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Fire Design Methods for Structures with Timber Framework

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Abstract. Timber structures are environmentally friendly in terms of decreasing the impact of human activity on the planet. Reliability of timber structures can be provided by the corresponding fire resistance so as fire risk is one of the most significant disadvantages of timber structures. Development of new method for calculation of actual fire resistance is a topical issue. The first draft of method for calculation of actual fire resistance and classes of fire risk for load-bearing timber structures based on Russian national standard, was compared with fire design methods based on European norm. The structure of one-storey glued load-bearing timber framework for sports hall of Corporative University of Sberbank was used as an example for the comparison. The main load bearing structure of the framework in the transversal direction is two-hinge glued laminated timber frame with the span equal to 24 m, consisting of the two columns with rectangular glued cross-sections and trapezium truss with triangular lattice system and two cantilevers. It was stated, that methods based on European norm and method based on Russian national standard enable to obtain comparable results for evaluation of fire resistance for glued laminated load-bearing timber frameworks structure. However, the method based on Russian national standard did not contain information for evaluation of fire resistance of joints of timber structures, which often is determinant for the timber frameworks. Therefore, adding of the chapter including approach to evaluation of fire resistance of the joints of timber structures is possible direction for further development of the method based on Russian national standard.

1. Introduction

The problem of limited raw material and energy resources is one of the most actual in the world. It can be solved by decreasing the structural dead weight, increase of span and durability of load carrying structures so as by the replacement of non-renewable structural materials by renewable ones [1–3]. The rapid rise of CO2 emissions is caused by the technological development mainly. The production of the concrete, which is the most widely used structural material, is already responsible for from 5 % to 8 % of global greenhouse gas emissions. The production of steel requires about 4 % of global energy use [4]. Replacement of reinforced concrete and steel structures by the timber ones is one of the modern tendencies in civil engineering. Timber structures are environmentally friendly. Using of timber structures enables to decrease the anthropogenic impact on the planet [5]. Structural members from glued laminated timber, cross-laminated timber and other timber-based materials are widely used for one-storey and multi-storey buildings [4, 6–9]. Timber as a structural material has a potential for substitution of concrete and steel in it major applications. Not so far timber use for the multi-storey buildings was mentioned as the most significant limitation of it use as a structural material. However, nowadays, this limitation was deleted due to the development of new timber based structural materials such as cross-laminated timber. At Norway has completed the frame of the tallest timber building in the world – The Mjos Tower, which has 18 floors. However, possibility to create more than 30-storey timber building was stated [10]. The structure of sports hall of Corporative University of Sberbank can be considered as an example of the modern one-storey

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residential building with the timber framework (Figure 1.*a)*). Residential building in London can be considered as an example of multi-storey timber buildings [4] (Figure 1.*b)*).

a) *b)*

Figure 1. One-storey and multi-storey timber buildings: *a)* The structure of sports hall of Corporative University of Sberbank; *b)* Residential building in London.

However, at the same time reliability of timber structures can be provided by the corresponding fire resistance so as fire risk is one of the most significant disadvantages of timber structures. Timber fire safety assessment is complicating due to it anisotropy [6]. Fire resistance of timber structures can be determined by fire tests, advanced calculations and simplified design methods [11–13]. Typically, fire test is very expensive, but advanced calculations require a lot of additional parameters that complicate calculations and make calculations more work and time-consuming, so, development of new simplified design method for calculation of actual fire resistance and classes of fire risk for load-bearing timber structures is a topical issue [14–20].

The first draft of method based on Russian national standard for calculation of actual fire resistance and classes of fire risk for load-bearing timber structures, which is based on the Federal Law No. 123-FZ “Technical regulations on fire safety requirements” in part of fire resistance and fire risk of building structures, was prepared up till now [21]. A certain interest cause comparison of the developed variant of method for calculation of actual fire resistance and classes of fire risk for load-bearing timber structures with just existing ones and exactly with the reduced section and reduced properties methods reflected in the European norm EN 1995-1-2, which are used in the most of European states now. The methods of reduced section and reduced properties are used in the case of standard fire action and recommended for practical application. Parametric design method, which also is used in European states, should be used in the case of full fire analyse.

Therefore, the aim of the paper is comparison of the methods for fire resistance evaluation on the base of European norm EN 1995-1-2, such as reduced cross-section method, known also as effective cross-sectional method and reduced properties method, and method based on Russian national standard for calculation of actual fire resistance and classes of fire risk for load-bearing timber structures, which is based on the Federal Law No. 123-FZ «Technical regulations on fire safety requirements” [21]. The comparison can indicate possible supplements and improvements for the developed method. The structure of one-storey glued load-bearing timber framework for sports hall of Corporative University of Sberbank will be used as an example for the comparison of fire resistances, obtained by the method based on Russian national standard and methods based on European norm.

2. Methods

2.1. Determination of fire resistance for timber structures by EN 1995-1-1 and EN 1995-1-2

The fire resistance of structures of glued load-bearing timber framework for sports hall of Corporative University of Sberbank should be analysed by the reduced section method and reduced propertied methods so as both the methods are mentioned in the EN 1995-1-2 and reduced section method was mentioned as a recommended one. The main load bearing structure of the framework in the transversal direction is two-hinge glued laminated timber frame consisting of the two columns with rectangular glued cross-sections and trapezium truss with triangular lattice system and two cantilevers (Figure 2).

Top and bottom chords of the truss so as the elements of the lattice have rectangular glued sections. The spatial stability of the frame is provided by the moment cornice joints of the columns with the truss in plane of the frame so as by horizontal and vertical bracings in zone of the trusses so as by glued timber decking. The structure of the roof including a glued timber decking joined with the top chord of the truss and providing it from the lateral torsional buckling. The system of vertical bracings between the trusses and columns provide stability of the frame in the perpendicular plane. The fire resistance of the glued laminated timber frame is evaluated as the minimum fire resistance of it load-bearing members and joints. Influence of the timber decking and system of bracing is taken into account.

