948 \frac{kN}{m}$$

4. Checking of the member's strength with simultaneous action of the bending and compression

$\frac{M_{max}}{W_d} \le f_{m.d}$ - formula for checking of the member's strength (see formula 17 in [2] or in [4])

$M_{max} := q_d \cdot \frac{L_x^2}{8} = 6.974 \cdot kN \cdot m$ - bending moment in the middle cross-section

$W_d := \frac{b \cdot h^2}{6} = 3.4 \times 10^{-4} \cdot m^3$ - section modulus about y-axle

$m_b := 0.9$ - operation factor, that depends on environmental conditions (see table 5 in [2] or in [4])

$m_h := 1$ - factor, that takes into account height of the beam (see paragraph 3.2д and table 7 in [2] or in [4])

$\gamma_n := 0.95$ - reliability factor (see table 9a in [2] or app 7 in [3])

$$\boxed{\frac{\frac{M_{max}}{W_d}}{f_{m.d}\cdot\frac{m_b\cdot m_h}{\gamma_n}} = 86.607\cdot\%}$$

5. Checking of the shear strength

$$\frac{V\cdot S_b}{I_b\cdot b} \le f_{v.d}$$ - formula for checking of the shear strength (see formula 18 in [2] or in [4])

$$F_d := h\cdot b = 0.01\,m^2$$ - cross-section area

$$V := q_d\cdot\frac{L_x}{2} = 13.948\cdot kN$$ - support reaction

$$S_b := \frac{F_d}{2}\cdot\frac{h}{4}$$ - static moment of the sliding part of the cross-section about neutral axle

formula for checking of the shear strength converts into the following:

$$\boxed{\frac{3\cdot V}{2\cdot F_d\cdot\left(f_{v.d}\cdot\frac{m_b}{\gamma_n}\right)} = 100.239\cdot\%}$$

6. Checking of the member's stability

$$\frac{M_{max}}{\phi_M\cdot W_d} \le f_{m.d}$$ - formula for checking of the member's stability with acting of bending moment (see formula 22 in [2] or in [4])

$$k_\phi := 1.13$$ - factor, that depends on the shape of the bending moment curve (see table Г.2 of app Г in [2] or table 2 of app 4 in [4])

$$\phi_M := 140\cdot\frac{b^2}{L_{k1}\cdot h}\cdot k_\phi = 2.286$$ - factor for elements with acting of the bending moment (see formula 23 in [2] or in [4])

$$k_{nM} := \left(0.142\cdot\frac{L_{k1}}{h} + 1.76\cdot\frac{h}{L_{k1}}\right) = 1.03$$ - factor for $\phi_{\cdot M}$ for taking into account fastenings in the tensile side of the member (see formula 24 in [2] or in [4])

$$\boxed{\frac{\frac{M_{max}}{\phi_M\cdot W_d}}{f_{m.d}\cdot\frac{m_b\cdot m_h}{\gamma_n}} = 37.886\cdot\%}$$

Note! The present stability calculation of the member is made with taking into account fastenings in the tensile side of the member.

7. Checking of the member's stiffness

$f_{max} \le f_{lim.}$ - formula for checking of the member's stiffness

$S_k := 2.5 \frac{kN}{m^2}$ - characteristic value of the snow load

$S_{k.red} := S_k \cdot 0.7 = 1.75 \cdot \frac{kN}{m^2}$ - reduced characteristic value of the snow load (see paragraph 5.7 in [1])

$g_k := \frac{q_k}{1m} = 10.16 \frac{kN}{m^2}$

$q_{k.red} := (g_k + S_{k.red}) \cdot \frac{k}{\cos(\alpha)} = 12.552 \cdot \frac{kN}{m}$ - reduced characteristic value of the whole line load

$I := \frac{b \cdot h^3}{12} = 3.4 \times 10^{-5} m^4$ - moment of area about y-axle

$l_{k.red} := L_x - S_1 = 1.878 m$ - distance between the internal surfaces of the walls

$m_d := 0.8$ - load duration factor (see paragraph 3.2.в in [2] or in [4])

$c := 19.2$ - shear deflection factor (see table Г.3 of app Г in [2] or table 3 of app 4 in [4])

$f_1 := \left(\frac{5}{384} \cdot \frac{q_{k.red} \cdot l_{k.red}^4}{m_b \cdot m_d \cdot E_{0.mean} \cdot I}\right) \cdot \left[1 + c \cdot \left(\frac{h}{l_{k.red}}\right)^2\right] \cdot \frac{\gamma_n}{l_{k.red}} = 3.707 \times 10^{-3}$ - see formulas 50, 51 in [2] or in [4]

