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PUB-WOOD

Structural Timber design according to Eurocode 5

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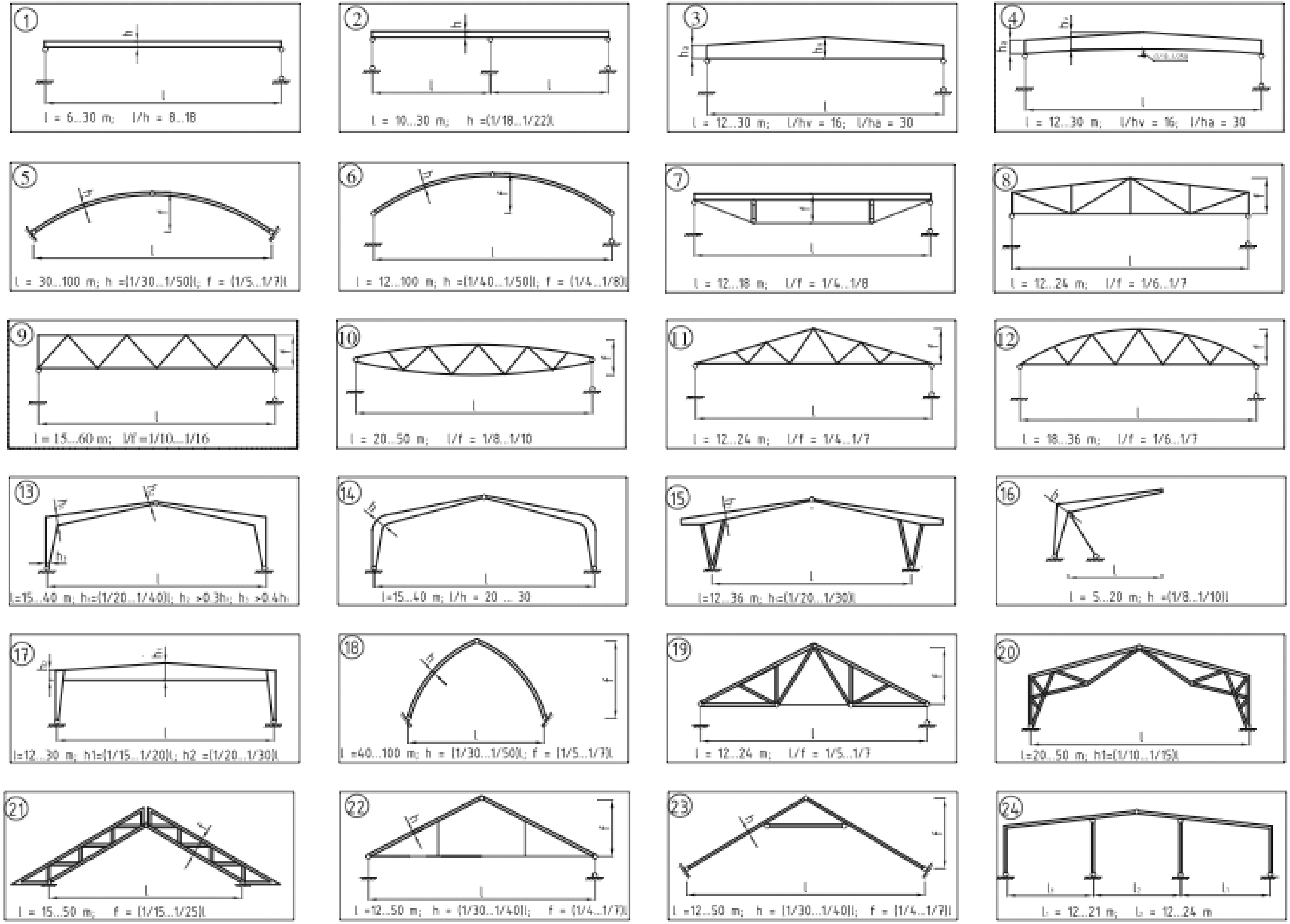
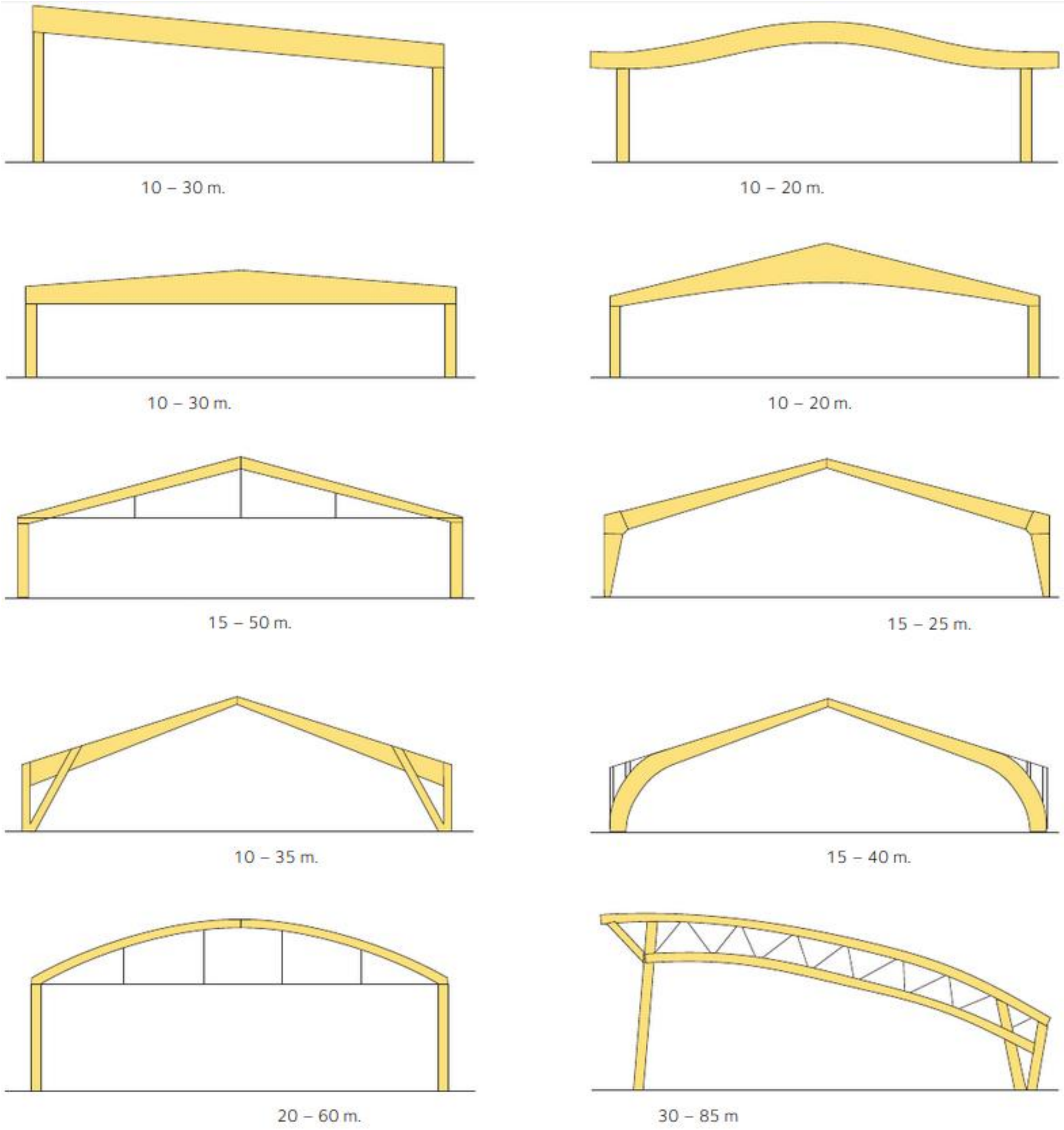
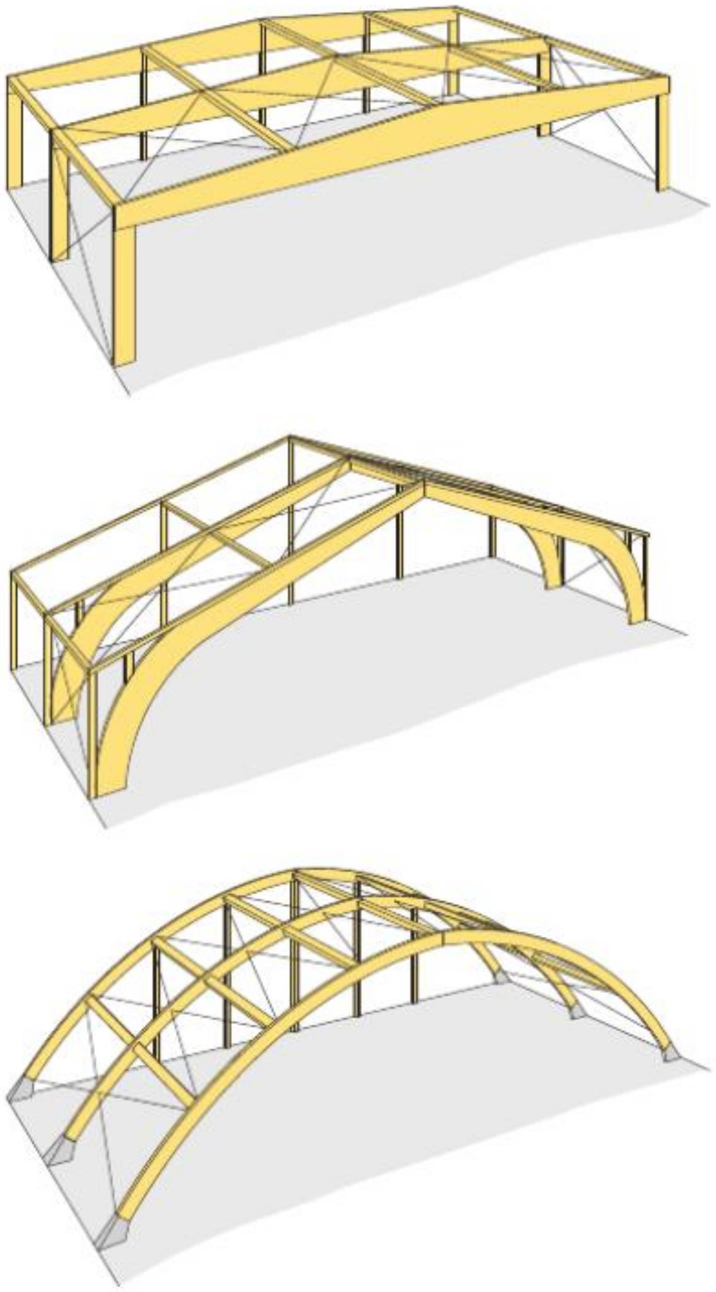
Main structural Systems of Timber frames and elements



Main structural Systems of Glulam





Structural Timber design is done according to EN 1995-1-1 Eurocode 5: Design of timber structures-Part 1-1: General-Common rules and rules for buildings:

1. **Ultimate Limit States** *according to Section 6;*

2. **Serviceability Limit States** *according to Section 7;*

3. **Connections with metal fasteners** *according to Section 8*

Ultimate Limit States of timber structures

Ultimate limit states:

1) Design of elements subjected to stresses in one principal direction:

1.1. tension parallel to grain;

1.2. tension perpendicular to the grain;

1.3. compression parallel to the grain;

1.4. compression perpendicular to the grain;

1.5. bending;

1.6. shear.

2) Design of elements subjected to combined stresses

2.1) combined bending;

2.2) compression stresses at an angle to the grain;

2.3) combined bending and axial tension;

2.4) combined bending and axial compression.

3) Stability of members.

4) Design of cross-sections in members with varying cross-section.