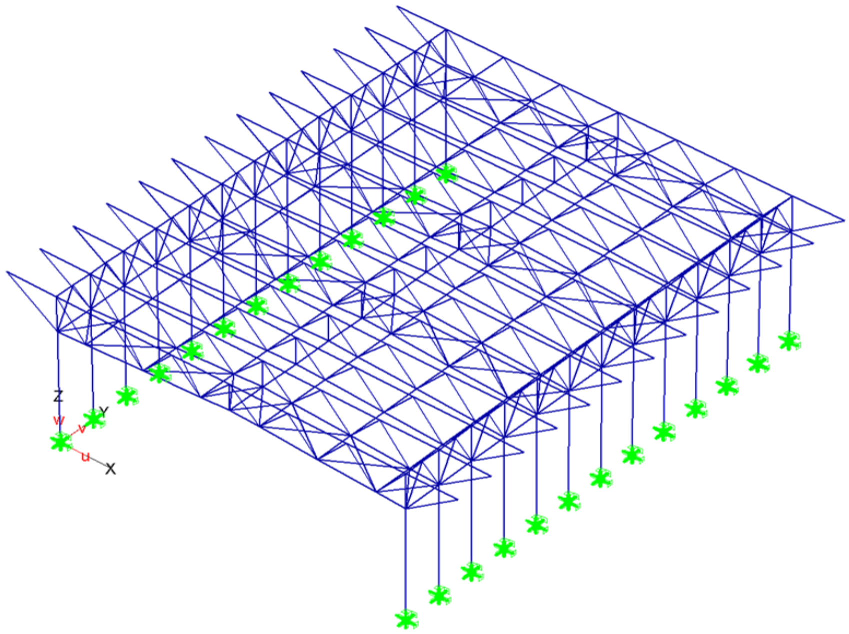


Figure 2. Structures of load-bearing framework of sports hall of Corporative University of Sberbank.

The members of glued laminated timber frame are subjected to the action of axial force in combination with the bending moment and axial force. The top chord of the truss and the columns are subjected to compression with the bending. Bottom chord of the truss is subjected to tension with the bending. Elements of the lattice of the truss are subjected to axial tension or compression. Glued timber decking is subjected to flexure. Therefore, conditions of strength and stability, which are explained in the points 6.1.6., 6.2.3, 6.2.4, 6.3.2 and 6.3.3 of EN 1995-1-1 are used for determination of fire resistances of glued timber decking, columns, chords of the truss so as elements of the lattice of the truss.

The fire resistance of joints of the truss must be checked by the controlling of the constructional requirements satisfaction for the bolted connections in course of the fire exposure. The structural requirements to the bolted connections are explained in the point 8.5 of EN 1995-1-1. Guidelines for evaluation of fire resistance for unprotected connections with the unprotected side members of wood by the simplified method are given in point 6.2.1.1. of EN 1995-1-2.

2.2. Reduced cross-section method

An effective cross-section of the members of glued timber frame should be determined by reducing the initial cross-section by the effective charring depth d_{ef} [22].

$$d_{ef} = d_{char,n} + k_0 d_0, \tag{1}$$

where: d_0 is thickness of pyrolyzed layer after 20 minutes of fire exposure (d_0 = 7 mm);

$d_{\mathrm{char,n}}$ is the notional design charring depth, which should be determined as a product of β_n notional design charring rate, the magnitude of which includes for the effect of corner rounding and fissures, and *t* time of fire exposure;

k_0 is parameter, which take into account decrease the thickness of pyrolyzed layer in the case of fire exposure less than 20 minutes.

The cases, when the structural members are subjected to fire action from four, three and one sides should be considered (Figure 3). The columns of the frame, bottom chord of the truss and elements of the lattice are subjected to fire action from four sides. Top chord of the truss is subjected to fire action from three sides so as it top side is protected from the fire action by the glued timber decking.

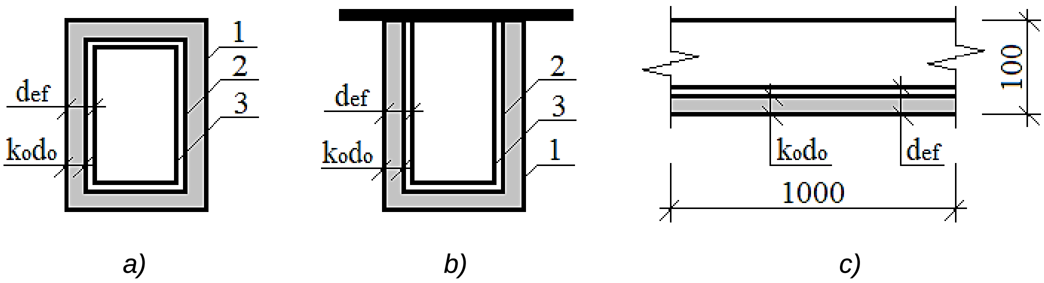


Figure 3 Reduced cross-section of the member for the cases [22]: *a)* structural member is subjected to the fire action from four sides; *b)* structural member is subjected to the fire action from three sides; *c)* structural member is subjected to the fire action from one side; 1 – initial surface of the member; 2 – border of residual cross-section; 3 – border of effective cross-section; d_{ef} – effective charring depth; $k_0 d_0$ – thickness of pyrolyzed layer.

The glued timber decking is considered as a section with the width equal to 1 m. It is subjected to fire action from the bottom only. The column and top chord of the truss are analysed at the fire action taking in to account strength and stability conditions so as the members are subjected to the action of axial compression forces and bending moment. The values of bending moment and axial forces in case of fire action are determined as a product of the forces obtained from the analysis at normal temperature for the fundamental combination of actions [23] and the reduction factor for the design load in the fire situation. As a simplification, the recommended value for the reduction factor for the design load in the fire situation was taken equal to 0.6 [22]. The strength condition is checked by the equations (2), (3) [24].

$$\left(\frac{\sigma_{c,0,d,fi}}{f_{c,0,d,fi}}\right)^2+\frac{\sigma_{m,y,d,fi}}{f_{m,y,d,fi}}+k_m\cdot\frac{\sigma_{m,z,d,fi}}{f_{m,z,d,fi}}\le 1; \tag{2}$$

$$\left(\frac{\sigma_{c,0,d,fi}}{f_{c,0,d,fi}}\right)^2+k_m\cdot\frac{\sigma_{m,y,d,fi}}{f_{m,y,d,fi}}+\frac{\sigma_{m,z,d,fi}}{f_{m,z,d,fi}}\le 1, \tag{3}$$

where $\sigma_{c,0,d,fi}$ is design compressive stress along the grain in case of fire action,

$\sigma_{m,y,d,fi}$ is design bending stress about the principal *y*-axis in case of fire action,

$\sigma_{m,z,d,fi}$ is design bending stress about the principal *z*-axis in case of fire action,

$f_{c,0,d,fi}$ is design compressive strength in case of fire action,

$f_{m,y,d,fi}$ is design bending strength about the principal *y*-axis in case of fire action,

$f_{m,z,d,fi}$ is design bending strength about the principal *z*-axis in case of fire action,

k_m is factor, considering redistribution of bending stress in cross-section.

