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Guidelines for a Finite Element Based Design of Timber Structures

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0. Introduction

(1) The Guidelines are compiled in the light of the revision of the Eurocodes and the upcoming discussions about Finite Element based design in general and especially in timber construction. Their development is largely based on prEN 1993-1-14 [68].

As the discussions are ongoing, this document is supposed to be a basis for discussion and handout for FE based design, without claiming completeness and representativeness of all expert knowledge.

Since the revision of the Eurocodes is in constant progress and not yet completed when these guidelines are formulated, inconsistencies with the new generation of Eurocodes cannot completely be ruled out. References to Eurocodes are to the Eurocodes of the new generation which are currently under preparation.

(2) Placeholders are inserted in the document and marked with *, where additional information is still to be given.

(3) The authors call for critical reflection on the document. Please feel free to send your comments and suggestions to janusch.toepler@ke.uni-stuttgart.de.

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1. Scope

1.1 Scope of design guidelines

(1) This document gives principles and requirements for the use of numerical methods in the design of timber structures, more specifically for the ultimate limit state and serviceability limit state verifications. It also gives principles and requirements for the application of advanced Finite Element and similar modelling techniques for numerical simulation which also covers safety assessment. [68]

(2) This document currently covers the Finite Element Method (FEM) for modelling, analysis and design of timber structures made of the following materials [68]:

a. solid timber,

b. glulam made,

c. cross-laminated timber,

d. laminated veneer lumber

and joints and connections, especially rules are given for:

e. bolts and dowels,

f. direct contact (e.g. notches).

(3) This document contains rules on the topics as follows:

a. modelling of structural systems and components,

b. modelling of actions and boundary conditions,

c. modelling of joints and connections,

d. modelling of material properties,

e. modelling of imperfections,

f. levels of modelling,

g. type of analysis,

h. verification and validation of numerical models,

i. representation of limit state criteria,

j. partial factors to be applied (where appropriate),

k. choice of software and documentation,

l. benchmark cases. [68]

(4) This document only contains rules for static analysis. If applicable, dynamic effects should be considered according to the relevant parts of EN 1995. [68]

(5) Analogous to EN 1995-1-1 this document only applies for the design of structures that are **not** exposed to temperatures above 60 °C for a longer period of time. Structural fire design is not covered in this document.

(6) The provisions of this document consider the following design criteria:

a. fracture (stress limits),

b. plastic failure (strain limits),

c. buckling,

d. serviceability. [68]

(7) This document defines the characteristic and design values of the resistance of the modelled structure. [68]

1.2 Assumptions

(1) In this document, all discretization methods are generally referred to as FE. [68]

(2) This document gives guidelines intended for users who are experienced in the use of FE (clause 4.1 (4)). [68]

(3) It is recognised that structural analysis, based upon the laws of physics, has been successfully researched, developed, historically or currently used for the design and verification of elements or whole structures. This remains appropriate for many structural solutions. However, when a more detailed understanding of structural behaviour is required, the methods described in this document can be useful for the professional design. [68]

(4) Unless specifically stated, EN 1990, EN 1991 (all parts) and the other relevant parts of EN 1995 (all parts) apply.

(5) The design methods given in these guidelines are applicable if

a. the execution quality is as specified in EN 1995-3, and

b. the construction materials and products used are as specified in the relevant parts of EN 1995 (all parts), or in the relevant material and product specifications.

2. Normative references

The following documents, in whole or in part, are normatively referenced in this document and are indispensable for its application. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies. [68]

EN 1990, *Basis of structural and geotechnical design*

EN 1991-1-1, *Actions on structures – Part 1-1: General actions - Densities, self-weight, imposed loads for buildings*

EN 1991-1-2, *Actions on structures – Part 1-2: General actions - Actions on structures exposed to fire*

EN 1991-1-3, *Actions on structures – Part 1-3: General actions - Snow loads*

EN 1991-1-4, *Actions on structures – Part 1-4: General actions - Wind actions*

EN 1991-1-5, *Actions on structures – Part 1-5: General actions – Thermal actions*

EN 1991-1-6, *Actions on structures – Part 1-6: Actions during execution*

EN 1991-1-7, *Actions on structures – Part 1-7: Accidental actions from impact and explosions*

EN 1991-2, *Actions on structures – Part 2: Traffic loads on bridges*

EN 1991-3, *Actions on structures – Part 3: Actions induced by cranes and machinery*

EN 1991-4, *Actions on structures – Part 4: Silos and tanks*

EN 1995-1-1, *Design of timber structures - Part 1-1: General - Common rules and rules for buildings*

EN 1995-1-2, *Design of timber structures - Part 1-2: General - Structural fire design*

EN 1995-1-2, *Design of timber structures - Part 2: Bridges*

EN 338:2016, *Structural timber – Strength classes*

EN 14080:2013, *Timber structures – Glued laminated timber and glued solid timber – Requirements*

3. Terms, definitions and symbols

3.1 Terms and definitions

Benchmark case: Offers inputs and outputs of analytical or numerical solutions (simplified models) for verification or validation of a numerical model. Or inputs and outputs of experimental tests for validation of a numerical model and its quality (see Annex E).