$\begin{pmatrix} a \cdot 3 + c - \frac{1}{150} = 0 \\ a \cdot 1 + c - \frac{1}{120} = 0 \end{pmatrix} \text{solve}, a, c \rightarrow \begin{pmatrix} -\frac{1}{1200} & \frac{11}{1200} \end{pmatrix}$ - see paragraph 2a in table Г.1 of app Г in [1] or in table 19 in [3]

$l_{k.red.} := 1.878$ - distance between the internal surfaces of the walls (from 1 to 3 meters)

$f_2 := \frac{-1}{1200} \cdot l_{k.red.} + \frac{11}{1200} = 7.602 \times 10^{-3}$ - formula for interpolation of the limit values of deflections

$\boxed{f_{max} := f_1 \cdot l_{k.red} = 6.962 \times 10^{-3} m}$ should be less than: $\boxed{f_{lim.} := f_2 \cdot l_{k.red} = 0.014 m}$

$\boxed{\frac{f_{max}}{f_{lim.}} = 48.765 \cdot \%}$

8. Checking of the local compression on the support (bearing)

$V_{max} \le T_{max.1}$ - formula for checking of the bearing strength of the member (see paragraph 5.1 in [2] or in [4])

$V_{max} := V = 13.948 \cdot kN$ - vertical support reaction of the member

$T_{max.1} := \frac{f_{c.90.d} \cdot m_b}{\gamma_n} \cdot b \cdot S_1 = 18.686 \cdot kN$ - bearing capacity of the joint on the support (see formula 56 in [2] or 52 in [4])

$\boxed{V_{max} = 13.948 \cdot kN}$ should be less than: $\boxed{T_{max.1} = 18.686 \cdot kN}$

$\boxed{\frac{V_{max}}{T_{max.1}} = 74.647 \cdot \%}$

Tables of factors

All values in tables below are taken from Russian building norms and regulations SNiP II-25-80*- “Timber structures”; SNiP 2.01.07-85*- “Loads and Influences” and Standards STO 36554501-002-2006- “Glued and solid wood structures”; STO 36554501-015-2008- “Loads and Influences”.

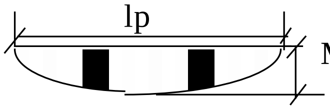
Types of structures and soils	Service factor γ_f
Steel	1,05
Concrete (with general density more 1600 kg/m^3), reinforced concrete, stone, timber	1,1
Concrete (with general density 1600 kg/m^3 and less), insulating, finishing layers (slabs, rolled materials, cement covering), fabricating prefabricated	1,2
The same on a building site	1,3
Natural soil	1,1
Filled-up soil	1,15

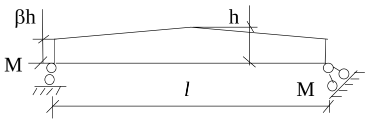
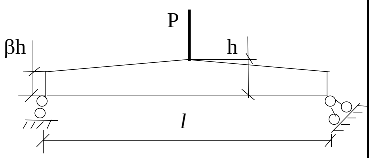
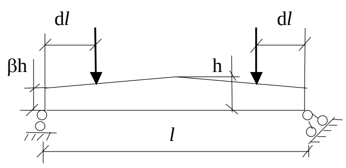
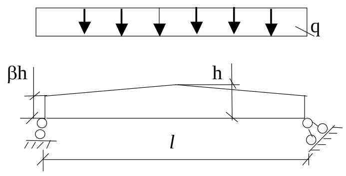
Snow regions in Russian Federation	I	II	III	IV	V	VI	VII	VIII
S_g кPa (кg/ m^2)	0,8 (80)	1,2 (120)	1,8 (180)	2,4 (240)	3,2 (320)	4,0 (400)	4,8 (480)	5,6 (560)

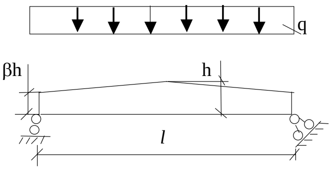
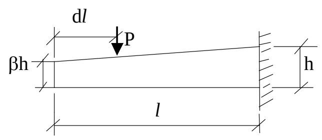
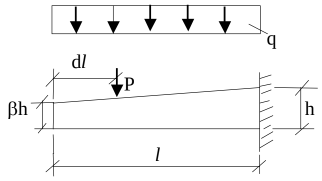
Combinations of loads	Duration of load $\tau_{пр}$, s	Factor m_{dl}	Service factor $m_н$
Linear increasing load	1 - 10	1	1,5
Permanent+ long-term	10^8 - 10^9	0,53	0,8
Permanent+ short-term snow	10^6 - 10^7	0,66	1
Permanent+ short-term wind	10^3 - 10^4	0,8	1,2
Permanent + seismic	10 - 10^2	0,92	1,4
Impact load	10^{-1} - 10^{-6}	1,1 - 1,35	1,7 - 2,0