Main axis according to EC5

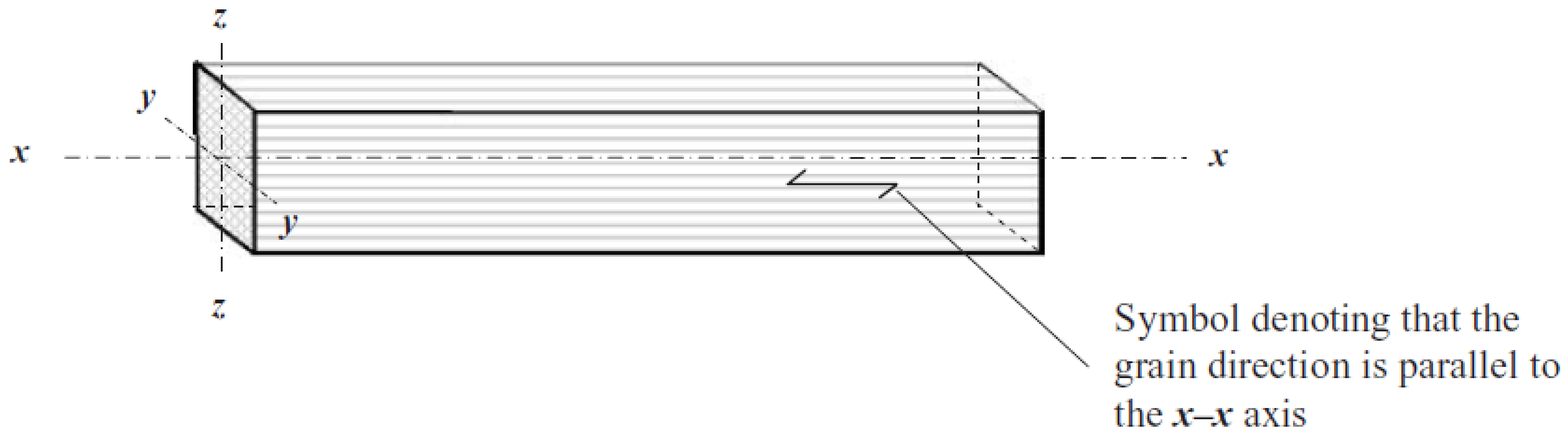


Fig. 1. Member axes

Design of cross-sections subjected to stress in one principal direction

Tension parallel to the grain (1)

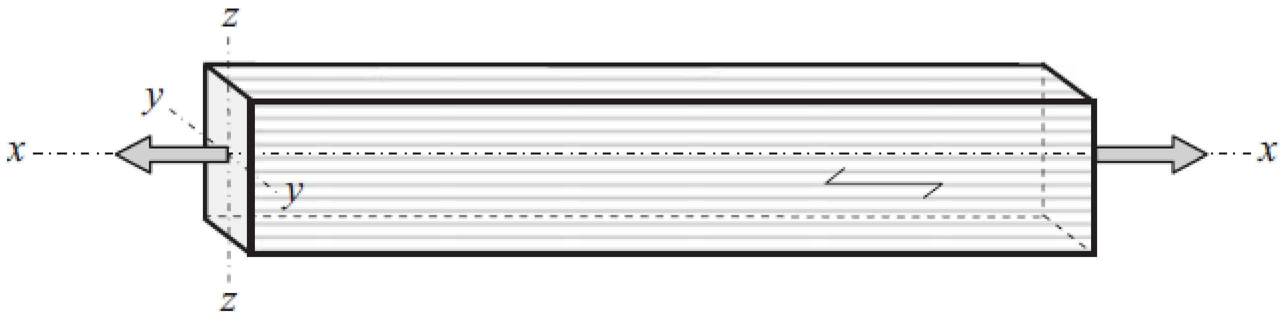


Fig. 1. Timber element subjected to axial tension

Strength condition of axially loaded timber element according to EN 1995-1-1, Eq. 6.1:

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

where:

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

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

Tension parallel to the grain (2)

The design tensile stress along the grain:

$$\sigma_{t,0,d} = \frac{N_{t,d}}{A_{net}}$$

where:

$N_{t,Ed}$ – the design tension force in the element;

A_{net} – netto cross section.

Determining A_{net} should be evaluated:

- If holes and cuts are in the distance of less than 200 mm, the Net cross section should be taken as one;
- Holed for nails and screws with diameter less of equal to 6 mm can be neglected.

Tension parallel to the grain (3)

The design tension strength parallel to the grain according to EN 1995-1-1, Eq. 2.17:

$$f_{t,0,d} = \frac{k_{\mathrm{mod}} \cdot k_h \cdot f_{t,0,k}}{\gamma_M}$$

where:

k_{mod} – is a modification factor taking into account the effect of the duration of load and moisture content;

$f_{t,0,k}$ – is the characteristic value of a tension strength;

γ_M – is the partial factor for a material property;

k_h – the cross section height coefficient.

Modification factor

Table NA.3 – Structures to service classes

Structure	Service class
Heated building roofs, floors, walls and partitions constructions	1
Protected from direct humidification unheated buildings roofs, walls, floors and partition constructions	2
In wet areas direct protected from humidification and well ventilated heated roofs, walls, floors and partition constructions	2
Floors installed above the ground	2
External structures exposed to external atmospheric effects	3
The internal structures in wet areas direct no protected from humidification	3

Structures shall be assigned to one of the service classes given below:

Service class ***1*** is characterized by moisture content in the materials corresponding to a temperature of 20^0C and the relative humidity of the surrounding air only exceeding 65% for a few weeks per year.

Service class ***2*** is characterized by moisture content in the materials corresponding to a temperature of 20^0C and the relative humidity of the surrounding air only exceeding 85% for a few weeks per year.

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

Modification factor
Load duration classes LDC

Table NA.2.2 – Examples of load-duration assignment

Load-duration class	Examples of loading
Permanent	self-weight; machinery, equipment and lightweight guards, permanently attached to the construction; gravitation
Long-term	storage load (category E); load of water tank
Medium-term	snow; equivalent distributed imposed floor load, balcony load (category A, B, C and D); garage and vehicle load of traffic area (category F and G)
Short-term	wind; staircase and balcony load; mounting or man on the roof load; mobile vehicle load
Instantaneous	accidentals actions; horizontal guards walls, barriers and parapets effects

The partial factor for a material property

Table NA.2.3 – Partial factors γ_M for material properties and resistances

Fundamental combinations	Partial factors γ_M
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 anchorage strength	1,3
punched metal plate fasteners metal strength	1,1
Accidental combinations	1,0

Mechanical properties of softwood

	Class	C14	C16	C18	C20	C22	C24	C27	C30	C35	C40	C45	C50
Strength properties in N/mm²													
Bending	$f_{m,k}$	14	16	18	20	22	24	27	30	35	40	45	50
Tension parallel	$f_{t,0,k}$	7,2	8,5	10	11,5	13	14,5	16,5	19	22,5	26	30	33,5
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
Compression parallel	$f_{c,0,k}$	16	17	18	19	20	21	22	24	25	27	29	30
Compression perpendicular	$f_{c,90,k}$	2,0	2,2	2,2	2,3	2,4	2,5	2,5	2,7	2,7	2,8	2,9	3,0
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
Stiffness properties in kN/mm²													
Mean modulus of elasticity parallel bending	$E_{m,0,mean}$	7,0	8,0	9,0	9,5	10,0	11,0	11,5	12,0	13,0	14,0	15,0	16,0
5 percentile modulus of elasticity parallel bending	$E_{m,0,k}$	4,7	5,4	6,0	6,4	6,7	7,4	7,7	8,0	8,7	9,4	10,1	10,7
Mean modulus of elasticity perpendicular	$E_{m,90,mean}$	0,23	0,27	0,30	0,32	0,33	0,37	0,38	0,40	0,43	0,47	0,50	0,53
Mean shear modulus	G_{mean}	0,44	0,50	0,56	0,59	0,63	0,69	0,72	0,75	0,81	0,88	0,94	1,00
Density in kg/m³													
5 percentile density	ρ_k	290	310	320	330	340	350	360	380	390	400	410	430
Mean density	ρ_{mean}	350	370	380	400	410	420	430	460	470	480	490	520

NOTE 1 Values given above for tension strength, compression strength, shear strength, char. modulus of elasticity in bending, mean modulus of elasticity perpendicular to grain and mean shear modulus have been calculated using the equations given in EN 384.

NOTE 2 The tension strength values are conservatively estimated since grading is done for bending strength.

NOTE 3 The tabulated properties are compatible with timber at moisture content consistent with a temperature of 20 °C and a relative humidity of 65 %, which corresponds to a moisture content of 12 % for most species.

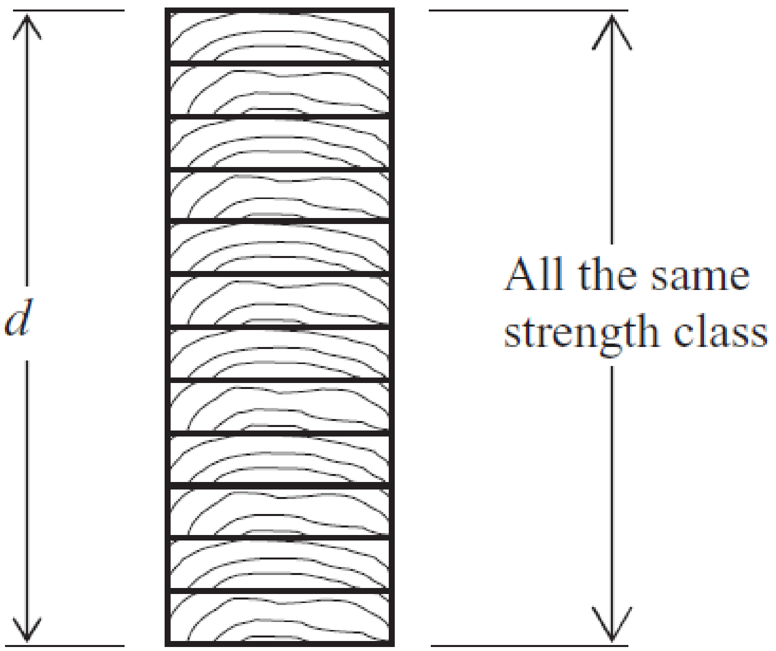
NOTE 4 Characteristic values for shear strength are given for timber without fissures, according to EN 408.

NOTE 5 These classes may also be used for hardwoods with similar strength and density profiles such as e.g. poplar or chestnut.

NOTE 6 The edgewise bending strength may also be used in the case of flatwise bending.

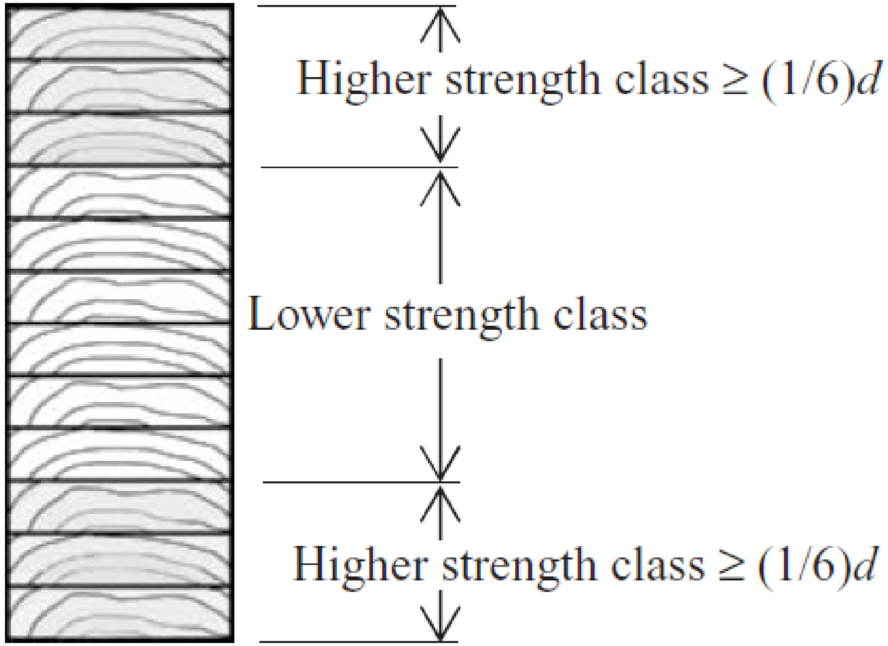
Mechanical properties of Glulam (homogenous)

Property	Symbol	Glulam strength class						
		GL 20h	**GL 22h**	**GL 24h**	**GL 26h**	**GL 28h**	**GL 30h**	**GL 32h**
Bending strength	$f_{m,g,k}$	20	22	24	26	28	30	32
Tensile strength	$f_{t,0,g,k}$	16	17,6	19,2	20,8	22,3	24	25,6
	$f_{t,90,g,k}$	0,5						
Compression strength	$f_{c,0,g,k}$	20	22	24	26	28	30	32
	$f_{c,90,g,k}$	2,5						
Shear strength (shear and torsion)	$f_{v,g,k}$	3,5						
Rolling shear strength	$f_{r,g,k}$	1,2						
Modulus of elasticity	$E_{0,g,mean}$	8 400	10 500	11 500	12 100	12 600	13 600	14 200
	$E_{0,g,05}$	7 000	8 800	9 600	10 100	10 500	11 300	11 800
	$E_{90,g,mean}$	300						
	$E_{90,g,05}$	250						
Shear modulus	$G_{g,mean}$	650						
	$G_{g,05}$	540						
Rolling shear modulus	$G_{r,g,mean}$	65						
	$G_{r,g,05}$	54						
Density	$\rho_{g,k}$	340	370	385	405	425	430	440
	$\rho_{g,mean}$	370	410	420	445	460	480	490



Mechanical properties of Glulam (combined)