Coordinate system: In Fig. 3.1 the generally used coordinate system is displayed. Where *x* is parallel to the grain direction (longitudinal), *y* is horizontal and *z* is vertical. A distinction of the directions radial and tangential to the grain is usually not made for engineered timber products.

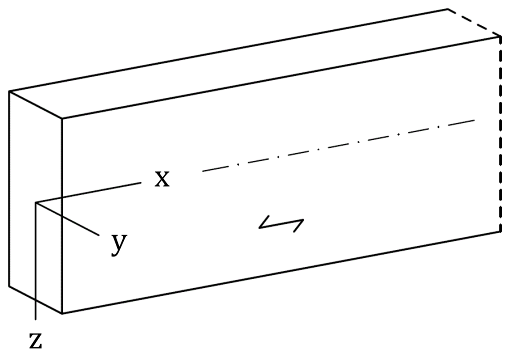


Fig. 3.1: Generally used coordinate system.

Degree of freedom (DOF): The number of independent motions that are allowed to the structure. DOF can be defined as DOF per node (1 to 7 – maximum 3 translational, 3 rotational and 1 warping) and total number of DOFs for the whole structure as sum of all node's DOFs. [68]

Discretization check: A mesh study that shows that the chosen mesh size, element type and size are accurate for the analysed problem and the calculation results are not significantly influenced by the discretization.

Direct resistance check: Analysis performed for design checks, which result is the ultimate resistance of the analysed structure.

Effective stiffness values: Global stiffness values used to define the mechanical properties of an entire composite cross-section built of several members with possibly varying material properties and a flexible connection of the different members. As a simplification often used to characterize CLT's load bearing capacity.

Finite Element based design methods: Numerical design calculation and numerical simulation (clause 4.2).

Finite Element type: type of numerical approximation of the finite elements such as quadratic beam elements or linear solid elements with reduced integration.

Follower load: A load changing direction as a function of the deformation of the analysed structure in a non-linear analysis. [68]

Imperfection sensitivity analysis: Check whether the results of the numerical solution (SRQs) are sensitive to imperfections in general and the chosen imperfection type, shape and magnitude.

Level of modelling: Describes the reduction degrees of a structural system starting from the global model (whole load-bearing structure of a building) to the local model (e.g. joint).

Multi-level or combined model: Modelling of the entire structure using different types of elements within one model, making the DOFs compatible at the intersection regions (e.g. coupling of beam, plate, shell or solid elements). [68]

Numerical design calculation with direct resistance check: FE based design method used for design checks resulting in the ultimate resistance of the analysed structure. [68]

Numerical design calculation requiring subsequent design check: FE based design method used for design checks, resulting in system response quantities (SRQs) to be further used in the static check of the analysed structure according to parts of EN 1995. [68]

Numerical model: Numerical idealization to simulate and predict aspects of the mechanical behaviour of the analysed structure or a part of it. [68]

Numerical simulation: FE based design method where the numerical model is used for complementation or extension of physical experiments to directly determine the resistance of a structure. [68]

Partial factor for modelling: The partial factor for modelling γ_{FE} covers the uncertainties of the numerical model and the executed type of analysis.

Proportionality limit: Transition point from linear elastic to non-linear behaviour.

Sensitivity check: Variation of the relevant input parameters to determine which parameters are crucial to the relevant SRQ and whether these parameters should be defined with higher precision or not.

Solution technique: Load- or displacement-controlled analysis.

Solver convergence study: Used to ensure that the numerical results have converged when the computation of a load or displacement step is finished.

Standard design case: Design check of failure modes based on numerical calculations for which also Eurocode based design resistance model exist. [68]

Sub-model: A part of the entire structure modelled using equivalent support conditions representing the neglected part of the structure. [68]

System response quantity (SRQ) is the relevant output value resulting from a certain analysis; it reflects the main objective of the analysis by selecting the major parameters and the limitation of their errors in both validation and verification. [68]

Type of action: Forces, displacements, strains or temperatures.

