Riga Technical University

Institute of Structural Engineering and Reconstruction

Scientific Seminar
Fire Design of Timber Structures by EN 1995-1-2
SPbU, March 26, 2016

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Content

1. **Design of timber members by EN 1995-1-1;**
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5. **Reduced cross-section method;**
6. **Reduced properties method;**
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Limit State design

The limit state design philosophy, which was formulated for reinforced concrete desig[n] in Russia during the 1930s, achieves the objectives by considering two „types" of limit state under which a structure may become unfit for its intended purpose. They are:

the ***Ultimate Limit State*** in which the structure, or some part of it, is unsafe for its intended purpose, e.g. compressive, tensile, shear or flexural failure or instability leading to partial or total collapse;

the ***Serviceability Limit State*** in which a condition, e.g. deflection, vibration or cracking, occurs to an extent, which is unacceptable to the owner, occupier, client etc.

The basis of the approach is statistical and lies an assessing the probability of reaching a given limit state and deciding upon as acceptable level of that probability for design purposes. The method in most codes is based on the use of ***characteristic values*** and ***partial safety factors***.

Partial Safety Factors

The use of **partial safety factors**, which are applied separately to individual parameters, enables the degree of risk for each one to be varied. This reflects the differing degrees of control which are possible in the manufacturing process of building structural materials/units (e.g. steel, concrete, timber, mortar and individual bricks) and construction process such as steel fabrication, pre-cast concrete, or building in masonry.

Characteristic Values

The use of **characteristic values** enables the statistical variability of various parameters such as materials strength, different load types, etc., to be incorporated in an assessment of the acceptable probability that the value of the parameter will be exceeded during the life of structure. The term **characteristic** in current design codes normally refers to a value of such magnitude that statistically for loads, there is a 5% probability of it being exceeded, whilst for strength, there is a 5% probability of the actual strength being less.

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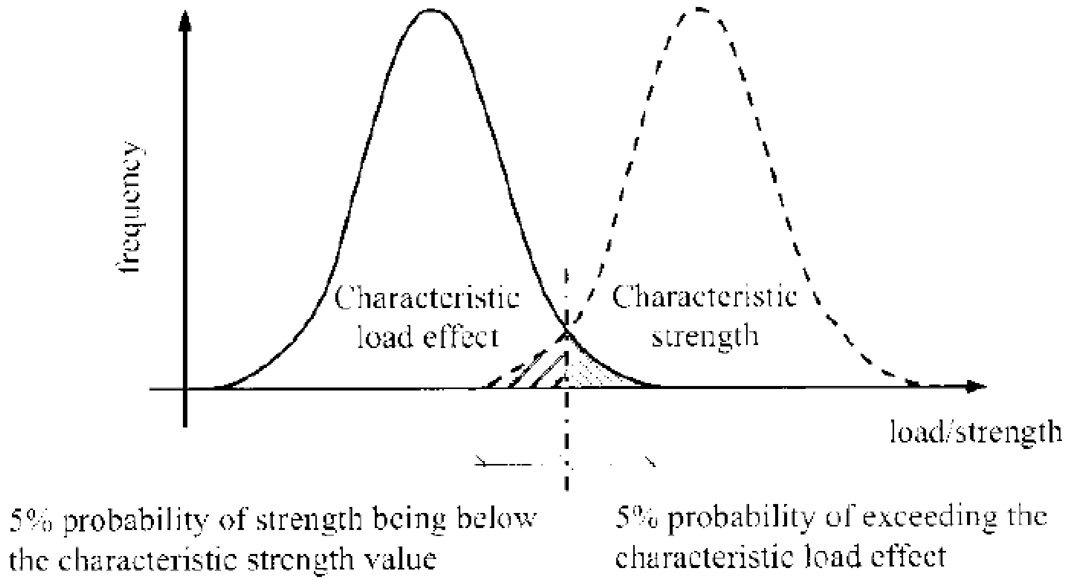
Limit State Philosophy



The shaded area represents the probability of failure, i.e. the level of design load effect, which can be expected to be exceeded by 5%, and the level of design strength which 5% of samples can be expected to fall below. The point of intersection of these two distribution curves represents the ultimate limit state, i.e. the design strength equals the design load effect.

Structural Design According to Eurocodes

The European Standarts Organisation, **CEN**, is the umbrella organisation under which a set of common structural design standarts (EC1, EC2, EC3, etc.) are being developed. The structural Eurocodes are the result of attempts to eliminate barriers to trade throughout the European Union. Separate codes exist for each structural material.

EN 1991 Eurocode 1 : Actions on structures
EN 1992 Eurocode 2 : Design of concrete structures
EN 1993 Eurocode 3 : Design of steel structures
EN 1994 Eurocode 4 : Design of composite steel and concrete structures
EN 1995 Eurocode 5 : Design of timber structures
EN 1996 Eurocode 6 : Design of masonry structures
EN 1997 Eurocode 7 : Geotechnical design
EN 1998 Eurocode 8 : Design of structures for earthquake resistance
EN 1999 Eurocode 9 : Design of aluminium structures

Basic requirements of Structural Design

A structure shall be designed to have adequate :

– structural resistance,

– serviceability,

– durability.

In the case of fire, the structural resistance shall be adequate for the required period
of time.

EN1995-1-1 Scope and structure

- **Section 1:** General definitions, terminology
- **Section 2:** Basis of design: Timber specific supplement to EN1990
- **Section 3:** Material properties to be used for design
- **Section 4:** Durability concept
- **Section 5:** Basis of structural analysis
- **Section 6:** Ultimate limit state design principles
- **Section 7:** Serviceability limit states
- **Section 8:** Fasteners
- **Section 9:** Design of components and assemblies
- **Section 10:** Workmanship, structural detailing and control

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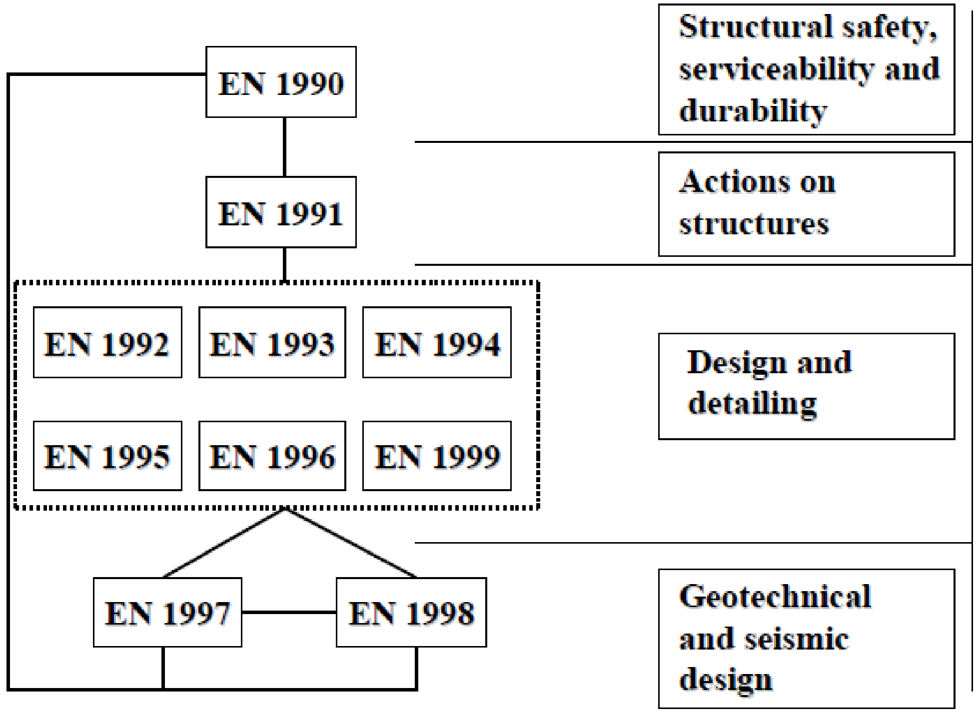
Link of EN 1995-1-1 to EN1990 and EN1991



Design value of material properties X_d

$$X_d = k_{\mathrm{mod}} \frac{X_k}{\gamma_M}$$

X_k – characteristic value of a strength property

γ_M – partial factor for a material property

k_{mod} – modification factor, taking into account duration of load and moisture content

Strength classes - Characteristic values

		Softwood species												Hardwood species							
		C14	C16	C18	C20	C22	C24	C27	C30	C35	C40	C45	C50	D18	D24	D30	D35	D40	D50	D60	D70
Strength properties (in N/mm^2)																					
Bending	$f_{m,k}$	14	16	18	20	22	24	27	30	35	40	45	50	18	24	30	35	40	50	60	70
Tension parallel	$f_{t,0,k}$	8	10	11	12	13	14	16	18	21	24	27	30	11	14	18	21	24	30	36	42
Tension perpendicular	$f_{t,90,k}$	0,4	0,4	0,4	0,4	0,4	0,4	0,4	0,4	0,4	0,4	0,4	0,4	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6
Compression parallel	$f_{c,0,k}$	16	17	18	19	20	21	22	23	25	26	27	29	18	21	23	25	26	29	32	34
Compression perpendicular	$f_{c,90,k}$	2,0	2,2	2,2	2,3	2,4	2,5	2,6	2,7	2,8	2,9	3,1	3,2	7,5	7,8	8,0	8,1	8,3	9,3	10,5	13,5
Shear	$f_{v,k}$	3,0	3,2	3,4	3,6	3,8	4,0	4,0	4,0	4,0	4,0	4,0	4,0	3,4	4,0	4,0	4,0	4,0	4,0	4,5	5,0
Stiffness properties (in kN/mm^2)																					
Mean modulus of elasticity parallel	$E_{0,mean}$	7	8	9	9,5	10	11	11,5	12	13	14	15	16	9,5	10	11	12	13	14	17	20
5 % modulus of elasticity parallel	$E_{0,05}$	4,7	5,4	6,0	6,4	6,7	7,4	7,7	8,0	8,7	9,4	10,0	10,7	8	8,5	9,2	10,1	10,9	11,8	14,3	16,8
Mean modulus of elasticity perpendicular	$E_{90,mean}$	0,23	0,27	0,30	0,32	0,33	0,37	0,38	0,40	0,43	0,47	0,50	0,53	0,63	0,67	0,73	0,80	0,86	0,93	1,13	1,33
Mean shear modulus	G_{mean}	0,44	0,5	0,56	0,59	0,63	0,69	0,72	0,75	0,81	0,88	0,94	1,00	0,59	0,62	0,69	0,75	0,81	0,88	1,06	1,25
Density (in kg/m^3)																					
Density	ρ_k	290	310	320	330	340	350	370	380	400	420	440	460	475	485	530	540	550	620	700	900
Mean density	ρ_{mean}	350	370	380	390	410	420	450	460	480	500	520	550	570	580	640	650	660	750	840	1080

Indexes:

c – compression;
t – tension;
m – bending (M - moment);

v – shear (V – shear force);
0 – parallel to grain;
90 – perp. to grain;

k – characteristic value;
d – design value.

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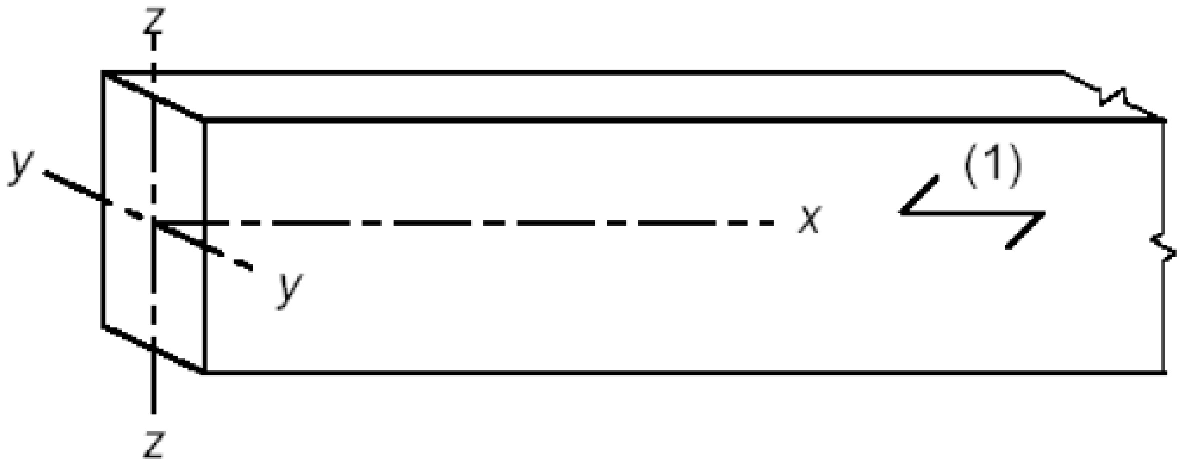
EN1995-1-1 - Definition of axes



Partial safety factor γ_M

Fundamental combinations:	
Solid timber	1,3
Glued laminated timber	1,25
LVL, plywood, OSB,	1,2
Particleboards	1,3
Fibreboards, hard	1,3
Fibreboards, medium	1,3
Fibreboards, MDF	1,3
Fibreboards, soft	1,3
Connections	1,3
Punched metal plate fasteners	1,25
Accidental combinations	1,0

Recommended material safety factor γ_M = 1,3

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Strength modification factor k_{mod}

Material	Standard	Service class	Load-duration class				
			Permanent action	Long term action	Medium term action	Short term action	Instantaneous action
Solid timber	EN 14081-1	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90
Glued laminated timber	EN 14080	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90
LVL	EN 14374, EN 14279	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90
Plywood	EN 636						
	Part 1, Part 2, Part 3	1	0,60	0,70	0,80	0,90	1,10
	Part 2, Part 3	2	0,60	0,70	0,80	0,90	1,10
	Part 3	3	0,50	0,55	0,65	0,70	0,90
OSB	EN 300						
	OSB/2	1	0,30	0,45	0,65	0,85	1,10
	OSB/3, OSB/4	1	0,40	0,50	0,70	0,90	1,10
	OSB/3, OSB/4	2	0,30	0,40	0,55	0,70	0,90
Particle-board	EN 312						
	Part 4, Part 5	1	0,30	0,45	0,65	0,85	1,10
	Part 5	2	0,20	0,30	0,45	0,60	0,80
	Part 6, Part 7	1	0,40	0,50	0,70	0,90	1,10
	Part 7	2	0,30	0,40	0,55	0,70	0,90
Fibreboard, hard	EN 622-2						
	HB.LA, HB.HLA 1 or 2	1	0,30	0,45	0,65	0,85	1,10
	HB.HLA1 or 2	2	0,20	0,30	0,45	0,60	0,80
Fibreboard, medium	EN 622-3						
	MBH.LA1 or 2	1	0,20	0,40	0,60	0,80	1,10
	MBH.HLS1 or 2	1	0,20	0,40	0,60	0,80	1,10
	MBH.HLS1 or 2	2	–	–	–	0,45	0,80
Fibreboard, MDF	EN 622-5						
	MDF.LA, MDF.HLS	1	0,20	0,40	0,60	0,80	1,10
	MDF.HLS	2	–	–	–	0,45	0,80

Load duration classes and examples

Load-duration class	Order of accumulated duration of characteristic load	Examples of loading
Permanent	more than 10 years	self-weight
Long-term	6 months – 10 years	storage
Medium-term	1 week – 6 months	imposed floor load, snow
Short-term	less than one week	snow, wind
Instantaneous		wind, accidental load

Service Classes

Service class 1 is characterised by a moisture content in the materials corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeding 65 % for a few weeks per year.

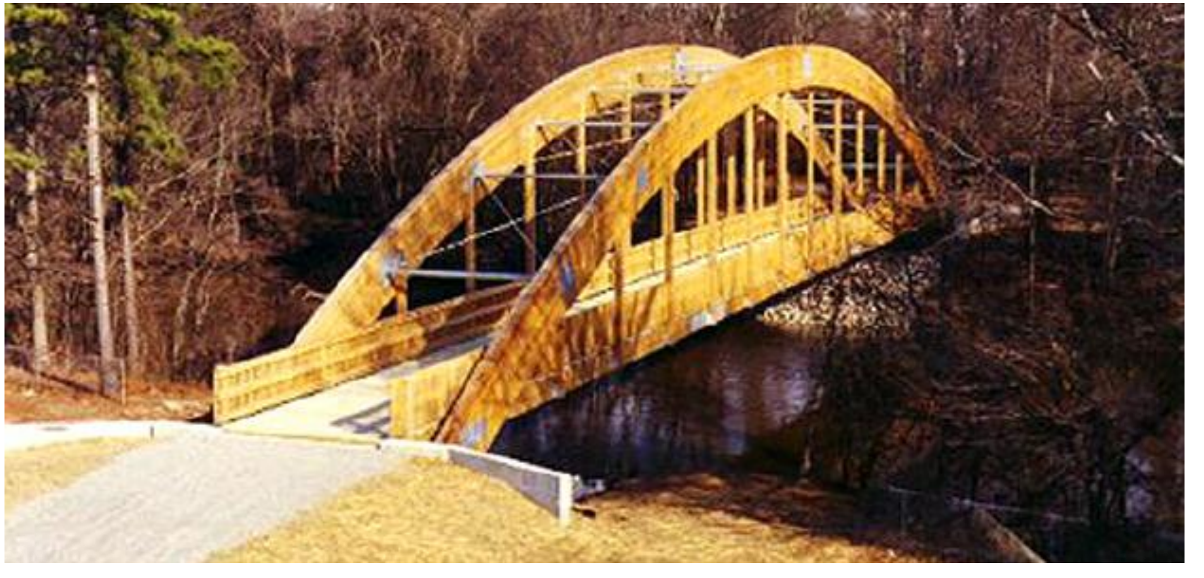
In service class 1 the average moisture content in most softwoods will not exceed 12 %.

