Advanced training

Timber

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Chapter 1: EN 1995 Code check , 1D element

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1.1 Basics of design

Within article **2.2.2** the possible procedures for carrying out the analysis of a Timber structure are specified. The following procedures are supported:

1. First order linear elastic analysis using mean values E_{mean} and G_{mean} for the stiffness properties.
2. Second order linear elastic analysis, using values E_d and G_d for the stiffness properties.

The design values are determined as given in **article 2.4.1(2).**

$$E_d = \frac{E_{mean}}{\gamma_m} \text{ and } G_d = \frac{G_{mean}}{\gamma_m}$$

> **Note: The design values of the stiffness properties are used only in case of 2nd order analysis, not in case of regular non-linear analysis.**

> **Note: The design values of the stiffness properties are used for both non-linear combination types i.e. 'Ultimate' and ' Service'**

For the partial safety factor γ_m specified in **article 2.4** reference is made to the Theorical Background for the National Annex to EN 1995

Table 2.3 – Recommended partial factors γ_M for material properties and resistances

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

1.2   Material Properties

The characteristic values of the material properties which are provided by default have been taken from the following references:

- Solid Timber: EN 338 Ref.[4]
- Glued Laminated Timber: EN 1194 : Ref.[5] and Ref.[7]

> **Note : The shear modulus $G_{0,05}$ is not given in any material code reference but is required for the Timber Code Check. This value is calculated during the Check as $E_{0,005}/16$ as given in Ref [7] pp 109 & 209**

1.2.1 Strength modification factor k_{mod}

The values of the modification factor k_{mod} are taken by default from **Table 3.1.**

Table 3.1 – Values of k_{mod}

Material	Standard	Service class	Load-duration class				
			Permanent action	Long term action	Medium term action	Short term action	Instanta-neous 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

As specified in **article 3.1.3(2)**, if a load combination consists of actions belonging to different load-duration classes a value of k_{mod} is used which corresponds to the action with the shortest duration.

1.2.2 Deformation modification factor k_{def}

The values of the deformation factor k_{def} are taken by default from **Table 3.2.**

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			
	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	–

In addition, as specified in **article 3.2(4)** the values given in **Table 3.2.** can be increased by **1.00** for timber which is installed at or near its fibre saturation point. This can be set in the Timber settings.

1.2.3 Depth factor k_h

In case the proper setting is activated in the Timber Setup the depth factor k_h is taken into account for both Solid Timber and Glued Laminated Timber.

According to **articles 3.2(3)** and **3.3(3)** the effect of the member size may only be accounted for in case a rectangular section is used. A rectangular section is defined as one of the following:
- **RECT** (Timber)
- **Rectangle** (Geometric shapes)
- **Full rectangular (Form code 7)**

In the following tables **b** is taken as the <u>smallest</u> dimension of the rectangle and **h** as the <u>biggest</u> dimension of the rectangle.
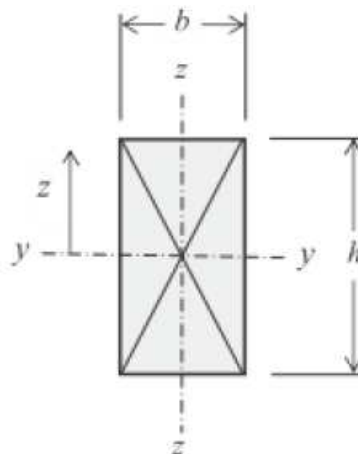
The factor k_h is calculated based on the type of timber:

Solid timber

Action	Conditions	k_h
Tension N_{Ed}	Material density is ≤ **700 kg/m³** Dimension **h < 150 mm**	$k_h = \min\begin{cases}(\frac{150}{h})^{0,2}\\1,3\end{cases}$
Bending $M_{y,Ed}$	Material density is ≤ **700 kg/m³** Dimension **h < 150 mm**	$k_h = \min\begin{cases}(\frac{150}{h})^{0,2}\\1,3\end{cases}$
Bending $M_{z,Ed}$	Material density is ≤ **700 kg/m³** Dimension **b < 150 mm**	$k_h = \min\begin{cases}(\frac{150}{b})^{0,2}\\1,3\end{cases}$

In all other cases k_h remains **1,00**

> **Note: The code clearly defines the width as the '' maximum cross-sectional dimension'' thus h is used.**

Glued Laminated timber

Action	Conditions	k_h
Tension N_{Ed}	Dimension **h < 600 mm**	$k_h = \min\begin{cases}(\frac{600}{h})^{0,1}\\1,1\end{cases}$
Bending $M_{y,Ed}$	Dimension **h < 600 mm**	$k_h = \min\begin{cases}(\frac{600}{h})^{0,1}\\1,1\end{cases}$
Bending $M_{z,Ed}$	-	$k_h = 1$

In all other cases k_h remains **1,00**

> **Note: According to Ref.[7] pp 84 for glued laminated members ''…(k_h) will only apply to the beam depth where the section is loaded perpendicular to the plane of the wide faces of the laminations'' therefore k_h is not accounted for in $M_{z,Ed}$ bending.**

1.3 Basis of Structural Analysis

For plane frames, the following imperfections can be defined according to **article 5.4.4(2):**

- Global Imperfections Φ using formula **(5.1)**
  The value of Φ in radians should as a minimum be taken as:
  $\Phi = 0,005$ for h ≤ 5 m
  $\Phi = 0,005\sqrt{5lh}$ for h >5 m
  Where h is the height of the structure or the length of the member, in m.

- Bow imperfections **e** using formula **(5.2)**
  The value of **e** should as a minimum be taken as:
  e= 0,0025 l

In addition, the shape of the elastic critical buckling mode can be used as unique global and local imperfection.

1.4 Ultimate Limit State

The Ultimate Limit State verifications are executed according to **Section 6.**

1.4.1 Tension parallel to the grain

The tension parallel to the grain check is executed according to **article 6.1.2.**

This check is executed only in case a tensile normal force N_{Ed} is present.

1.4.2 Compression parallel to the grain

The compression parallel to the grain check is executed according to **article 6.1.4.**

This check is executed only in case a compressive normal force N_{Ed} is present.

1.4.3 Compression perpendicular to the grain

The compression perpendicular to the grain check is executed according to **article 6.1.4.**

This check was revised in the addendum A1 to EN 1995-1-1 , see Ref [3]

This check is executed only in case a shear force $V_{z,Ed}$ is present. The check is executed on positions where there is a jump in the $V_{z,Ed}$ diagram.
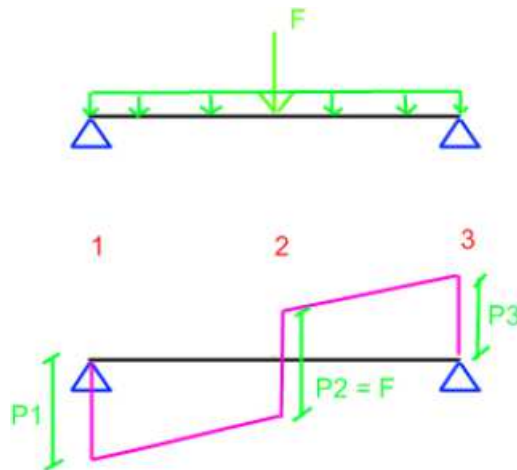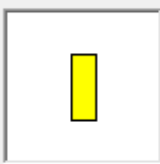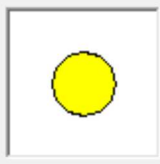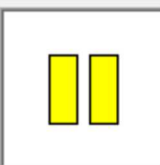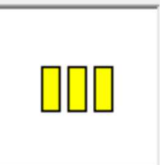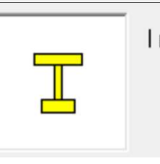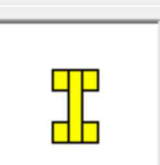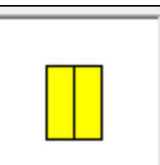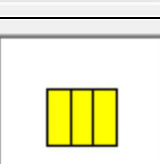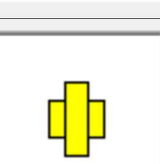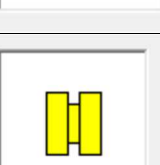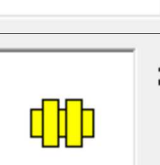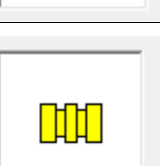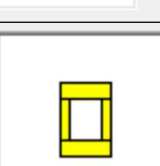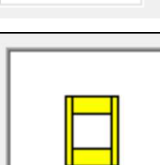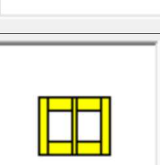

When evaluating the jumps in the shear force diagram of a given member, also the shear diagrams of the adjacent members are accounted for. Adjacent members are defined as neighboring within the same buckling system.