<table>
<tr><th rowspan="2">Property[a]</th><th rowspan="2">Symbol</th><th colspan="7">Glulam strength class</th></tr>
<tr><th>GL 20c</th><th>GL 22c</th><th>GL 24c</th><th>GL 26c</th><th>GL 28c</th><th>GL 30c</th><th>GL 32c</th></tr>
<tr><td>Bending strength</td><td>$f_{m,g,k}$</td><td>20</td><td>22</td><td>24</td><td>26</td><td>28</td><td>30</td><td>32</td></tr>
<tr><td rowspan="2">Tensile strength</td><td>$f_{t,0,g,k}$</td><td>15</td><td>16</td><td>17</td><td>19</td><td>19,5</td><td>19,5</td><td>19,5</td></tr>
<tr><td>$f_{t,90,g,k}$</td><td colspan="7">0,5</td></tr>
<tr><td rowspan="2">Compression strength</td><td>$f_{c,0,g,k}$</td><td>18,5</td><td>20</td><td>21,5</td><td>23,5</td><td>24</td><td>24,5</td><td>24,5</td></tr>
<tr><td>$f_{c,90,g,k}$</td><td colspan="7">2,5</td></tr>
<tr><td>Shear strength
(shear and torsion)</td><td>$f_{v,g,k}$</td><td colspan="7">3,5</td></tr>
<tr><td>Rolling shear strength</td><td>$f_{r,g,k}$</td><td colspan="7">1,2</td></tr>
<tr><td rowspan="4">Modulus of elasticity</td><td>$E_{0,g,mean}$</td><td>10 400</td><td>10 400</td><td>11 000</td><td>12 000</td><td>12 500</td><td>13 000</td><td>13 500</td></tr>
<tr><td>$E_{0,g,05}$</td><td>8 600</td><td>8 600</td><td>9 100</td><td>10 000</td><td>10 400</td><td>10 800</td><td>11 200</td></tr>
<tr><td>$E_{90,g,mean}$</td><td colspan="7">300</td></tr>
<tr><td>$E_{90,g,05}$</td><td colspan="7">250</td></tr>
<tr><td rowspan="2">Shear-modulus</td><td>$G_{g,mean}$</td><td colspan="7">650</td></tr>
<tr><td>$G_{g,05}$</td><td colspan="7">540</td></tr>
<tr><td rowspan="2">Rolling shear modulus</td><td>$G_{r,g,mean}$</td><td colspan="7">65</td></tr>
<tr><td>$G_{r,g,05}$</td><td colspan="7">54</td></tr>
<tr><td rowspan="2">Density[b]</td><td>$\rho_{g,k}$</td><td>355</td><td>355</td><td>365</td><td>385</td><td>390</td><td>390</td><td>400</td></tr>
<tr><td>$\rho_{g,mean}$</td><td>390</td><td>390</td><td>400</td><td>420</td><td>420</td><td>430</td><td>440</td></tr>
</table>



The cross section height coefficient according to EN 1995-1-1, Eq. 3.1 and Eq. 3.2

For rectangular solid timber with a characteristic timber density ≤70 kg/m^3, the reference depth in bending or width (maximum cross-sectional dimension) in tension is 150 mm. For depths in bending or widths in tension of solid timber less than 150 mm the characteristic values for $f_{m,k}$ and $f_{t,k}$ may be increased by factor k_h:

For rectangular glued laminated timber, the reference depth in bending or width in tension is 600 mm. For depths in bending or widths in tension of glued laminated timber less than 60 mm the characteristic values for $f_{m,k}$ and $f_{t,k}$ may be increased by factor k_h:

$$k_h = \min\begin{cases}\left(\dfrac{150}{h}\right)^{0,2}\\ 1,3\end{cases}$$

$$k_h = \min\begin{cases}\left(\dfrac{600}{h}\right)^{0,1}\\ 1,1\end{cases}$$

Tension perpendicular to the grain

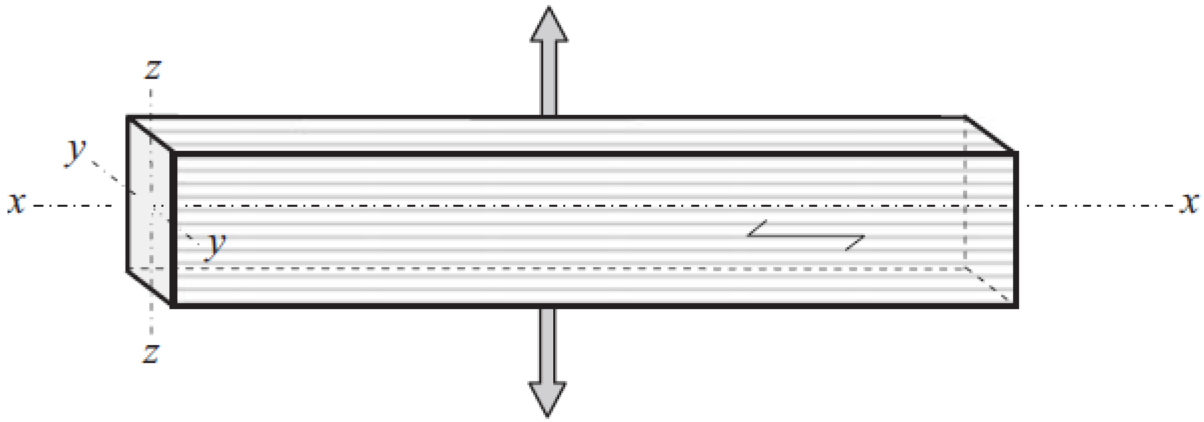


Fig. 1. Element subjected to tension perpendicular to the grain

Strength condition of the element subjected to tension perpendicular to the grain according to EN 1995-1-1, Section 6.1.4:

$$\sigma_{t,90,d} \leq f_{t,90,d}$$

Remark:

The effect of member size shall be taken into account.

Compression parallel to the grain

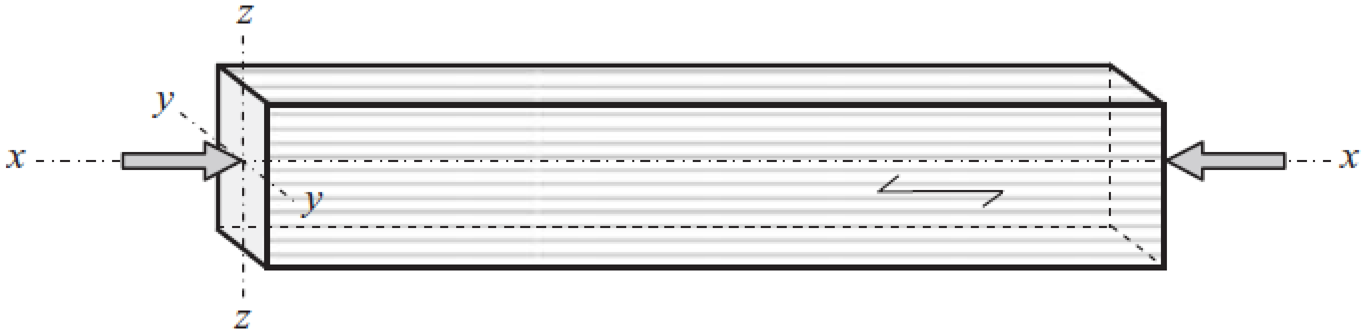


Fig. 1. Element subjected to axial compression stress

The following condition should be satisfied according to EN 1995-1-1, Eq. 6.2:

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

The design compressive strength:

$$f_{c,0,d} = \frac{f_{c,0,k} \cdot k_{\mathrm{mod}}}{\gamma_M}$$

Compression perpendicular to the grain (1)

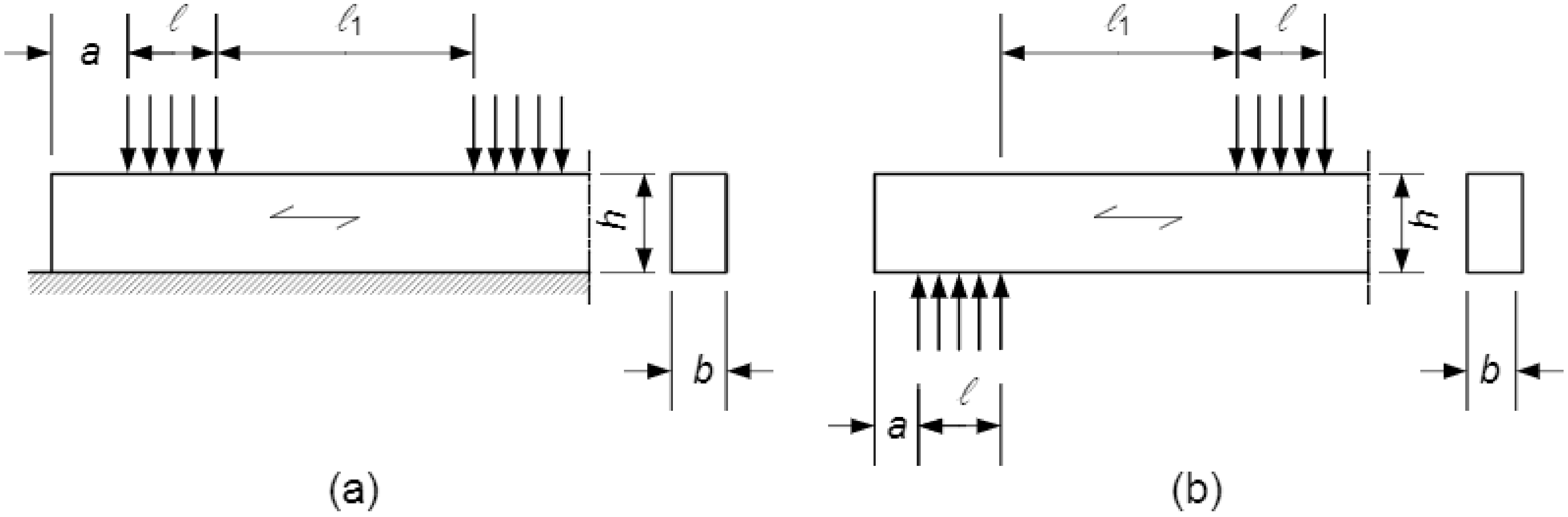


Fig. 1. Timber member subjected to compression perpendicular to the grain:

a) Member on continuous supports;

b) Member on discrete supports

Compression perpendicular to the grain (2)

The strength condition of member subjected to compression perpendicular to the grain according to EN 1995-1-1, Eq. 6.3 and 6.4:

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

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{ef}}$$

where:

A_{ef} – the effective contact area in compression perpendicular to the grain;

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

Compression perpendicular to the grain (3)

The effective contact area perpendicular to the grain A_{ef} should be determined taking into account an effective contact length parallel to the grain, where the actual length l at each side is increased by 30 mm, but not more than a, l ir $l_1/2$.

Condition should be satisfied $l_1 \geq 2h$.

For members coefficient is equal to:

Members on continuous supports:

$$\begin{cases} k_{c,90} = 1.25 \quad solid\ /\ sawn\ timber \\ k_{c,90} = 1.5 \quad glulam \end{cases}$$

Members on discrete supports:

$$\begin{cases} k_{c,90} = 1.5 \quad solid\ /\ sawn\ timber \\ k_{c,90} = 1.75 \quad glulam, l \leq 400mm \end{cases}$$

Bending

The following *expressions shall be satisfied (when relative slenderness for bending is ≤0,75) according to EN 1995-1-1, Eq. 6.11 and 6.12:*

$$\frac{\sigma_{m,y,d}}{f_{m,d}} \leq 1,0$$ *About axis y-y*

$$\frac{\sigma_{m,z,d}}{f_{m,d}} \leq 1,0$$ *About axis z-z*

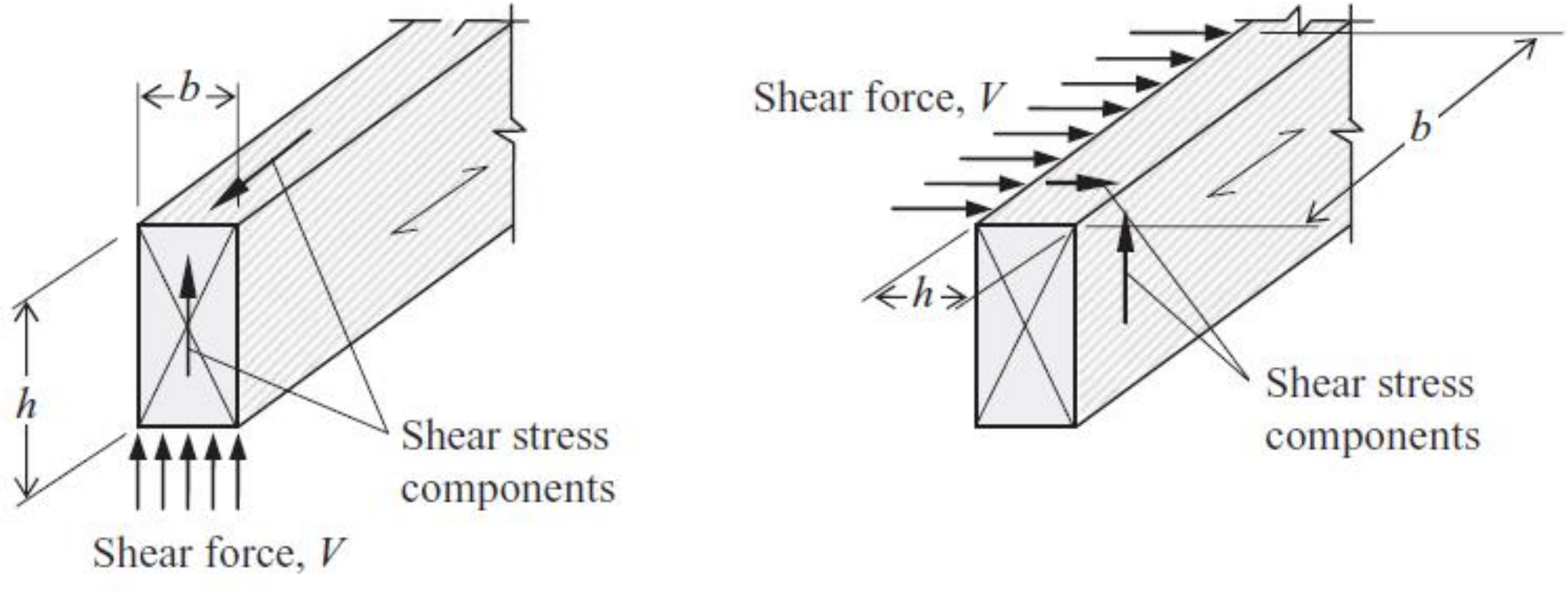
$$\sigma_{m,y,d} = \frac{M_{y,d}}{W_y}$$

$$\sigma_{m,z,d} = \frac{M_{z,d}}{W_z}$$

$$f_{m,d} = \frac{f_{m,k} \cdot k_h \cdot k_{\text{mod}}}{\gamma_M}$$

Determined in the same way as for elements subjected to tension

Shear (1)