Service Classes

Service class 2 is characterised by a moisture content in the materials corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeding 85 % for a few weeks per year.
In service class 2 the average moisture content in most softwoods will not exceed 20 %.

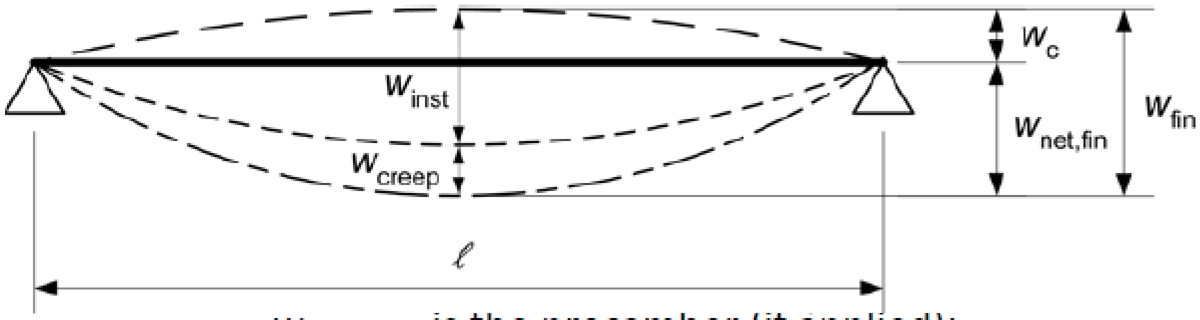
Service Classes

Service class 3 is characterised by climatic conditions leading to higher moisture contents than in service class 2.



Serviceability limit states

Beam example



- w_c is the precamber (if applied);
- w_{inst} is the instantaneous deflection;
- w_{creep} is the creep deflection;
- w_{fin} is the final deflection;
- $w_{net,fin}$ is the net final deflection.

$$w_{net,fin} = w_{inst} + w_{creep} - w_c = w_{fin} - w_c$$

Serviceability limit states

Beam example

Examples of limiting values for deflections of beams

	w_{inst}	$w_{net,fin}$	w_{fin}
Beam on two supports	$\ell/300$ to $\ell/500$	$\ell/250$ to $\ell/350$	$\ell/150$ to $\ell/300$
Cantilevering beams	$\ell/150$ to $\ell/250$	$\ell/125$ to $\ell/175$	$\ell/75$ to $\ell/150$

Deformation modification factor k_{def}

Material	Standard	Service class		
		1	2	3
Solid timber	EN 14081-1	0,60	0,80	2,00
Glued Laminated timber	EN 14080	0,60	0,80	2,00
LVL	EN 14374, EN 14279	0,60	0,80	2,00
Plywood	EN 636			
	Part 1	0,80	–	–
	Part 2	0,80	1,00	–
	Part 3	0,80	1,00	2,50
OSB	EN 300			
	OSB/2	2,25	–	–
	OSB/3, OSB/4	1,50	2,25	–
Particleboard	EN 312			
	Part 4	2,25	–	–
	Part 5	2,25	3,00	–
	Part 6	1,50	–	–
	Part 7	1,50	2,25	–
Fibreboard, hard	EN 622-2			
	HB.LA	2,25	–	–
	HB.HLA1, HB.HLA2	2,25	3,00	–
Fibreboard, medium	EN 622-3			
	MBH.LA1, MBH.LA2	3,00	–	–
	MBH.HLS1, MBH.HLS2	3,00	4,00	–
Fibreboard, MDF	EN 622-5			
	MDF.LA	2,25	–	–
	MDF.HLS	2,25	3,00	–

Axially loaded timber elements

Tension parallel to the grain

$$\sigma_{t,0,d} \leq f_{t,0,d}$$

$\sigma_{t,0,d}$ is the design tensile stress along the grain;

$f_{t,0,d}$ is the design tensile strength along the grain.

Effective cross-section area

Effective area A_{ef} of tensioned members is calculated by combining of all weakenings (holes), which are situated on the part of element with length (to both sides from the design section), which is ½ of allowed minimal distance between connectors. For bolts: 5d.

The weakenings can be ignored if:
-weakenings are caused by nails and screws with diameter less then 6 mm and if they are worked into without predrilling.
-weakenings are situated in the compressed zone and are filled with material with larger stiffness characteristics, than wood.
-for stability calculations, if weakenings not cause external grains.

Tension perpendicular to the grain

The effect of member size shall be taken into account.

Solid timber: $\sigma_{t,90,d} \leq f_{t,90,d}$

Glued timber: $\sigma_{t,90,d} \leq f_{t,90,d} \left(V_0 / V\right)^{0.2}$

V_0 – base volume in tension, for glued timber V_0 = 0.01 m^3;
V – actual volume of stressed timber perpendicular to the grains

Compression parallel to the grain

$$\sigma_{c,0,d} \leq f_{c,0,d}$$

$\sigma_{c,0,d}$ is the design compressive stress along the grain;

$f_{c,0,d}$ is the design compressive strength along the grain.

Compression perpendicular to the grain

$$\sigma_{c,90,d} \leq k_{c,90} f_{c,90,d}$$

$\sigma_{c,90,d}$ is the design compressive stress in the contact area perpendicular to the grain;

$f_{c,90,d}$ is the design compressive strength perpendicular to the grain;

$k_{c,90}$ is a factor taking into account the load configuration, possibility of splitting and degree of compressive deformation.

Stability of members

Stability shall be verified using the characteristic properties, e.g. $E_{0,05}$

The relative slenderness ratios:

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \qquad \lambda_{rel,z} = \frac{\lambda_z}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}}$$

λ_y and $\lambda_{rel,y}$ - slenderness ratios corresponding to bending about the *y*-axis (deflection in the *z*-direction);

If both $\lambda_{rel,z} \leq 0{,}3$ and $\lambda_{rel,y} \leq 0{,}3$, stability can not be checked.

Rectangular:

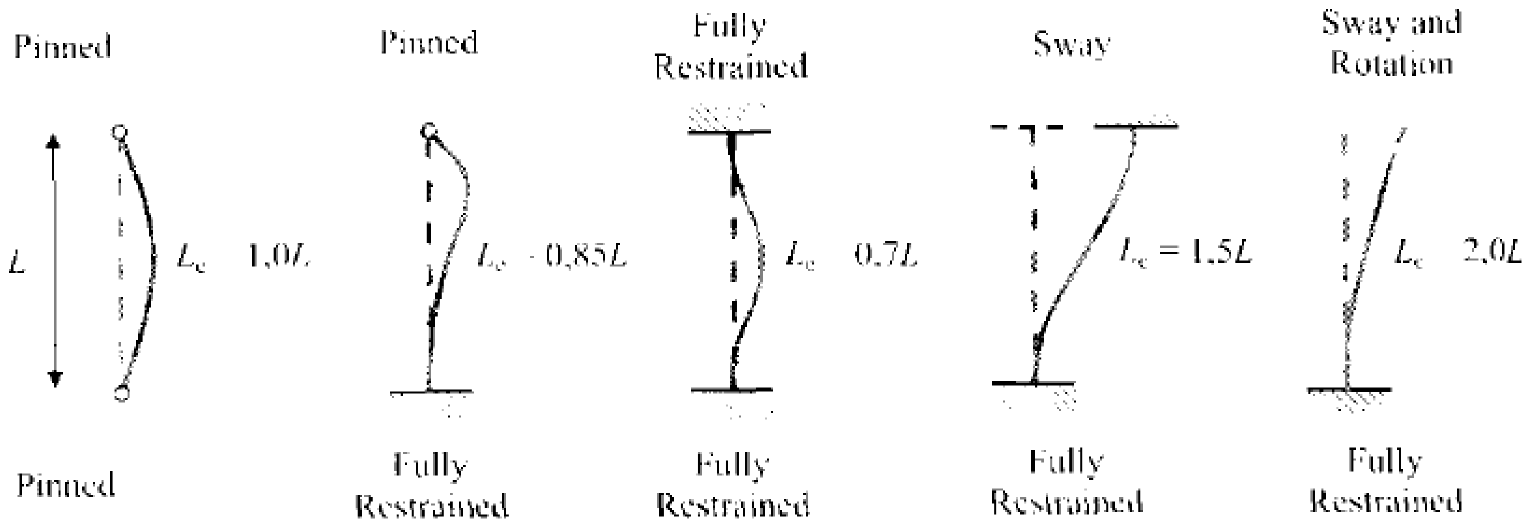
$$\lambda_y = \frac{\ell_{ef,y}}{i_y} = \frac{\ell_{ef,y}}{h/\sqrt{12}}$$

$$\lambda_z = \frac{\ell_{ef,z}}{i_z} = \frac{\ell_{ef,z}}{b/\sqrt{12}}$$

Circle:

$$\lambda = \frac{\ell_{ef}}{i} = \frac{\ell_{ef}}{D/4}$$

Stability of members



Stability of members

$$\frac{\sigma_{c,0,d}}{k_{c,y} f_{c,0,d}} \leq 1 \qquad \frac{\sigma_{c,0,d}}{k_{c,z} f_{c,0,d}} \leq 1$$

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}} \qquad k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}}$$

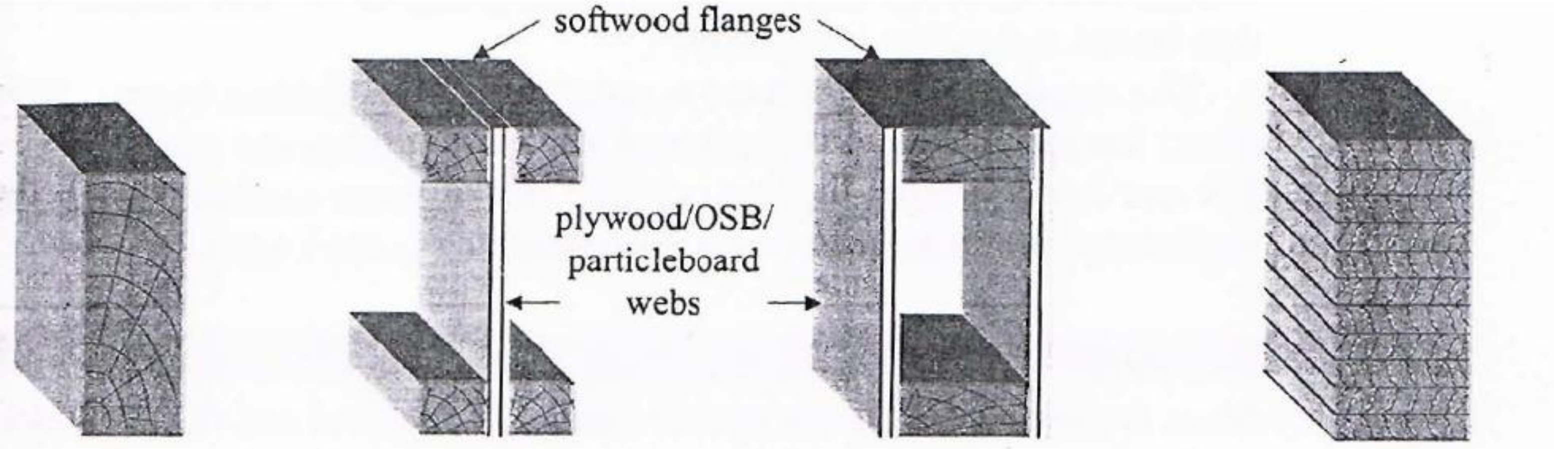
$$k_y = 0,5\left(1 + \beta_c\left(\lambda_{rel,y} - 0,3\right) + \lambda_{rel,y}^2\right)$$

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

$$\beta_c = \begin{cases} 0,2 & \text{for solid timber} \\ 0,1 & \text{for glued laminated timber and LVL} \end{cases}$$

Timber elements subjected to flexure

Flexural members are those elements in a structure that are subjected to bending, and several types and forms of such members are used in timber construction. Typical examples are solid section rectangular beams, floor joists, rafters and purlins.
Other examples include glulam beams and composites (thin webbed beams and thin flanged beams).



(a) solid beam (b) composite I-beam (c) composite box-beam (d) glued laminated beam

Timber elements subjected to flexure

The principal considerations in the design of all beams and floor systems comprise both ultimate and serviceability limit states as follows:

Ultimate Limit States

bending;
shear;
torsion;
bearing;
lateral torsional stability.

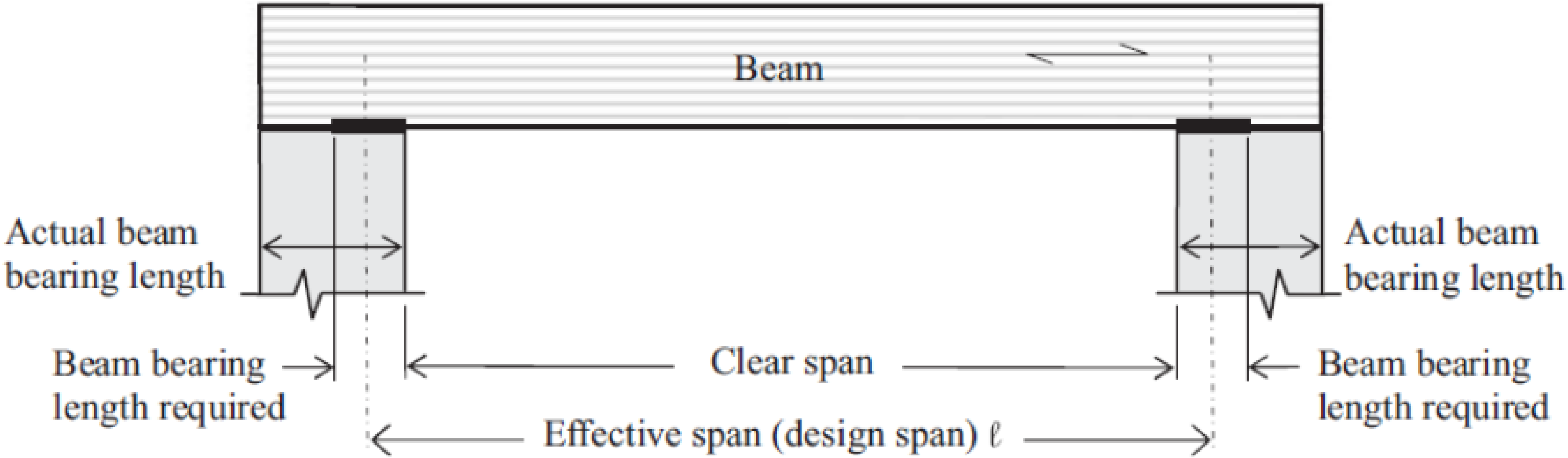
Serviceability Limit States

deflection;
vibration.

Span

The design span, i.e. the effective span of the beam, will be the clear span plus half the bearing length at each end.

For **solid timber beams and built-up flooring beams** required bearing length is **50 mm**.
For **built-up beams with spans up to 12 m** required bearing length is **100 mm**.