Type of analysis: LA, LBA, MNA, GNA, GMNA, GNIA, GMNIA (clause 6.1.2).

Validation: Comparison of the numerical results to known accurate solutions (benchmark) to demonstrate that the model correctly or conservatively captures the physical phenomena to be modelled. [68]

Verification: Demonstrates that the numerical model is properly implemented, understood and used and that the numerical solution is a good approximation of the exact mathematical solution. [68]

3.2 Symbols and abbreviations

3.2.1 Latin upper-case symbols

$\boldsymbol{C}$	elastic material stiffness matrix
$C_{11/\ldots}$	entries of the elastic material stiffness matrix
E_0	modulus of elasticity parallel to the grain
E_{90}	modulus of elasticity perpendicular to the grain
$E_{1/\mathrm{L}}$	modulus of elasticity parallel to the grain / in longitudinal direction
$E_{2/\mathrm{R}}$	modulus of elasticity in radial direction
$E_{3/\mathrm{T}}$	modulus of elasticity in tangential direction
G	shear modulus
G_0	shear modulus parallel to the grain
G_{90}	shear modulus perpendicular to the grain (rolling shear modulus)
$G_{12/\mathrm{LR}}$	shear modulus in longitudinal-radial plane
$G_{13/\mathrm{LT}}$	shear modulus in longitudinal-tangential plane
$G_{23/RT}$	shear modulus in radial-tangential plane
G_{xz}	shear modulus of CLT, for parallel layers with $G_{\mathrm{xz}} = G_0$, for cross layers with $G_{\mathrm{xz}} = G_{90}$
$I_{\mathrm{n,x}}$	moment of inertia of the net cross-section of CLT in x-direction
I_{p}	polar moment of inertia with respect to rotation pole of a joint
$K_{??}$	joint/connection stiffness – This still has to be edited
$L/R/T$	abbreviation for longitudinal/radial/tangential
M_{x}	moment for determination of bending stress in x-direction
f_{check}	computed numerical strength for the checked structural resistance case
f_{FE}	numerically calculated strength
f_{k}	characteristic strength
$f_{\mathrm{k,known}}$	calculated or known characteristic structural strength
$f_{\mathrm{test,known}}$	known strength from experiments
$\boldsymbol{S}$	elastic compliance matrix
$S_{11/\ldots}$	entries of the elastic compliance matrix
$S_{\mathrm{n,x}}$	static moment of the net cross-section of CLT in x-direction
V_{x}	coefficient of variation of the ratio of the measured (or known) and computed results ($f_{\mathrm{k,known}}/f_{\mathrm{check}}$ or $f_{\mathrm{test,known}}/f_{\mathrm{check}}$) for n samples
V_{xz}	shear force for the determination of shear stress in the xz-plane

3.2.2 Latin lower-case symbols

a	distance of the two outermost parallel layers of CLT (concerning their centroidal axes)
d	depth of the CLT plate
d_1	depth of the first / upper outermost layer
d_i	depth of layers between the outermost layers
d_{n}	depth of the last / under outermost layer
$d_{\mathrm{x,i}}$	depth of one single layer in x-direction (span direction)
$f_{c,1,0}$	strength proportionality limit parallel to the grain, in N/mm^2
$f_{c,1,90}$	strength proportionality limit perpendicular to the grain, in N/mm^2
$f_{\mathrm{c,2,0}}$	compression strength parallel to the grain, in N/mm^2
$f_{\mathrm{c,2,90}}$	compression strength perpendicular to the grain, in N/mm^2
$f_{\mathrm{c,3,0}}$	compressive strength parallel to the grain after softening, in N/mm^2
$f_{\mathrm{c,0,k}}$	characteristic compression strength parallel to the grain, in N/mm^2
$f_{\mathrm{c,90,k}}$	characteristic compression strength perpendicular to the grain, in N/mm^2
f_{k}	characteristic strength, in N/mm^2
$f_{\mathrm{m,k}}$	characteristic bending strength, in N/mm^2
$f_{\mathrm{t,0,k}}$	characteristic tensile strength parallel to the grain, in N/mm^2
$f_{\mathrm{t,90,k}}$	characteristic tensile strength perpendicular to the grain, in N/mm^2
$f_{\mathrm{t/v,2}}$	tensile or shear strength, in N/mm^2
$f_{\mathrm{t/v,3}}$	residual tensile or shear strength, in N/mm^2
k_{def}	factor for the evaluation of creep deformation taking into account the relevant service class
$k_{\mathrm{lin,c,0}}$	factor describing the proportionality limit for compression parallel to the grain exceeding which plasticising occurs
$k_{\mathrm{lin,c,90}}$	factor describing the proportionality limit for compression perpendicular to the grain exceeding which plasticising occurs
$k_{\mathrm{end,c,0}}$	factor describing the compressive strength parallel to the grain after softening
k_{mod}	modification factor taking into account the effect of the duration of load and moisture content
k_{n}	characteristic fractile factor according to EN 1990, Annex D, Table D.1
$k_{\mathrm{pl,2,0}}$	factor describing the plastic strain when reaching the compression strength parallel to the grain
$k_{\mathrm{pl,2,90}}$	factor describing the plastic strain when reaching the compression strength perpendicular to the grain