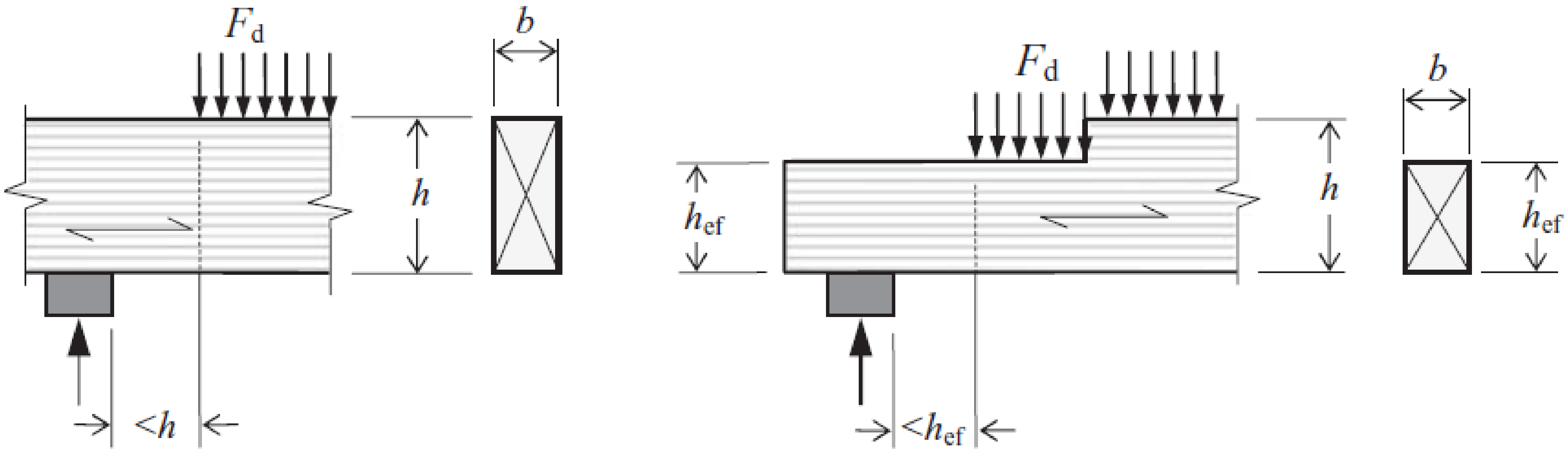
(a) A shear component parallel to the grain

(b) Both shear components perpendicular to the grain (rolling shear situation)

For shear *with a stress component parallel to the grain, as well as for shear with both stress component s perpendicular to the grain, the following expression shall be satisfied*, according to EN 1995-1-1, Eq. 6.13:

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

Shear (2)

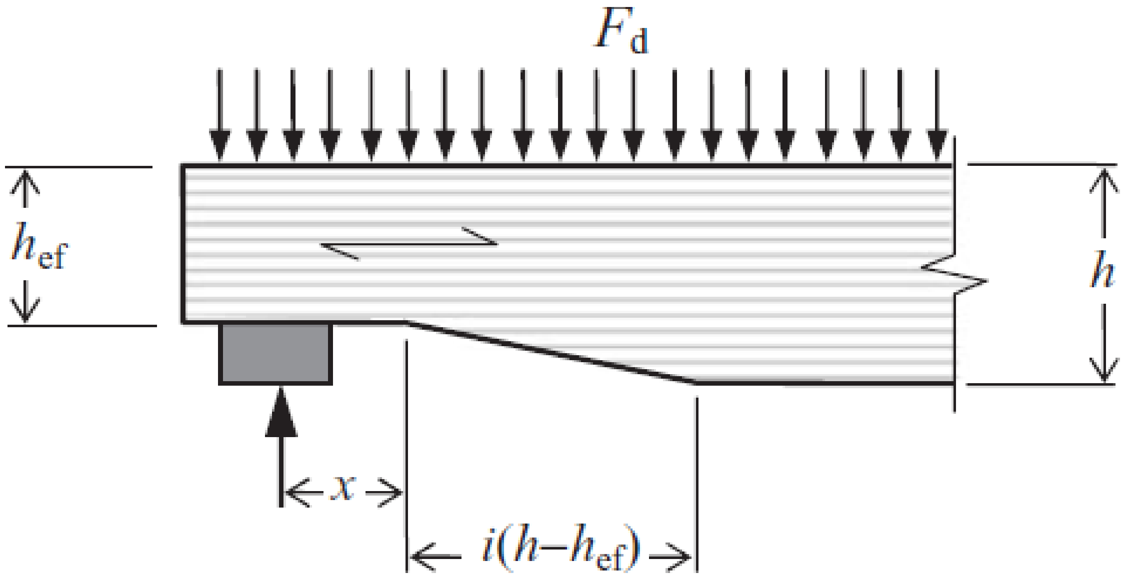


(a) Without a notch (b) With a notch

$$\tau_d = \frac{3 \cdot V_d}{2 \cdot k_{cr} \cdot b \cdot h} \qquad f_{v,d} = \frac{k_{\text{mod}} \cdot f_{v,k}}{\gamma_M}$$

k_{cr} is the coefficient which is used for verification of shear resistance for members in bending, the influence of cracks should be taken into account. For solid timber and for glued laminated timber is equal to 0,67.

Shear (3)



Beams with an end notch.

The strength condition should be verified according to EN 1995-1-1, Eq. 6.60:

$$\tau_d = \frac{1{,}5 \cdot V_d}{k_{cr} \cdot b \cdot h_{ef}} \le k_v \cdot f_{v,d}$$

Shear (4)

Reduction factor k_v according to EN 1995-1-1, Eq. 6.61 and 6.62***:***

$$k_v = 1,0$$

For beams notched at the opposite side to the support

$$k_v = \min \begin{cases} 1 \\ \dfrac{k_n\left(1+\dfrac{1,1\cdot i^{1,5}}{\sqrt{h}}\right)}{\sqrt{h}\left(\sqrt{\alpha(1-\alpha)}+0,8\dfrac{x}{h}\sqrt{\dfrac{1}{\alpha}-\alpha^2}\right)} \end{cases}$$

For beams notched on the same side as the support

$$\alpha = \frac{h_{ef}}{h}$$

$$k_n = \begin{cases} 4,5 & for\ LVL \\ 5 & for\ solid\ timber \\ 6,5 & for\ Glulam \end{cases}$$

Torsion (1)

The following *strength condition shall be satisfied* according to EN 1995-1-1, Eq. 6.14:

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

$$\tau_{tor,d} = \frac{2 \cdot T}{\pi \cdot r^3}$$ *For a circular cross section*

$$\tau_{tor,d} = \frac{T}{k_2 \cdot b \cdot h^2}$$ *For a rectangular cross section*

Torsion (2)

Coefficient k_2 takes into account the ratio between height and width of the cross section:

$$h / b = 1{,}0 \rightarrow k_2 = 0{,}208$$

$$h / b = 1{,}2 \rightarrow k_2 = 0{,}219$$

$$\ldots$$

Factor depending on the shape of the cross-section:

$$k_{shape} = 1{,}2$$

For a circular cross section

$$k_{shape} = \min \begin{cases} 1 + 0{,}15 \dfrac{h}{b} \\ 2{,}0 \end{cases}$$

For a rectangular cross section

Design of cross-sections subjected
to combined stresses

Bending

The following condition (when relative slenderness for bending ≤0,75) should be verified according to EN 1995-1-1, Eq. 6.11 and 6.12:

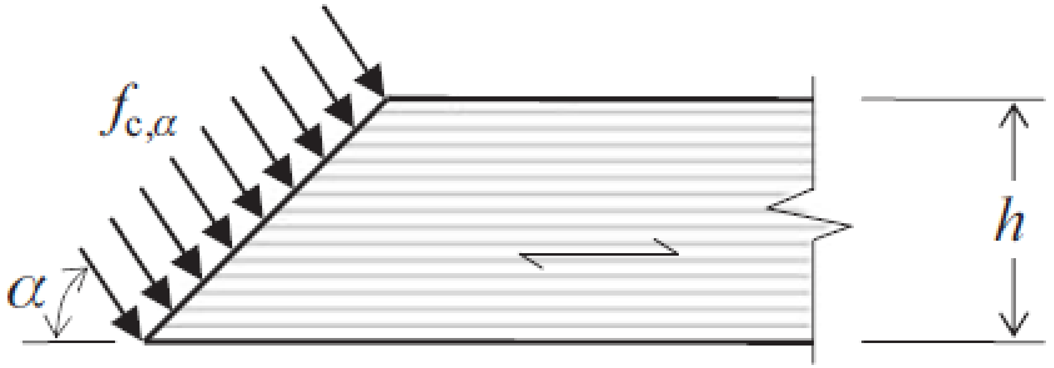
$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1,0 \quad about\ y-y\ axis$$

$$k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1,0 \quad about\ z-z\ axis$$

k_m=0,7 (for rectangular cross section)

k_m=1,0 (for other cross sections)

Compression stresses at an angle to the grain



Compressive strength of a member loaded at an angle α to the grain.

The compressive stresses at an angle to the grain, should satisfy the following expression according to EN 1995-1-1, Eq. 6.16:

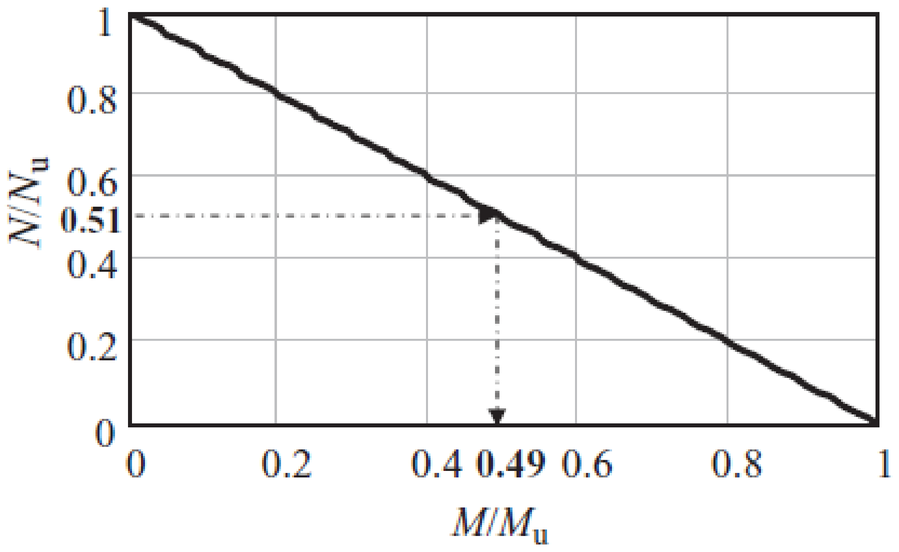
$$\sigma_{c,\alpha,d} \leq \frac{f_{c,0,d}}{\dfrac{f_{c,0,d}}{k_{c,90} \cdot f_{c,90,d}} \sin^2\alpha + \cos^2\alpha}$$

Combined bending and axial tension

The following expressions shall be satisfied according to EN 1995-1-1, Eq. 6.17 and 6.18:

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1,0 \quad about\ y-y\ axis$$

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1,0 \quad about\ z-z\ axis$$



Combined bending and axial compression (1)

The following expressions shall be satisfied, when relative slenderness of elements are $\lambda_{rel,y}$ *and* $\lambda_{rel,z} \le 0,3$ according to EN 1995-1-1, Eq. 6.19 and 6.20:

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \le 1,0 \quad about\ y-y\ axis$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \le 1,0 \quad about\ z-z\ axis$$

Combined bending and axial compression (2)