Service classes	Factor m_b	Service classes	Factor m_b
А1, А2, Б1, Б2	1	В2, В3, Г1	0,85
А3, Б3, В1	0,9	Г2, Г3	0,75

Service classes	Description of service conditions	Maximum relative humidity, %	
		Glued timber	Non glued
	Inside heated places with temperature 35 °C and less, and relative humidity, %:		
$\left.\begin{matrix}A1\\A2\end{matrix}\right\}(C1)$	60 and less	9	20
	from 60 to 75	12	20
А3 (С2)	from 75 to95	15	20
	Inside not heated spaces:		
$\left.\begin{matrix}Б1\\Б2\end{matrix}\right\}(C3)$	In dry zone	9	20
	In normal zone	12	20
Б3 (С3.1)	In dry and normal zones with constant inside humidity more 75 % and in wet zone	15	25
	In open air:		
$\left.\begin{matrix}B1\\B2\\B3\end{matrix}\right\}(C3.2)$	In dry zone	9	20
	In normal zone	12	20
	In wet zone	15	25
	In parts of building:		
Г1 (С4)	Adjoined to soil or situated in soil	-	25
Г2 (С4.1, С4.2)	Permanently wet	-	Not limited
Г3	Located in water	-	Also

Shape of the moment curve	Value of k_{ϕ} for hinge-supported element
	1.13

Cross section	Scheme	*k*	*c*
Rectangular		β	0
The same		0,23 + 0,77β	16,4 + 7,6β
The same		0,5d + (1 - 0,5d) β	[45 - 24d(1 - β) + 3β] × $\frac{1}{3-4d^2}$
The same		0,15 + 0,85β	15,4 +3,8β

Cross section	Scheme	*k*	*c*
I-section		0,4 + 0,6β	(45,3 - 6,9β)γ
Rectangular		0,23 + 0,77β + 0,6d(1 - β)	[8,2 + 2,4(1 - β)d + 3,8β $\times \frac{1}{(2+a)(1-a)}$
То же		0,35 + 0,65β	5,4 + 2,6β

СИСТЕМА СЕРТИФИКАЦИИ ГОСТ Р
ГОССТАНДАРТ РОССИИ

Добровольная сертификация

СЕРТИФИКАТ СООТВЕТСТВИЯ

№ РОСС FI.AE95.H00234

Срок действия с 05.03.2008 по 04.03.2011

0877696

ОРГАН ПО СЕРТИФИКАЦИИ рег. № РОСС RU.0001.11AE95
ПРОДУКЦИИ ОБЩЕСТВО С ОГРАНИЧЕННОЙ ОТВЕТСТВЕННОСТЬЮ "НИИ-ТЕСТ"
Юридический адрес: 127591, Москва, ул. Дубнинская, д.44а
Фактический адрес: 119121, Москва, Ружейный пер., д. 6, стр. 1, тел. (495) 241-51-36, факс (495) 241-51-36

ПРОДУКЦИЯ Конструкции деревянные клееные Kerto-S, Kerto-Q, Glulam GL24, GL28, GL32 согласно приложений на 2х листах.
Серийный выпуск .

код ОК 005 (ОКП):
53 6660

СООТВЕТСТВУЕТ ТРЕБОВАНИЯМ НОРМАТИВНЫХ ДОКУМЕНТОВ

Спецификации изготовителя.

код ТН ВЭД:
4418 90 100 0

ИЗГОТОВИТЕЛЬ
Фирма "Metsaliitto Osuuskunta Finnforest"
FMO Tapiola, Tuulikuja 2, FI-02100 Espoo (P.O. box 50 FI-02020 Metsa), Финляндия
Филиалы изготовителя: «Finnforest», PL 24, 08101 LOHJA, Finland; «Finnforest», PL 25, 19601 HARTOLA, Finland.

СЕРТИФИКАТ ВЫДАН
Фирма "Metsaliitto Osuuskunta Finnforest"
FMO Tapiola, Tuulikuja 2, FI-02100 Espoo (P.O. box 50 FI-02020 Metsa), Финляндия

НА ОСНОВАНИИ
Протокола испытаний № 820-261 от 29.02.2008 г. ИЛ Лесопромышленной продукции и тары ЗАО "Региональный орган по сертификации и тестированию" "РОСТЕСТ-МОСКВА", рег. № РОСС RU.0001.21ДМ30, адрес: 117418, Москва, Нахимовский пр., д. 31, санитарно-эпиддемиологического заключения № 78.01.06.536.П.004431.08.06 от 18.08.2006 до 18.08.2011 г. Территориальное управления федеральной службы по надзору в сфере защиты прав потребителей и благополучия человека по городу Санкт-Петербургу.