Effective contact area

The effective contact area **A$_{ef}$** is determines as follows: $A_{ef} = l_{ef} * b$

The width **b** concerns the contact width of the cross-section and is determined by default as follows:

Cross-section	Contact Width b	Cross-section	Contact Width b
RECT	B	CIRC	10 mm
2 Rect	2* tha	3 Rect	3*tha
T profile	tha	I non-symm	Min (Bb;Bc)
I symm	tha+ 2*Bb	2 Rect	2*B
3 Rect	3*B	Cross	tha
2+1 El.fill	2* Bb	2+3 El.fill	2* Bb
3+2 El.fill	3*Ba	Box	Ba
Box 1	Bb + 2* Ba	Box 2	2*Bb + 3* Ba
Any other section	10 mm		

The length l_{ef} concerns the effective contact length and is calculated from the contact length **L** as specified in **article 6.1.5(1).**

> **Note: Both the contact width and the contact length can be overruled at any position for any cross-section using 'Compression perpendicular to the grain' additional data.**

Factor $k_{c,90}$

The factor $k_{c,90}$ is by default determined for a member on 'discrete supports' as follows:
- In case $l_1 \geq 2 * h$ AND **Solid timber** with name starting with "**C**" (= Softwood) => $k_{c,90}$ **= 1,50**
- In case $l_1 \geq 2 * h$ AND **Glulam timber** with **l ≤ 400 mm** => $k_{c,90}$ **= 1,75**
- In all other cases **kc,90 = 1,00**.

For a member on 'continuous supports', the factor $k_{c,90}$ is determined as follows:
- In case $l_1 \geq 2 * h$ AND **Solid timber** with name starting with "**C**" (= Softwood) => $k_{c,90}$ **= 1,25**
- In case $l_1 \geq 2 * h$ AND **Glulam timber** => **kc,90 = 1,50**
- In all other cases **$k_{c,90}$ = 1,00**

> **Note: By default the support condition is taken as 'discrete'. Using 'Compression perpendicular to the grain' additional data the support condition can be set to 'continuous'.**

In the above, in case there are neighbouring jumps to the left and right of the considered position, the length l_1, between the contact lengths of two jumps, is taken as the minimal value of the length to the left and the length to the right.
The distance **a** is measured to the end of the buckling system, not the end of the actual member.
The depth h of a member is determined as **h = (y_max – y_min)**
With:
- y_max = maximal vertical fiber coord.
- y_min = minimal vertical fiber coord.

1.4.4 Bending

The Bending check is executed according to **article 6.1.6**

This check is executed only in case a bending moment $M_{y,Ed}$ and/or $M_{z,Ed}$ is/are present.

For the determination of the stress redistribution factor k_m, a rectangular section is defined as one of the following:
- **RECT** (Timber)
- **Rectangle** (Geometric shapes)
- **Full rectangular (Form code 7)**

1.4.5 Shear

The Shear check was executed only in case a shear force $V_{y,Ed}$ and/or $V_{z,Ed}$ is/are present.

The design shear stress τ_d is calculated as follows:

$$\tau_{y,d} = \frac{V_{y,Ed} * (\tau_{y,unit})}{k_{cr}}$$

$$\tau_{z,d} = \frac{V_{z,Ed} * (\tau_{z,unit})}{k_{cr}}$$

With $\tau_{y,unit}$ and $\tau_{z,unit}$ the maximal unit shear stresses per fibre as taken from the cross-section.

The shear modification factor k_{cr} is determined depending on the National Annex.
Reference is made to the Theorical Background for the National Annex to EN 1995.
By default this factor is taken as **0,67** for both Solid and Glued Laminated Timber.

Interaction of combined shear
EN 1995-1-1 does not give an interaction equation in case of combined shear.
The following interaction check is executed as NCCI:

$$\left(\frac{\tau_{y,d}}{f_{v,d}}\right)^2 + \left(\frac{\tau_{z,d}}{f_{v,d}}\right)^2 \leq 1$$

The interaction equation is taken from the German National Annex Ref .[8].

1.4.6 Torsion

The Torsion is executed according to **article 6.1.8**

This check is executed only in case a torsional moment **T$_{Ed}$** is present.

The design shear stress $\tau_{tor,d}$ is calculated as follows:

$$\tau_{tor,d} = T_{Ed} * \left(\tau_{tor,unit}\right)$$

With $\tau_{tor,unit}$ the maximal unit torsion stresses per fibre as taken from the cross-section.

> **Note: In case the unit torsion stress is not calculated in the cross-section and a torsional moment is present, the torsion check cannot be executed. In this case the proper analysis should be set in the cross-section.(For example activating the 2D FEM analysis)**

For the determination of the shape factor **k$_{shape}$**, a distinction is made between rectangular and circular cross sections.
A rectangular section is defined as one of the following:
- **RECT** (Timber)
- **Rectangle** (Geometric shapes)
- **Full rectangular** (**Form code 7**)

With **b** taken as the smallest dimension of the rectangle and **h** as the biggest dimension of the rectangle.
A circular section is defined as one of the following:
- **CIRC** (Timber)
- **Circle** (Geometric shape)
- **Full circle** (**Form code 11**)

For any sections which are nor rectangular nor circular **k$_{shape}$** is taken as **1,00.**

Interaction of combined shear and torsion
EN 1995-1-1 does not give an interaction equation for combined shear and torsion.
The following interaction check is executed as NCCI:

$$\left(\frac{\tau_{tor,d}}{k_{shape} * f_{v,d}}\right) + \left(\frac{\tau_{y,d}}{f_{v,d}}\right)^2 + \left(\frac{\tau_{z,d}}{f_{v,d}}\right)^2 \leq 1$$

This interaction equation is taken from the German National Annex Ref .[8].as well as Ref .[7].pp 125.

1.4.7 Combined Bending and Axial Tension

The Combined Bending and Axial Tension check is executed according to **article 6.2.3**.

This check is executed only in case the following conditions are met:
- A tensile normal force **N$_{Ed}$** is present
- A bending moment **M$_{y,Ed}$** and/or **M$_{z,Ed}$** is/are present

1.4.8 Combined Bending and Axial Compression

The Combined Bending and Axial Tension check is executed according to **article 6.2.4**.

This check is executed only in case the following conditions are met:
- A compressive normal force **N$_{Ed}$** is present
- A bending moment **M$_{y,Ed}$** and/or **M$_{z,Ed}$** is/are present

1.4.9 Columns subjected to compression or combined compression and bending

The stability check for members subjected to compression or combined compression and bending is executed according to **article 6.3.2**.

This check is executed only in case a compressive normal force **N$_{Ed}$** is present.

For the calculation of the buckling ratio several methods are available:
- General formula (standard method)
- From Stability Analysis
- Manual input

Calculation of Buckling ratio – General Formula

For the calculation of the buckling ratios, some approximate formulas are used. These formulas are treated in reference [9], [10] and [11].