Ultimate limit state

Strength according to maximal normal stresses

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

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

$\sigma_{m,y,d}$ and $\sigma_{m,z,d}$ – the design bending stresses about the principal axes

$f_{m,y,d}$ and $f_{m,z,d}$ – the corresponding design bending strengths

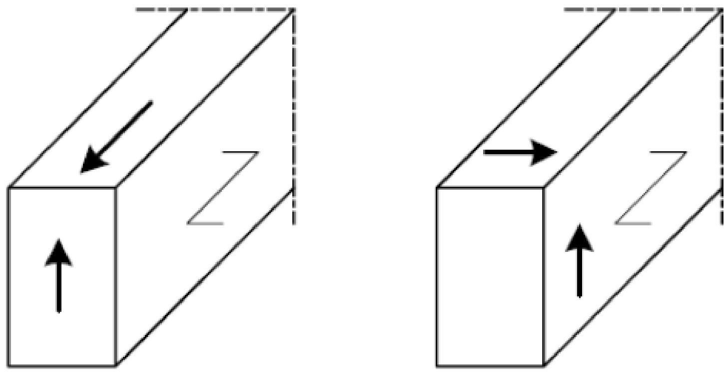
k_m – factor, that makes allowance for re-distribution of stresses and the effect of inhomogeneities of the material in a cross-section

Ultimate limit state

Strength according to maximal shear stresses

For shear with a stress component parallel to the grain as well as for shear with both stress components perpendicular to the grain the following expression shall be satisfied:

$$\tau_d \leq f_{v,d}$$



Member with a shear stress component parallel to the grain

Member with both stress components perpendicular to the grain

Ultimate limit state

Strength according to torsion stresses

The following expression shall be satisfied:

$$\tau_{tor,d} \leq k_{shape} f_{v,d}$$

$$k_{shape} = \begin{cases} 1{,}2 & \text{for a circular cross section} \\ \min \begin{cases} 1+0{,}15 \dfrac{h}{b} \\ 2{,}0 \end{cases} & \text{for a rectangular cross section} \end{cases}$$

$\tau_{tor,d}$ is the design torsional stress;
$f_{v,d}$ is the design shear strength;
k_{shape} is a factor depending on the shape of the cross-section;
h is the larger cross-sectional dimension;
b is the smaller cross-sectional dimension.

Torsional stress

Circle cross-section: $\tau_{tor} = \frac{T}{\pi r^2}$ Rectangular cross-section: $\tau_{tor} = \frac{T}{\alpha h b^2}$

T – torsional moment;
r – radius of cross-section;
h ≥ b;
α – coefficient.

h/b	1,00	1,50	1,75	2,00	2,50	3,00	4,00	6,00	8,00	10,0	∞
α	0,208	0,231	0,239	0,246	0,258	0,267	0,282	0,299	0,307	0,313	0,333

Bearing

The behavior of timber under the action of concentrated loads, e.g. at position of support, is complex and influenced by both the length and locations of bearing. The design compressive strength perpendicular to the grain $f_{c,90,d}$ is used to determine the suitability of the bearing strength.

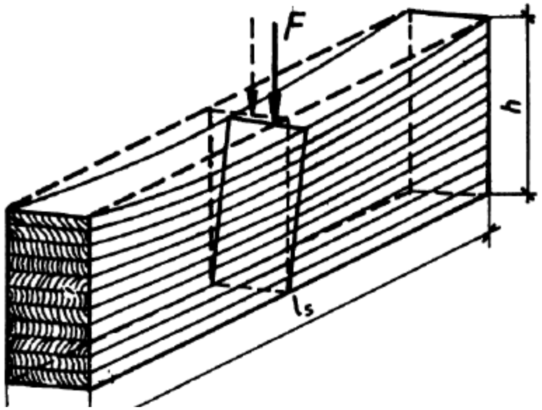
Ultimate limit state

Lateral Stability

The stresses should satisfy the following expression:

$$\sigma_{m,d} \leq k_{crit} f_{m,d}$$

$\sigma_{m,d}$ – is the design bending stress;
$f_{m,d}$ – is the design bending strength;
k_{crit} – is a factor which takes into account the reduced bending strength due to lateral buckling.



Ultimate limit state

Lateral Stability

$$k_{crit} = \begin{cases} 1 & \text{for } \lambda_{rel,m} \le 0{,}75 \\ 1{,}56 - 0{,}75\lambda_{rel,m} & \text{for } 0{,}75 < \lambda_{rel,m} \le 1{,}4 \\ \dfrac{1}{\lambda_{rel,m}^2} & \text{for } 1{,}4 < \lambda_{rel,m} \end{cases}$$

The relative slenderness for bending:

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}}$$

The critical bending stress should:

$$\sigma_{m,crit} = \frac{0{,}78 b^2}{h \ell_{ef}} E_{0,05}$$

$E_{0,05}$ – is the fifth percentile value of modulus of elasticity parallel to grain;
b – is the width of the beam;
h – is the depth of the beam;
l_{ef} – is the effective length of the beam, depending on the support conditions and the load configuration

Ultimate limit state

Lateral Stability

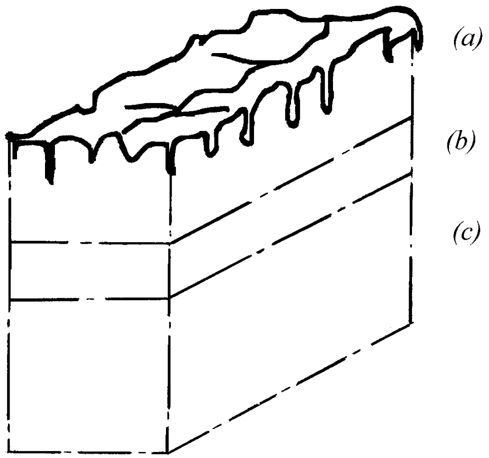
Effective length as a ratio of the span

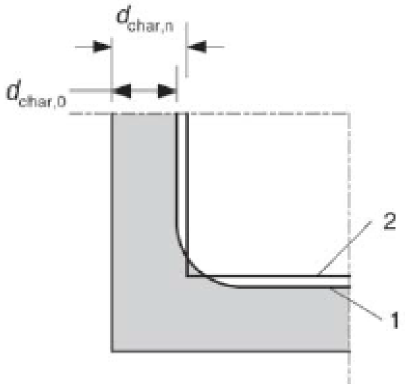
Beam type	Loading type	ℓ_{ef}/ℓ[a]
Simply supported	Constant moment Uniformly distributed load Concentrated force at the middle of the span	1,0 0,9 0,8
Cantilever	Uniformly distributed load Concentrated force at the free end	0,5 0,8

[a] The ratio between the effective length ℓ_{ef} and the span ℓ is valid for a beam with torsionally restrained supports and loaded at the centre of gravity. If the load is applied at the compression edge of the beam, ℓ_{ef} should be increased by $2h$ and may be decreased by $0,5h$ for a load at the tension edge of the beam.

Basic requirements

Changes in timber in case of fire





Key:

1. Border of residual cross-section (real shape)
2. Border of equivalent rectangular residual cross-section

(a) layer of charcoal; *(b)* pyrolised layer; *(c)* timber which is not subjected to fire.

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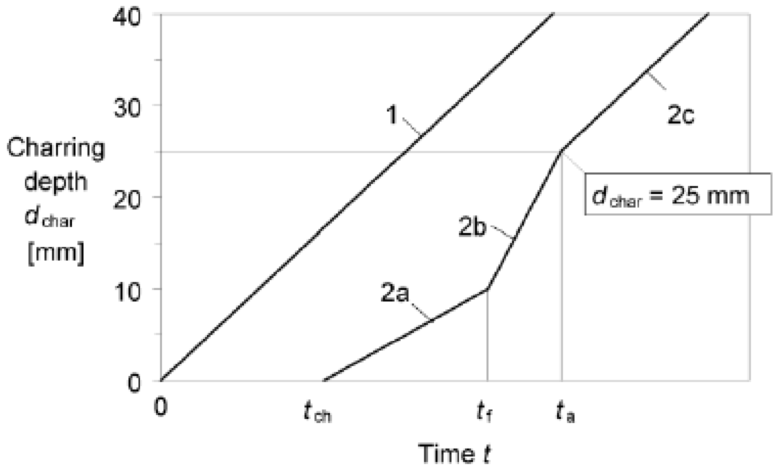
General description of charring for initially protected timber surfaces according to EN 1995-1-2



Key:

1. Relationship for initially unprotected members for charring rate β_0 and β_n
2. Relationship for initially protected members where charring t_{ch} starts before failure of protection t_f:

2a. Charring starts at t_{ch} at a reduced rate when protection is still in place

2b. After protection has fallen off charring increased at double rate β_0 and β_n

2c. After char depth exceeds 25 mm charring rate reduces to β_0 and β_n

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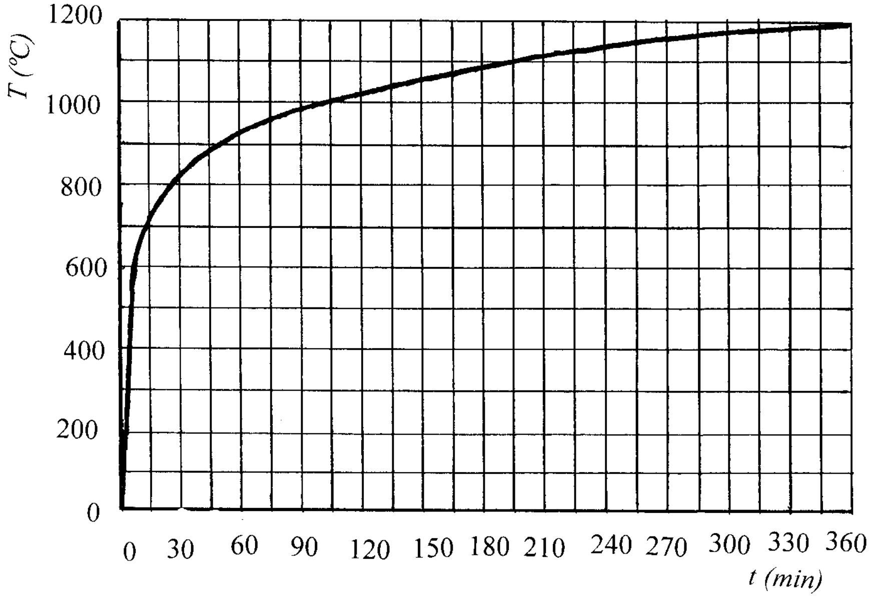
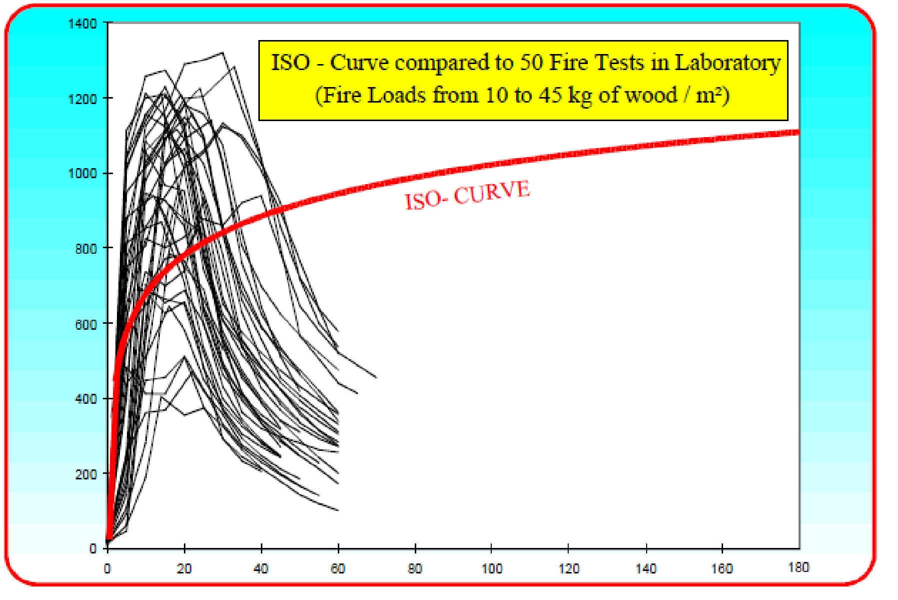
Basic requirements

Standard time/tempirature curve (ISO 834)



Temperatures-time curves from natural fire



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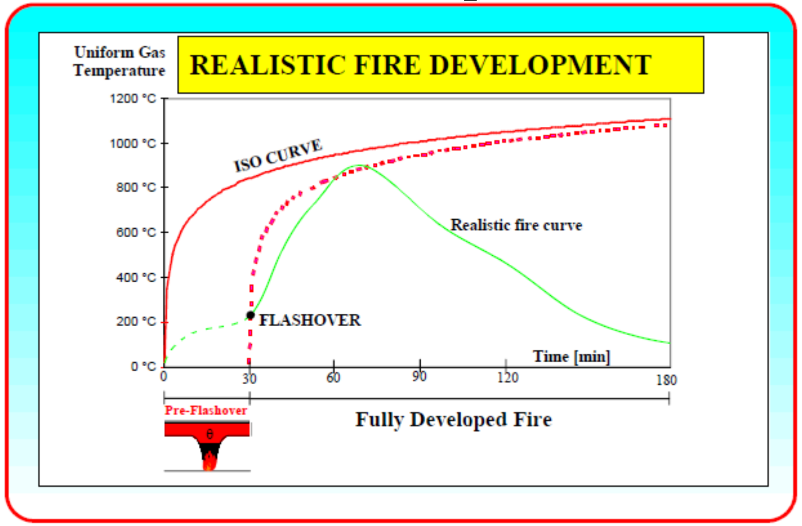
Natural fire phases



The characteristics of real fire include:

A smouldering phase;
A growing phase;
A flashover;
A post flashower phase;
A decreasing phase.

Basic requirements

(1)P Where mechanical resistance in the case of fire is required, structures shall be designed and constructed in such a way that they maintain their load-bearing function during the relevant fire exposure.

(2)P Where fire compartmentation is required, the elements forming the boundaries of the fire compartment, including joints, shall be designed and constructed in such a way that they maintain their separating function during the relevant fire exposure. This shall include, when relevant, ensuring that:

– integrity failure does not occur;
– insulation failure does not occur;.
– thermal radiation from the unexposed side is limited.

Basic requirements

(3)P Deformation criteria shall be applied where the means of protection, or the design criteria for separating elements, require that the deformation of the load-bearing structure is taken into account.

(4) Consideration of the deformation of the load-bearing structure is not necessary in the following cases, as relevant:

– the efficiency of the means of protection has been proved according to 3.4.3 or 5.2;

– the separating elements fulfil the requirements of a nominal fire exposure.

Nominal fire exposure

(1)P For standard fire exposure, elements shall comply with criteria R, E and I as follows:

– separating function only: integrity (criterion E) and, when requested, insulation (criterion I);

– load-bearing function only: mechanical resistance (criterion R);

– separating and load-bearing functions: criteria R, E and, when requested, I.

(2) Criterion R is assumed to be satisfied when the load-bearing function is maintained during the required time of fire exposure.

(3) Criterion I may be assumed to be satisfied where the average temperature rise over the whole of the non-exposed surface is limited to 140 K, and the maximum temperature rise at any point of that surface does not exceed 180 K.

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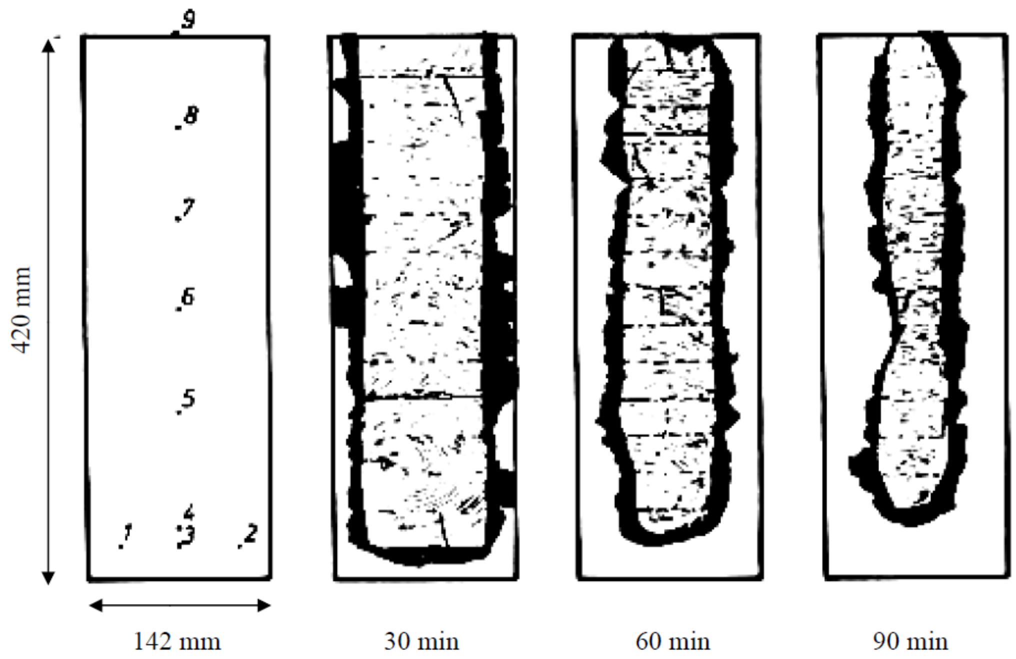
Nominal fire exposure

Laminated beam exposed to 90 minutes standard fire



Design values of materials properties and resistances

(1)P For verification of mechanical resistance, the design values of strength and stiffness properties shall be determined from

$$f_{d,fi} = k_{mod,fi} \frac{f_{20}}{\gamma_{M,fi}}$$

$$S_{d,fi} = k_{mod,fi} \frac{S_{20}}{\gamma_{M,fi}}$$

f_{20} is the 20 % fractile of a strength property at normal temperature;

S_{20} is the 20 % fractile of a stiffness property (modulus of elasticity or shear modulus) at normal temperature;

$k_{mod,fi}$ is the modification factor for fire;

$\gamma_{M,fi}$ is the partial safety factor for timber in fire.