ДОПОЛНИТЕЛЬНАЯ ИНФОРМАЦИЯ
Схема сертификации 3.

М.П.

Руководитель органа ________ подпись Т.В. Заболотная инициалы, фамилия

Эксперт ________ подпись Т.В.Раденкая инициалы, фамилия

Сертификат не применяется при обязательной сертификации

Приложение к сертификату соответствия
№ РОСС FI.AE95.H00234 от 29.02.2008
(на двух листах)

характеристика	значение					
	Glulam GL 24h	Glulam GL 28c	Glulam GL 32c	Kerto-S Толщина 21 – 90 мм	Kerto-Q Толщина 21 – 24 мм	Kerto-Q Толщина 27 – 69 мм
Допуски для размеров при содержании влаги 10±2%						
Толщина	h≤400 +4 -2 h>400 +1.0 - 0.5%	h≤400 +4 -2 h>400 +1.0 - 0.5%	h≤400 +4 и -2 h>400 +1.0 - 0.5%	+ (0.8+0.03t) – (0.4+0.03t)	+0.8 – 0.4 – (0.4+0.03t)	+0.8 – 0.4 – (0.4+0.03t)
Ширина	±2	±2	±2	<400± 2.0 ≥400± 0.5 % ± 5.0	<400± 2.0 ≥400± 0.5 % ± 5.0	<400± 2.0 ≥400± 0.5 % ± 5.0
Длина	h≤2м ±2 2<h≤20м 0.1% h>20m ±20	h≤2м ±2 2<h≤20м 0.1% h>20m ±20	h≤2м ±2 2<h≤20м 0.1% h>20m ±20			
Пятые процентильные значения						
Прочность на изгиб:						
В боковом направлении (глубина 300 мм)	24	28	32	44.0	28.0	32.0
Размерный эффект	-	-	-	0.12	0.12	0.12
Перпендикулярно слоям (толщ. от 21 до 90 мм)	-	-	-	50.0	32.0	36.0
Прочность на растяжение:						
Параллельно волокнам (длина 3000 мм)	16.5	16.5	19.5	35.0	19.0	26.0
Перпендикулярно волокнам, вбок	0.4	0.4	0.45	0.8	6.0	6.0
Перпендикулярно волокнам, перпендикулярно слоям	-	-	-	-	-	-
Прочность при сжатии:						
Параллельно волокнам	24.0	24.0	26.5	35.0	19.0	26.0
Перпендикулярно волокнам, вбок	2.7	2.7	3.0	6.0	9.0	9.0
Перпендикулярно волокнам, перпендикулярно слоям	-	-	-	1.8	1.8	1.8
Прочность при сдвиге:						
Параллельно волокнам	2.7	2.7	3.2	4.1	4.5	4.5
Перпендикулярно волокнам	-	-	-	2.3	1.3	1.3
Модуль упругости:						

Параллельно волокнам	9400	10200	11100	11600	8300	8800
Перпендикулярно волокнам, вбок	-	-	-	350	2000	2000
Перпендикулярно волокнам, перпендикулярно слоям	-	-	-	100	100	100
Перпендикулярно волокнам	-	-	-	-	-	-
Модуль сдвига:						
Параллельно волокнам	720	720	780	400	400	400
Перпендикулярно волокнам	-	-	-	400	-	-
Плотность	380	380	410	480	480	480
Средние значения						
Модуль упругости:						
Параллельно волокнам	11600	12600	13700	13800	10000	10500
Перпендикулярно волокнам, вбок	390	390	420	430	2400	2400
Перпендикулярно волокнам, перпендикулярно слоям	-	-	-	130	130	130
Модуль сдвига:						
Параллельно волокнам	-	-	-	600	600	600
Перпендикулярно волокнам	-	-	-	600	-	-
Плотность	-	-	-	510	510	510
Коэффициенты изменения размеров						
Толщина	-	-	-	0.0024	0.0024	0.0024
Ширина	-	-	-	0.0032	0.0003	0.0003
Длина	-	-	-	0.0001	0.0001	0.0001
Коэффициент сопротивления водяному пару						
В направлении толщины	-	-	-	80	62	62
В направлении ширины	-	-	-	82	9.5	9.5
В продольном направлении	-	-	-	3.9	4.7	4.7

Характеристики Kerto-S, Kerto-Q и Glulam

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