The following formulas are used for the buckling ratios (Ref.[9],pp.21):

For a non-sway structure:

$$l/L = \frac{(\rho_1\rho_2+5\rho_1+5\rho_2+24)(\rho_1\rho_2+4\rho_1+4\rho_2+12)2}{(2\rho_1\rho_2+11\rho_1+5\rho_2+24)(2\rho_1\rho_2+5\rho_1+11\rho_2+24)}$$

For a sway structure:

$$l/L = x\sqrt{\frac{\pi^2}{\rho_1 x}+4|}$$

With:
- L: System length
- E Modulus of Young (mean)
- I Moment of inertia
- C$_i$ Stiffness in node i
- M$_i$ Moment in node i
- Φ$_i$ rotation in node i

$$x = \frac{4\rho_1\rho_2+\pi^2\rho_1}{\pi^2(\rho_1+\rho_2)+8\rho_1\rho_2}$$

$$\rho_i = \frac{C_i L}{EI}$$

$$C_i = \frac{M_i}{\phi_i}$$

The values for M$_i$ and Φ$_i$ are approximately determined by the internal forces and the deformations, calculates by load cases which generate deformation forms, having an affinity with the buckling shape.

The following load cases are considered:
- load case 1: on the beams, the local distributed loads qy=1 N/m and qz=-100 N/m are used, on the columns the global distributed loads Qx = 10000 N/m and Qy =10000 N/m are used.
- load case 2: on the beams, the local distributed loads qy=-1 N/m and qz=-100 N/m are used, on the columns the global distributed loads Qx = -10000 N/m and Qy= -10000 N/m are used.

In addition, the following limitations apply (Ref[9],pp.21):
- The values of ρ_i are limited to a minimum of 0.0001
- The values of ρ_i are limited to a maximum of 1000
- The indices are determined such that $\rho_1 \geq \rho_2$
- Specifically for the non-sway case, if $\rho_1 \geq 1000$ and $\rho_2 \leq 0,34$ the ratio l/L is set to 0,7

The used approach gives good results for frame structures with perpendicular rigid or semi-rigid beam connections. For other cases, you have to evaluate the presented bucking ratios. In such cases a more refined approach (from stability analysis) can be applied.

> **Note: The following ryle applies specifically to ky: in case both the calculation for load case 1 and load case 2 return ky=1 then ky is taken as kz. This rule is used to account for possible rotations of the cross-section.**

Calculation of Buckling ratio – From Stability Analysis

When member buckling data from stability are defined, the critical buckling load **Ncr** for a prismatic member is calculated as follows:

$$N_{cr} = \lambda * N_{Ed}$$

Using Euler's formula, the buckling ratio k can then be determined:

$$N_{cr} = \frac{\pi^2.E.I}{(k.s)^2} \Rightarrow k = \frac{1}{s}.\sqrt{\frac{\pi^2.E.I}{N_{cr}}}$$

With:
- λ Critical load factor for the selected stability combination
- N_{Ed} Design loading in the member
- E   Modulus of Young (mean)
- I   Moment of inertia
- s   Member length

1.4.10 Beams subjected to bending or combined bending and compression

The stability check for members subjected to bending or combined bending and compression check is executed according to **article 6.3.3.**

This check is executed only in case a bending moment **$M_{y,Ed}$** is present.

The critical moment **$M_{y,crit}$** is determined using the general formula **(6.31).**

> **Note: The shear modulus $G_{0,05}$ is not given in any material code reference but is required for the Timber Code Check. This value is calculated during the Check as $E_{0,05}/16$ as given is Ref.[7] pp 109 and 209**

Effective length L_{ef}

The effective length for Lateral Torsional Buckling is modified depending on the loading type and load position.

Loading Type

The ratio between the effective length **L_{ef}** and the span length is determined according to **Table 6.1.**

The current moment distribution is compared with several standard moment distributions.

These standard moment distributions are:
- Moment line generated by a distributed q load
- Moment line generated by a concentrated F load
- Moment line which has a maximum at the start or at the end of the beam

The standard moment distribution which is closest to the current moment distribution determines the loading type.

Table 6.1 distinguishes between simply supported and cantilever beams. A cantilever is defined as a member at the end of a buckling system which has free ends for both buckling about the y-y and z-z axis. In addition, the LTB length should correspond to the full system length of the buckling system.

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 2*h* and may be decreased by 0,5*h* for a load at the tension edge of the beam.

Loading position

As specified in **Table 6.1** the effective length L_{ef} is modified depending on the load position.

In case **Influence of load position** in the buckling data is set to '**Destabilizing**':
$$L_{ef} = L_{ef} + 2h$$

In case **Influence of load position** in the buckling data is set to '**Stabilizing**':
$$L_{ef} = L_{ef} - 0,5h$$

With **h** taken as the maximal vertical dimension of the cross-section:
- $h = (z_{max} - z_{min})$
- z_{max} = maximal vertical fibre coordinate
- z_{min} = minimal vertical fibre coordinate

Interaction

In case, beside a bending moment $M_{y,Ed}$, also a compressive normal force N_{Ed} is present the following additional check according to formula **(6.35)** is executed:

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

> **Note: This interaction only accounts for lateral torsional buckling and weak axis flexural buckling. Strong axis flexural buckling and bending about the weak axis are not accounted for.**
>
> **In case the German National Annex is chosen, additional interaction checks are executed which do account for these effects. Reference is made to the Theoretical Background for the National Annex to EN 1995.**

1.4.11 Members with varying cross-section or curved shape

Specific checks for members with varying cross-sections or curved shapes are executed according to **article 6.4.**

In general, any non-uniform member is checked in each section using the actual cross-section properties at that section.

In addition, specific rules are applied for members which are detected as tapered or curved:
- Rules for single tapered beams are given in **article 6.4.2**
- Rules for double tapered beams are given in **article 6.4.3**
- Rules for curved beams are given in **article 6.4.3**

Note: Pitches cambered members are not supported

All additional checks for tapered and curved members are only executed in case a bending moment $M_{y,Ed}$ is present.

Single tapered beams

The specifics for single tapered beams are given in **article 6.4.2**
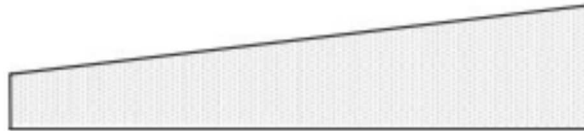
Definition

A single tapered member is defined as a member with the following characteristics:
- A rectangular cross-section
  - **RECT** (Timber)
  - **Rectangle** (Geometric shapes)
  - **Full rectangular (Form code 7)**
- Material type **Glued, laminated**
- Only the height varies linearly <u>over the total member length</u>
- The height at the biggest end has to be larger than the width of the cross-section
- The haunch/arbitrary alignment is one of the following:
  - Top surface
  - Top left
  - Top right
  - Bottom surface
  - Bottom left
  - Bottom right

Note: EN 1995-1-1 only gives rules in case of members with a single tapered edge thus a default alignment (both faces tapered) is not supported

Only tapered members which have a geometry which complies with the above definition are seen as actual tapered beams for which the additional rules of EN 1995-1-1 apply.

Stresses at the tapered edge

Depending on the sign of the moment, the tapered edge is either in compression or in tension.

Alignment top & $M_{y,Ed} > 0$ → Tapered edge in **tension**

Alignment top & $M_{y,Ed} < 0$ → Tapered edge in **compression**

Alignment bottom & $M_{y,Ed} > 0$ → Tapered edge in **compression**

Alignment bottom & $M_{y,Ed} < 0$ → Tapered edge in **Tension**

Note: The stresses at the tapered edge are based solely on My,Ed, not on the combination of N_{Ed} , $M_{y,Ed}$, $M_{z,Ed}$ since their interaction is taken into account in the interaction formulae.

Strength Reduction factor

Depending on the stresses at the tapered face, the strength reduction factor $k_{m,\alpha}$ is determined.