Design values of materials properties and resistances

(3) The 20 % fractile of a strength or a stiffness property should be calculated as:

$$f_{20} = k_{fi} f_k$$

$$S_{20} = k_{fi} S_{05}$$

where:

f_{20} is the 20 % fractile of a strength property at normal temperature;

S_{20} is the 20 % fractile of a stiffness property (modulus of elasticity or shear modulus) at normal temperature;

S_{05} is the 5 % fractile of a stiffness property (modulus of elasticity or shear modulus) at normal temperature

k_{fi} is given in table 2.1.

Design values of materials properties and resistances

Table 2.1 — Values of k_{fi}

	k_{fi}
Solid timber	1,25
Glued-laminated timber	1,15
Wood-based panels	1,15
LVL	1,1
Connections with fasteners in shear with side members of wood and wood-based panels	1,15
Connections with fasteners in shear with side members of steel	1,05
Connections with axially loaded fasteners	1,05

Member analysis

(1) The effect of actions should be determined for time $t = 0$ using combination factors $\psi_{1,1}$ or $\psi_{2,1}$ according to EN 1991-1-2:2002 clause 4.3.1.

(2) As a simplification to (1), the effect of actions $E_{d,fi}$ may be obtained from the analysis for normal temperature as:

$$E_{d,fi} = \eta_{fi} E_d$$

where:

E_d is the design effect of actions for normal temperature design for the fundamental combination of actions, see EN 1990:2002;

η_{fi} is the reduction factor for the design load in the fire situation.

Member analysis

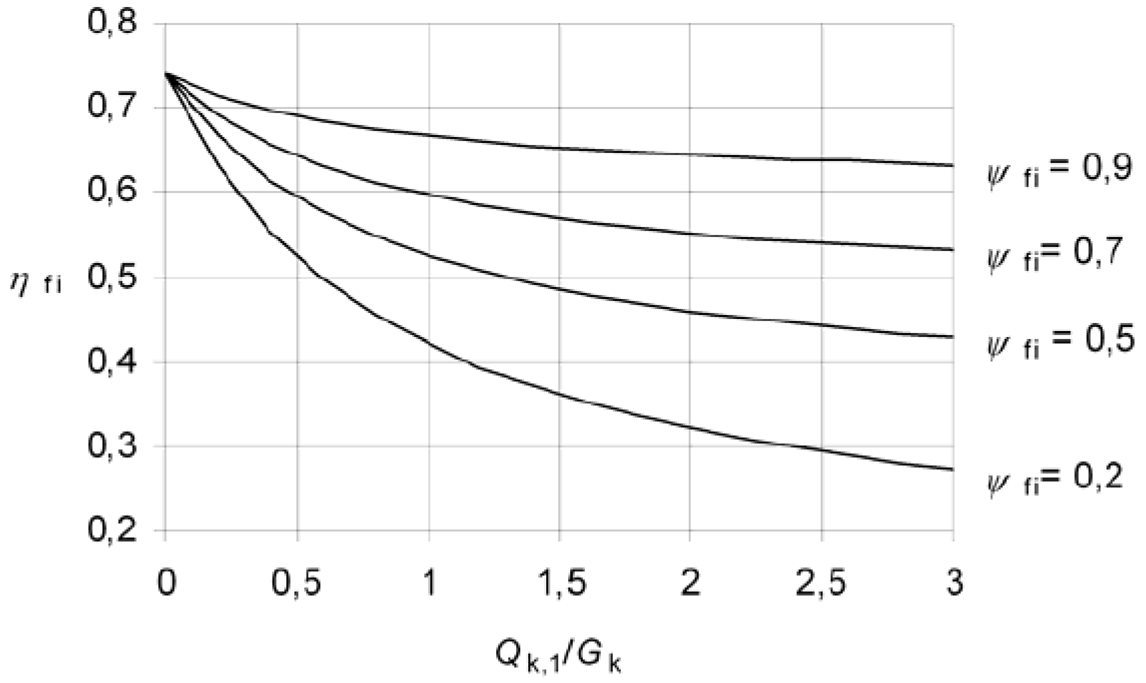
(3) The reduction factor η_{fi} for load combination (6.10) in EN 1990:2002 should be taken as

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}}$$

where:

$Q_{k,1}$	is the characteristic value of the leading variable action;
G_k	is the characteristic value of the permanent action;
γ_G	is the partial factor for permanent actions;
$\gamma_{Q,1}$	is the partial factor for variable action 1;

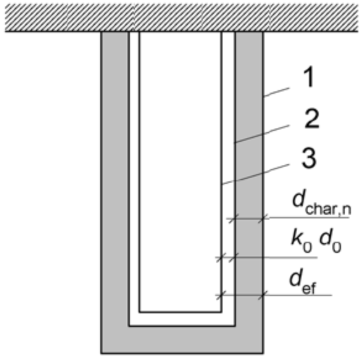
Examples of reduction factor η_{fi} versus load ratio $Q_{k,1}/G_k$



Design procedures for mechanical resistances

Reduced cross-section method

(1) An effective cross-section should be calculated by reducing the initial cross-section



1. Initial surface of member
2. Border of residual cross-section
3. Border of effective cross-section

$d_{char,n}$	notional charring depth	
d_0	zero strength layer: d_0 = 7 mm	
k_0	$k_0 = 1{,}0$	for $t \geq 20$ minutes;
	$k_0 = t/20$	for $t < 20$ minutes
t	time of fire exposure	
d_{ef}	effective charring depth	

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

with:

d_0 = 7 mm

$$d_{char,n} = \beta_n t$$

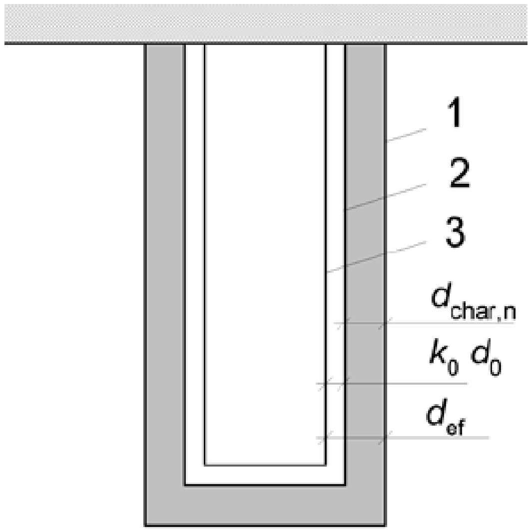
where:

$d_{char,n}$ is the notional design charring depth, which incorporates the effect of corner roundings;

β_n is the notional design charring rate, the magnitude of which includes for the effect of corner roundings and fissures.

Reduced cross-section method

Definition of residual cross-section and effective cross-section



Key

1 Initial surface of member

2 Border of residual cross-section

3 Border of effective cross-section

Reduced cross-section method

Definition of residual cross-section and effective cross-section

For unprotected surfaces, k_0 should be determined from table

	k_0
t < 20 minutes	$t/20$
$t \geq$ 20 minutes	1,0

Reduced properties method

(1) The following rules apply to rectangular cross-sections of softwood exposed to fire on three or four sides and round cross-sections exposed along their whole perimeter.

(2) The residual cross-section should be determined

(3) For t ≥ 20 minutes, the modification factor for fire $k_{mod,fi}$,

– for bending strength:

$$k_{mod,fi} = 1{,}0 - \frac{1}{200}\frac{p}{A_r}$$

– for compressive strength:

$$k_{mod,fi} = 1{,}0 - \frac{1}{125}\frac{p}{A_r}$$

– for tensile strength and modulus of elasticity:

$$k_{mod,fi} = 1{,}0 - \frac{1}{330}\frac{p}{A_r}$$

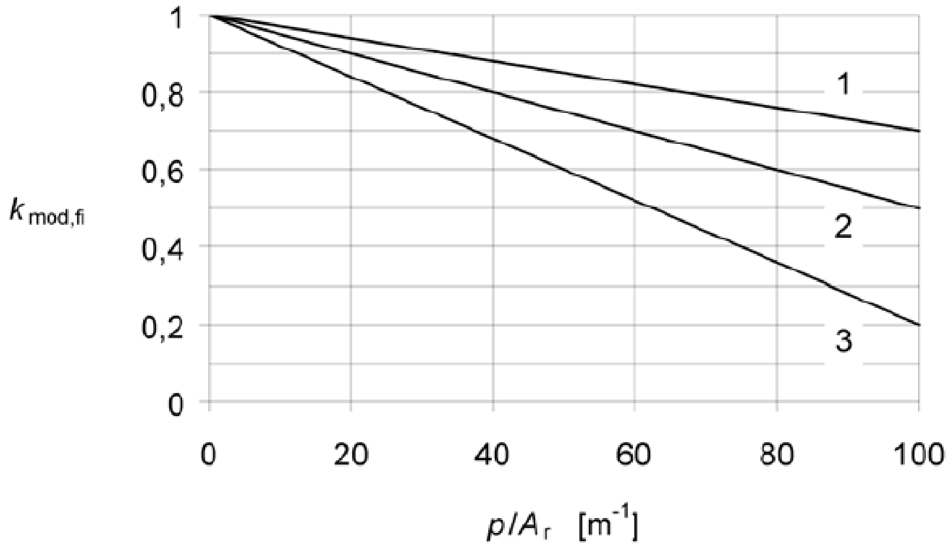
where:

p is the perimeter of the fire exposed residual cross-section, in metres;

A_r is the area of the residual cross-section, in m^2.

Reduced properties method

(4) For unprotected and protected members, for time t = 0 the modification factor for fire should be taken as $k_{mod,fi}$ = 1. For unprotected members, for $0 \leq t \leq 20$ minutes the modification factor may be determined by linear interpolation.



Key:

1 Tensile strength, Modulus of elasticity
2 Bending strength
3 Compressive strength

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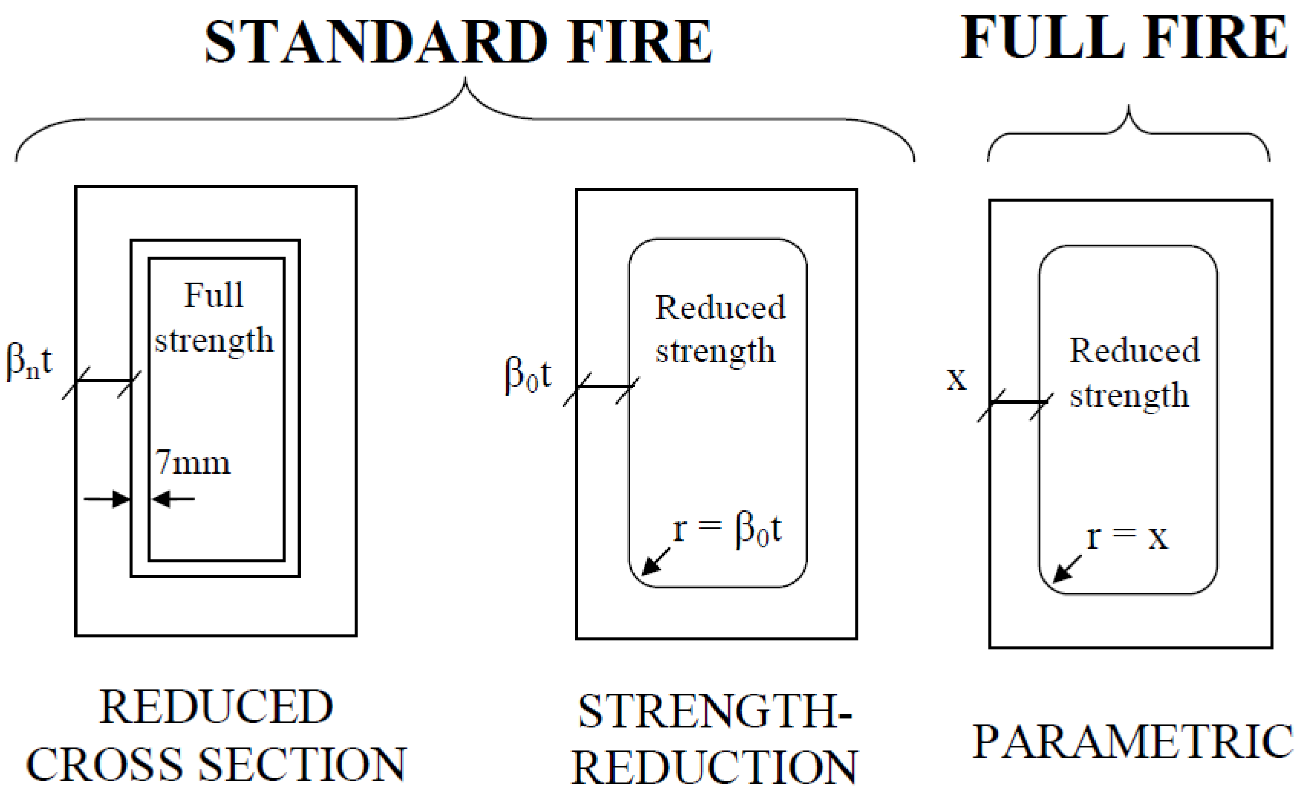
Survey of calculation methods



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Survey of calculation methods

REDUCED CROSS SECTION

$\beta_n = 0.70$ mm/min for laminated
$\beta_n = 0.08$ mm/min for conifer

$x = \beta_n t + 7$mm pyrolysis

Full strength and stiffness
in the reduced cross section

STRENGTH-REDUCTION / PARAMETRIC

$\beta_0 = 0.65$ mm/min for laminated and conifer

STRENGTH-REDUCTION: $x = \beta_0 t$

PARAMETRIC: $x = x(t, O, q)$

For both:

$k_{mod,fi} = 1 - k_{fi}\,(P/A_r)$

P = Perimetre, A_r = Residual Area

$k_{fi} = 1/125$ for compression
$k_{fi} = 1/200$ for bending
$k_{fi} = 1/330$ for tension, shear and stiffness

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T.Saknite, D.Serdjuks

Fire Design of Arch Typr Timber Roof

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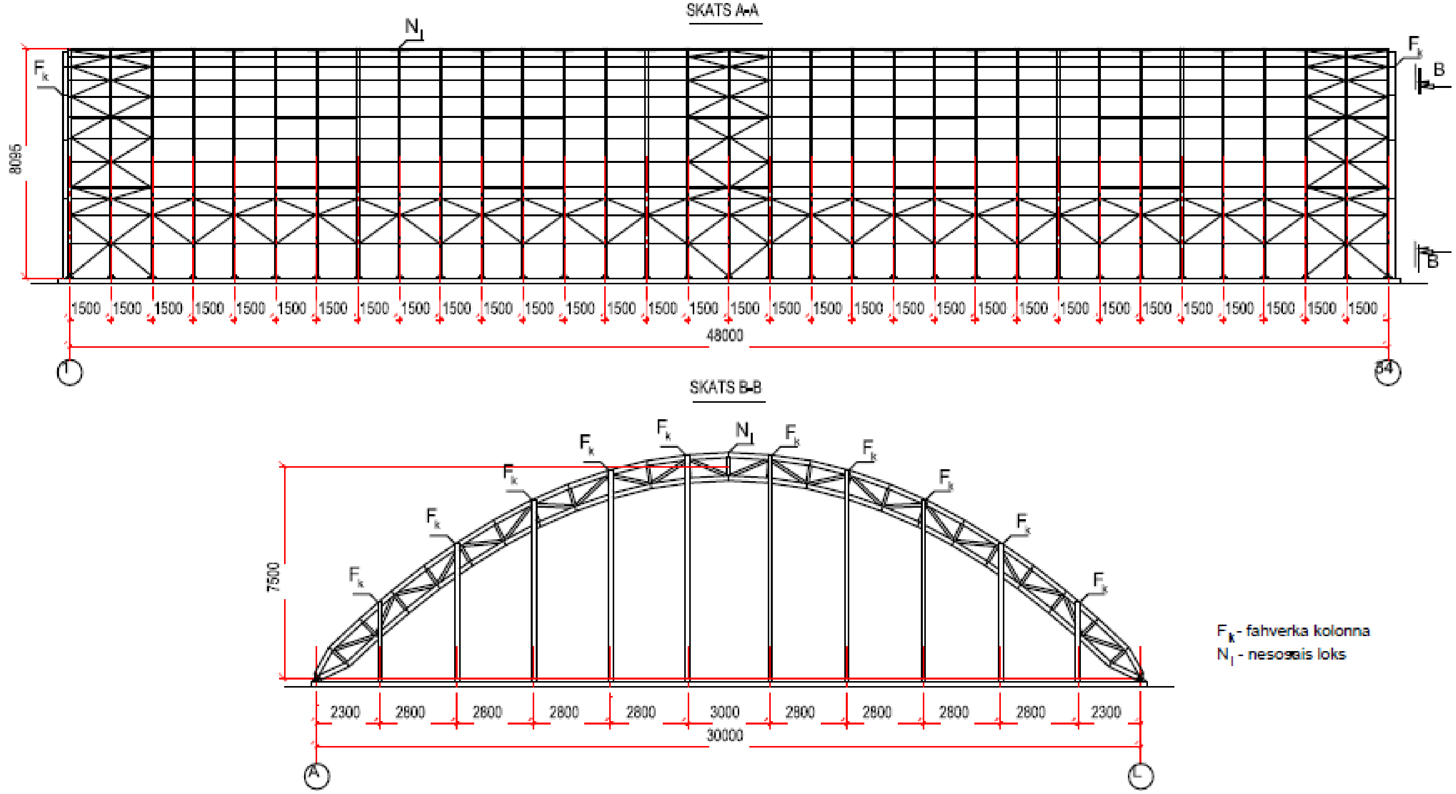
Object of investigation