$k_{\mathrm{pl},3,0}$ factor describing the plastic strain when reaching the softening strength parallel to the grain

m_{x} mean value of the ratio of the measured (or known) and computed results ($R_{\mathrm{k,known}}/R_{\mathrm{check}}$ or $R_{\mathrm{test,known}}/R_{\mathrm{check}}$) for n samples

u moisture content, in %

3.2.3 Greek upper-case symbols

None.

3.2.4 Greek lower-case symbols

$\alpha_{\mathrm{pl,c},90}$ slope of the stress-strain curve perpendicular to the grain after reaching the compressive strength

γ_{FE} partial factor for modelling covering the uncertainties of the numerical model and the executed type of analysis

γ_{Rd} partial factor associated with the uncertainty of the numerical model and geometric deviations

$\varepsilon_{\mathrm{el},2,0}$ elastic strain when reaching the compressive strength parallel to the grain

$\varepsilon_{\mathrm{el},2,90}$ elastic strain when reaching the compressive strength perpendicular to the grain

$\varepsilon_{\mathrm{el},3,0}$ elastic strain when reaching the softening strength parallel to the grain

$\varepsilon_{\mathrm{el+pl},2,0}$ total strain when reaching the compressive strength parallel to the grain

$\varepsilon_{\mathrm{el+pl},2,90}$ total strain when reaching the compressive strength perpendicular to the grain

$\varepsilon_{\mathrm{el+pl},3,0}$ total strain when reaching the softening strength parallel to the grain

$\varepsilon_{\mathrm{pl},2,0}$ plastic strain when reaching the compressive strength parallel to the grain

$\varepsilon_{\mathrm{pl},2,90}$ plastic strain when reaching the compressive strength perpendicular to the grain

$\varepsilon_{\mathrm{pl},3,0}$ plastic strain when reaching the softening strength parallel to the grain

λ_{rel} relative slenderness ratio

ν Poisson’s ratio

$\nu_{12/\ldots \text{ or LR}/\ldots}$ Poisson’s rations

σ_{cr} lowest elastic critical bifurcation stress of the examined structure

σ_{fin} stress at time t = 50 a due to relaxation

τ_{xz} shear stress

$\tau_{\mathrm{xz,r}}$ rolling shear stress

4. Basis of design and modelling

4.1 General

(1) The basis of Finite Element based design shall be in accordance with the general rules given in EN 1990 and EN 1991 (all parts) and the specific design provisions for timber structures given in EN 1995 (all relevant parts). [68]

(2) Application of Finite Element based design methods should not bring significant resistance increases or decreases of SQRs compared to well-established traditional design methods, unless shown to be reasonable through verification and validation of the employed numerical model according to clause 7 covering all relevant failure modes. [68]

(3) The rules in this document are independent of the software used. However, it should be checked and confirmed by the designer that the software is capable of modelling the relevant physical phenomena (see clause 7). [68]

(4) To ensure design quality, the applicability of the Finite Element based design should be linked to design qualification and experience levels (DQLs). [68]

NOTE Minimum appropriate qualifications and experience of personnel designing structures using numerical models can be defined by national regulations based on EN 1990 Table B.1. [68]

4.2 Finite Element based design methods

(1) Finite Element analysis based design may be executed by one of the following two methods:

a. numerical design calculation,

b. numerical simulation. [68]

(2) The FE based design methods differ regarding the (i) applied geometrical and material properties, (ii) results of the analysis, (iii) validation and verification process, (iv) further evaluation method of output data and (v) reliability assessment of the calculation results. [68]

(3) Numerical design calculations may be different based on the type of analysis (clause 6.1.2) and results. Design rules are given based on the following two categories:

a. requiring subsequent design check,

b. direct resistance check. [68]

NOTE Fig. 4.1 displays the basic design process and differences using the different design methods.