Tapered face in **tension:**

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

Tapered face in **compression:**

$$k_{m,\alpha} = \frac{1}{\sqrt{1 + \left(\frac{f_{m,y,d}}{1,5\,f_{v,d}}\tan\alpha\right)^2 + \left(\frac{f_{m,y,d}}{f_{c,90,d}}\tan^2\alpha\right)^2}}$$

The slope angle of the tapered edge is calculated as follows:
$$\tan(\alpha) = \frac{H_L - H_S}{L}$$

With:
- H_L Height at the largest end
- H_S Height at the smallest end
- L Member length

> **Note: Both reduction factors are limited to a maximum of 1,00**

Influence on the Section checks

For the bending check as well as the combined bending and axial tension/compression checks the following modifications are made:
- The strong axis bending resistance $f_{m,y,d}$ is multiplied by $k_{m,\alpha}$.
  For example:

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} \longrightarrow \frac{\sigma_{m,y,d}}{k_{m,\alpha} * f_{m,y,d}}$$

- The depth factor $k_{h,y}$ is calculated using the cross-section properties at the LARGEST end of the beam (H_L).

Influence on the Section checks for buckling

For the buckling check (Columns subjected to compression or combined compression and bending), the EN 1995-1-1 code does not give any specific rules. This implies the following:
- The strong axis bending resistance $f_{m,y,d}$ is multiplied by $k_{m,\alpha}$.
- The slenderness in each section is calculated using the actual area and inertia at that given section.

Influence on the Stability check for LTB

For the LTB check (Beams subjected to bending or combined bending and compression) the following modifications are made based on the stresses at the tapered edge:

Tapered edge in tension

In case the tapered edge is in tension it means LTB does not occur at that side. In that case the critical moment $M_{y,crit}$ and the critical bending stress $\sigma_{m,crit}$ are calculated using the cross-section properties at the LARGEST end of the beam (**H_L**). This implies that I_t, I_z and W_y as well as h used in L_{ef} are all taken from the largest cross-section of the tapered member as opposed to the actual section.

Tapered edge in compression

In case the tapered edge is in compression it means LTB occurs at that side. In that case the following modifications are made:

- The strong axis bending resistance $f_{m,y,d}$ is multiplied by $k_{m,\alpha}$.

$$\sigma_{m,y,d} \le k_{crit}\, f_{m,y,d} \longrightarrow \sigma_{m,y,d} \le k_{crit}\, k_{m,\alpha}\, f_{m,y,d}$$

- The relative slenderness is calculated including $k_{m,\alpha}$.

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

- The critical moment $M_{y,crit}$ and the critical bending stress $\sigma_{m,crit}$ are calculated using the cross-section properties at the LARGEST end of the beam (HL). This implies that I_t, I_z and W_y as well as h used in L_{ef} are all taken from the largest cross-section of the tapered member as opposed to the actual section.

For background information to the above modifications reference is made to Ref.[7].

Double tapered beams

The specifics for double tapered beams are given in **article 6.4.3**

Definition

A double tapered member is defined as a member with the following characteristics:

- A rectangular cross-section
    - **RECT** (Timber)
    - **Rectangle** (Geometric shapes)
    - **Full rectangular (Form code 7)**
- Material type **Glued, laminated**
- One member with an arbitrary definition of two spans:
    - Each span length is half of the member length
    - Only the height varies linearly over the span length
    - On the first span the height varies from small to large, on the second span from large to small.
    - The height at the beginning of the first span and the end of the second span is equal.
    - The height at the end of the first span and the beginning of the second span is equal.
- The height at the end of the first span has to be larger than the width of the cross-section
- The arbitrary alignment of both spans is the same and is one of the following:
    - Top surface
    - Top left
    - Top right
   - Bottom surface
    - Bottom left
    - Bottom right

Only tapered members which have a geometry which complies with the above definition are seen as actual double tapered beams for which the additional rules of EN 1995-1-1 apply.

In each section of the double tapered beam also the modifications as listed in the paragraph for single tapered beams apply.

The member length L used for the determination of the slope angle is in this case taken as the span length.

Apex zone

The 'Apex Zone' for a double Tapered member is defined on the following picture:
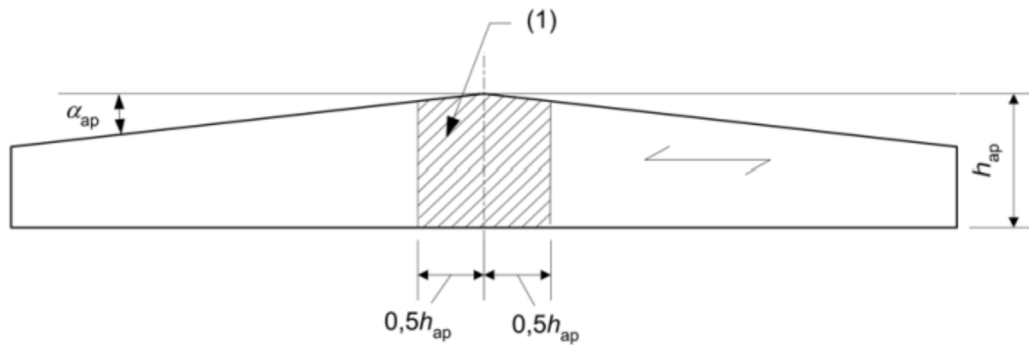
For the sections located at the Apex (all sections in the hatched zone) additional checks are executed:
- Bending stress in the Apex zone
- Tension perpendicular to the grain in the Apex zone
- Combined tension perpendicular to the grain and shear in the Apex zone

Note: For a Double tapered beam the radius r = ∞

Bending stress in the Apex zone

For a double tapered beam, the bending stress at the Apex is checked according to **article 6.4.3(3).** As indicated in Ref.[7] this formula is enhanced to include also the reduction factor for LTB:

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

Note: Following Ref.[12] pp.6 the influence of the tapered edge of a single tapered beam (km,α) is not included in the verification formula for the bending stress in the apex zone. The k_{crit} used in the above verification can include this factor.

As indicated in Ref.[13] pp.B8/8 the EN 1995-1-1 rules assume a constant moment acts over the Apex zone. Therefore $M_{ap,d}$ is taken as the maximal moment from all sections in the Apex zone.

Tension perpendicular to the grain in the Apex zone

For a Double Tapered member, the tension perpendicular to the grain at the Apex is checked according to **article 6.4.3(6).**

The total volume of the beam V_b is calculated as follows for a double tapered beam:

$$b \cdot L \cdot \left[h_{ap} - 0{,}25 \cdot L \cdot \tan(\alpha_{ap}) \right]$$

The stressed volume V of the Apex zone is calculated as follows for a double tapered beam, see Ref.[7]:

$$b \cdot h_{ap}^2 \cdot \left[1 - 0{,}25 \cdot \tan(\alpha_{ap}) \right] \text{ and limited to } (2/3)V_b$$

With:
- b   Width of the beam
- h_{ap} Height at the Apex zone
- α_{ap} Slope angle of the tapered beam
- L   Member length

Note: In case the alternative formula (6.55) is used the line load p_d is calculated as the equivalent line load for the given moment diagram. In addition, this line load is taken as positive in case it causes compression at the tapered edge.

### Combined Tension perpendicular to the grain and shear in the Apex zone

For a double tapered beam, the combined shear and tension perpendicular to the grain at the Apex is checked according to **article 6.4.3(7).**

This check is executed only in case a shear force $V_{z,Ed}$ is present.
This shear force is taken for the actual section in which the check is executed.