Object of investigation

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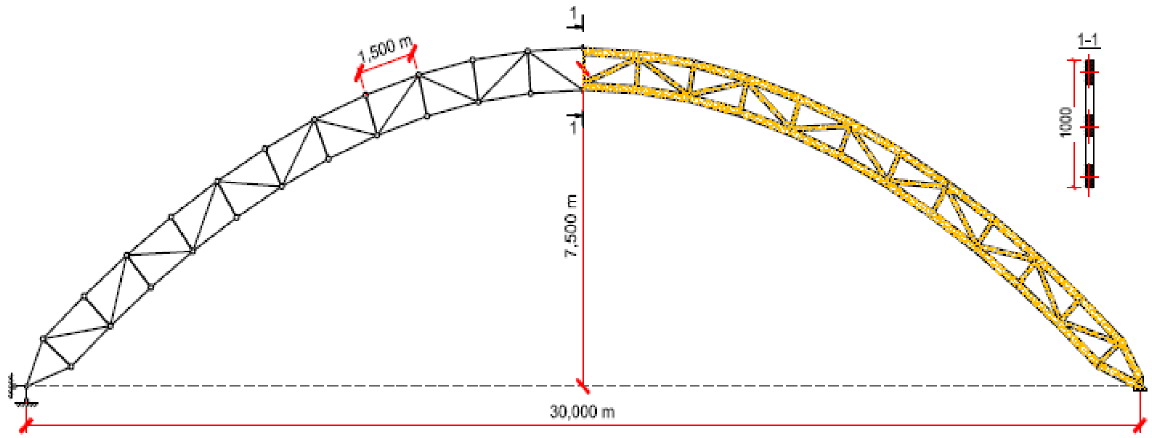
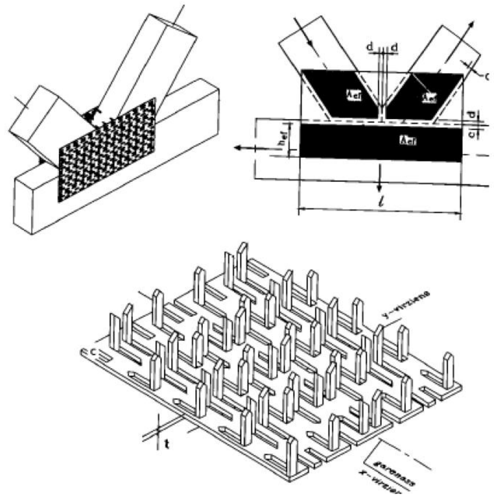
The aim of investigation

The aim of the current investigation is to evaluated rational geometrical parameters of the two hinge lattice arch in case of fire action. The dependence of material consumption on the geometrical parameters of the lattice arch must be obtained for the purposes.



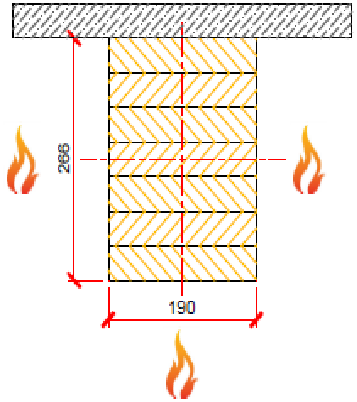


Choice of design method for arch-type timber roof

Design procedures for mechanical resistance explained in EN 1995-1-2 include:

- reduced cross-section method;
- reduced properties method;
- parametric design method.

Choice of the method for fire resistance analise was carried out on the base of results comparison, which were obtained for the glued beam:



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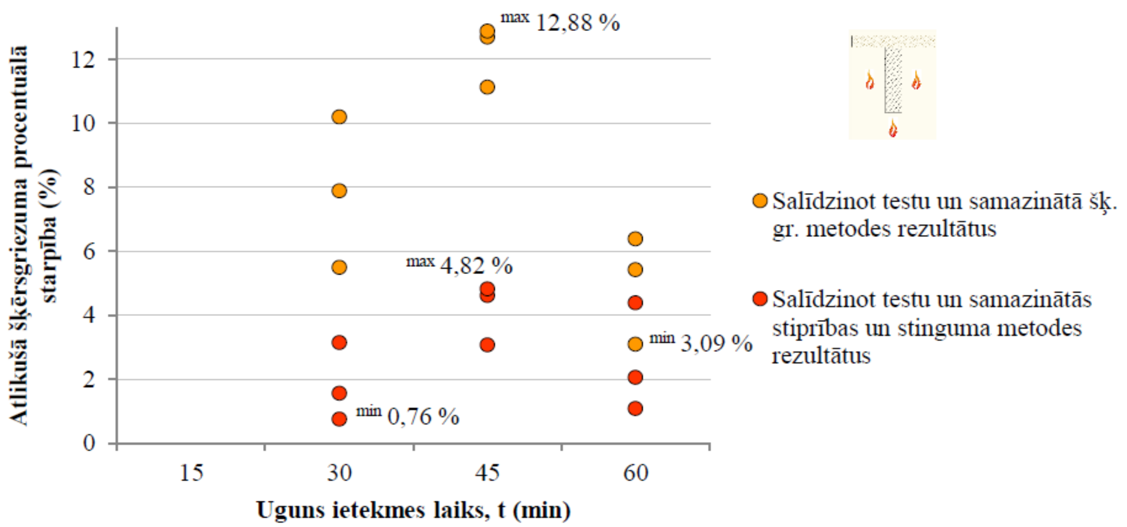
The differences in % between the values of residual cross-section determined by the experiment and by reduced cross-section and reduced strength and stiffness method



Comparison of results obtained analytically and by the experiment indicates that the values of residual cross-sections that were obtained by the reduced stress-stiffness method differed from the experimentally obtained values by 0.76-4.82%. Therefore, for the fire resistance analyse of arch-type timber roof will be used reduced stress-stiffness method.

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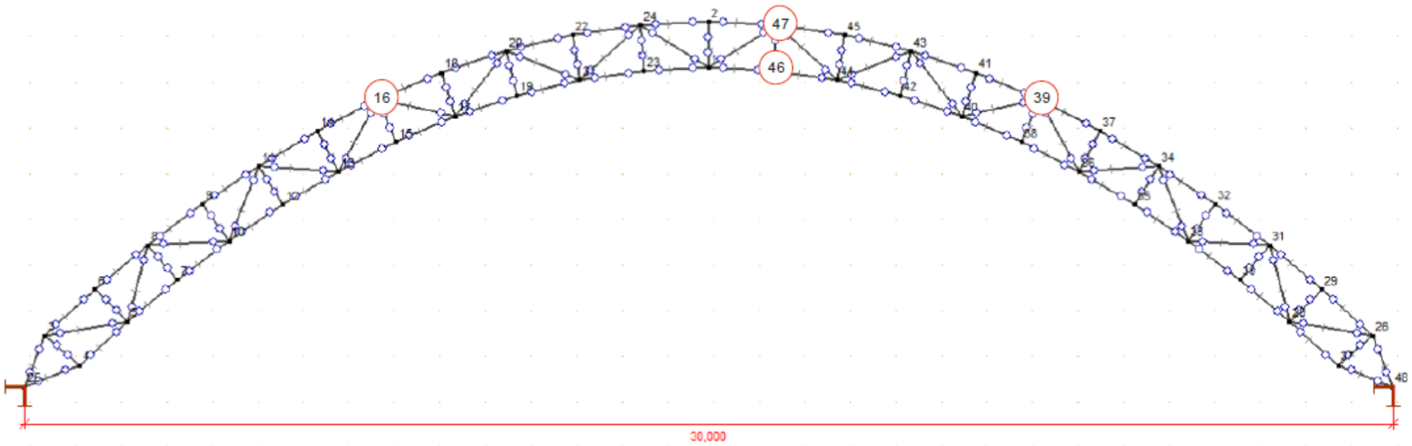
Fire design of arch-type timber roof

Fire design of purlins

Nodes numbering of lattice timber arch and position of the most heavy loaded purlins



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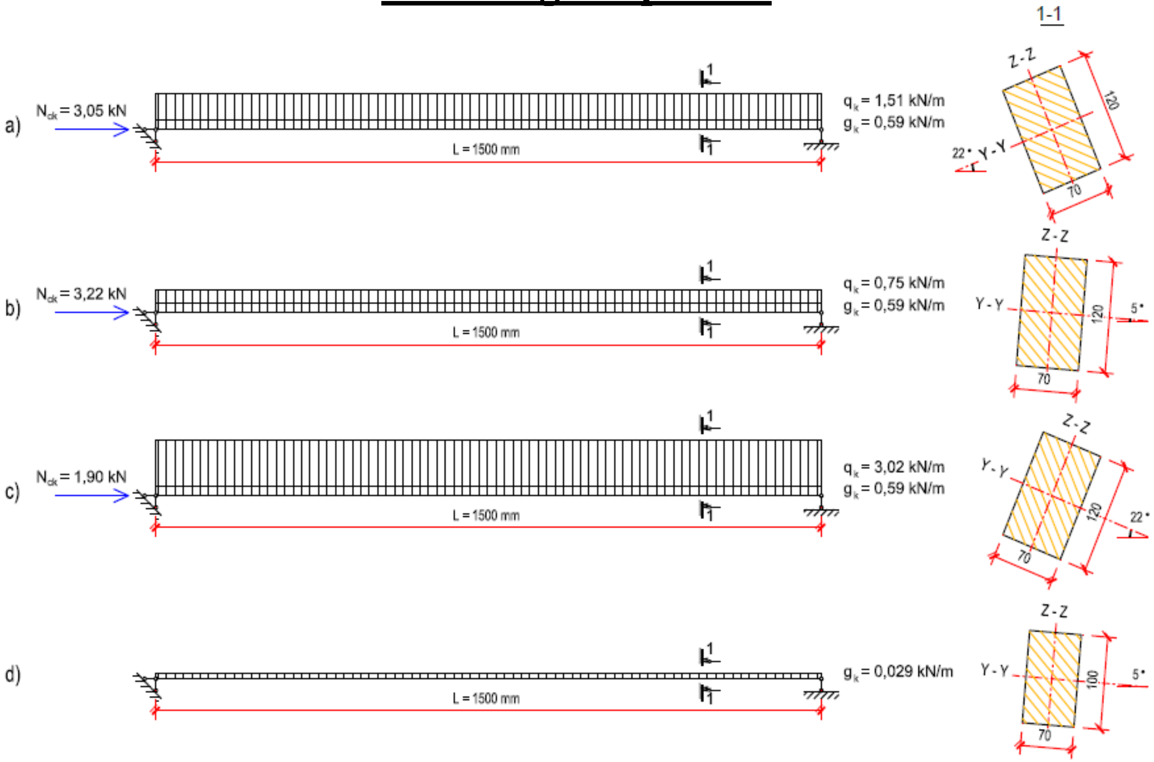
Fire design of arch-type timber roof

Fire design of purlins



Fire design of arch-type timber roof

Fire design of purlins

The strength of the purlins as the member subjected to compressing with the bending, was checked by the equations:

$$\left(\frac{\sigma_{c,o,d.fi}}{f_{c,o,d.fi}}\right)^2 + \frac{\sigma_{m,y,d.fi}}{f_{m,y,d.fi}} + k_m \cdot \frac{\sigma_{m,z,d.fi}}{f_{m,z,d.fi}} \leq 1$$

$$\left(\frac{\sigma_{c,o,d.fi}}{f_{c,o,d.fi}}\right)^2 + k_m \cdot \frac{\sigma_{m,y,d.fi}}{f_{m,y,d.fi}} + \frac{\sigma_{m,z,d.fi}}{f_{m,z,d.fi}} \leq 1$$

Fire design of arch-type timber roof

Fire design of purlins

The stability of the purlins as the member subjected to compressing with the bending, was checked by the equations:

$$\frac{\sigma_{c,o,d.fi}}{k_{c,y.fi} \cdot f_{c,o,d.fi}} + \frac{\sigma_{m,y,d.fi}}{f_{m,y,d.fi}} + k_m \cdot \frac{\sigma_{m,z,d.fi}}{f_{m,z,d.fi}} \leq 1$$

$$\frac{\sigma_{c,o,d.fi}}{k_{c,z.fi} \cdot f_{c,o,d.fi}} + k_m \cdot \frac{\sigma_{m,y,d.fi}}{f_{m,y,d.fi}} + \frac{\sigma_{m,z,d.fi}}{f_{m,z,d.fi}} \leq 1$$

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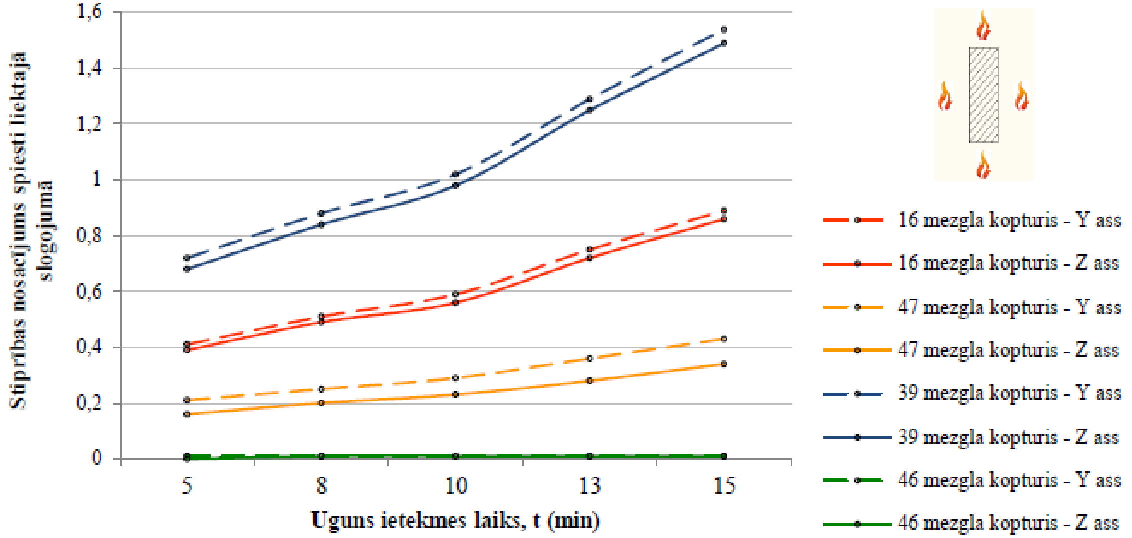
Fire design of purlins

The dependence of safety storage from the time of fire exposure for the four most heavy loaded purlins obtained by the strength condition



It was shown, that the strength condition was not satisfied after 9.8 minutes of fire exposure.