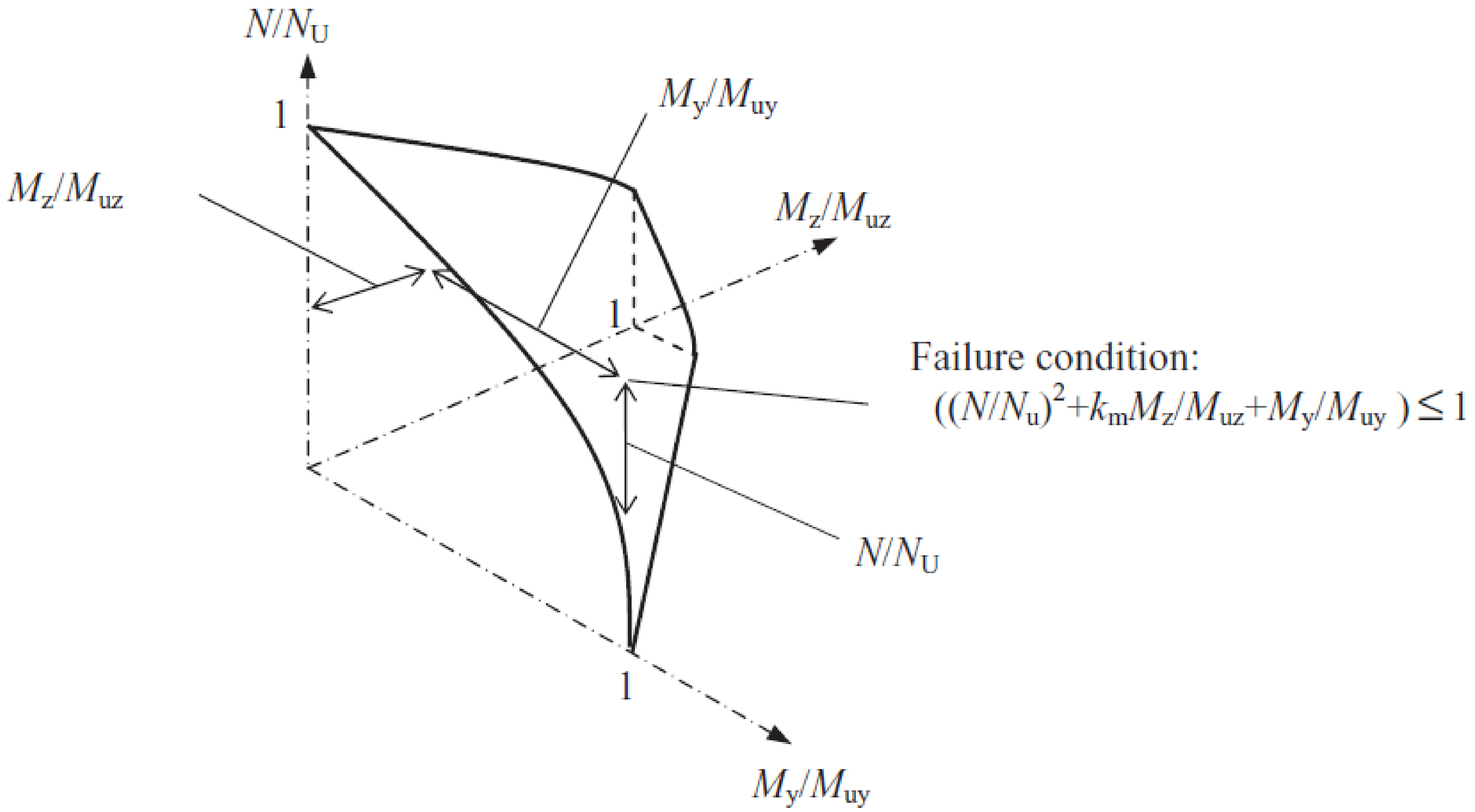


Fig. 5.11. Axial force–moment interaction curve for bi-axial bending when both $\lambda_{rel,y}$ and $\lambda_{rel,z} \leq 0.3$ and with factor k_m applied to the ratio of moments about the z–z axis.

Stability of members

Bending (1)

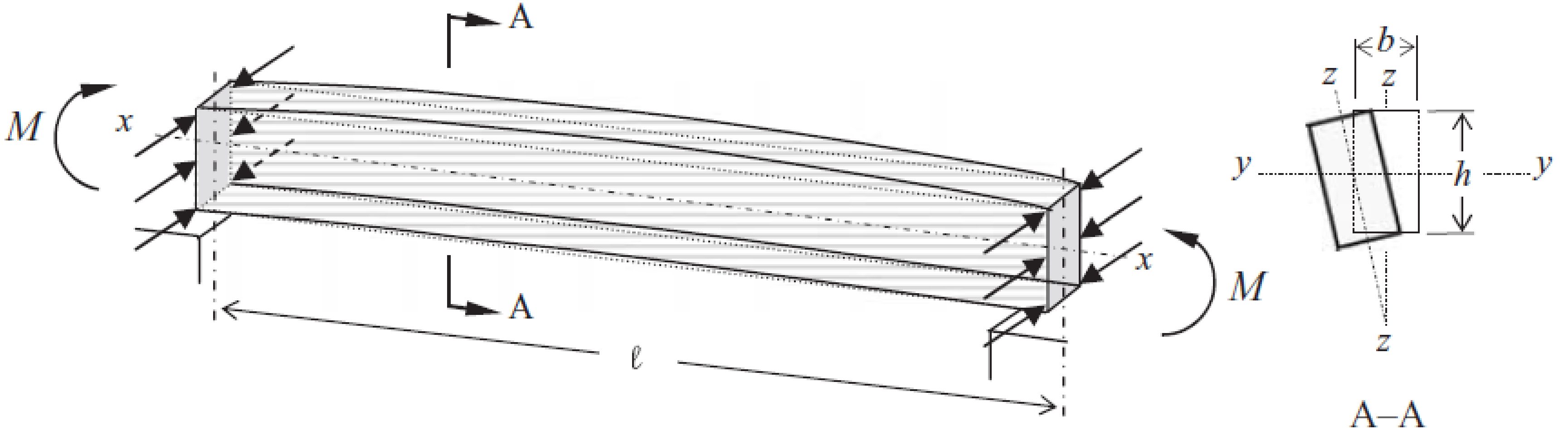


Fig. 1. Lateral torsional buckling of bending element

***In the case** where only moment M_y exists about the strong axis y, the stresses should satisfy the following condition* according to EN 1995-1-1, Eq. 6.33:

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

Bending (2)

***For beams** with an initial lateral deviation from straightness within the limits defined in Section 10, the coefficient may be determined from expressions* according to EN 1995-1-1, Eq. 6.34*:*

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

The factor may be taken as 1,0 for beam where lateral displacement of its compressive edge is prevented throughout its length and where torsional rotation is prevented at its supports.

Bending (3)

The relative slenderness for bending should be taken as according to EN 1995-1-1, Eq. 6.30:

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

Critical bending stress according to EN 1995-1-1, Eq. 6.31:

$$\sigma_{m,crit} = \frac{0,78 \cdot b^2}{h \cdot l_{ef}} E_{0,05} \qquad for\,solid\,timber$$

$$\sigma_{m,crit} = \frac{\pi \cdot b^2}{h \cdot l_{ef}} \sqrt{E_{0,05} \cdot G_{0,05} \cdot \left(1 - 0,63\frac{b}{h}\right)} \qquad for\,Glulam$$

Bending (4)

Table 6.1 – 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.

Centrically loaded elements (1)

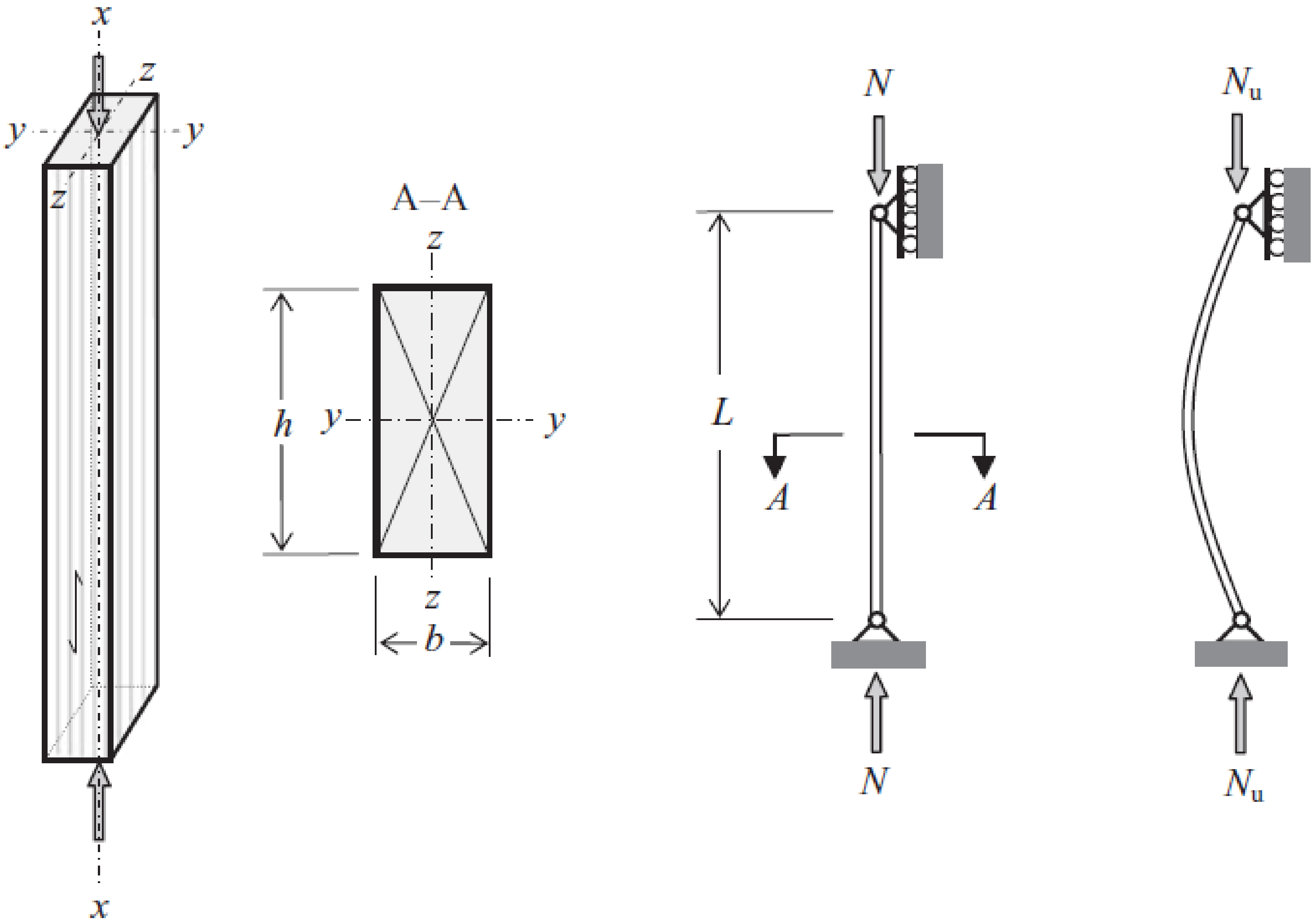


Fig. 1. Buckling of centrically loaded element

Centrically loaded elements (2)

Centrically loaded elements when relative slenderness is

$\lambda_{rel,y}\ ir\ \lambda_{rel,z} > 0,3$ according to EN 1995-1-1, Eq. 6.23 and 6.24:

$$\frac{\sigma_{c,0,d}}{k_{c,y} \cdot f_{c,0,d}} \leq 1,0 \quad about\ y-y\ axis$$

$$\frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} \leq 1,0 \quad about\ z-z\ axis$$

Centrically loaded elements (3)

Buckling coefficients according to EN 1995-1-1, Eq. 6.25-6.29***:***

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}} \quad about\ \ y-y\ axis$$

$$k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}} \quad about\ \ z-z\ axis$$

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

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

$$\beta_c = 0{,}2 \quad solide\,timber$$

$$\beta_c = 0{,}1 \quad glulam$$

Centrically loaded elements (4)

Relative slenderness according to EN 1995-1-1, Eq. 6.21 and 6.22:

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \quad about\ \ y - y$$

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \quad about\ \ z - z$$

Slenderness:

$$\lambda_y = \frac{L_{e,y}}{i_y} = \frac{L_{e,y}}{h/\sqrt{12}} \quad about\ \ y - y$$

$$\lambda_z = \frac{L_{e,z}}{i_z} = \frac{L_{e,z}}{b/\sqrt{12}} \quad about\ \ z - z$$

Centrically loaded elements (5)

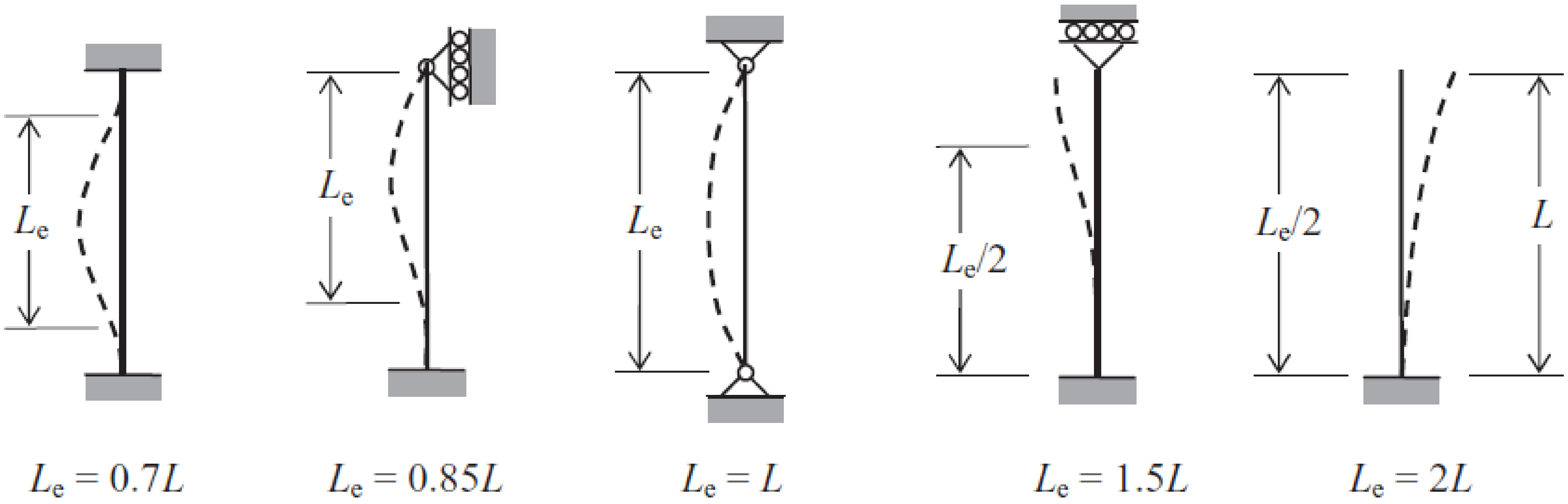


Fig. 5.4. Effective length and end conditions.