Curved beams

The specifics for curved members are given in **article 6.4.3.**
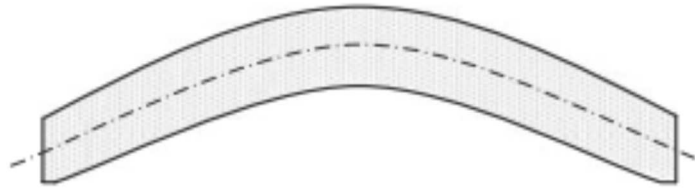
Definition

A curved beam is defined as a member with the following characteristics:
- A rectangular cross-section
  - **RECT** (Timber)
  - **Rectangle** (Geometric shapes)
  - **Full rectangular (Form code 7)**
- The height has to be large than the width of the cross-section
- Material type **Glued, laminated**
- The member is uniform i.e. no haunch or arbitrary data
- The member has shape type **Polyline** with following characteristics:
  - All nodes are in one plane
  - The polyline has a sequence of type 'Line' , 'Circle arc', 'Line'
  - The polyline is 'symmetrical'

Apex zone

The 'Apex zone' for a curved beam is defined on the following picture:
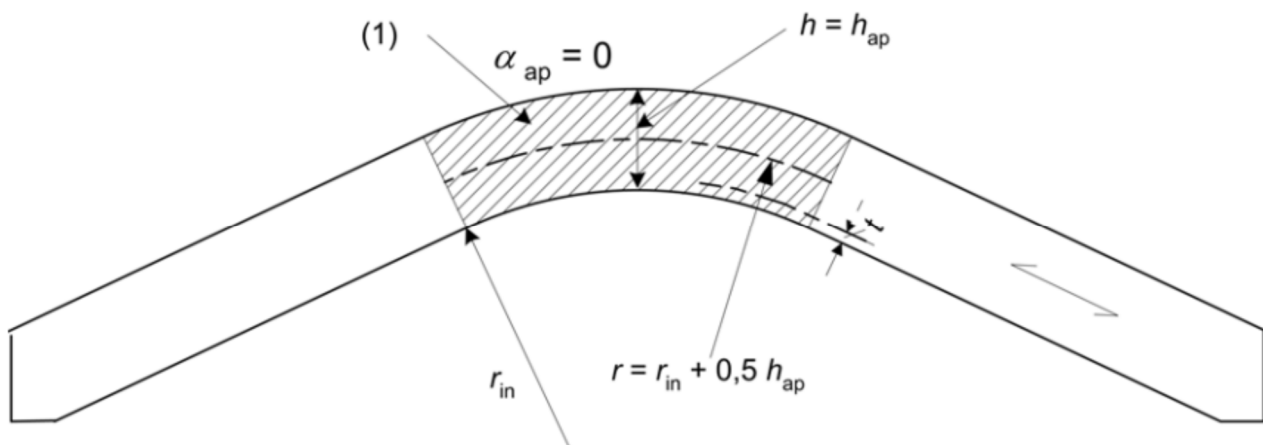
For the sections located at the Apex (all sections in the hatched zone) additional checks are executed:
- Bending stress in the Apex zone
- Tension perpendicular to the grain in the Apex zone
- Combined tension perpendicular to the grain and shear in the Apex zone

Bending stress in the Apex zone

For a curved beam, the bending stress at the Apex is checked according to **article 6.4.3(3).** As indicated in Ref.[7] this formula is enhanced to include also the reduction factor for LTB:

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

> **Note: As indicated in Ref.[13] pp.B8/8 the EN 1995-1-1 rules assume a constant moment acts over the Apex zone. Therefore $M_{ap,d}$ is taken as the maximal moment from all sections in the Apex zone.**

Tension perpendicular to the grain in the Apex zone

For a curved beam, the tension perpendicular to the grain at the Apex is checked acc. to article **6.4.3(6):**

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

The stressed volume **V** of the Apex zone is calculated as follows for a curved beam, see Ref.[7]:

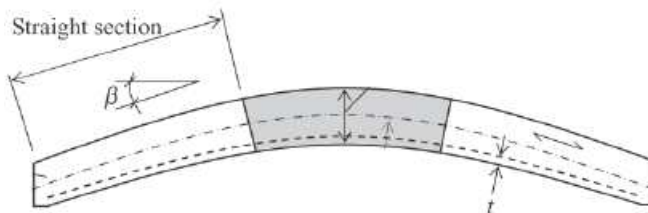$$\frac{\beta\pi}{180} b\left(h_{ap}^2 + 2h_{ap}r_{in}\right)$$

This is limited to **(2/3)V_b**
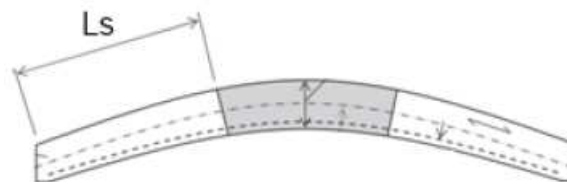
The total volume of the beam **V_b** is calculated as follows for a curved beam:

$$V + 2 \cdot b \cdot h_{ap} \cdot L_s$$

With:
- b   width of the beam
- h_{ap} heigth of the beam
- β   slope angle of the straight parts

- L_s length of one straight part

- r_{in} inner radius of the curved beam

Combined tension perpendicular to the grain and shear in the Apex zone

For a Curved beam, the combined shear and tension perpendicular to the grain at the Apex is checked according to **article 6.4.3(7).**

This check is executed only in case a shear force **V_{z,Ed}** is present.

This shear force is taken for the actual section in which the check is executed.

System Strength

As specified in **article 6.6,** when several equally spaced similar members, components or assemblies are laterally connected by a continuous load distribution system, the member strength properties may be multiplied by a system strength factor **k_{sys}.**

By default, the factor **k_{sys}** is taken as **1,00** so no system strength is used. However, through the use of Timber member data, the system strength factor **k_{sys}** can be defined for a given member.

In this case the design strengths for this member are increased by the inputted factor.

1.5 Serviceability Limit State

The Serviceability Limit State verifications are executed according to **article 2.2.3**.
Both the instant and the final relative deflections (including creep) are checked.
The final deformation is calculated according to **formula (2.2)**.

$$u_{fin} = u_{fin,G} + u_{fin,Q,1} + \sum u_{fin,Q,i}$$

> **Note: As specified in article 2.2.3(5) this formula assumes a linear relation between the actions and corresponding deformations. Therefore, this relative deformation check cannot be executed for non-linear combinations.**

For each load case, the final deformation with creep is calculated based on the type of action, as given in **formulas (2.3), (2.4), (2.5)**:

- Permanent action **(G)**

$$u_{fin,G} = u_{inst,G}(1 + k_{def})$$

- Leading Variable action **(Q1)**

$$u_{fin,Q,1} = u_{inst,Q,1}(1 + \psi_{2,1}k_{def})$$

- Accompanying Variable action **(Qi)**

$$u_{fin,Q,i} = u_{inst,Q,i}(\psi_{0,i} + \psi_{2,i}k_{def})$$

The above is determined automatically in case Serviceability combinations according to the code are used:
- EN-SLS Characteristic
- EN-SLS Frequent
- EN-SLS Quasi-Permanent

> **Note:**
>
> **In case the check is executed for a single variable load case, this is seen as a Leading Variable action.**
>
> **In case the check is executed for a Linear serviceability combination or an Envelope serviceability combination , it is not possible to determine if the variable load cases are leading or accompanying. Therefore, every variable load case in such a combination is seen as a Leading Variable action.**
>
> **Precamber according to article 7.2 is not supported**