The stability of column and top chord of the truss was checked by the equations (4) and (5) [24] as for members subjected to compression with the bending.

$$\left(\frac{\sigma_{c,0,d,fi}}{k_{c,y,fi}\cdot f_{c,0,d,fi}}\right)^2+\frac{\sigma_{m,y,d,fi}}{f_{m,y,d,fi}}+k_m\cdot\frac{\sigma_{m,z,d,fi}}{f_{m,z,d,fi}}\le 1; \tag{4}$$

$$\left(\frac{\sigma_{c,0,d,fi}}{k_{c,z,fi}\cdot f_{c,0,d,fi}}\right)^2+k_m\cdot\frac{\sigma_{m,y,d,fi}}{f_{m,y,d,fi}}+\frac{\sigma_{m,z,d,fi}}{f_{m,z,d,fi}}\le 1, \tag{5}$$

where $k_{c,y,fi}$ – instability factor about the principal *y*-axis in case of fire action,

$k_{c,z,fi}$ – instability factor about the principal *z*-axis in case of fire action; other designations as for equations (2) and (3).

The bottom chord of the truss is analysed at the fire action by the strength condition taking in to account axial tension force and bending moment by the equation (6), (7) [24].

$$\left(\frac{\sigma_{t,0,d,fi}}{f_{t,0,d,fi}}\right)^2+\frac{\sigma_{m,y,d,fi}}{f_{m,y,d,fi}}+k_m\cdot\frac{\sigma_{m,z,d,fi}}{f_{m,z,d,fi}}\le 1; \tag{6}$$

$$\left(\frac{\sigma_{t,0,d,fi}}{f_{t,0,d,fi}}\right)^2 + k_m \cdot \frac{\sigma_{m,y,d,fi}}{f_{m,y,d,fi}} + \frac{\sigma_{m,z,d,fi}}{f_{m,z,d,fi}} \leq 1, \tag{7}$$

where $\sigma_{t,0,d,fi}$ is design tensile stress along the grain in case of fire action,

$f_{t,0,d,fi}$ is design tensile strength in case of fire action, other designations as for the equations (2) and (3).

The members of the lattice of the truss are analysed at the fire action taking in to account strength and stability conditions as the members subjected to the action of axial forces. The strength conditions are checked by the equations (8) and (9) for the axially tensioned and compressed elements of the lattice, correspondingly [24].

$$\frac{\sigma_{t,0,d,fi}}{f_{t,0,d,fi}} \leq 1; \tag{8}$$

$$\frac{\sigma_{c,0,d,fi}}{f_{c,0,d,fi}} \leq 1, \tag{9}$$

where designations as for the equations (2) and (6).

Stability of lattice of the truss, was checked by the equations (10) and (11) [24] as for the members subjected to compression.

$$\frac{\sigma_{c,0,d,fi}}{k_{c,y,fi} \cdot f_{c,0,d,fi}} \leq 1; \tag{10}$$

$$\frac{\sigma_{c,0,d,fi}}{k_{c,z,fi} \cdot f_{c,0,d,fi}} \leq 1, \tag{11}$$

where designations as for the equations (4) and (5).

The strength of the glued timber decking subjected to flexure, was checked by the equations (12) and (13).

$$\frac{\sigma_{m,y,d,fi}}{f_{m,y,d,fi}} + k_m \cdot \frac{\sigma_{m,z,d,fi}}{f_{m,z,d,fi}} \leq 1; \tag{12}$$

$$k_m \cdot \frac{\sigma_{m,y,d,fi}}{f_{m,y,d,fi}} + \frac{\sigma_{m,z,d,fi}}{f_{m,z,d,fi}} \leq 1, \tag{13}$$

where designations as for the equations (4) and (5).

The design strength and stiffness properties of the effective cross-section should be calculated with factor $k_{\mathrm{mod,fi}}$ = 1.0.

2.3. Reduced properties method

The design strength and stiffness properties of the effective cross-section by the reduced properties method should be calculated with factor $k_{\mathrm{mod,fi}}$, which should be determined by the dependences (14)–(16) or taken by the graphical dependence 4.3 [22].The values of the factor $k_{\mathrm{mod,fi}}$ will be differed dependently from the inner forces acting in the considered structural members. The value of $k_{\mathrm{mod,fi}}$ should be determined for bending strength:

$$k_{mod,fi} = 1.0 - \frac{1}{200} \cdot \frac{p}{A_r}. \tag{14}$$

The value of $k_{\mathrm{mod,fi}}$ should be determined for compression strength:

$$k_{mod,fi} = 1.0 - \frac{1}{125} \cdot \frac{p}{A_r}. \tag{15}$$

The value of $k_{\mathrm{mod,fi}}$ should be determined for tensile strength:

$$k_{mod,fi} = 1.0 - \frac{1}{330} \cdot \frac{p}{A_r}, \tag{16}$$

where p is the perimeter of the fire exposed residual cross-section, in meters, A_r is the area of the residual cross-section, in m[2].

The dependences (14)–(16) should be used in case if time of fire exposure is bigger or equal to 20 minutes. For time of fire exposure equal to zero $k_{\mathrm{mod,fi}}$ = 1.0. The value of $k_{\mathrm{mod,fi}}$ should be determined by the linear interpolation in case if time of fire exposure will be within the limits from 0 to 20 minutes.