Columns subjected to either compression of combined compression and bending (1)

In all other cases *the stresses, which will be increased due to deflection, should satisfy the following expressions* according to EN 1995-1-1, Eq. 6.23 and 6.24*:*

$$\frac{\sigma_{c,0,d}}{k_{c,y} \cdot f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1,0$$

$$\frac{\sigma_{c,0,d}}{k_{c,y} \cdot f_{c,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1,0$$

Columns subjected to either compression of combined compression and bending (2)

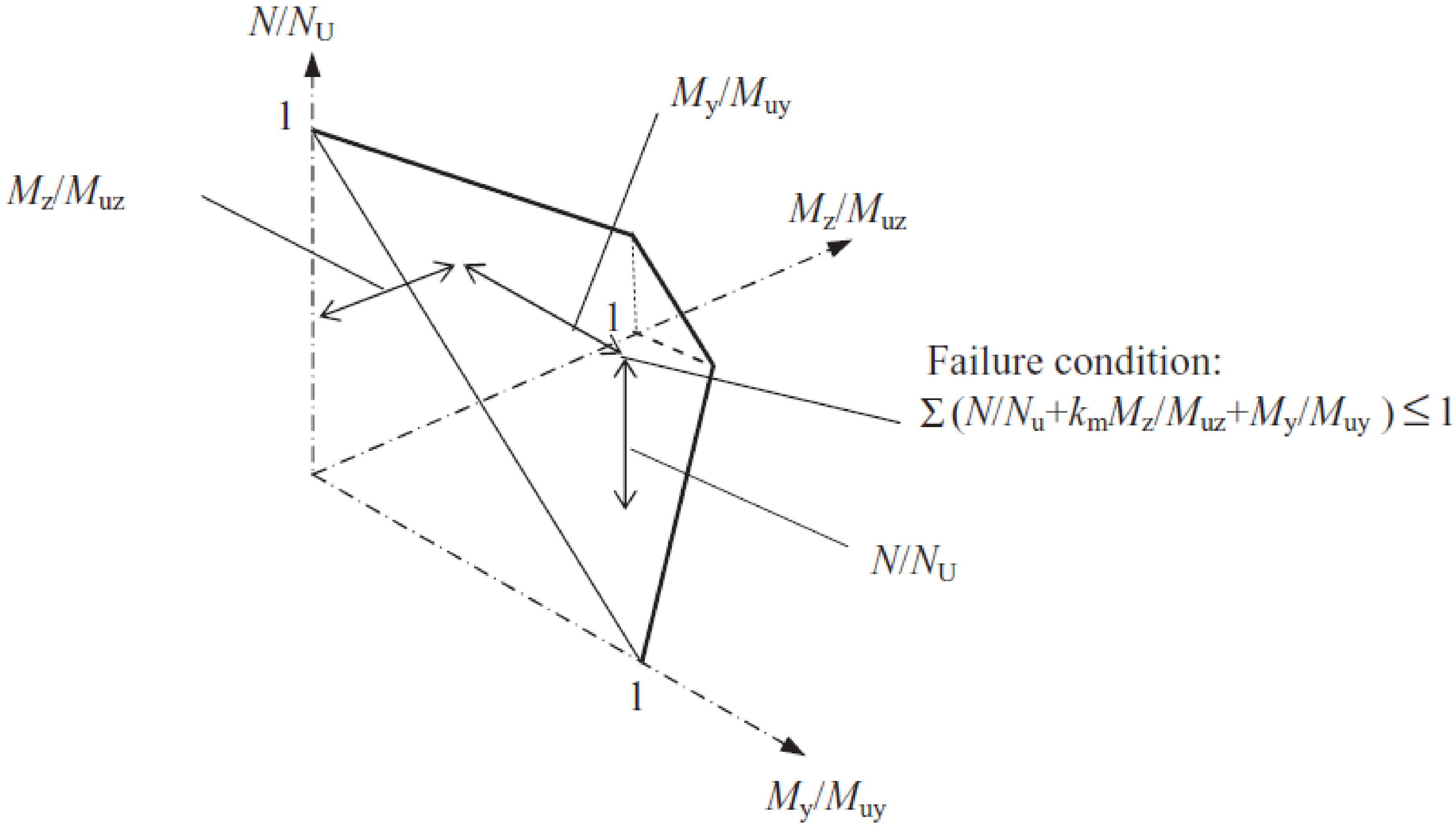


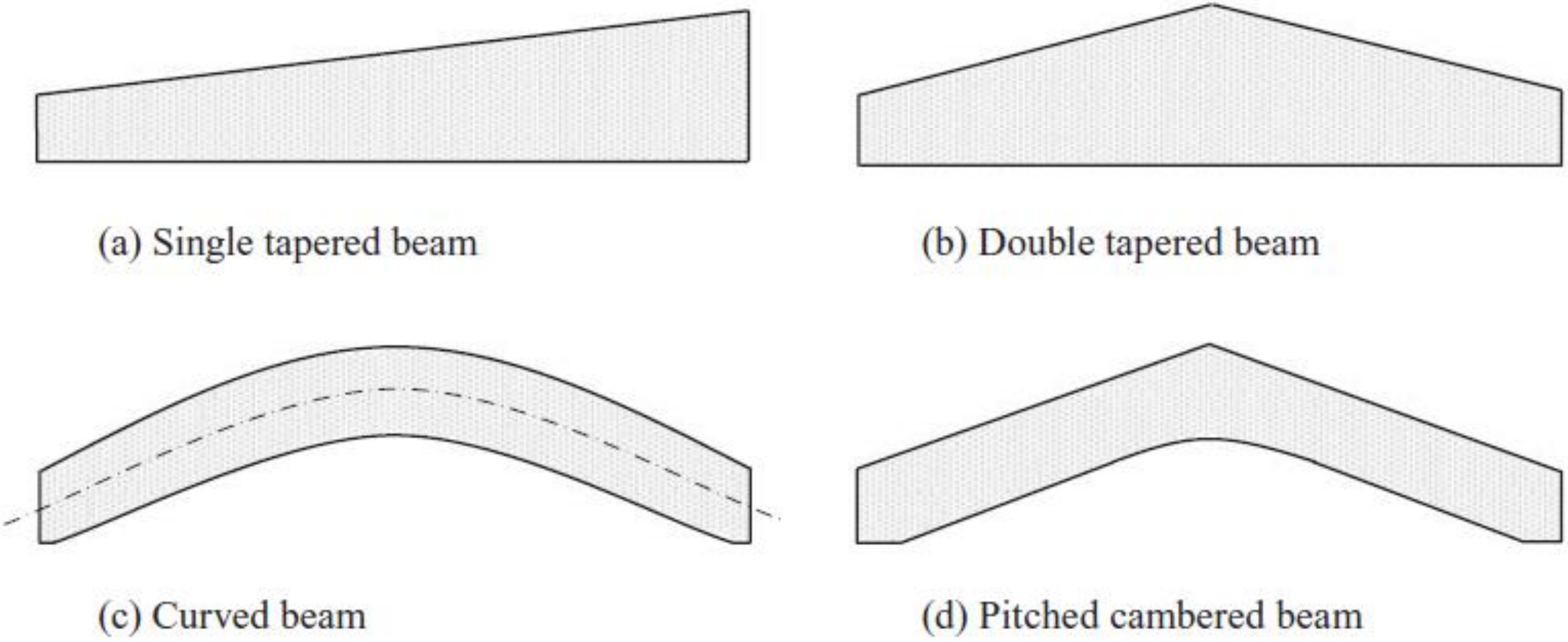
Fig. 5.12. Axial force–moment interaction curve for bi-axial bending when either $\lambda_{\mathrm{rel},y}$ or $\lambda_{\mathrm{rel},z} > 0.3$, and with factor k_m applied to the ratio of moments about the z–z axis.

Beams subjected to either bending or combined bending and compression

In the case *where a combination of moment M about the strong axis y and compressive force N exists, the stresses should satisfy the following expression* according to EN 1995-1-1, Eq. 6.35:

$$\left(\frac{\sigma_{m,d}}{k_{crit} \cdot f_{m,d}}\right)^2 + \frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} \leq 1,0$$

Design of glued laminated members with tapered, curved or pitched curved profiles



(a) Single tapered beam

(b) Double tapered beam

(c) Curved beam

(d) Pitched cambered beam

Single tapered beam (1)

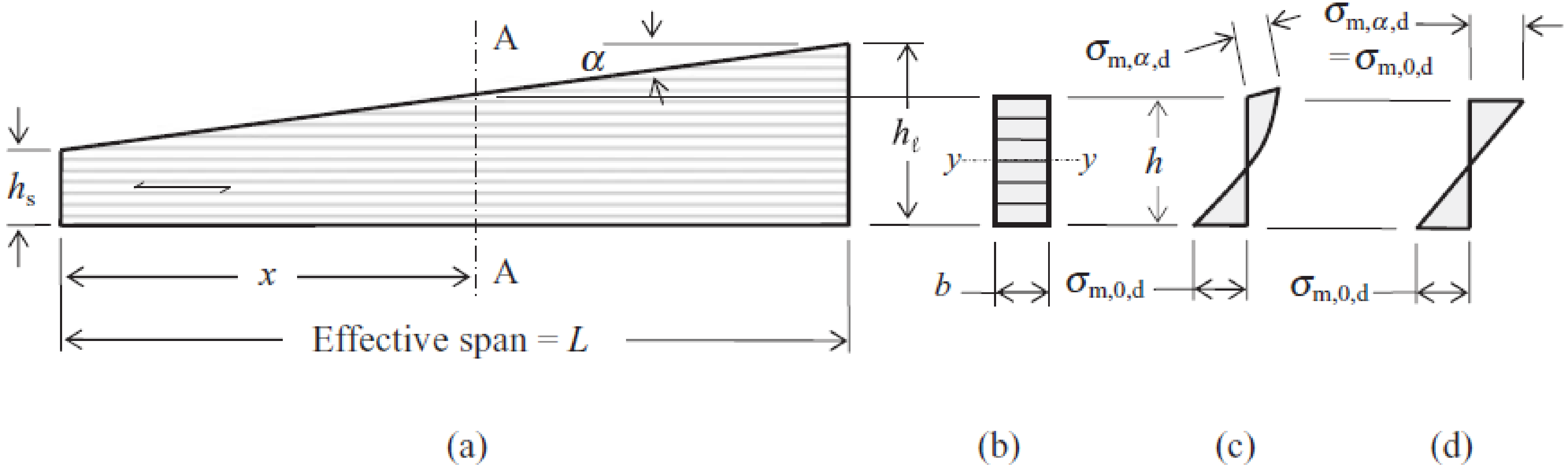


Fig. 6.5. A single tapered beam: (a) elevation; (b) section A–A; (c) bending stress; and (d) bending stress distribution used in EC5 at section A–A.