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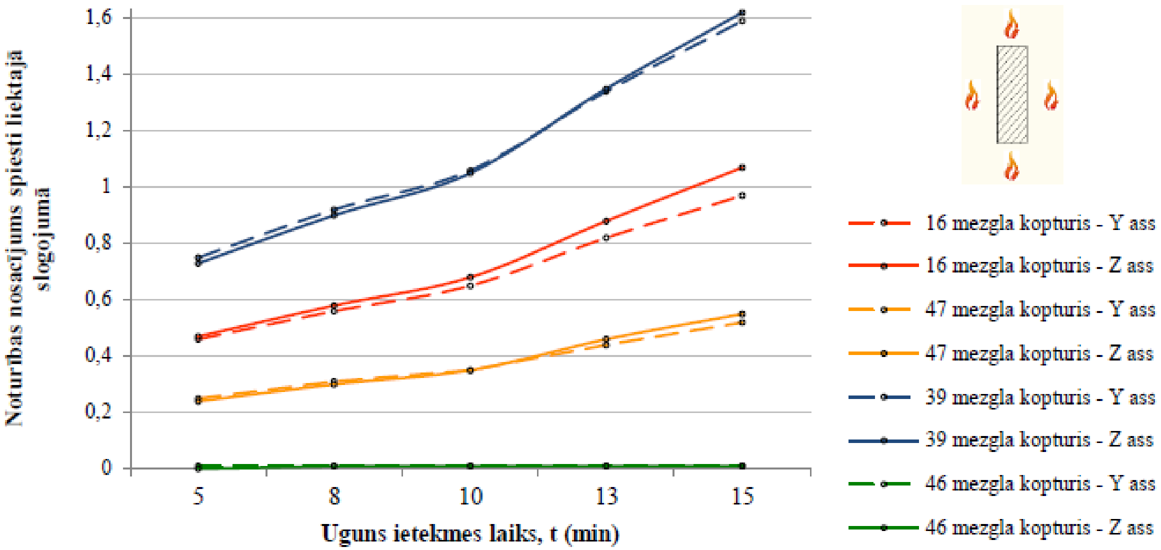
Fire design of purlins

The dependence of safety storage from the time of fire exposure for the four most heavy loaded purlins obtained by the stability condition



It was shown, that the stability condition was not satisfied after 9.4 minutes of fire exposure.

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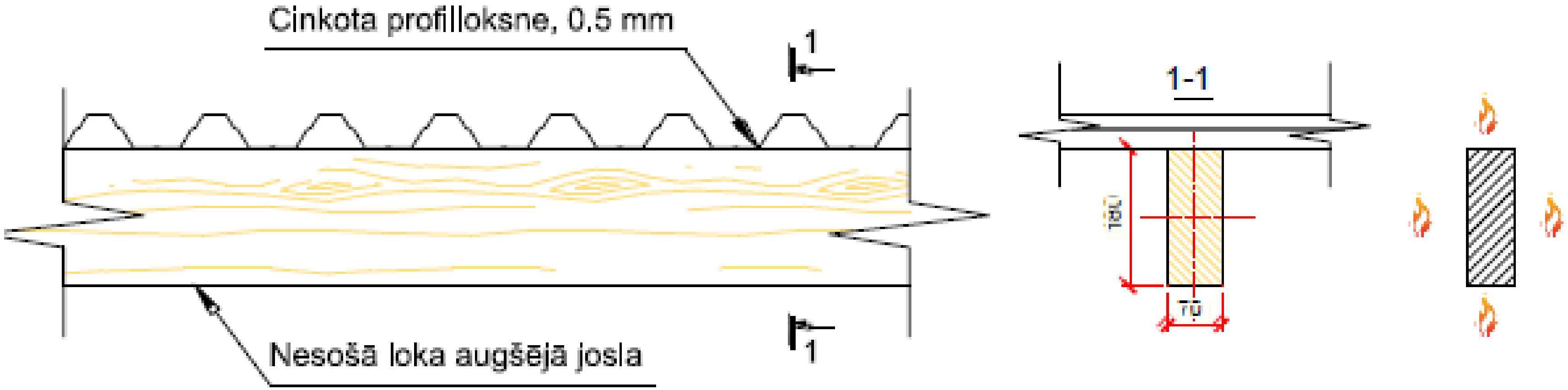
Scheme of the fire action at the top chord of the lattice arch



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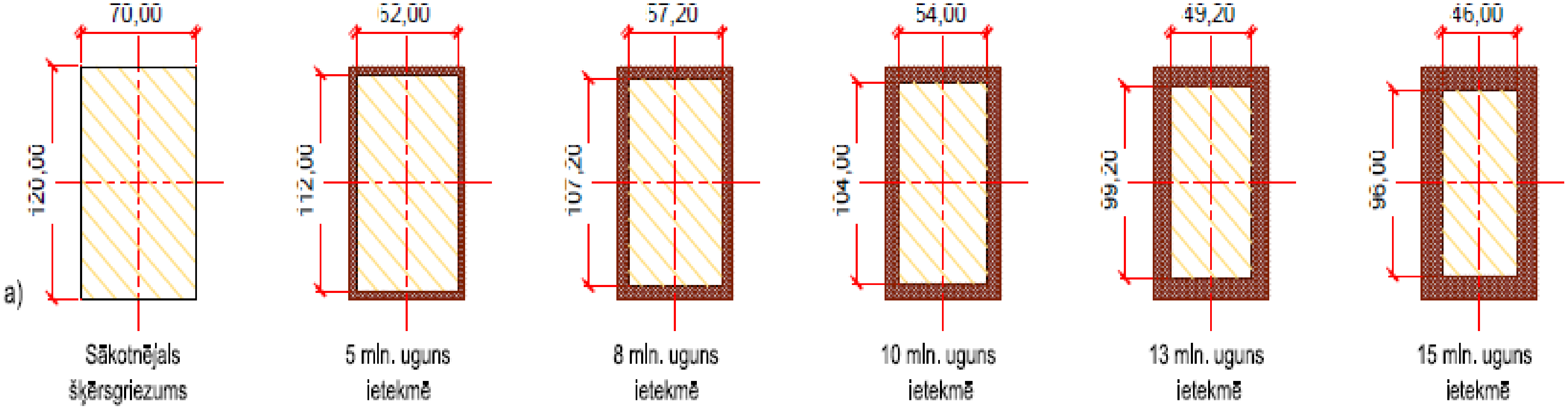
Decrease of geometrical parameters of the top chord elements in case of fire action



The dependences indicates, that strength condition for the top chord of the timber lattice arch is not satisfied just after 11.1 minutes of fire exposure. The buckling of the top chord occurs after 10.5 minutes of fire exposure relatively y-axis.

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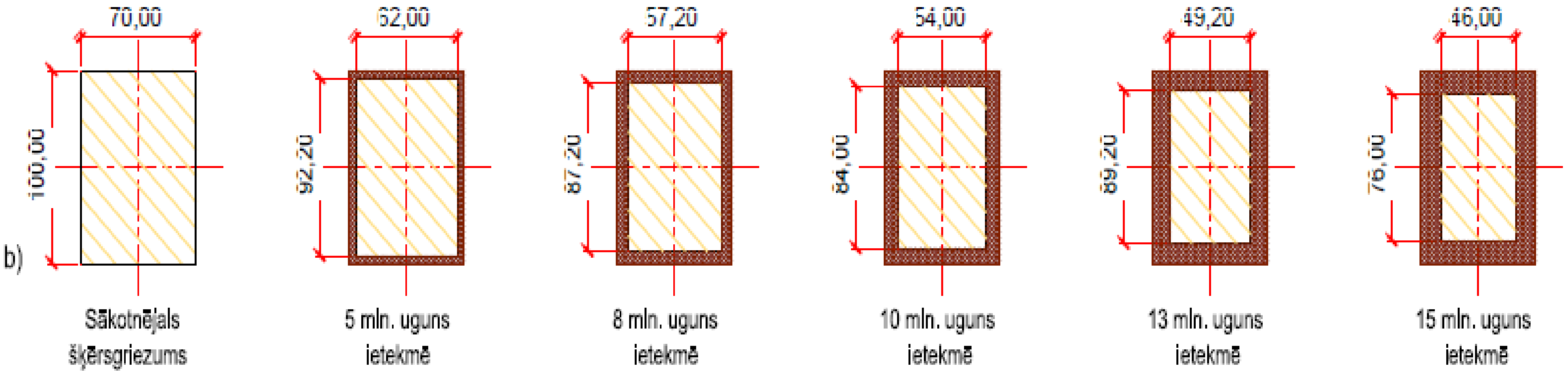
Decrease of geometrical parameters of the bottom chord elements in case of fire action



The buckling of the bottom chord occurs after 7.2 minutes of fire exposure relatively z-axis.

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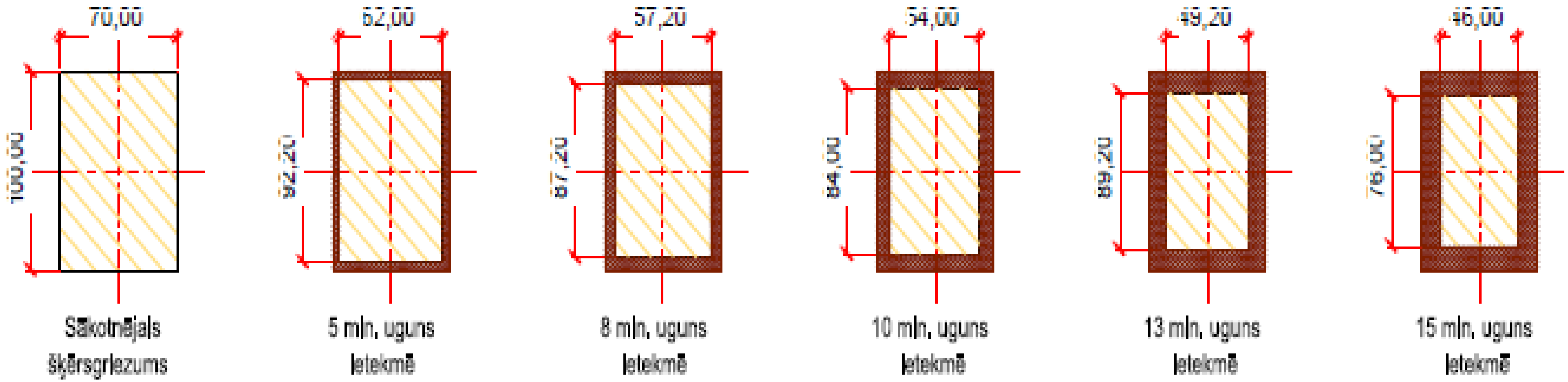
Decrease of geometrical parameters of the lattice elements in case of fire action



It means that the fire resistance of the lattice timber arch is equal to R7.2, what is more than two times less, than the minimum required value R15.

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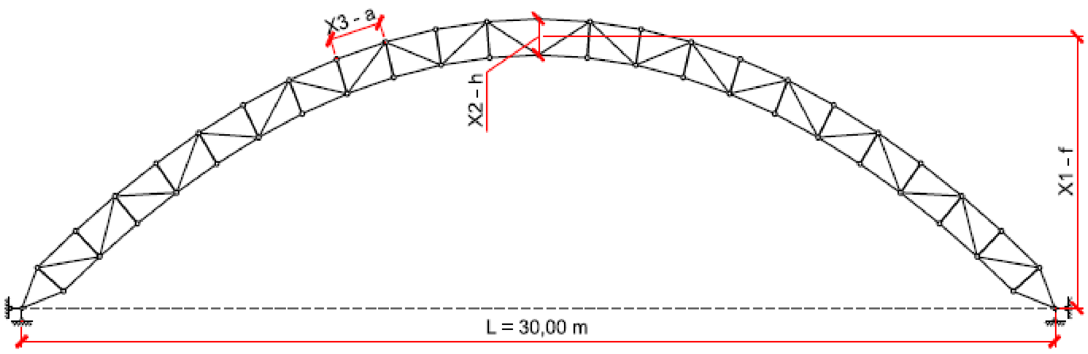
Main geometrical parameters of the lattice arch

Vadāmais faktors		Līmenis			Variācijas intervāls (m)
		Augšējais +1	Nulles 0	Zemākais -1	
$X_1 - f$	Loka ass līnijas izliece (m)	7,5	6,5	5,5	1,0
$X_2 - h$	Loka šķērsgriezuma augstums (m)	1,0	0,75	0,5	0,25
$X_3 - a$	Loka augšējās joslas paneļa garums (m)	1,5	1,25	1,0	0,25



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The dependence of material consumption on the geometrical parameters of the lattice arch

$$Y' = b_0 + b_1 \cdot X_1 + b_2 \cdot X_2 + b_3 \cdot X_3 + b_{12} \cdot X_1 \cdot X_2 + b_{13} \cdot X_1 \cdot X_3 + b_{23} \cdot X_2 \cdot X_3 + b_{123} \cdot X_1 \cdot X_2 \cdot X_3 + b_{11} \cdot {X_1}^2 + b_{22} \cdot {X_2}^2 + b_{33} \cdot {X_3}^2 \implies$$

Values of the coefficients

	Izejot no materiāla patēriņa		**Izejot no maksimāliem ass spēkiem loka augšējā joslā**	
	Neievērojot uguns ietekmi	**Ņemot vērā uguns ietekmi**	**Neievērojot uguns ietekmi**	**Ņemot vērā uguns ietekmi**
b_0	9,46184	10,9741	547,358	551,152
b_1	−1,50739	−1,37964	−22,7047	−23,5849
b_2	−5,69789	−6,27883	−411,36	−407,507
b_3	−1,44156	−3,74211	−123,181	−124,552
b_{12}	0,304	0,397667	−0,262333	−0,711667
b_{13}	0,07	0,231333	1,528	1,30067
b_{23}	−1,956	−3,01333	−5,87333	−5,21467
b_{123}	0,00	0,00	0,00	0,00
b_{11}	0,0872222	0,0575	0,9335	1,03778
b_{22}	3,52622	4,776	190,216	190,946
b_{33}	1,13156	1,992	46,5093	47,4604

Evaluation of the rational geometrical parameters of the lattice arch

$$\begin{cases} \frac{\partial G}{\partial f} = b_1 + b_{12} \cdot X_2 + b_{13} \cdot X_3 + b_{123} \cdot X_2 \cdot X_3 + 2 \cdot b_{11} \cdot X_1 = 0 \\ \frac{\partial G}{\partial h} = b_2 + b_{12} \cdot X_1 + b_{23} \cdot X_3 + b_{123} \cdot X_1 \cdot X_3 + 2 \cdot b_{22} \cdot X_2 = 0 \\ \frac{\partial G}{\partial a} = b_3 + b_{13} \cdot X_1 + b_{23} \cdot X_2 + b_{123} \cdot X_1 \cdot X_2 + 2 \cdot b_{33} \cdot X_3 = 0 \end{cases}$$

X_1 – height of the lattice arch; X_2 – height of the lattice arch cross-section; X_3 – distance between the nodes of the top chord

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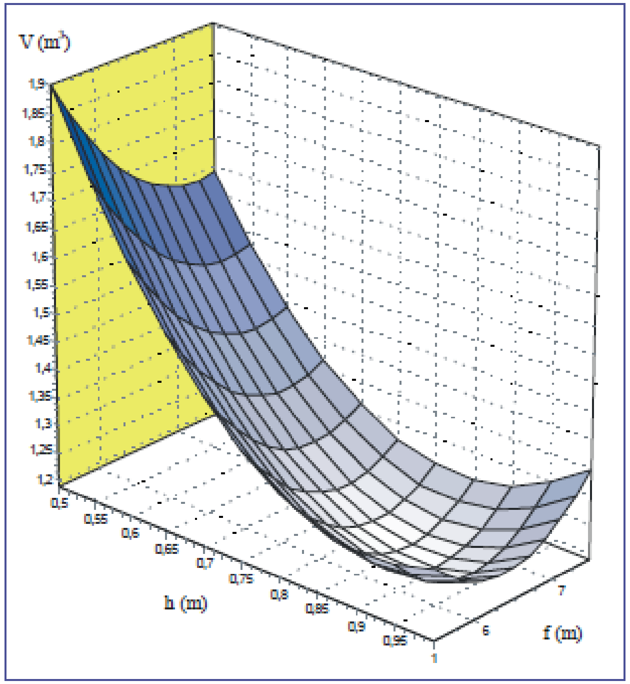
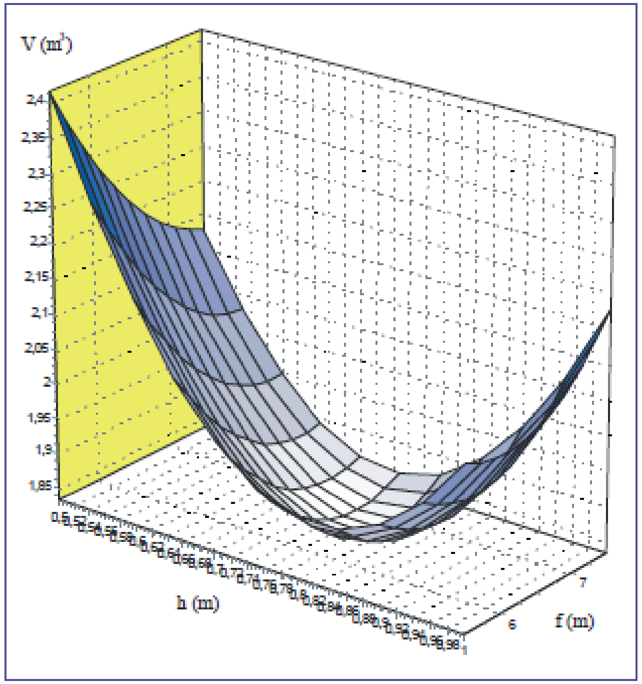
Evaluation of the rational geometrical parameters of the lattice arch

The dependence of material consumption on the geometrical parameters of the lattice arch

a)



b)



a) – fire action is not taked in to account; *b)* – taking fire action in to account.