The residual cross-section of the considered structural members should be determined by the point 3.4 [22]. The fire resistance of the joints was evaluated as for connections with the side members of timber, which are described in the point 6.2 [22]. The fire resistance of unprotected timber-to-timber connections where spacings, edge and end distances and side member dimensions satisfy the requirements given in EN 1995-1-1 section 8, were taken from the table 6.1 [22].

2.4. Determination of fire resistance for timber structures by method for calculation of actual fire resistance and classes of fire risk

The method contains the major requirements for calculation of actual fire resistance and classes of fire risk for load-bearing timber structures with solid and glued cross-sections. Changing of mechanical properties and geometrical parameters of the members cross-sections are taken into account. The method can be used for the development of technical solutions during designing of timber structures in accordance with the requirements of the Federal Law No. 123-FZ “Technical regulations on fire safety requirements”. The method was developed on the base of results of experimental tests [25]. The statements of existing analytical methods for evaluation of fire resistance of building constructions were taken into account [26]. The design statements of the method are a number of semi-empirical dependences which were obtained basing on the results of fire testing of structures under load by the method, which is described in GOST 30247.0-94 and GOST 30247.1-94. Evaluation of fire resistance of timber structures joined with the determination of time of losing of it load-bearing capacity by taking into account time-temperature curve in case of standard fire [27]. The charring rates were taken constant and equal to 0.7 mm/min if the cross-sections linear dimensions of the member are not less than 120 mm and 1 mm/min for the members with linear dimensions of cross-sections less than 120 mm. The time of loosing of load-bearing capacity of timber structural members in general case is calculated by formula (17).

$$\tau_{\mathrm{p}} = \tau_{\mathrm{kp}} + \tau_{з} = \tau_{\mathrm{kp}} + 5\ \mathrm{min}, \tag{17}$$

where τ_{kp} is time till the moment of critical cross-section development;

$\tau_{з}$ is time of delay, which is taken equal to 5 min.

The time till the moment of critical cross-section development dependently from the heating conditions for the structural members should be determined by the formula (18) in case, if the structure is subjected to fire action from three sides.

$$\tau_{\mathrm{kp}} = B - \frac{b}{2\beta} = H - \frac{h}{\beta}, \tag{18}$$

where B is initial width of cross-section before heating, mm;

H is initial depth of cross-section before heating, mm;

b is width of critical cross-section, mm;

h is depth of critical cross-section, mm;

β is charring rate.

The time till the moment of critical section development should be determined by the formula (19) in case if structure is subjected to fire action from four sides.

$$\tau_{\mathrm{kp}} = B - \frac{b}{2\beta} = H - \frac{h}{2\beta}, \tag{19}$$

where designations as for formula (18).

The rounding of corners and decrease of mechanical properties of timber are taken into account by the coefficient ξ, which should be taken from the table 1 of the method dependently from the internal forces acting in the considered structural member [21]. The coefficient ξ is equal to 0.6, 0.7 and 0.85 for the members subjected to flexure, compression parallel to fibres and tension parallel to fibres, correspondingly. The depths and widths of critical cross-sections should be determined by the formulas (20) and (21).

$$R_{\mathrm{m,d}} \cdot \xi = \frac{6M_{\mathrm{d}}}{bh^2}, \tag{20}$$

where M_d is maximum bending moment from the action of actual load;

ξ is coefficient, which takes into account decrease of mechanical properties of timber and rounding of corners;

$R_{m,d}$ is design resistance of timber in bending;

b is width of critical cross-section, mm;

h is depth of critical cross-section, mm.

$$R_{c/t,0,d} \cdot \xi = \frac{N_{c/t,\,d}}{bh}, \tag{21}$$

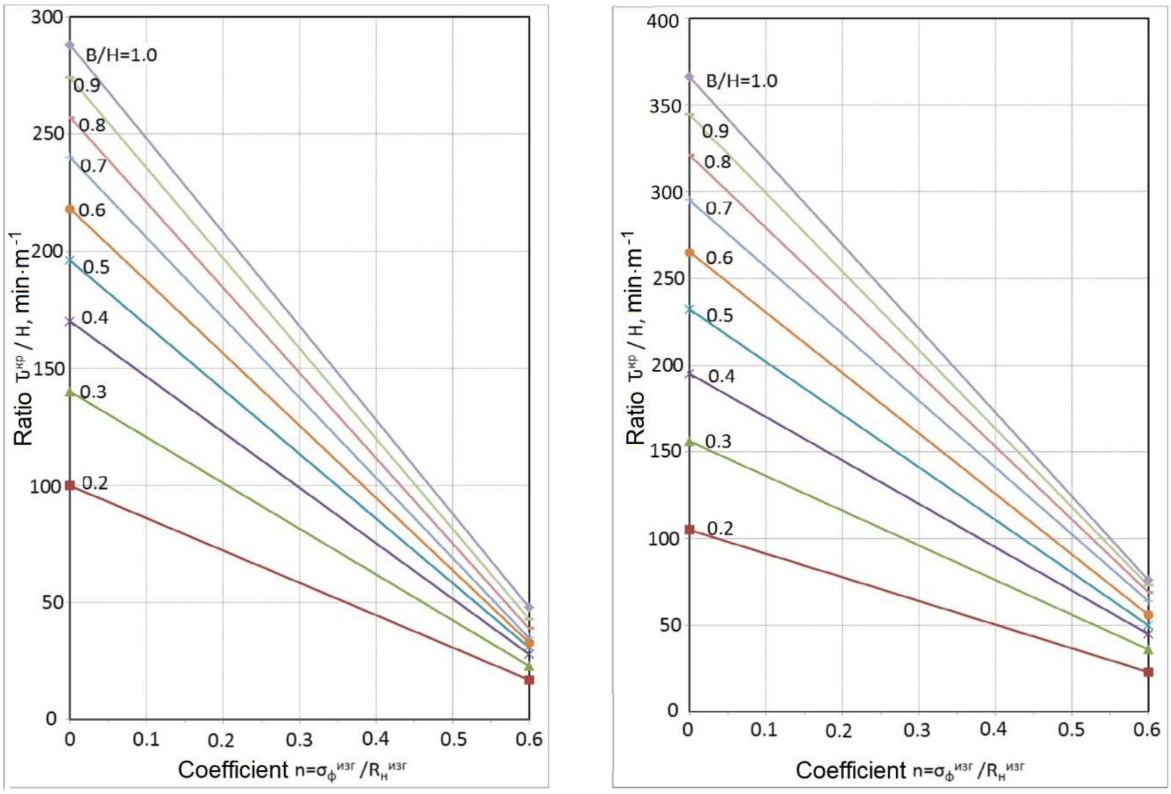
where $N_{c/t,0,d}$ is maximum axial force (compression/tension) acting parallel to the grain from the action of actual load;

$R_{c/t,0,d,}$ is design resistance of timber in compression or tension; other designations as in formula (18).