(4) In the case of **numerical design calculations requiring a subsequent design check** the results of the analysis are different system response quantities (SQRs) which are to be further used in design verification formulas according to relevant parts of EN 1995. [68]

(5) In the case of **numerical design calculations with direct resistance check** the result of the analysis is the ultimate resistance of the analysed structure (determined according to clause 8.1.4). [68]

(6) **Numerical simulations** may be applied to complement, extent or replace physical experiments for design assisted by testing. [68]

NOTE National regulations can give further rules and limitation on the application of numerical

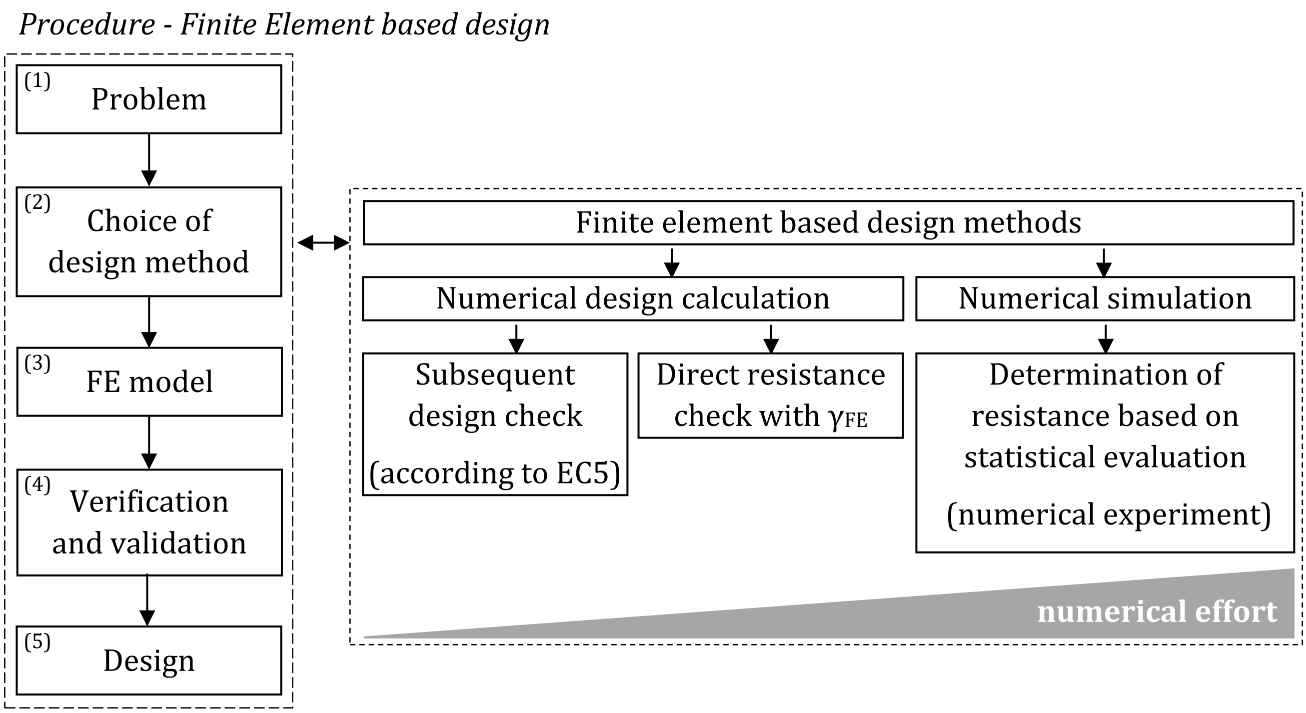


Fig. 4.1: Procedure for a FE based design

simulation.

(7) In the case of numerical design calculations, nominal values according to EN 1995-1-1, relevant product standards or technical approvals should be used for geometrical and material properties, e.g. cross-section dimensions.

(8) In the case of numerical simulations, measured, mean or scattering values should be used for geometrical and material properties. Scattering values should be chosen along experiments, literature or experience.

(9) Application rules for the different methods of analysis are given in clause 5 to 8.

5. Modelling

5.1 General

(1) Numerical models for analysis (clause 6) shall be appropriate to determine effects of actions and compute the relevant SQRs, involving all relevant variables. [70]

(2) Numerical models should as a minimum consider deformations due to bending, shear and axial forces of members, deformations and stiffnesses of connections, joints and supports. [70]

(3) Numerical models should account for support, connection, joint and load eccentricities (clause 5.3). [70]

(4) In numerical models the ideally planned structure is reduced and simplified by beam, plate, shell or volume elements and with possibly changed position to form a coherent (im)perfect model. All reductions and simplifications should be considered where relevant.