At the outermost fibre of the tapered edge, the stresses should satisfy the following expression according to EN 1995-1-1, Eq. 6.38:

$$\sigma_{m,\alpha,d} \leq k_{m,\alpha} \cdot f_{m,d}$$

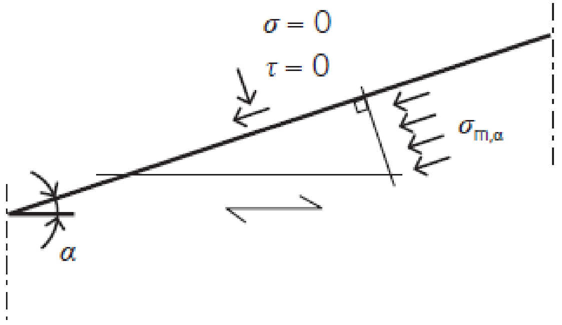
Single tapered beam (2)

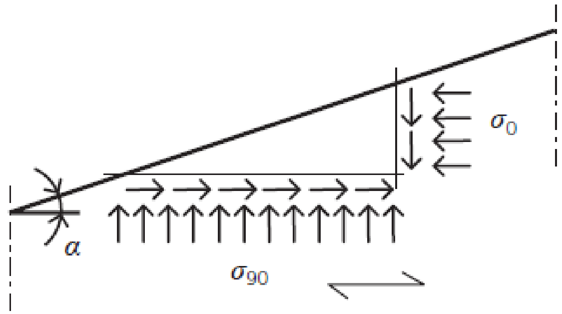
For tensile stresses parallel to the tapered edge according to EN 1995-1-1, Eq. 6.39:

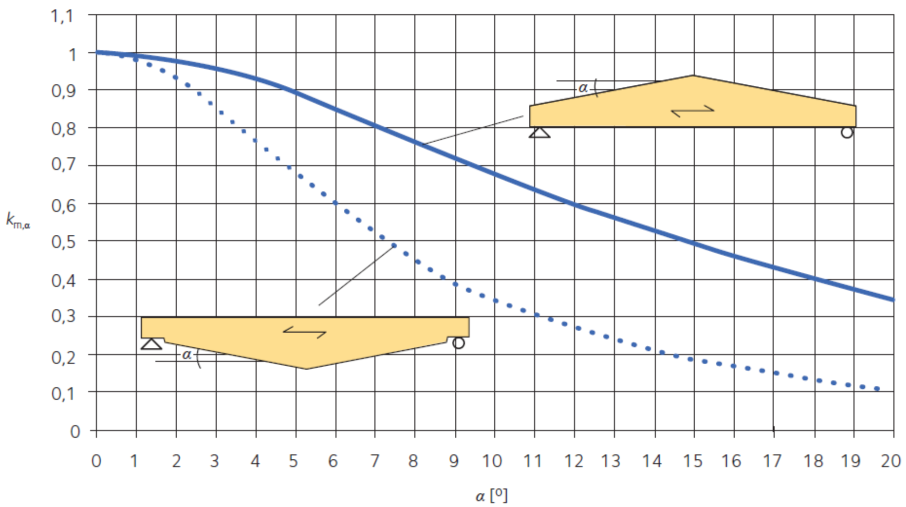
$$k_{m,\alpha}=\frac{1}{\sqrt{1+\left(\frac{f_{m,d}}{0,75\cdot f_{v,d}}\tan\alpha\right)^2+\left(\frac{f_{m,d}}{f_{t,90,d}}\tan^2\alpha\right)^2}}$$

For compressive stresses parallel to the tapered edge according to EN 1995-1-1, Eq. 6.40:

$$k_{m,\alpha}=\frac{1}{\sqrt{1+\left(\frac{f_{m,d}}{0,75\cdot f_{v,d}}\tan\alpha\right)^2+\left(\frac{f_{m,d}}{f_{t,90,d}}\tan^2\alpha\right)^2}}$$







Double tapered beams

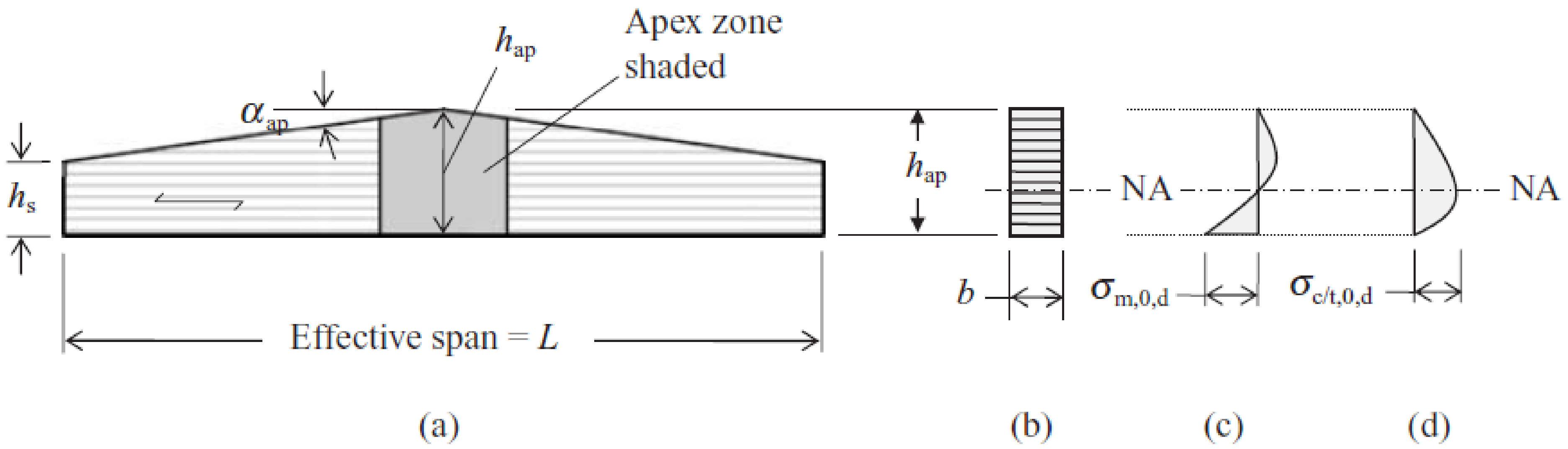


Fig. 6.8. Bending and radial stresses in the apex zone of a double tapered beam: (a) elevation of a double tapered beam; (b) section at apex; (c) bending stress at apex; (d) radial stress at apex.

The apex *bending stress should be calculated as follows:*

$$\sigma_{m,d} \leq f_{m,d} \qquad\qquad \sigma_{m,d} = k_l \frac{6 \cdot M_{ap,d}}{b \cdot h_{ap}^2}$$

$$k_l = 1 + 1,4 \tan \alpha_{ap} + 5,4 \tan^2 \alpha_{ap}$$

Curved and pitched cambered beams (1)

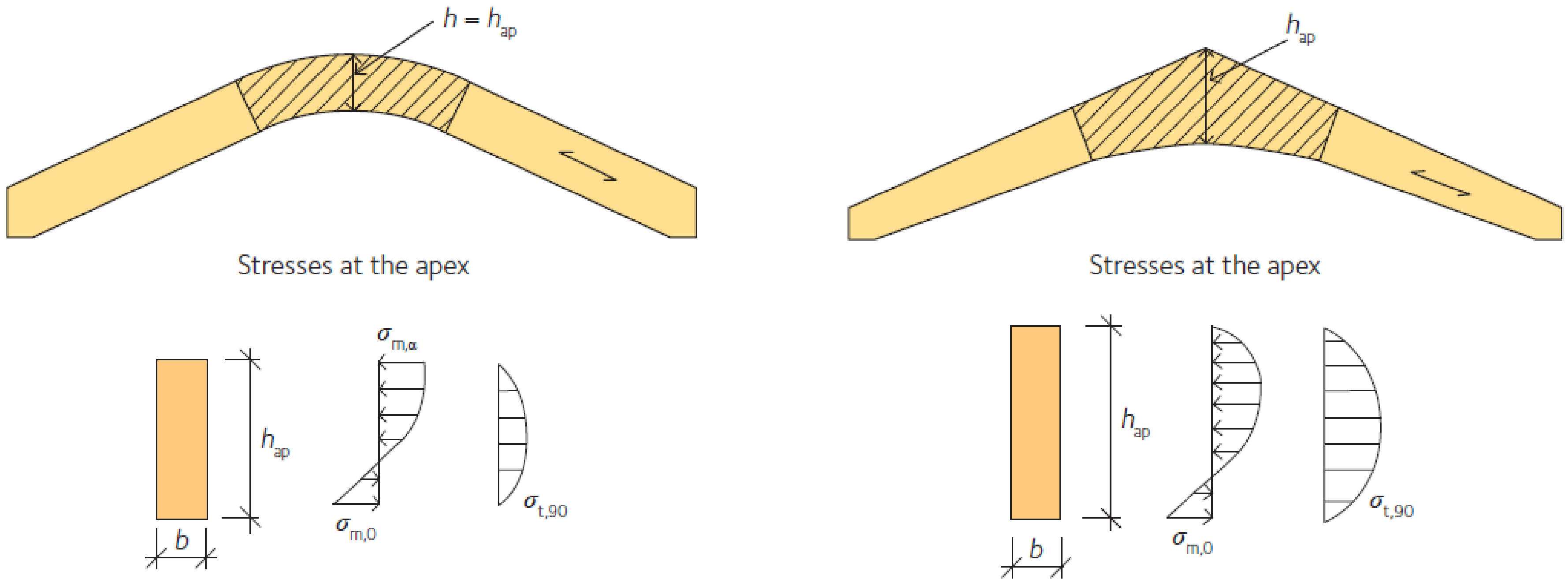


Figure 3.36: Bending stresses and tension stresses perpendicular to the grain for: curved beam (left) and pitched cambered beam (right).

In the apex *zone, the bending stresses should satisfy the following expression:*

$$\sigma_{m,d} \leq k_r \cdot f_{m,d}$$

Curved and pitched cambered beams (2)

The apex bending *stress should be calculated as follows:*

$$\sigma_{m,d} = k_l \frac{6 \cdot M_{ap,d}}{b \cdot h_{ap}^2}$$

$$k_l = k_1 + k_2\left(\frac{h_{ap}}{r}\right) + k_3\left(\frac{h_{ap}}{r}\right)^2 + k_4\left(\frac{h_{ap}}{r}\right)^3$$

$$k_1 = 1 + 1,4\tan\alpha_{ap} + 5,4\tan^2\alpha_{ap}$$

$$k_2 = 0,35 - 8\tan\alpha_{ap}$$

$$k_3 = 0,6 + 8,3\tan\alpha_{ap} - 7,8\tan^2\alpha_{ap}$$

$$k_4 = 6\tan^2\alpha_{ap}$$

$$k_r = \begin{cases} 1; & kai\ {r_{in}}/{t} \geq 240 \\ 0,76 + 0,001\dfrac{r_{in}}{t}; & kai\ {r_{in}}/{t} < 240 \end{cases}$$

$M_{ap,d}$ – *bending moment in the apex*

Tension perpendicular to the grain in the apex (1)

The greatest *tensile stress perpendicular to the grain due to the bending moment should be calculated as follows:*

$$\sigma_{t,90,d} \leq k_{dis} \cdot k_{vol} \cdot f_{t,90,d}$$

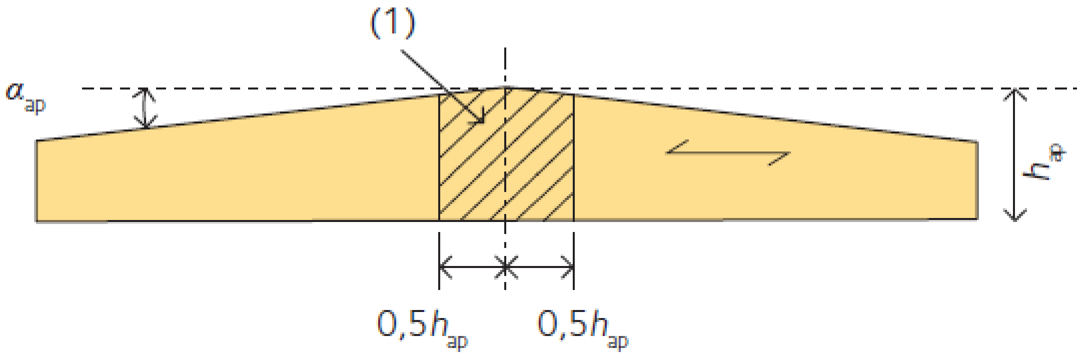
$$k_{vol} = \begin{cases} 1; & solid\ timber \\ \left(\dfrac{V_0}{V}\right)^{0,2}; & glulam \end{cases}$$

$$k_{dis} = \begin{cases} 1{,}4 & \text{for double tapered and curved beams} \\ 1{,}7 & \text{for pitched cambered beams} \end{cases}$$

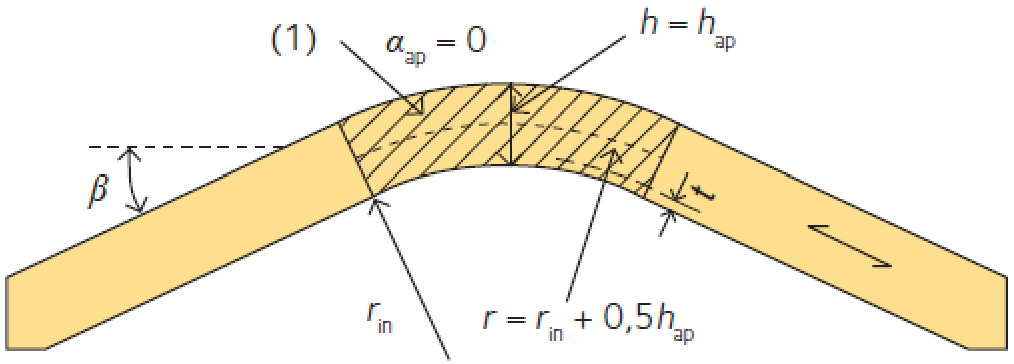
V_0 is the reference volume of 0,01m^3;

V is the stressed volume of the apex zone, in m^3, (see Figure 6.9) and should not be taken greater than $2V_b/3$, where V_b is the total volume of the beam.