The depth and width of critical cross-sections for elements subjected to compression with the bending should be calculated by the summing of right parts of formulas (20) and (21).

The time till the moment of critical section development can be determined by the simplified approach with using of nomographs.

The nomographs were obtained on the base of experimental data for glued timber members subjected to axial compression parallel to grains and flexure. The nomographs obtained for the beams are shown on Figure 4. The nomographs obtained for the columns have the similar shapes. The rounding of the corners of the member and decrease of mechanical properties of timber are taken into account. The nomographs are linear dependences of coefficient of cross-section use before heating and relation of critical time to the initial depth of cross-section, which changes within the limits from 0.2 to 1.0



a) *b*)

Figure 4. Nomograph obtained for the beams: *a)* subjected to fire action from three sides; *b)* subjected to fire action from four sides [21].

The nomographs were obtained for the beams and columns subjected to fire action from three and four sides.

3. Results and discussions

3.1. Determination of fire resistance of timber framework by EN 1995-1-1 and EN 1995-1-2

The fire resistance of structure of glued laminated load-bearing framework for sports hall of Corporative University of Sberbank was determined by the reduced cross-section and reduced properties methods. The fire resistance of structure of glued laminated load-bearing framework was determined as the smallest from the fire resistances of columns, trusses and glued timber decking. The fire resistance of trapezium truss was determined as a minimum from the fire resistances of top and bottom chords so as members of lattice and joints. The scheme of the two-hinge glued laminated timber frame is shown on the Figure 5. Span of the frame is equal to 24 m and its bay is equal to 2.4 m. Heights of the truss at supports and in the middle of the span are equal to 2.9 and 2.5 m, correspondingly.

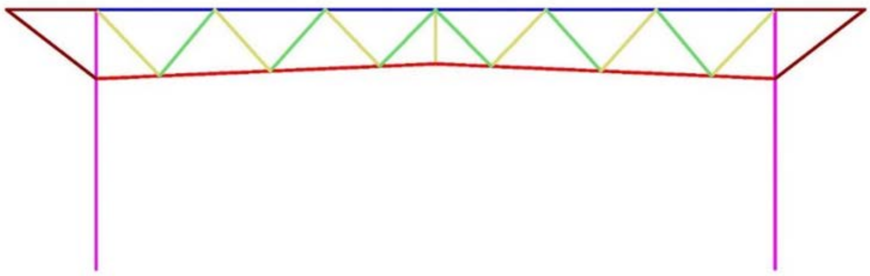


Figure 5. Scheme of the two-hinge glued laminated timber frame with coloured indication of members cross-sections, the colours descriptions are given in the Table 1.

The dimensions of cross-sections of all members of the two-hinge glued laminated timber frame so as some mechanical properties of used glued laminated timber are given in the Table 1.

Table 1. Parameters of members of the frame.

Member of the frame	Colour identification	Cross-section ($b*h$), mm	Modulus of elasticity, kN/m^2	Density, kg/m^3	Poisson ratio
Top chord		240*400	1e+007	600.00	0.45
Cantilevers		300*300	1e+007	600.00	0.45
Rising braces		200*200	1e+007	600.00	0.45
Lowering braces and central strut		150*220	1e+007	600.00	0.45
Bottom chord		320*420	1e+007	600.00	0.45
Columns and support struts		240*600	1e+007	600.00	0.45

3.1.1. *Fire resistance of glued laminated timber decking*

The glued timber decking is a structural member which is supported by the top chords of the trapezium trusses. The glued timber decking is developed by the squared boards with the depth of cross-section equal to 100 mm. Thickness of the glued timber decking is also equal to 100 mm. The softwood with the design resistance in 13 MPa was considered as a structural material for the glued timber decking. It was stated, that corresponding strength class of the timber is C20 [28]. The design scheme of the glued timber decking was five span continuous beam with the spans in 2.4 m. It was loaded by the uniformly distributed load with intensity in 7 kN/m (Figure 6).

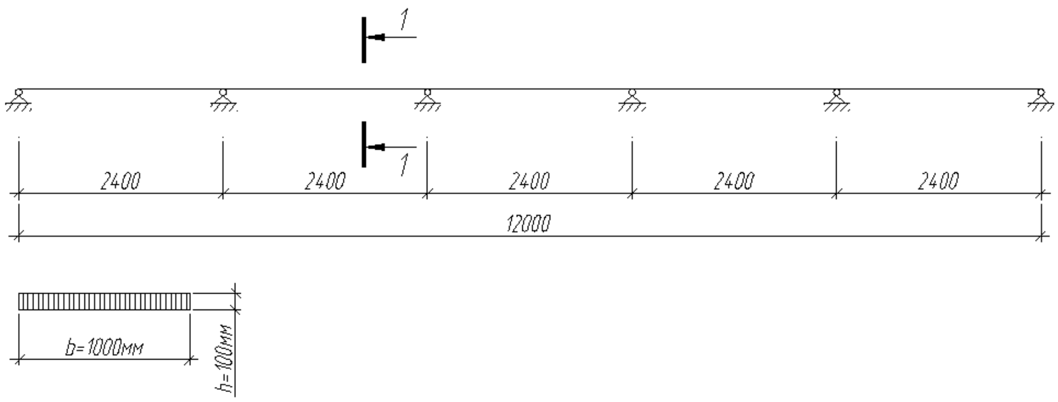


Figure 6. Design scheme of glued timber decking.