Evaluation of the rational geometrical parameters of the lattice arch

Mainīgais faktors		Izejot no materiāla patēriņa viedokļa	Izejot no maksimālās ass spēka vērtības loka augšējā joslā
Neievērojot uguns ietekmi			
$X_1 - f$	Loka ass līnijas izliece (m)	6,70	11,30 *
$X_2 - h$	Loka šķērsgriezuma augstums (m)	0,80	1,10 *
$X_3 - a$	Loka augšējās joslas paneļa garums (m)	1,15	1,20
Ņemot vērā uguns ietekmi			
$X_1 - f$	Loka ass līnijas izliece (m)	7,85 *	10,95 *
$X_2 - h$	Loka šķērsgriezuma augstums (m)	0,60	1,10 *
$X_3 - a$	Loka augšējās joslas paneļa garums (m)	0,95 *	1,20

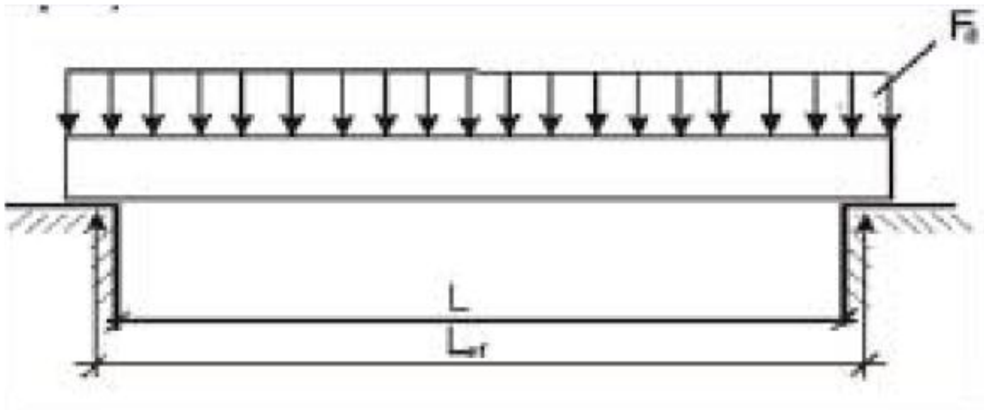
Acknowledgement

The research leading to these results has received the funding from Latvia state research programme under grant agreement "Innovative Materials and Smart Technologies for Environmental Safety, **IMATEH**". **Project ID 1854 (task 3)**.

5. Examples

Design of beam subjected to fire

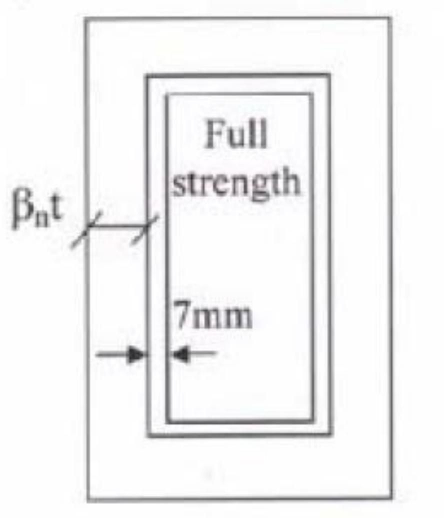
Reduced cross-section method (example 1)



Data: $L = 5$ m; $L_{atb} = 0{,}1$m; $b = 100$ mm; $d = 300$ mm; $G_k = 0{,}13$ kN/m; $Q_k = 1{,}8$ kN/m; C16; S.C. 2.

Task

To determine fire resistance of the beam. The beam is subjected to fire from the **four sides**.



<u>Solution</u>

1. **<u>Loads</u>**

$$F_{d,fi} = G_{k,j} + \psi_{1,1} Q_{k,1} = 0,13 + 0,7 \cdot 1,8 = 1,39 kN/m$$

2. **<u>Internal forces</u>**

$$M_{y,d,fi} = \frac{F_{d,fi} \cdot L_{ef}^2}{8} = \frac{1,39 \cdot 5,1^2}{8} = 4,52 kN \cdot m = 4,52 \cdot 10^6 N \cdot mm \;;$$

$$V_{d,fi} = \frac{F_{d,fi} \cdot L_{ef}}{2} = \frac{1,39 \cdot 5,1}{2} = 4,92kN$$;

$$L_{ef} = L + L_{atb} = 5 + 0,1 = 5,1\ m$$.

3. Geometrical parameters of the reduced section

$$X = \beta_n \cdot t + 7mm = 0,8t + 7mm$$
$$d_{f,i} = d - 2 \cdot X = 300 - 2 \cdot (0,8t + t) = 286 - 1,6t, \quad mm$$
$$b_{f,i} = b - 2 \cdot X = 100 - 2 \cdot (0,8t + t) = 86 - 1,6t, \quad mm$$

Moment of inertia relatively y axis:

$$\Im_{y.fi} = \frac{b_{fi} \cdot d_{fi}^3}{12} = \frac{(86 - 1,6t)(286 - 1,6t)^3}{12} = 0,0833(86 - 1,6t)(286 - 1,6)^3$$

Šķērsgriezuma laukums:

$$A_{fi} = b_{fi} \cdot d_{fi} = (86 - 1,6t)(\mathbf{286} - 1,6t)$$, mm^2

Modulus of section relatively y axis:

$$W_{y,fi} = \frac{b_{fi} \cdot d_{fi}^3}{6} = 0,167(86 - 1,6t)(286 - 1,6t)^2$$, mm^3

4. Fire resistance of the beam basing on the normal stresses strength condition:

$$\frac{\sigma_{m,y,d,fi}}{f_{m,y,d,fi}} \leq 1,0$$

$$f_{m,y,d,fi} = k_{mod,fi} \cdot \frac{f_{20}}{\gamma_{M,fi}} = 1 \cdot \frac{20}{1} = \frac{20N}{mm^2}$$

$$f_{20} = k_{fi} \cdot f_{m,y,k} = 1,25 \cdot 16 = 20\frac{N}{mm^2}$$

$k_{fi} = 1,25$ ("masīva koksne)"

$k_{mod,fi} = 1$ (aprēķinam pēc reducētā šķērsgriezuma metodes)

5. The normal stresses strength condition:

$$\frac{4,52 \cdot 10^6 / 0,167(86-1,6t)(286-1,6t)^2}{20} \leq 1,0$$

<u>At t = 37 min</u>:

$$\frac{\sigma_{m,y,d,fi}}{f_{m,y,d,fi}} = 0,974 \leq 1,0$$

It means, that fire resistance of the beam basing on the normal stresses strength condition is **R30.**

5. Fire resistance of the beam basing on the shear stresses strength condition:

$$\frac{\tau_{d,fi}}{f_{v,d,fi}} \leq 1,0$$

$$\tau_{d,fi} = \frac{3V_{d,fi}}{2A_{fi}} = \frac{3 \cdot 3,54 \cdot 10^3}{2(86 - 1,6t)(286 - 1,6t)};$$

$$f_{v,d,fi} = k_{mod,fi} \cdot \frac{f_{20}}{\gamma_{M,fi}} = 1 \cdot \frac{2,25}{1} = 2,\frac{25N}{mm^2}$$

$$f_{20} = k_{fi} \cdot f_{v,k} = 1,25 \cdot 1,8 = 2,25\frac{N}{mm^2}$$

The shear stresses strength condition:

$$\frac{3 \cdot 3,54 \cdot 10^3 / 2(86 - 1,6t)(\mathbf{286} - 1,6t)}{2,25} \leq 1,0$$

<u>At t = 45 min</u>

$$\frac{\tau_{d,fi}}{f_{v,d,fi}} = 0,8 \leq 1,0$$

It means, that fire resistance of the beam basing on the shear stresses strength condition is **R45.**

6. **Fire resistance of the beam basing on the global stability condition (prevention of the lateral torsional bucling):**

$$\frac{\sigma_{m,d,fi}}{k_{critfi} f_{m,d,fi}} \leq 1,0$$

$k_{crit} = 1$ **at slenderness** $\lambda_{vel,m} \leq 0,75$

$k_{crit} = 1,56 - 0,75\lambda_{vel,m}$ **at slenderness** $0,75 \leq \lambda_{vel,m} \leq 1,4$

$k_{crit} = \frac{1}{\lambda_{vel,m}^2}$ **at slenderness** $\lambda_{vel,m} > 1,4$

$$\lambda_{vel,mfi} = \sqrt{\frac{f_{20}}{\sigma_{m,crit,fi}}}; \qquad \sigma_{m,crit,fi} = \frac{0,78 b_{fi}^2 \cdot E_{20}}{d_{fi} \cdot \ell_{ef}}$$

$$E_{20} = E_{0,05} \cdot k_{fi}$$

At t = 18 min

$$\frac{\sigma_{m,d,fi}}{k_{critfi} f_{m,d,fi}} = 0,86 \leq 1,0$$

It means, that fire resistance of the beam basing on the global stability condition is **R15.**

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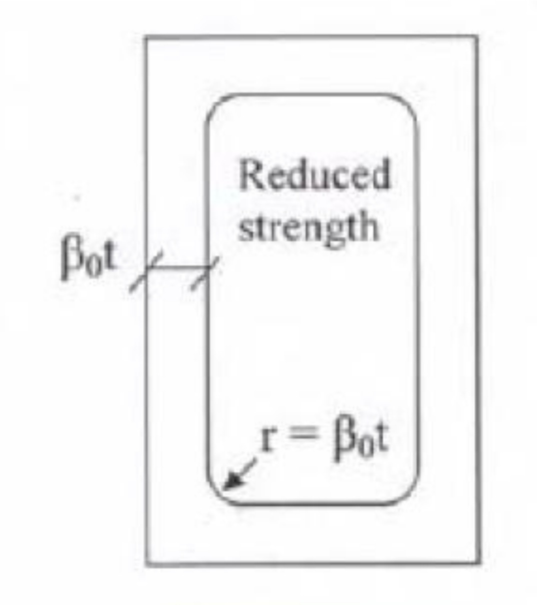
So, we can conclude, that the fire resistance of considered beam is R15.

Design of the beam
<u>Reduced properties method</u> (example 2)

Let us to consider previous example only by the using of the **<u>reduced properties method</u>**



Solution

1. Reduced cross-section parameters:

$\beta_0 = 0.65$

$$d_{fi} = d - 2 \cdot \beta_0 \cdot t$$

$$b_{fi} = b - 2 \cdot \beta_0 \cdot t$$

Moment of inertia relatively y axis **:**

$$\mathcal{I}_{fi,y} = \frac{d_{fi}^3 \cdot b_{fi}}{12} - 4\left[7,42 \cdot 10^{-3} \cdot (\beta_0 \cdot t)^4 + \left(d_{fi} \cdot 0,5 - 0,223 \cdot \beta_0 t\right)^2 \cdot A_i\right], \text{ mm}^4$$

$$\mathcal{I}_{fi,y} = \frac{(300 - 2 \cdot 0,65t)^3 \cdot (100 - 2 \cdot 0,65t)}{12} - 4\left[7,42 \cdot 10^{-3} (0,65t)^4 + \left(0,5(300 - 2 \cdot 0,65t) - -0,223 \cdot (0,65t)\right)^2 (0,65t)^2 \left(1 - \frac{\pi}{4}\right)\right]$$

$$A_i = r^2 \left(1 - \frac{\pi}{4}\right), \text{ mm}^2$$

Modulus of section relatively y axis :

$$W_{fi,y} = \frac{J_{fi,y}}{0,5d_{fi}}$$, mm^3

Area of cross-section:

$$A_{fi} = d_{fi} \cdot b_{fi} - 4(0,65t)^2 \left(1 - \frac{\pi}{4}\right)$$, mm^2

2. Fire resistance of the beam basing on the normal stresses strength condition:

$$\frac{\sigma_{m,y,d,fi}}{f_{m,y,d,fi}} \leq 1,0$$

$$f_{m,y,d,fi} = k_{mod,fi} \cdot \frac{f_{20,mk}}{\gamma_{M,fi}} = 20 - 0,1\left(\frac{2(d_{fi} + b_{fi})}{d_{fi} \cdot b_{fi}}\right)$$

$$f_{20,m,k} = k_{fi} \cdot f_{m,y,k} = 1,25 \cdot 16 = 20 \frac{N}{mm^2}$$

$k_{\downarrow}fi = 1,25$ ("masiva koksne)"

$$k_{mod,fi} = 1 - k_{fi}\left(\frac{P}{A_r}\right) = 1 - \frac{1}{200}\left(\frac{2(d_{fi} + b_{fi})}{d_{fi} \cdot b_{fi}}\right)$$

$\gamma_{M,fi} = 1$ (recommended value for mechanical properties).

The normal stresses strength condition:

$$\frac{4,52 \cdot \frac{10^6}{W_{fi,y}}}{20 - 0,1\left(\frac{2(d_{fi} + b_{fi})}{d_{fi} \cdot b_{fi}}\right)} \leq \mathbf{1,0}$$

<u>Pie t = 45 min</u>

$$\frac{\sigma_{m,y,d,fi}}{f_{m,y,d,fi}} = 0,98 \leq 1,0$$

It means, that fire resistance of the beam basing on the normal stresses strength condition is **R45.**

3. Fire resistance of the beam basing on the shear stresses strength condition:

$$\frac{\tau_{d,fi}}{f_{v,d,fi}} \le 1,0$$

$$\tau_{d,fi} = \frac{3V_{d,fi}}{2A_{fi}} = \frac{3 \cdot 3,54 \cdot 10^3}{2A_{fi}};$$

$$f_{v,d,fi} = k_{mod,fi} \cdot \frac{f_{20v,k}}{\gamma_{M,fi}};$$

$$k_{mod,fi} = 1 - k_{fi}\left(\frac{P}{A_r}\right) = 1 - \frac{1}{330}\left(\frac{2(d_{fi} + b_{fi})}{d_{fi} \cdot b_{fi}}\right).$$

<u>At t =60 min</u>

$$\frac{\tau_{d,fi}}{f_{v,d,fi}} = 0,94 \le 1,0$$

It means, that fire resistance of the beam basing on the shear stresses strength condition is **R60**.

4. **Fire resistance of the beam basing on the global stability condition (prevention of the lateral torsional bucling):**

$$\frac{\sigma_{m,d,fi}}{k_{critfi} f_{m,d,fi}} \leq 1,0 \quad ,$$

$$\lambda_{vel,mfi} = \sqrt{\frac{f_{20}}{\sigma_{m,crit,fi}}} \; ; \quad \sigma_{m,crit,fi} = \frac{0,78 b_{fi}^2 \cdot E_{d,fi}}{d_{fi} \cdot \ell_{ef}}$$

$$E_{dfi} = k_{mod,fi} \frac{E_{20}}{\gamma_{M,fi}}, \qquad E_{20} = E_{0,05} \cdot k_{fi} \, .$$

At t = 32 min

$$\frac{\sigma_{m,d,fi}}{k_{critfi} f_{m,d,fi}} = 0,9 \leq 1,0$$

It means, that fire resistance of the beam basing on the global stability condition is **R30.**

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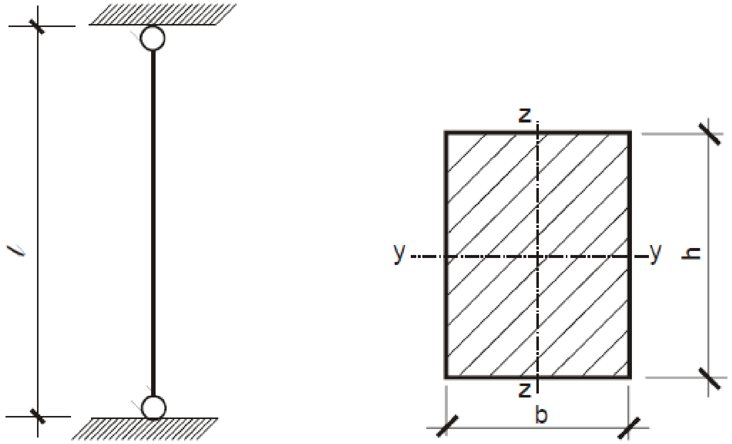
So, we can conclude, that the fire resistance of considered beam is R30**.**

Fire design of column

<u>Reduced cross-section method</u> (example 3)

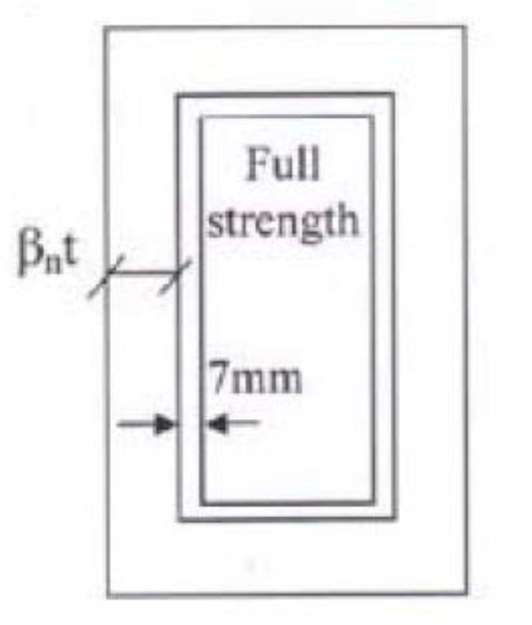


<u>Given</u>:

L = 3000 mm; $N_{c,d}$=10 kN; C 14;   S.C.-2.