## 1.5.1 Timber 1D SLS Design in SCIA Engineer

Camber definition

The camber can be defined on two places within SCIA Engineer:
- **Material setup:**
  The camber definition defined in the material setup are applied on the members of that given material that are using the default buckling groups or are using a buckling group in which the setting "Camber definition" = From setup.
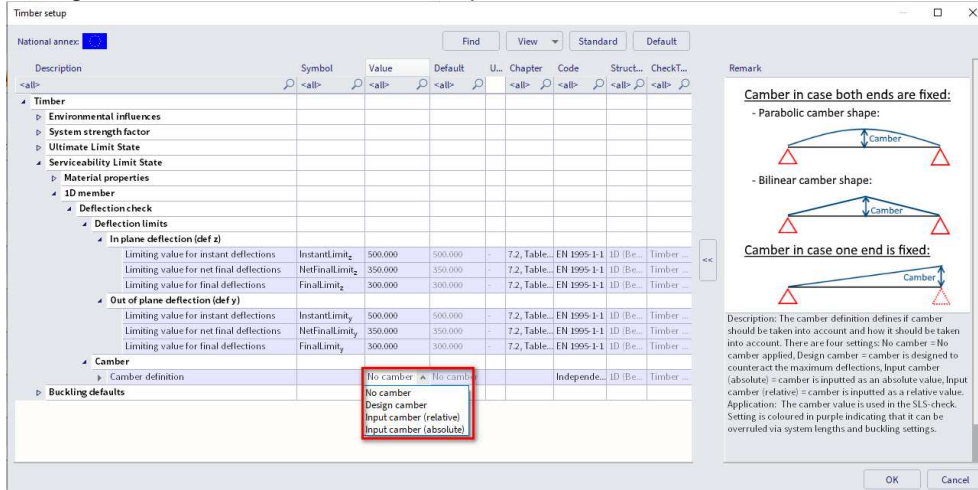
- **Buckling groups** (via system lengths and buckling settings):
  The camber definition can also be defined within a buckling group for deflection z only.
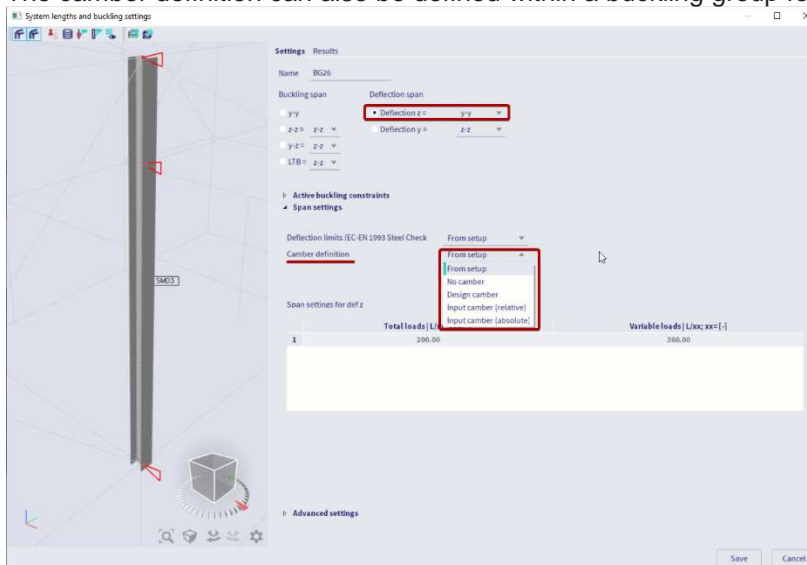
Camber curves

Within SCIA Engineer three possible camber shapes are supported depending on the type of span:
- Deflection z span with one end fixed (=cantilever span):
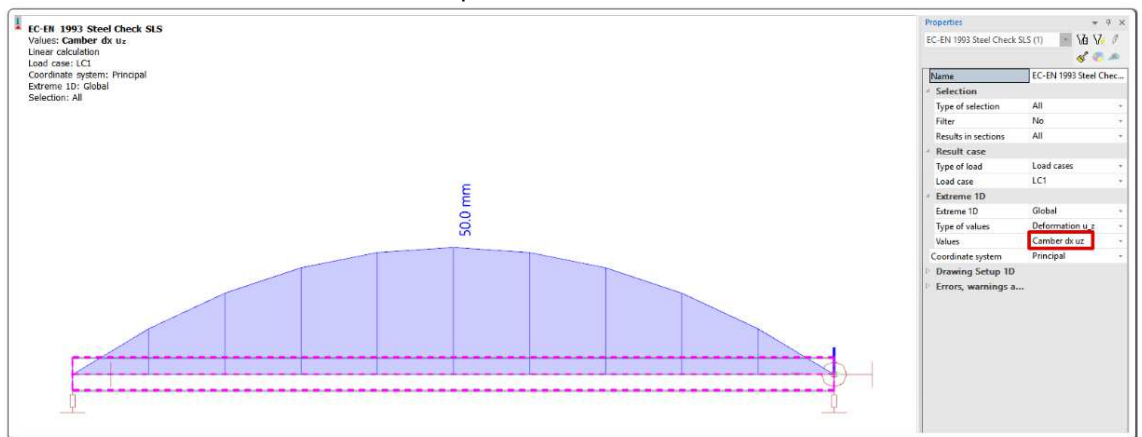 **Linear camber shape**
  In case the deflection z span only has one fixed end, a linear camber shape is used in which the camber value at the fixed end is 0mm and the camber value at the free end has the maximum value.
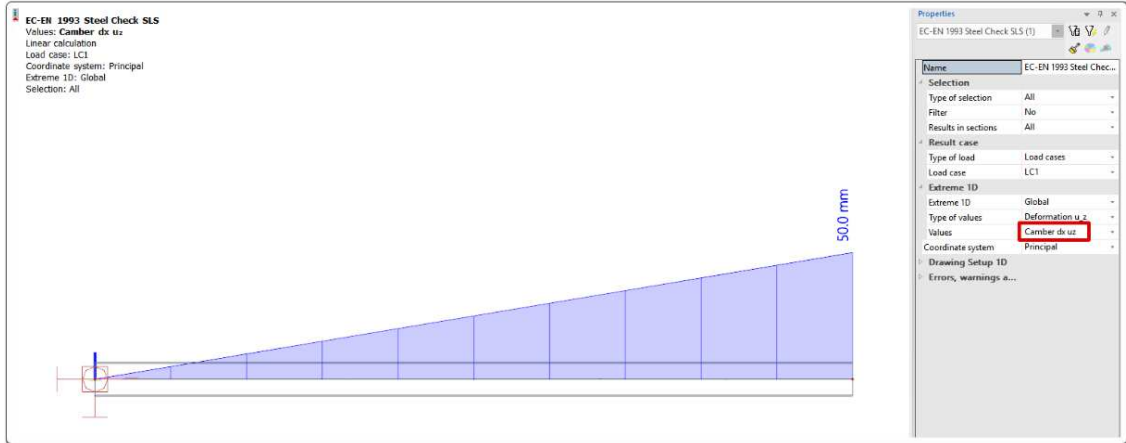
- Deflection z span with both ends fixed (=simply supported span):
  a) **Parabolic camber shape**
     In case the deflection z span has two fixed ends and the camber shape is set to "parabolic", a parabolic camber shape is used in which the camber value at the fixed ends are 0mm and the camber value in the middle of the span has the maximum value.

 b) **Bilinear camber shape (since SCIA Engineer version 19.1)**
     In case the deflection z span has two fixed ends and the camber shape is set to "bilinear", a bilinear camber shape is used in which the camber value at the fixed ends are 0mm and the camber value in the middle of the span has the maximum value.
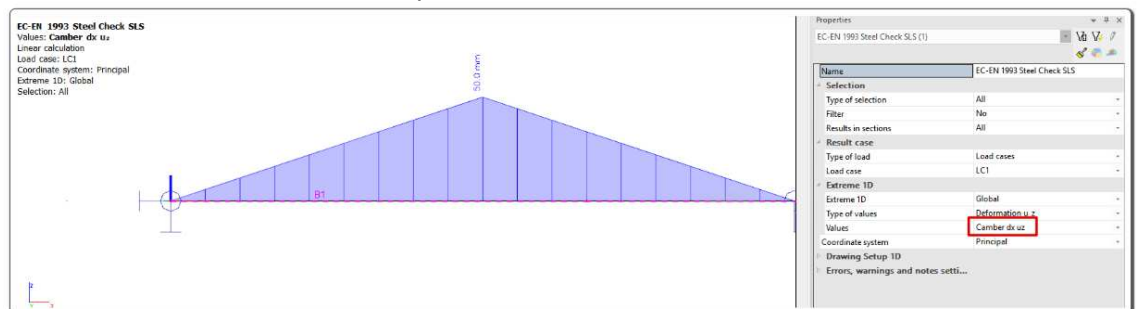
Camber design

Camber design parameters

Before the camber design can be executed the user needs to define some settings in the steel settingsp or in the buckling group depending where the camber definition is set to "Design camber".
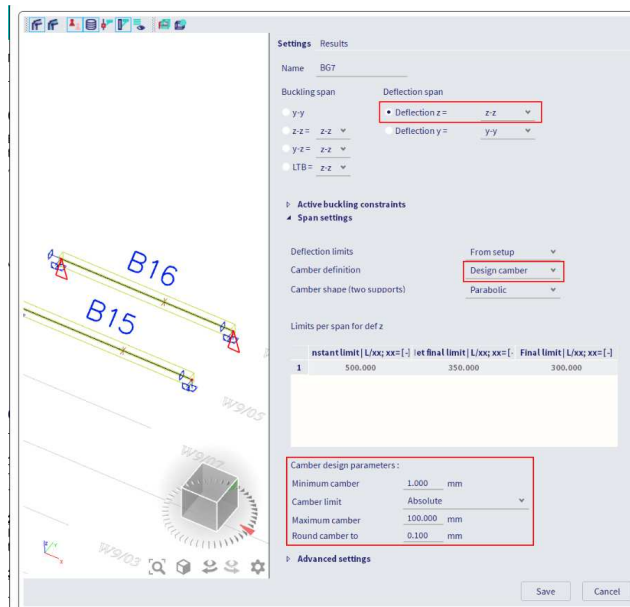
Minimum camber: you can input a minimum (absolute) camber value. If the camber design value is lower than this minimum value, the minimum camber is used.