The intensities of surface permanent and snow loads are equal to 5.2 and 1.8 kPa, correspondingly. The growing of normal stresses in the cross-section, where act the maximum bending moment, was analysed in the case of normal fire by the reduced properties and reduced section methods. The maximum bending moment acts on the second support and was equal to 4.24 kNm when the structure is loaded by the snow and permanent load and fire action is not taken in to account. The value of bending moment in case of fire action was determined as a product of the forces obtained from the analysis at normal temperature and the reduction factor for the design load in the fire situation. The reduction factor was taken equal to 0.6 [22]. The value of bending moment in case of fire action was equal to 2.54 kNm. Relation of the maximum normal stresses acting in the cross-sections to the design resistance of the timber was considered as a level of cross-section use in case of fire. The cross-section of glued timber decking was considered as subjected to fire from one side as it is shown on the Figure 3, c). The dependences of the level of cross-section use, as a function from the time of the fire action, were obtained and shown on the Figure 7.

Fire resistance of the glued timber decking was evaluated as R85 and R106 by the reduced section and reduced properties methods, correspondingly. Fire resistance of cantilever part of the glued timber decking was evaluated as R82 and R103 by the reduced section and reduced properties methods, correspondingly. Therefore, the fire resistance of glued timber decking can be evaluated as R85, so as cantilever parts of the decking did not influence on the behaviour of the structure in general.

3.1.2. Fire resistance of glued timber columns

The glued laminated timber column with the dimensions of cross-section 600x200 mm with the hinge joints with the truss and foundation was analysed. Length of the column is equal to 11030 mm. The strength class of glued laminated timber was taken as GL28h [29]. Transversal frame of the building was loaded by the permanent, snow and wind loads. Characteristic value of wind pressure was taken as 0.23 kPa. The maximum values of bending moment and axial compression force in case of fire action were equal to 172.8 kN and 37.35 kNm, correspondingly. The cross-section of glued laminated timber column was considered as a subjected to fire from four sides, as it is shown on the Figure 3 a). Fire resistance of glued laminated timber column was analysed by the equations (2)–(5). The dependences of the level of cross-section use of the glued laminated timber column as a function from the time of the fire action were obtained and shown on the Figure 8.

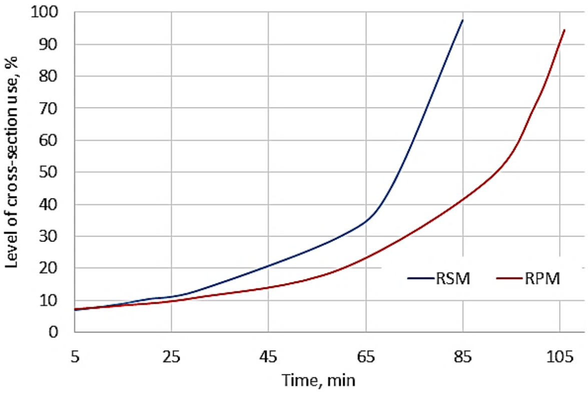


Figure 7. The dependences of the level of cross-section use as a function from the time of the fire action for glued timber decking: RSM – obtained by the reduced section method; RPM – obtained by the reduced properties method.

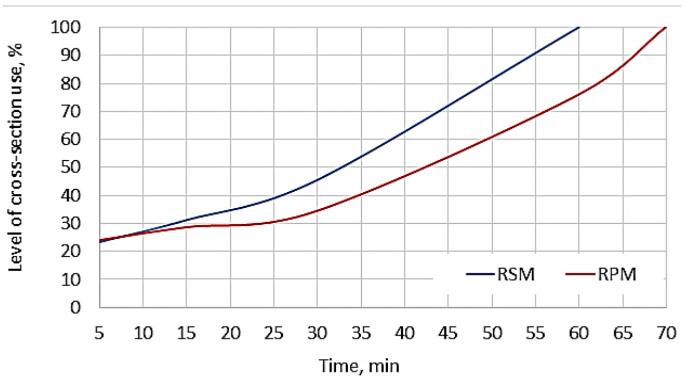


Figure 8. The dependences of the level of cross-section use as a function from the time of the fire action for glued laminated timber column: designations as in Figure 7.

Fire resistance of the glued laminated timber column was evaluated as R60 and R70 by the reduced section and reduced properties methods, correspondingly.

3.1.3. Fire resistance of timber truss

Fire resistance of the timber truss was evaluated as the minimum fire resistance of glued laminated timber top and bottom chords and members of the lattice. The timber truss has a span equal to 24 m. The glued laminated timber top chord of the truss has cross-section in 400x240 mm. It effective length is equal to the length of the panel in the both planes. The length of the panel of the top chord of the truss is equal to 4 m. The maximum values of the bending moment and axial compression forces in case of fire action were equal to 15.16 kNm and 332.12 kN, correspondingly. The cross-section of glued laminated timber top chord was considered as a subjected to fire from three sides, as it is shown on the Figure 3 b). The upper side of the top chord was protected by the decking. Fire resistance of glued timber top chord was analysed by the equations (2)–(5). Fire resistance of the glued laminated timber top chord was evaluated as R85 and R106 by the reduced section and reduced properties methods, correspondingly.

The glued laminated timber bottom chord of the truss has cross-section in 420×320 mm. The maximum values of the bending moment and axial tension forces in case of fire action were equal to 6 kNm and 342.60 kN, correspondingly. The cross-section of glued laminated timber bottom chord was considered as a subjected to fire from four sides, as it is shown on the Figure 3, a). Fire resistance of glued laminated timber bottom chord was analysed by the equations (6)–(7). It is significantly bigger than that of the top chord so as bottom chord is not subjected to the action of compressive forces. Safety storage in 68 % will take place for the bottom chord at the moment, when the buckling of the top chord occurs.

The dependences of the level of cross-section use, as a function from the time of the fire action for the top chord and most compressed brace of the truss, were obtained and shown on the Figure 9.