<u>Task</u>:

To determine fire resistance of the column. The column is subjected to fire from the **<u>four sides</u>**.



Solution

1. Reduced cross-sections parameters

$$x = \beta_n t + 7mm = 0{,}8t + 7mm$$

(β_n = 0,8 – solid timber with characteristic density ≥ 290kg/m^3)

$$d_{fi} = b_{fi} = 100 - 2 \cdot (0{,}8t + 7) = 86 - 1.6t, \quad mm$$

Moments of inertia relatively y and z axises:

$$\Im_{y,fi} = \Im_{z,fi} = \frac{b_{fi}^4}{12} = (86 - 1{,}6t)^4 \cdot 0{,}0833, \quad mm^4$$

Area of cross-section:

$$A_{fi} = b_{fi}^2 = (86 - 1{,}6t)^2, \quad mm^2$$

2. Determination of the columns fire resistance basing on the stability condition:

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

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

The previous conditions can be rewritten, so as $k_{c,y,fi} = k_{c,z,fi}$,:

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

$$k_{cfi} = \frac{1}{k_{fi} + \sqrt{k_{fi}^2 + \lambda_{rel,fi}^2}}; \quad k = 0{,}5\left(1 + \beta_c\left(\lambda_{rel,fi} - 0{,}3\right) + \lambda_{rel,fi}^2\right)$$

β_c =0,2 (solid timber);

$$\lambda_{rel,fi} = \frac{\lambda_{fi}}{\pi}\sqrt{\frac{f_{20}}{E_{20}}}; \quad \lambda_{fi} = \frac{\ell_{ef}}{i_{fi}}$$

$$i_{fi} = \sqrt{\frac{\Im_{fi}}{A_{fi}}} = \sqrt{\frac{(86 - 1{,}6t)^4 \cdot 0{,}0833}{(86 - 1{,}6t)^2}} = 0{,}289(86 - 1{,}6t)$$

$$\lambda_{fi} = \frac{3000}{0{,}289(86-1{,}6t)};$$

$$\lambda_{rel,fi} = \frac{3000/0{,}289(86-1{,}6t)}{3{,}14}\sqrt{\frac{20}{5{,}875\cdot 10^3}} = \frac{55{,}74}{0{,}289(86-1{,}6t)}$$

$$f_{20} = f_{c,0,k}\cdot k_{fi} = 16\cdot 1{,}25 = 20\ \frac{N}{mm^2}$$

$$E_{20} = E_{0,05}\cdot k_{fi} = 4{,}7\cdot 10^3\cdot 1{,}25 = 5{,}875\cdot 10^3$$

$$\lambda_{rel,fi} = \frac{3000/0{,}289(86-1{,}6t)}{3{,}14}\sqrt{\frac{20}{5{,}875\cdot 10^3}} = \frac{55{,}74}{0{,}289(86-1{,}6t)}$$

$$f_{20} = f_{c,0,k}\cdot k_{fi} = 16\cdot 1{,}25 = 20\ \frac{N}{mm^2}$$

$$E_{20} = E_{0,05}\cdot k_{fi} = 4{,}7\cdot 10^3\cdot 1{,}25 = 5{,}875\cdot 10^3$$

At t = 10 min

$$i_{fi} = 0{,}289(86 - 1{,}6 \cdot 10) = 20{,}23 \;\; mm$$

$$\lambda_{fi} = \frac{3000}{0{,}289(86-1{,}6 \cdot 10)} = 148{,}29 \;;$$

$$\lambda_{rel,fi} = \frac{148{,}29}{3{,}14}\sqrt{\frac{20}{5{,}875 \cdot 10^8}} = 2{,}76;$$

$$k = (1 + 0{,}2(2{,}76 - 0{,}3) + 2{,}76^2) = 4{,}546$$

$$k_{cfi} = \frac{1}{4{,}546 + \sqrt{4{,}546^2 + 2{,}76^2}} = 0{,}101$$

$$\sigma_{c,0,d,fi} = \frac{N_{Ed}}{A_{fi}} = \frac{N_{Ed}}{(86 - 1{,}6t)^2} = \frac{10 \cdot 10^3}{(86 - 1{,}6 \cdot 10)^2} = 2{,}04 \;\; N/mm^2$$

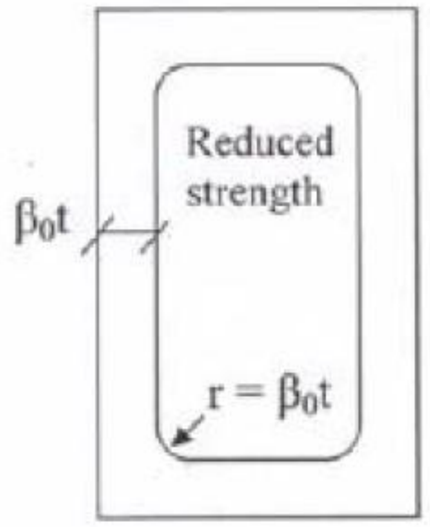
Stability condition:

$$\frac{\sigma_{c,0,d,fi}}{k_{c,fi} f_{d,fi}} = \frac{2{,}04}{0{,}101 \cdot 20} = 1{,}009 \approx 1{,}0 \leq 1{,}0$$

It means, that the column will colapse after 10 minutes basing on the stability condition.

Fire design of column (example 4)

Let us to consider the previous example by the **reduced properties method**. Lets check the conditions regarding b_{min}.



$$d_{char,0} = \beta_0 \cdot t = 0{,}65 \cdot 10 = 6{,}5\ mm$$

From the previous example, t = 10 min

$$b_{min} = 8{,}15 \cdot d_{char,0} = 8{,}15 \cdot 6{,}5 = 53\ \ mm$$

$$b > b_{min} = 53\ mm$$

<u>Solution</u>

1. Reduced cross-section parameters determination:

$d_{fi} = b_{fi} = b - 2 \cdot \beta_0 \cdot t;$ $\qquad r = \beta_0 \cdot t$

Moment of inertia:

$$\Im_{fi,y(z)} = \frac{b_{fi}^4}{12} - r^2(4-\pi)\left[\frac{b_{fi}}{2} - (0{,}223) \cdot r\right]^2 - 4 \cdot 0{,}00742 \cdot r^4 =$$
$$= 0{,}083(100 - 1{,}3t)^4 - (0{,}065t)^2 \cdot 0{,}86 \times$$
$$\times [0{,}5(100 - 1{,}3t) - (0{,}223) \cdot 0{,}65t]^2 - 0{,}0297 \cdot (0{,}65t)^4$$

Cross-sectional area:

$$A_{fi} = b_{fi}^2 = (100 - 1{,}3t)^2; \quad \text{mm}^2$$

2. Determination of the columns fire resistance basing on the stability condition:

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

$$f_{d,fi} = f_{c,0,d,fi} = k_{mod,fi} \frac{f_{20,c,0,k}}{\gamma_{M,fi}} \quad ;$$

$$f_{20,c,0,k} = k_{fi} f_{c,0,k} \cdot = 1{,}25 \cdot 16 = 20 \quad \frac{N}{mm^2}$$

$\gamma_{M,f,i} = 1{,}0$ (recommended value for determination of mechanical properties in the case of fire)

$$k_{mod,fi} = 1 - k_{fi}\left(\frac{P}{A_r}\right) = 1 - \frac{1}{125}\left(\frac{4 \cdot b_{fi}}{b_{fi}^2}\right) = 1 - \frac{1}{125}\left(\frac{4}{b_{fi}}\right) =$$

$$= 1 - \frac{1}{125}\left(\frac{4}{100 - 1{,}3t}\right)$$

It can be written:

$$f_{d,fi} = f_{c,0,d,fi} = 20 - 0,16\left(\frac{4}{100 - 1,3t}\right) = 20 - \frac{0,64}{100 - 1,3t}$$

$$k_{cfi} = \frac{1}{k_{fi} + \sqrt{k_{fi}^2 + \lambda_{rel,fi}^2}};$$

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

β_c =0,2 (solid timber);

$$\lambda_{rel,fi} = \frac{\lambda_{fi}}{\pi}\sqrt{\frac{f_{20}}{E_{20}}}; \lambda_{fi} = \frac{\ell_{ef}}{i_{fi}}$$

$$i_{fi} = \sqrt{\frac{\Im_{fi}}{A_{fi}}}$$

At t = 17 min

$$\Im_{fi} = 0{,}083(100 - 1{,}3 \cdot 17)^4 - (0{,}65 \cdot 17)^2 \cdot 0{,}86 \times$$
$$\times [0{,}5(100 - 1{,}3 \cdot 17) - (0{,}223) \cdot 0{,}65 \cdot 17]^2 - 0{,}0297(0{,}65 \cdot 17)^4 =$$
$$= 2{,}806 \cdot 10^6 \quad mm^4$$
$$A_{fi} = (100 - 1{,}3 \cdot 17)^2 = 6068{,}41 \;\; mm^2 = 0{,}00607 \cdot 10^6 \quad mm^2$$

$$i_{fi} = \sqrt{\frac{\Im_{fi}}{A_{fi}}} = \sqrt{\frac{2{,}806 \cdot 10^6}{0{,}00607 \cdot 10^6}} = 21{,}5 \;\; mm;$$

$$\lambda_{fi} = \frac{3000}{21{,}5} = 139{,}53 \;;$$

$$\lambda_{rel,fi} = \frac{139{,}53}{3{,}14} \sqrt{\frac{20}{5{,}875 \cdot 10^3}} = 2{,}593 \;;$$

$$k = 0{,}5(1 + 0{,}2(2{,}593 - 0{,}3) + 2{,}593^2) = 4{,}09;$$

$$k_{c,fi} = \frac{1}{4{,}09 + \sqrt{4{,}09^2 + 2{,}593^2}} = \frac{1}{8{,}934} = 0{,}112$$

$$\sigma_{c,0,d,fi} = \frac{N_{Ed}}{A_{fi}} = \frac{10 \cdot 10^3}{0{,}00607 \cdot 10^6} = 1{,}647 \ \ N/mm^2$$

$$\frac{\sigma_{c,0,d,fi}}{k_{c,fi} \cdot f_{c,0,d,fi}} = \frac{1{,}647}{0{,}112 \cdot 19{,}992} = 0{,}74 \leq 1{,}0$$

$$\left(f_{c,0,d,fi} = 20 - \frac{0{,}64}{100 - 1{,}3 \cdot 17} = 19{,}997 \ \ \frac{N}{mm^2}\right)$$

At t = 22 min the relation:

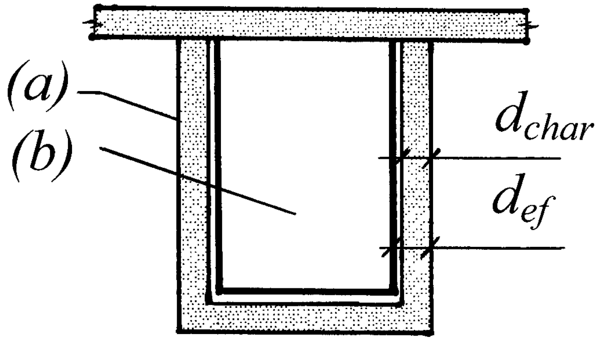
$$\frac{\sigma_{c,0,d,fi}}{k_{c,fi} \cdot f_{c,0,d,fi}} = 1{,}0$$

It means, that the column will colapse after 22 minutes basing on the stability condition. The Columns fire resistance can be evaluated as **R 15**.

Design of beam

<u>Reduced cross-section method</u> (piemērs 5)

Let us to consider solution of the example 1 in the case, if the beam is protected from one side.



***(a)* charring depth, *(b)* material, which is not subjected to fire**

1. determination of reduced cross-sections parameters

$$x = \beta_n t + 7mm = 0,8t + 7mm$$
$$d_{fi} = d - x = 300 - (0,8t + 7) = 293 - 0,8t, \qquad mm$$
$$b_{fi} = b - 2 \cdot x = 100 - 2(0,8t + 7) = 86 - 1,6t, \qquad mm$$

Moment of inertia relatively y axis:

$$\mathfrak{J}_{y,fi} = \frac{b_{fi} \cdot d_{fi}}{12} = \frac{(86 - 1,6t) \cdot (293 - 0,8t)^3}{12} = 0,083(86 - 1,6t)(293 - 0,8t)^3$$

Cross-sectional area:

$$A_{fi} = b_{fi} \cdot d_{fi} = (86 - 1{,}6t)(293 - 0{,}8t)$$

Madulus of section relatively y axis:

$$W_{y,fi} = \frac{b_{fi} \cdot d_{fi}^2}{12} = 0{,}167(86 - 1{,}6t)(293 - 0{,}8t)^2$$

2. Fire resistance of the beam basing on the normal stresses strength condition:

$$\frac{\sigma_{m,y,d,fi}}{f_{m,y,d,fi}} \leq 1,0$$

$$f_{m,y,d,fi} = \frac{20N}{mm^2}$$

The normal stresses strength condition in the current case:

$$\frac{4,52 \cdot 10^6 / 0,167(86 - 1,6t)(293 - 0{,}8t)^2}{20} \leq 1,0$$

At t = 42 min (For the example 1 (case without protection of one side)   t = 37 min):

$$\frac{\sigma_{m,y,d,fi}}{f_{m,y,d,fi}} = 1{,}0 \leq \mathbf{1,0}$$

3. Fire resistance of the beam basing on the shear stresses strength condition:

$$\frac{\tau_{d,fi}}{f_{v,d,fi}} \leq \mathbf{1,0}$$

$$\tau_{d,fi} = \frac{3V_{d,fi}}{2A_{fi}} = \frac{3 \cdot 3{,}54 \cdot 10^3}{2(86 - 1{,}6t)(293 - 0{,}8t)};$$

$$f_{v,d,fi} = 2{,}\frac{25N}{mm^2}$$

At t = 48 min (For the example 1 (case without protection of one side)   t = 45 min):

$$\frac{\tau_{d,fi}}{f_{v,d,fi}} = 0{,}86 \leq \mathbf{1,0}$$

4. Fire resistance of the beam basing on the global stability condition**:**

$$\frac{\sigma_{m,d,fi}}{k_{critfi} f_{m,d,fi}} \leq 1,0$$

$k_{crit} = 1$ in the case, if slenderness $\lambda_{rel,m} \leq 0,75$

$k_{crit} = 1,56 - 0,75\lambda_{rel,m}$ in the case, if slenderness $0,75 < \lambda_{rel,m} \leq 1,4$

$k_{crit} = \frac{1}{\lambda^2_{rel,m}}$ in the case, if slenderness $\lambda_{rel,m} > 1,4$

$$\lambda_{vel,mfi} = \sqrt{\frac{f_{20}}{\sigma_{m,crit,fi}}}\ ; \qquad \sigma_{m,crit,fi} = \frac{0,78 b^2_{fi} \cdot E_{20}}{d_{fi} \cdot \ell_{ef}}$$

<u>At t = 25 min</u> (For the example 1 (case without protection of one side) t = 18 min):

$$\frac{\sigma_{m,d,fi}}{k_{critfi} f_{m,d,fi}} = 0,988 \leq 1,0$$

Fire resistance of the beam is R15, bet t increase at 38%.

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