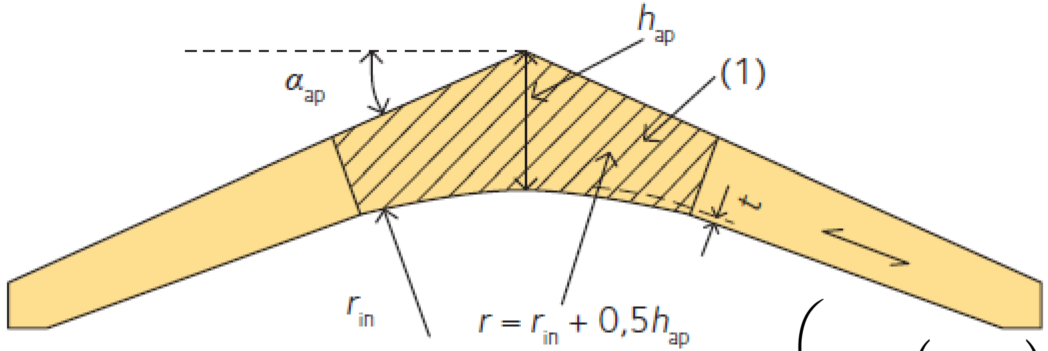
Tension perpendicular to the grain in the apex (2)



$$V = b \cdot \left(h_{ap}\right)^2$$



$$V = \frac{\beta \cdot \pi}{180} b\left(h_{ap}^2 + 2h_{ap} \cdot r_{in}\right)$$



$$V = b\left(\sin\left(\alpha_{ap}\right)\cos\left(\alpha_{ap}\right)\left(r_{in} + h_{ap}\right)^2 - r_{in}^2 \frac{\alpha_{ap}\pi}{180}\right)$$

Tension perpendicular to the grain in the apex (3)

(8) The greatest tensile stress perpendicular to the grain due to the bending moment should be calculated as follows:

$$\sigma_{t,90,d} = k_p \frac{6\, M_{ap,d}}{b\, h_{ap}^2} \tag{6.54}$$

with:

$$k_p = k_5 + k_6 \left(\frac{h_{ap}}{r}\right) + k_7 \left(\frac{h_{ap}}{r}\right)^2$$

$$k_5 = 0{,}2\ \tan\alpha_{ap}$$

$$k_6 = 0{,}25\ -\ 1{,}5\ \tan\alpha_{ap}\ +\ 2{,}6\ \tan^2\alpha_{ap}$$

$$k_7 = 2{,}1\ \tan\alpha_{ap}\ -\ 4\ \tan^2\alpha_{ap}$$

$M_{ap,d}$ is the design moment at apex resulting in tensile stresses parallel to the inner curved edge;

Serviceability Limit States
of timber structures

Serviceability limit states

General data

Deformations should be less than its limit values to ensure:

- Visual and functional requirements of the building;
- To avoid brittle failure of the decoration;
- Water drainage from roofs would be ensured;
- The proper construction should be ensured of the entire life of the building;
- Ensure a sufficient level of comfort due to vibration.

Calculation of timber structures. Serviceability Limit States

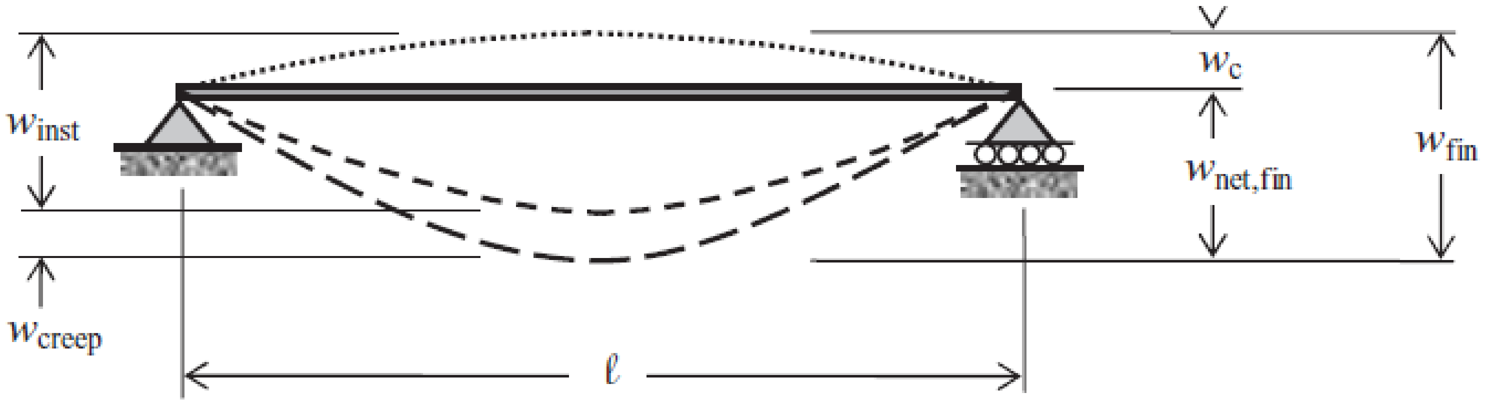


Fig. 1. Components of timber beam‘s deflection

w_c – is the pre-camber;

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.

Limiting values for deflections of beams according to Eurocode 5 (1)

The net deflection below a straight line between the supports, $w_{net,fin}$, should be taken as:

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

Limit deflection values for deflections of beams according to Eurocode 5:

Table 7.2 – Examples of limiting values for deflections of beams

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

Determination of deflections according to Eurocode 5 (1)

The instantaneous deformation, u_{inst} should be calculated for the characteristic combination of actions, see EN 1990, clause 6.5.3(2) a), using mean values of the appropriate moduli of elasticity, shear moduli and slip moduli.

The final deformation, u_{fin} should be calculated for the quasi-permanent combination of actions according to EN 1990, clause 6.5.3(2) c).

If structure consists of members or components having different creep behavior, the final deformation should be calculated using final mean values of the appropriate moduli of elasticity, shear moduli and slip moduli.

Determination of deflections according to Eurocode 5 (2)

For structures consisting of members, components and connections with the same creep behaviour and under the assumption of a linear relationship between the actions and the corresponding deformations, as a simplification of 2.2.2(3), the final deformation, u_{fin}, may be determined as:

$$u_{fin} = u_{fin,G} + u_{fin,Q_1} + u_{fin,Q_i}$$

where:

$$u_{fin,G} = u_{inst,G}\left(1 + k_{def}\right);$$

$$u_{fin,Q_1} = u_{inst,Q_1}\left(1 + \psi_{2,1} \cdot k_{def}\right);$$

$$u_{fin,Q_i} = u_{inst,Q_i}\left(\psi_{0,i} + \psi_{2,i} \cdot k_{def}\right).$$

Determination of deflections according to Eurocode 5 (3)

$u_{inst,G}$; u_{inst,Q_1}; u_{inst,Q_i} – are the instantaneous deformations for actions G, Q_1, Q_i, respectively;

$\psi_{2,1}$; $\psi_{2,i}$ – are the factors for quasi-permanent value of variable actions;

$\psi_{0,i}$ – are the factors for the combination value of variable actions;

k_{def} – factor which takes into account the influence of the service class on the creep deflection.

Determination of deflections according to Eurocode 5 (4)

Recommended values for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A : domestic, residential areas	0,7	0,5	0,3
Category B : office areas	0,7	0,5	0,3
Category C : congregation areas	0,7	0,7	0,6
Category D : shopping areas	0,7	0,7	0,6
Category E : storage areas	1,0	0,9	0,8
Category F : traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
– Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
– Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
– Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
NOTE The ψ values may be set by the National annex.			
* For countries not mentioned below, see relevant local conditions.			

Determination of deflections according to Eurocode 5 (5)

Table 3.2 – Values of k_{def} for timber and wood-based materials

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			
	Type EN 636-1	0,80	–	–
	Type EN 636-2	0,80	1,00	–
	Type EN 636-3	0,80	1,00	2,50
OSB	EN 300			
	OSB/2	2,25	–	–
	OSB/3, OSB/4	1,50	2,25	–
Particleboard	EN 312			
	Type P4	2,25	–	–
	Type P5	2,25	3,00	–
	Type P6	1,50	–	–
	Type P7	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	–

The increase of deflection due to shear deformations (1)

For structural steel, the ratio $E_{0,mean}/G_{mean}$ is approximately 2 and consequently in steel design when considering normal sections, the shear deformation effect is generally ignored. With timber, however, $E_{0,mean}/G_{mean}$ is approximately 16 and for practical beam design, h/l will range between 0.1 and 0.05 resulting in a shear deformation between 5 and 20 % of the flexural value.

As this is significant percentage, the effect of shear deformation must be taken into account when designing timber structures.

Shear deformation can be expressed in terms of the flexural deflection multiplied by a shear amplification factor:

$$u_{inst} = u_{inst,M} + u_{inst,V} = u_{inst,M} \cdot k_{shear}$$

The increase of deflection due to shear deformations (2)

$$k_{shear} = \left(1 + 0,96\left(\frac{E_{0,mean}}{G_{0,mean}}\right)\left(\frac{h}{l}\right)^2\right)$$

Uniformly distributed load of a simply supported beam

$$k_{shear} = \left(1 + 1,20\left(\frac{E_{0,mean}}{G_{0,mean}}\right)\left(\frac{h}{l}\right)^2\right)$$

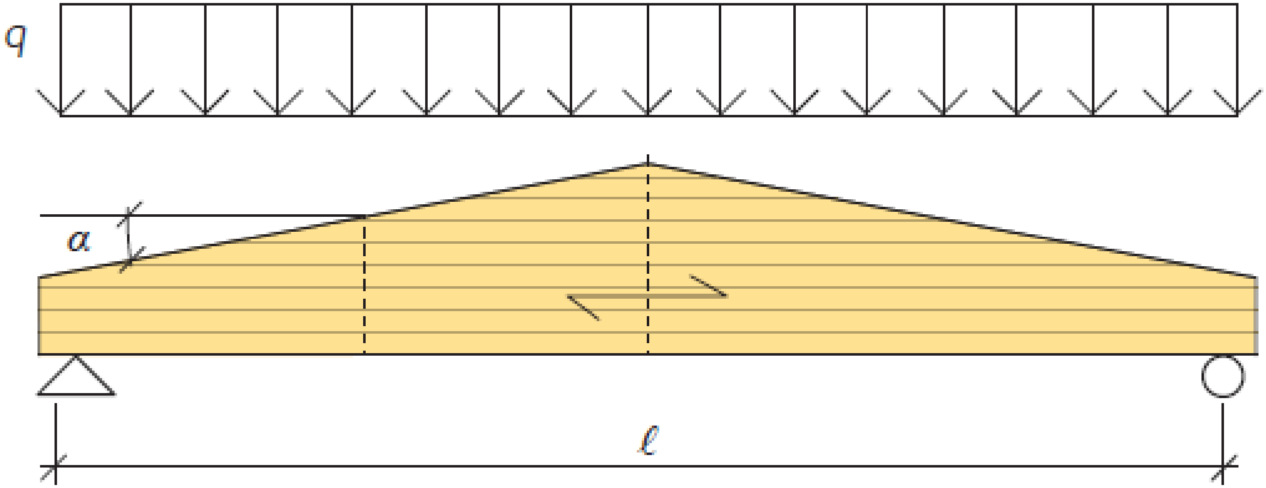
Point load at mid span of a simply supported beam

$$k_{shear} = \left(1 + 0,3\left(\frac{E_{0,mean}}{G_{0,mean}}\right)\left(\frac{h}{l}\right)^2\right)$$

Point load at the end of a cantilever

…

The increase of deflection due to shear deformations (3)



$$u_{inst} = u_{inst,M} + u_{inst,V}$$

$$\chi = 1,2$$

$$u_{inst} = u_{inst,M} + u_{inst,V} = \boxed{\frac{5 \cdot q \cdot L^4}{384 \cdot E_{0,mean} \cdot I} \cdot k_m} + \boxed{\chi \frac{q \cdot L^2}{8 \cdot G_{mean} \cdot A} \cdot k_v}$$

Deflection from flexure

Deflection from shear

$$k_m = \left(\frac{h}{H}\right)^3 \cdot \frac{1}{0,15 + 0,85 \cdot h/H}$$

$$k_v = \frac{2}{1 + (H/h)^{2/3}}$$

70

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PUB-WOOD

Structural Timber design according to Eurocode 5

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