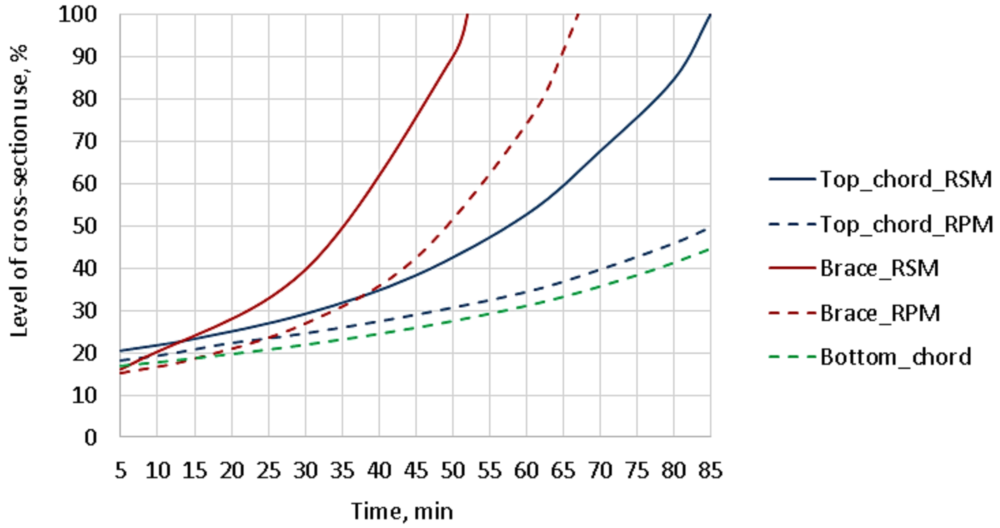


Figure 9. The dependences of the level of cross-section use as a function from the time of the fire action for glued laminated timber truss elements, where RSM – reduced section method, RPM – reduced properties method, Top_chord – the dependence obtained for the top chord, Brace – the dependence obtained for the most compressed brace; Bottom_chord – the dependence obtained for the bottom chord by the reduced properties method.

The lattice of the truss is presented by the compressed and tensioned braces. The most loaded compressed and tensioned braces of the truss are subjected to axial forces equal to 150.60 and 137.40 kN, correspondingly. The most loaded compressed and tensioned braces are the first and second braces from the support. The first from the support glued laminated timber brace has cross-section in 200x200mm and considered as a subjected to fire from four sides as it is shown on the Figure 3 a). Fire resistance of glued laminated timber brace was analysed by the equations (8)–(11) and evaluated as R52 and R65 by the reduced section and reduced properties methods, correspondingly. The fire resistance of tensioned brace is significantly bigger.

The fire resistance of the trusses joints was evaluated as R15 as bolted joints without fire protection. Therefore, overall fire resistance of the timber truss and glued laminated timber framework at all was determined as R15.

3.2. Determination of fire resistance of timber framework by method based on Russian national standard

Fire resistance of the timber structures is dependent from parameters of cross-section and density of timber. Therefore, more massive timber structure possesses a higher fire resistance. As it was mentioned before, determination of fire resistance of massive and glued laminated timber structures is joined with the determination of time R, when in case of fire cross-section of the structure decreases till the critical value, when the structure loose it load-carrying capacity. Decrease of the cross-section of the structure is joined with the development of the layer of coal on the surface of the structure in case of fire action.

The fire resistance of timber framework, described in the subchapter 3.2, was determined by the method based on Russian national standard [21]. The method was described in details in the sub-chapter 2.2. The structure in case of fire is subjected to the action of permanent and imposed loads – dead weight of structures and snow load (III snow region).

The fire action on the structure corresponds to the standard curve, which is described by the dependence (22).

$$T = T_0 + 345 \cdot \lg\left(8t + 1\right), \tag{22}$$

where T is temperature corresponding to time t, °C;

T_0 is initial temperature of fire action °C (T_0 =20 °C);

t is time from the initial moment of fire action.

Actual stresses acting in the structural members should be determined to find fire resistance which is reflected by the critical time from the initial moment of fire action till the moment, when structure loose it load-carrying capacity. Top and bottom chords of the truss so as support compressed brace were considered for evaluation of fire resistance of the glued laminated timber frame.

For support compressed brace coefficient of cross-section use before heating so as relation of initial width to depth of the cross-section are equal to 0.2 and 1.0, correspondingly. Relation of critical time to depth of the brace cross-section was determined by the nomograph and is equal to 230 min/m. Corresponding critical time is equal to 46 min. Therefore, the fire resistance of support compressed brace is equal to 51 min taking in to account time of charring delay, which is equal to 5 min.

The bottom chord of the truss is a member subjected to the action of tension force with the bending moment. For bottom chord coefficient of cross-section use before heating so as relation of initial width to depth of the cross-section are equal to 0.04 and 0.76, correspondingly as for the member subjected to flexure. The coefficient of cross-section use before heating and relation of initial width to depth of the cross-section are equal to 0.039 and 0.76, correspondingly, as for the member subjected to tension. Relations of critical time to depth of the top chord cross-section was determined by the nomographs and are equal to 241 min/m and 258 min/m by the nomograph for the bending moment and tension force, correspondingly. Corresponding critical times are equal to 101 and 108 min as for the member subjected to flexure and tension, correspondingly. Therefore, the fire resistance of the bottom chord of the truss is equal to 106 min taking in to account time of charring delay, which is equal to 5 min.

The fire resistance for the top chord of the truss was determined by the same approach as for the bottom once. For top chord coefficient of cross-section use before heating so as relation of initial width to depth of the cross-section are equal to 0.064 and 0.60, correspondingly. Relation of critical time to depth of the top chord cross-section was determined by the nomograph and is equal to 200 min/m by the nomograph for the bending moment. Corresponding critical time is equal to 80 min. Therefore, the fire resistance of support compressed brace is equal to 85 min.

The fire resistance for timber column was determined by the approach to compressed elements. For timber column coefficient of cross-section use before heating so as relation of initial width to depth of the cross-section are equal to 0.2 and 0.4, correspondingly. Relation of critical time to depth of the column cross-section was determined by the nomograph and is equal to 360 min/m. Corresponding critical time is equal to 72 min. Therefore, the fire resistance of the column is equal to 77 min taking in to account time of charring delay. Fire resistances for support compressed brace, top and bottom chords so as column are given on the Figure 10.