Camber limit & **Maximum camber**: contains two ways for defining the maximum camber value. If the camber design value is higher than the camber limit value, the camber limit value is used instead.
Camber limit = Percentage of final deflections: the maximum camber value is defined by means of a percentage of the final deflections.
Camber limit = Absolute: the maximum camber value is defined by means of an absolute value.

Round camber to: the obtained camber design value is rounded to the defined increments.

More information about camber design algorithm could be found in the link below:
https://help.scia.net/webhelplatest/en/#timber/setup_check_member_data/tb_timbercodecheck/en/timber_sls_check_camber_design.htm

Chapter 2: prEN 1995 Code Check, Cross Laminated Timber 2D

2.1 Basics of design

Within **article 4.4** the possible procedures for carrying out the analysis of a Timber structure are specified. The following procedures are supported:

- First order linear elastic analysis using mean values E_{mean} and G_{mean} for the stiffness properties.

The design values are determined as given in **article 4.5.1(1)**.

> **Note: It is important to emphasize that certain combined checks such as double bending as specified in art. 8.1.8.1 are not applied directly. Instead the bending checks in both directions are evaluated separately. Or with other words combined checks are supported if the acting internal forces are in the same direction.**
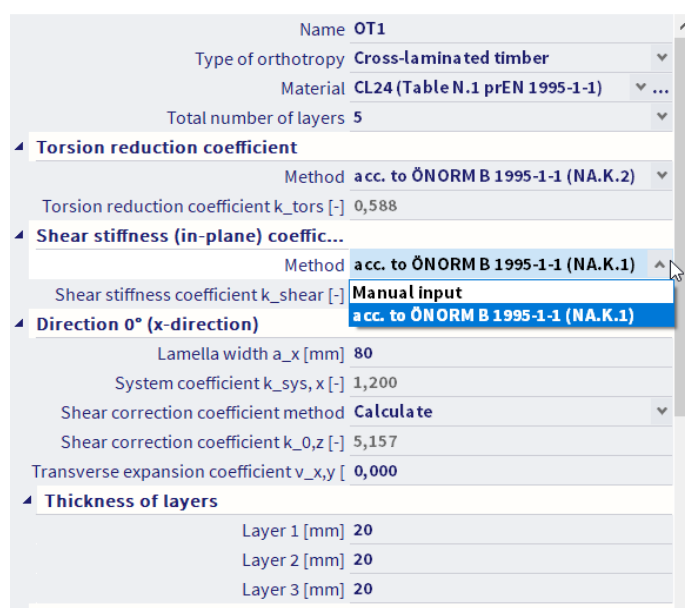
2.1.1 Partial safety factor

Partial safety factors γM used in the CLT-check are to be defined in the EN 1995-1-1 NA setup.
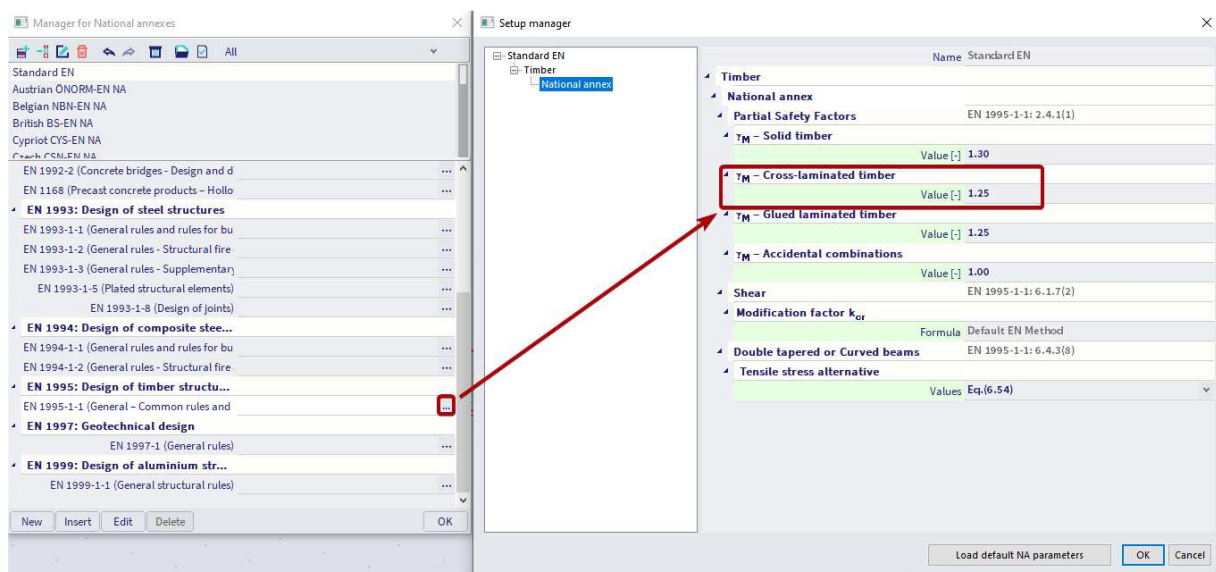
2.1.2 CLT orthotropy extension

In the new version there is possibility to choose between manual input and according to Önorm for the different coefficient

Calculation of k_{tors} and k_{shear}

According to Önorm B 1995-1-1 (NA.K.1)

$$k_{shear} = \frac{1}{1+6 \times p_S \times (\frac{d_{max}}{a})^{q_S}}$$

With:

$$a = \frac{a_x + a_y}{2}$$, with a_x and a_y the individual lamination widths for the given direction.

d_{max}: Thickness of the thickest individual layer.

	3 layers	5, 7+ layers
p_S	0.53	0.43
q_S	1.21	

$$k_{tors} = \frac{1}{1+6 \times p_D \times (\frac{d_{max}}{a})^{q_D}}$$

With:

$$a = \frac{a_x + a_y}{2}$$

d_{max}: Thickness of the thickest individual layer.

	3 layers	5 layers	7+ layers
p_D	0.89	0.67	0.55
q_D	1.33	1.26	1.23

More information about the orthotropy in the link below:
https://help.scia.net/webhelplatest/en/#modelling/orthotropy/library_of_orthotropy/3_type_cross_laminated_timber.htm

2.1.3 System strength Ksys

The system strength k_{sys} is calculated in the orthotropy dialog according to art.8.4 (4) of prEN 1995-1-1. Different individual laminations widths (a_x and a_y) can be defined per direction as a consequence k_{sys} can be different per direction ($k_{sys,x}$ and $k_{sys,y}$).

The number of loaded laminations in order to determine k_{sys} is calculated as the ratio of:

$$\frac{1000 \text{ mm}}{\text{individual laminations width (ax or ay)}}$$

The system strength k_{sys} is always calculated in the orthotropy dialog but is only applied when the setting "Apply system strength factor" is activated in the timber setup or in timber 2D member data. Once activated it is applied in the calculation of the design strengths for the following checks:
- Tension parallel to the grain check
- Bending check

2.2 Material properties

2.1.1 Strength modification factor kmod

The values of the modification factor k_{mod} are taken by default from Table 5.1.