(5) Geometrical and material properties should be chosen based on the applied FE based design method according to clause 4.2 (7) and (8).

(6) Structural behaviour may be modelled on a global (whole load-bearing structure of a building), reduced global (e.g. truss), member (e.g. beam) or local (e.g. joint) level of modelling.

(7) Where spatial interactions between members or parts of structures are known from experience and can be described, a spatial structural system may be subdivided into (plane) spatially unconnected parts. The effect on the structural system from adjacent structural systems should be considered (clause 5.3.1). [70]

NOTE If modelling on a reduced global, member or local level adjacent structures can be modelled as effective members with effective stiffnesses. E.g. if modelling a horizontally braced glulam beam, the horizontal bracing can be modelled be means of an effective beam element with effective bending stiffness connected to the glulam beams with springs.

(8) It should be taken in account that the structure on a global to local level of modelling can be sensitive to global, member and local imperfections (clause 5.5 and 7.2).

(9) When modelling timber elements material characteristics as the effect of fibre direction, brittle tensile failure, plasticizing and the size effect should be accounted for (clause 5.4). [70]

(10) Where the material behaviour is adequately ductile or connections of adequate ductility are used in structures, elastic-plastic methods may be used and ductile redistribution of forces may be assumed for the calculation of the internal forces in the members [70]. For elastic-plastic material modelling see clause 5.4 and for elastic-plastic connection modelling see clause 5.3.1.

(11) Where applicable rheological effects such as creep, relaxation, shrinkage and swelling should be considered (e.g. on the redistribution of internal forces). [70]

(12) For hybrid structural systems consisting of materials with significantly different rheological behaviour (e.g. timber-concrete-composite structures), effects such as differential creep, relaxation, shrinkage and swelling due to loading should be considered. [70]

(13) Where applicable the building progress, transport, assembly, operation and demolition should be considered.

5.2 Geometrical models

5.2.1 General

(1) The Finite Element type should be chosen depending the modelled physical phenomenon, boundary conditions and other relevant input parameters (e.g. loads, material model, …), geometry complexity (curvature), discretization, FE based design method, type of analysis, expected results and relevant limit states.

(2) Where adequate, a structure may be modelled using different element types within a model. [68]

(3) Contact elements or other interface elements may be used to couple the different model parts (clause 5.3.1). [68]

(4) The DOFs of the chosen elements should be made compatible within the modelled structure and with the chosen boundary conditions. [68]

(5) The discretization of the numerical model should be adequate and follow the geometrical properties of the structure. Element shape properties should be of suitable quality to ensure accuracy (element aspect ratio, Jacobian ratio, warping factor, ...). [68]

(6) At locations with stress or strain concentrations or large gradients of stresses or strains mesh refinement should be used to ensure the required accuracy. For further information about stress and strain concentrations and singularities see clause 5.4.3, clause 8 and Annex B. Geometrical stress and strain concentrations and numerical singularities may be distinguished according to Annex B. [68]

(7) At locations where failure of the structure is anticipated mesh refinement or other methods (e.g. fracture mechanic approaches, cohesive zones, continuum damage mechanics, …) for adequately capturing the relevant failure behaviour should be used. [68]

(8) The accuracy of the chosen FE mesh (density, chosen element types, …) should be proven by model verification according to clause 7.2. [68]

(9) Geometrical properties should be chosen based on the applied FE based design method according to clause 4.2 (7) and (8).

(10) In the case of analysis used for model verification and validation geometrical properties in deviation from (9) may be chosen (clause 7).

(11) The effects of changes of the cross-sectional area (e.g. notches, holes, hunches, …) on stiffness and load-bearing capacity of structural members should be considered if relevant. [70]

(12) If relevant, it should be taken in account that the structure on a global to local level of modelling can be sensitive to global, member and local imperfections (clause 5.5 and 7.2).

(13) For members with relevant combined bending and shear stresses in the same plane and small ratio of span / height, the influence of cross-sectional warping due to shear should be accounted for.

NOTE This includes e.g. uniformly or punctually loaded beams where both bending and shear stresses are implied due to transverse loading.

(14) Special care should be taken as stresses and strains in Finite Element Analysis are calculated

at the elements' integration points and all material definitions are therefore only applied/valid at the integration points (for e.g. 20 node brick elements CD20R in Abaqus these are one quarter of the element size away from the element edge [71]). Stress and strain concentrations might therefore not be captured with coarse meshes (see numerical singularities Annex B). Values of SQRs at element nodes are extrapolated and therefore might violate the material definitions as e.g. $f_{c,0,2}$ in Fig. 5.3.