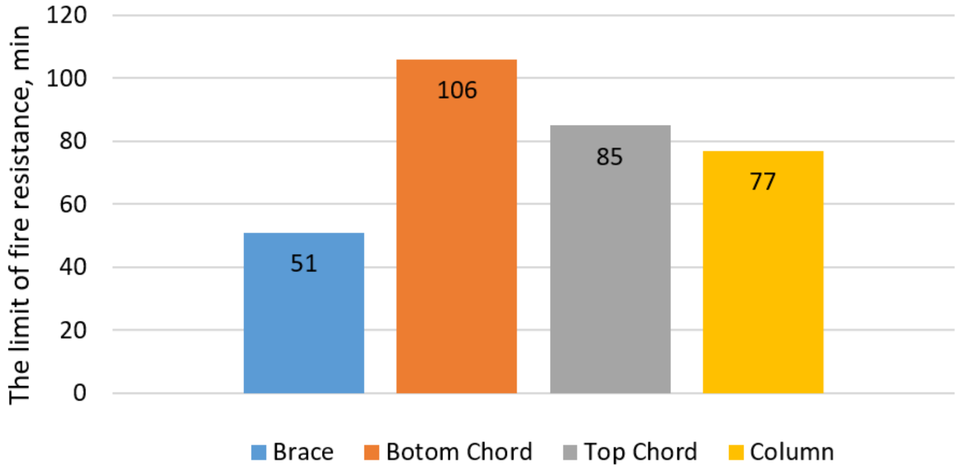


Figure 10. Value of the limit of fire resistance for part of timber frame.

Therefore, fire resistance of glued laminated timber frame is accepted as lowest value for timber frame and evaluated as R51.

3.3. Discussions

Comparison of results obtained for glued laminated load-bearing structures of timber framework for sports hall of Corporative University of Sberbank enables to make a conclusion that both methods allow to evaluate fire resistance of the major load bearing members with the comparable precision. The determinant structural member for evaluation of fire resistance for glued laminated load-bearing frameworks structure was timber truss in the case, if fire resistance of the joints is neglected. The fire resistance of glued timber top chord was evaluated by the reduced section method, reduced properties method and by method based on Russian national standard [21, 22]. The obtained values of fire resistance of glued timber top chord are evaluated as R85, R106 and R85, correspondingly. Fire resistance of glued laminated timber brace was analysed by the above-mentioned methods [21, 22] and evaluated as R52, R65 and R51 by the reduced section method, reduced properties method and by method based on Russian national standard. The fire resistances obtained for the glued laminated timber column were evaluated as R60, R70 and R77, correspondingly. Fire resistance of the glued timber bottom chord was evaluated as exceeding R106.

Therefore, it can be stated, that reduced section method, reduced properties method and method based on Russian national standard [21, 22] enable to obtain comparable results for evaluation of fire resistance for glued laminated load-bearing timber frameworks structure. The fire resistances obtained by the reduced section method and by method based on Russian national standard for the structure for sports hall of Corporative University of Sberbank were evaluated as R52 and R51, correspondingly. The differences between the results obtained for the glued laminated timber column by the method based on Russian national standard, reduced section method and reduced properties method were equal to 22.08 and 9.09 %, correspondingly. However, at the same moment, fire resistance of the trusses joints was evaluated as R15 as bolted joints without fire protection. Therefore, overall fire resistance of the timber truss and glued laminated timber framework was evaluated as R15 as for connections with the side members of timber, which are described in the point 6.2 [22]. However, the method based on Russian national standard did not contain information for evaluation of fire resistance of joints of timber structures. Therefore, adding of the chapter including approach to evaluation of fire resistance of the joints of timber structures is possible direction for further development of the method [21].

4. Conclusions

The methods for fire resistance evaluation on the base of European norm EN 1995-1-2 and method based on Russian national standard for calculation of actual fire resistance and classes of fire risk for load-bearing timber structures, which is based on the Federal Law No. 123-FZ «Technical regulations on fire safety requirements" were compared numerically for one-storey glued load-bearing timber framework. It was shown that the difference between the results obtained by the method based on Russian national standard and methods based on European norm is within the limits from 1,96 to 27.45 %. It was stated, that the method based on Russian national standard did not contain information for evaluation of fire resistance of joints of timber structures, which is determinant for the considered one-storey glued load-bearing timber framework so as it overall fire resistance was evaluated as R15 as for connections with the side members of timber. Therefore, adding of the chapter including approach for evaluation of fire resistance of the joints of timber structures is possible direction for further development of the method based on Russian national standard.

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Методы определения огнестойкости конструкций деревянного каркаса

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Ключевые слова: клееный несущий деревянный каркас, трапециевидная деревянная ферма, клееный деревянный настил, метод уменьшенных сечений, метод уменьшенной прочности, огнестойкость.

Аннотация. Использование деревянных конструкций позволяет снизить негативное воздействие человеческой деятельности на окружающую среду. Надежность деревянных конструкций может быть обеспечена путем достижения соответствующей огнестойкости так как опасность пожара является одним из главных недостатков деревянных конструкций. Разработка новых методов определения огнестойкости деревянных конструкция является актуальным вопросом. В настоящий момент на основе российского национального стандарта разработана первая редакция методики оценки пределов огнестойкости и классов пожарной опасности несущих деревянных конструкций. Результаты, полученные при помощи данной методики для определения огнестойкости несущих деревянных конструкций здания спортзала Корпоративного университета Сбербанка, сравнены с результатами, полученными при помощи расчетных методов, описанных в европейских нормах. Несущие деревянные конструкции здания спортзала Корпоративного университета Сбербанка представлены двухшарнирной клееной деревянной рамой , состоящей из трапециевидной фермы с треугольной решеткой, двумя консолями, а так же клееными колоннами и клееным деревянным настилом. В результате сравнения, получено, что методика оценки пределов огнестойкости и классов пожарной опасности несущих деревянных конструкций на основе российского национального стандарта дает результаты сопоставимые с результатами, полученными при помощи методов, описанных в европейских нормах. В то же время следует отметить, что отсутствие информации по определению огнестойкости узлов конструкций является существенным недостатком методики на основе российского национального стандарта, так как именно узлы часто являются звеном, определяющим огнестойкость деревянных конструкций. Дополнение методики на основе российского национального стандарта информацией по определению огнестойкости узлов является рекомендацией по ее улучшению.

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