As specified in article 5.1.3(2), if a load combination consists of actions belonging to different load-duration classes a value of k_{mod} is used which corresponds to the action with the shortest duration.

2.3   CLT ULS design

2.2.1 Section check

Tension parallel to the grain

The Tension parallel to the grain check is executed according to **article 8.1.2**.
This check is executed only in case a tensile normal force n_x or n_y is present.

Compression parallel to the grain

The Compression parallel to the grain check is executed according to **article 8.1.5**.
This check is executed only in case a compressive normal force n_x or n_y is present.

Bending

The Bending check is executed according to **article 8.1.8**.

This check is executed only in case a bending moment m_x or m_y is present. The bending stresses for the given direction are calculated on each side per layer (top- and bottom side), the maximum acting bending stress is then used in the bending check verification.

> **Note**: Formula 8.19 and 8.20 of art. 8.1.8.1 are tailored for 1D-members, in case of 2D-members such as CLT-panels the bending check is executed per direction individually.

Axial tension and bending

The Axial Tension and Bending check is executed according to **article 8.1.9**.

This check is executed only in case both a tension force n_x or n_y and a bending moment m_x or m_y are present for the same direction.

> **Note**: Formula 8.22 and 8.23 of art. 8.1.9 are tailored for 1D-members, in case of 2D-members such as CLT-panels the Axial Tension and Bending check is executed per direction individually.

Axial compression and bending

The Axial Compression and Bending check is executed according to **article 8.1.10**.

This check is executed only in case both a tension force n_x or n_y and a bending moment m_x or m_y are present for the same direction.

> **Note**: Formula 8.24 and 8.25 of art. 8.1.10 are tailored for 1D-members, in case of 2D-members such as CLT-panels the Axial Compression and Bending check is executed per direction individually

Transverse shear

The Transverse shear check due to out of plane loading is executed according to **article 8.1.11.1(1)**.

This check is executed only in case a shear force v_x or v_y is present. The shear stresses for the given direction are calculated on each side per layer (top- and bottom side) and also on the CLT-panel's centroid position, the maximum acting shear stress in the layers oriented in the given direction as well as in the perpendicular direction are then used in the transverse shear force check verification.

> **Note**: Rolling shear is taken into account by evaluating the maximum shear stress for the perpendicular oriented layer with the design rolling shear strength via formula 8.26.

In -plane shear

The In-plane shear check is executed according to **article 8.1.11.2(2)**.

This check is executed only in case a shear force n_{xy} is present.

Design in-plane shear stress for the given direction is calculated according to prEN 1995-1-1: 8.36 given by CEN/TC 250/SC 5 N 1489:

$$\tau_{v,xy,d} = \frac{6 \cdot n_{xy}}{a_x{}^2 \cdot n_{bond}} \cdot \left(\frac{1}{n_{x,l}{}^2} - \frac{1}{n_{x,l}{}^3} \right)$$

$$\tau_{v,yx,d} = \frac{6 \cdot n_{xy}}{a_y{}^2 \cdot n_{bond}} \cdot \left(\frac{1}{n_{y,l}{}^2} - \frac{1}{n_{y,l}{}^3} \right)$$

with:

n_{xy}: longitudinal shear force on mesh element

a_x: width of 1 lamella in the x-direction

a_y: width of 1 lamella in the y-direction

n_{bond}: number of interface layers (number of CLT layers-1)

$n_{x,l}$: number of laminations across all layers in the x-direction within 1 meter width ($1000mm/a_x$)

$n_{y,l}$: number of laminations across all layers in the y-direction within 1 meter width ($1000mm/a_y$)

The shear stresses for i-direction in the bond-lines between laminations of adjacent orthogonal layers is verified according to prEN 1995-1-1: 8.31:

$$\tau_{tor,node,i,d} = \frac{3}{2} \cdot \tau_{v,i,d} \cdot \left(\frac{t_l}{a_i} \right) \leq f_{tor,node,d}$$

with:

t_l: the maximum lamination thickness (the thickness of the outer layers is reduced by 20% according to article 8.1.11.2 (2a))

$f_{tor,node,d}$: is the design torsional shear strength of the glued area of crosswise bonded laminations

Torsion and combined shear and torsion

The Torsion and combined Shear and Torsion check is executed according to **article 8.1.12**.

This check will be executed only in case a torsional moment **m_{xy}** is present or in case a shear force **n_{xy}** is present together with transverse shear force **v_x** or **v_y**.

Maximum shear stress due to torsional moment is calculated as:

$$\tau_{tor,d} = \frac{m_{xy} \cdot t_{CLT}}{I_t}$$

with:

m_{xy}: torsional moment on mesh element

t_{CLT}: total thickness of the CLT-plate

I_t: torsional moment of inertia calculated according to prEN 1995-1-1: D.11

$$I_t = \frac{t_{CLT}{}^3 \cdot h_{CLT}}{3} \cdot \left(1 - 0.63 \cdot \frac{t_{CLT}}{h_{CLT}} \right)$$

h_{CLT}: width of equivalent cross-section (=1000mm)

In case no in-plane shear force **n_{xy}** is present, formula 8.12 of art. 8.1.12.1 is used for calculation of unity check.

However article 8.1.11.2(3) of prEN 1995-1-1specifies, that linear interaction of shear stresses is required in case of presence of in-plane and perpendicular to plane loading. Therefore if an in-plane shear stress is present on a mesh element ($n_{xy} \neq 0$), formula 8.12 of art. 8.1.12.1 is extended to:

$$UC_{V,xz+M,xy} = \frac{|\tau_{tor,d}|}{k_{shape} \cdot f_{v,d}} + \left(\frac{|\tau_{v,xz,d}|}{k_v \cdot f_{v,d}} \right)^2 + MAX \left(\frac{|\tau_{tor,node,x,d}|}{f_{tor,node,d}} ; \frac{|\tau_{tor,node,y,d}|}{f_{tor,node,d}} \right)$$

and

$$UC_{V,yz+M,xy} = \frac{|\tau_{tor,d}|}{k_{shape} \cdot f_{v,d}} + \left(\frac{|\tau_{v,yz,d}|}{k_v \cdot f_{v,d}} \right)^2 + MAX \left(\frac{|\tau_{tor,node,x,d}|}{f_{tor,node,d}} ; \frac{|\tau_{tor,node,y,d}|}{f_{tor,node,d}} \right)$$

Note: Formula 8.12 of art. 8.1.12.1 is tailored for 1D-members, in case of 2D-members such as CLT-panels the combined Torsion and Shear check is executed per direction individually. Coefficients kshape and kv for CLT panels are set to 1.

2.3.2 Stability check

Flexural buckling

The Flexural buckling check is executed according to **article 8.2.2.2**.

This check is executed only in case a compressive force n_x and/or n_y is present.

Coefficient of imperfection β_c used in equation 8.42 is set to value 0.1, as given by publication "Cross-Laminated Timber - Structural Design, Wallner-Novak M., 2014 (article 5.10).

Note: Formula 8.39 and 8.40 of art. 8.2.2.2(2) is tailored for 1D-members, in case of 2D-members such as CLT-panels theFlexural buckling check is executed per direction individually.

When the check-box "Flexural buckling accounted for by 2nd order calculation" is activated in timber setup, the "Geometrical nonlinearity" is activated and the check is executed for a nonlinear combination, the flexural buckling is skipped and a note is printed.

Errors and warnings regarding CLT in the link below:

https://help.scia.net/webhelplatest/en/#timber/setup_check_member_data/tb_timbercodecheck_clt/en/clt_ewn.htm

Prerequisites in SCIA Engineer

You should define:
- A CLT material should be defined in the project
- A Timber 2D element with CLT orthotropy assigned

How the CLT ULS design works in SCIA?

- Orthotropy defines a 'mean stiffness' used in the calculation
- Stiffness is used for calculation internal forces
- Stress distribution is recalculated based on internal forces and CLT section properties
- Stresses are checked according to the code

Note: The buckling length should be inputted manually in the Timber configuration

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