(15) Element types not covered by the following clauses may also be applied in the numerical model taking special care on their modelling specifics. [68]

5.2.2 Models using beam elements

(1) Beam elements should only be used if the material behaviour perpendicular to the beam axis is not relevant or is accounted for separately.

(2) The system axis should lie within the profile (or contour) of the modelled structural member [70]. The system axis of beam elements should be either chosen identical to that of the centroid of the cross-sections, or it should be chosen such that the effects of its displacement from the centroid of the cross-sections are sufficiently small to ignore, or if shear and gravity centres differ and are relevant in the calculation, either eccentricities should be included in the interpretation, or internal force adjustment should be made by the numerical analysis [68].

(4) Curved structural members may be modelled by polygons of straight beam elements. [70]

(5) Beam elements should include shear effects if relevant [68]. For members with combined bending and shear stresses and ratio of span / height < 6 the SQRs can be significantly influenced by cross-sectional warping due to shear, which can rule out the use of beam elements and require the use of shell or volume elements.

NOTE The calculation of elastic critical loads (for lateral torsional buckling) for members of varying height can require the use of specific beam elements. [68]

5.2.3 Models using plate or shell elements

(1) Plate or shell elements should only be used if the material behaviour perpendicular to the surface is not relevant or is accounted for separately.

(2) The middle surface of a plate or shell may be taken as the reference surface for modelling. Care should be taken that the effects of eccentricities and offsets from mid-surfaces are properly included in the model and that they realistically represent the structural behaviour of the modelled structure. [68]

(3) Eccentricities and steps in the middle surface should be included in the model, if they induce significant bending effects caused by the membrane stress resultants following an eccentric path. [68]

(4) For the modelling of 3D surface bodies (e.g. shell structures) shell elements should be used having 5 or 6 DOFs at each node. The chosen elements should be able to model thin or thick shells. Special shell elements with different DOFs may be used in specific shell problems (e.g. shells of revolution, cylindrical, conical or spherical shells, etc.). [68]

(5) Curved shells may be modelled as segmented shells with polygonal elements.

(6) Plate or shell elements should include shear effects if relevant. For members with combined bending and shear stresses and small ratios of span / height the SQRs can be significantly influenced by cross-sectional warping due to shear.

5.2.4 Models using solid elements

(1) For the modelling of solid bodies loaded by either in-plane loads or loads perpendicular to their plane, plane elements with 2 or 3 DOFs (only translational) at each node may be used for meshing (e.g. plane stress/strain or axisymmetric problems). [68]

NOTE Solid elements usually have 3 DOFs (only translational) at each node. Special care is needed that the application of forces, deformations, constraints and supports is defined in a compatible way with these three DOFs (i.e. bending moments cannot be applied). [68]

(2) The chosen mesh should be continuous at the intersection points, in the joint regions and at the location of thickness changes. [68]

5.2.5 Multi-level and combined models

(1) In multi-level or combined models, continuous load, displacement and rotation transfer should be provided at the interfaces between the different modelled parts. [68]

(2) If different levels of modelling are used and different structural elements are connected, their eccentricities should be included within the model. [68]

(3) In multi-level or combined models, contact elements or other interface elements may be used to couple the different model parts. The different DOFs of the different elements should be made compatible as well as the discretization of different parts. [68]

5.2.6 Additional rules for cross-laminated timber (CLT)

5.2.6.1 General

(1) The layered structure of CLT contributes to the anisotropic material behaviour of the whole cross-section, additionally to the anisotropic behaviour of the material wood itself.

(2) The rolling shear effect of cross layers should be included if relevant, as it can govern the physical behaviour. Therefore, some assumptions according to Euler-Bernoulli beam theory and Kirchhoff-Love plate theory do eventually not apply (e.g. flat surfaces remain flat).

(3) For structures outside the application limits of Annex C (5) and CLT plates without reinforcement perpendicular to the plate at the points of the maximum shear forces (e.g. reinforced with glued-in steel rods or fully threaded screws), the rolling shear deformations can significantly influence the stress distribution and solid elements should be used.

(4) The Finite Element type should be chosen according to the SRQs to be computed (Table 5.1).

NOTE Some Finite Element types are capable to directly numerically obtain stresses and strains and some are not (see Table 5.1). For element types that do not offer that possibility, a single cross-section (which covers the behaviour of the CLT panel in one direction with a defined width) with effective stiffness values can be used which can be calculated according to Annex C. Stress and strain distributions resulting from computed internal forces can be calculated analytically according to Annex C.

Table 5.1: Finite Element types for modelling CLT

SQR / Structural system	Computation of: deformations, internal forces	Computation of: deformations, internal forces, strains, stresses
Uni-axial spanning systems	- beam elements - shell elements - multilayer option	- volume elements
Two-axial spanning systems <u>neglecting</u> torsional bending	- beam elements [1] - shell elements - multilayer option	- volume elements
Two-axial spanning systems <u>considering</u> torsional bending	- beam elements [1,2] - shell elements [3,4] - multilayer option [3,4]	- volume elments[3,4]

[1] *edge-glued boards: a girder grid can be used but underestimates torsional bending stiffness significantly*
[2] *non edge-glued boards: beam elements can be used as torsional bending stiffness is conservatively set to zero*
[3] *edge-glued boards: without modelling joints between single boards*
[4] *non edge-glued boards :modelling the joints between the single boards is necessary to avoid overestimating torsional bending stiffness*

NOTE If a software is used where there is no option to directly enter effective stiffness values but only to directly enter E- and G-modulus plus the thickness of the plate, a recalculation of the effective stiffness values is required by taking the gross cross-section into account (see Annex C).

(5) The element type should be chosen depending on whether edge-glued or not edge-glued CLT elements are to be modelled (see Table 5.1).

(6) The element type should be chosen depending on whether uniaxial or not biaxial spanning CLT-plates are to be modelled. If biaxial spanning CLT-plates are to be modelled, it should be considered whether the plate is under significant torsional flexural loading, for example at loaded, unsupported edges (see Table 5.1).

(7) Single layers with varying height, an asymmetrical arrangement of the single layers, double layers in one direction or the combination of layers with different material quality can result in a high effort for calculating effective stiffness values. Using solid elements can be beneficial in these cases.

(8) If just deformations and internal forces are obtained numerically, stresses may be calculated in a second step analytically (see Annex C).

(9) In order to obtain a reduction of computational effort and solver time as well as a less complicated interpretation of calculation results, beam-, shell and multilayer-elements may be used instead of solid elements for standard structural engineering problems.

(10) If a software is used where there is no option to directly enter effective stiffness values but only to directly enter E- and G-modulus plus the thickness of the plate, effective E- and G-modulus and/or an effective plate thickness should be used to consider the effective stiffness (see Annex

C).

(11) If computing CLT-panels with in-plane loading, E_{90} should be set to zero and the net cross-section in force direction should be considered.

(12) In-plane buckling: For a ratio of span to depth > 20 the influence of the rolling shear effect on the longitudinal stress distribution may be neglected. If the increase of internal forces due to second order effects is less than 10 %, calculations may be done according to first-order theory. Verification of in-plane buckling may be done according the equivalent member (k_c) method in EN 1995-1-1-, considering the net cross-section [56].

(13) For computing shear walls under lateral loading the net cross-section of the weaker layer group should be considered. The width of the single boards should be used as input values for determining the torsional shear stress in the intersections of the longitudinal and crosswise boards. Thus, all single boards should be modelled. Alternatively, torsional shear stresses may be calculated analytically (see [56]).

5.2.6.2 Models using beam elements

CLT - out of plane loading

(1) Beam elements should include shear effects (clause 5.2.6.1 (2)).

NOTE Suitable beam elements can be based on Timoshenko beam theory [1].

(2) Multi-axial spanning slabs may be modelled using a grillage of beam elements in the direction of the CLT layers, where the beam elements in the respective direction should be modelled with their effective bending and shear stiffnesses [56]. This approach may be used if CLT elements are not edge glued by conservatively setting the torsional stiffness to zero [1]. If CLT elements are edge glued a grillage of beam elements should only be used if torsional bending stresses are not decisive.

NOTE The torsional stiffness of a grillage of beam elements is significantly lower than it actually can be assumed for edge glued boards.

(3) Effective stiffness values may be used which may be calculated according to Annex C. Alternatively, an effective beam height may be used.

(4) If beam elements are used, only a direct numerical computation of deformations and internal forces is possible, but no direct numerical computation of the stress or the strain distribution across the cross-section height in the FE model. Stress and strain distributions over the cross-section height may be obtained analytically according to Annex C, if the application limits of the analytical formulas are met.

(5) The used software should account for both E- and G-modulus in input, computation and post-processing.

NOTE Anisotropic / orthotropic material parameter input is not required as effective stiffness values and beam elements (in the direction of the CLT layers for multi-axial spanning slabs) are used.

5.2.6.3 Models using plate or shell elements

CLT - out of plane loading

(1) Plate or shell elements should include shear effects (clause 5.2.6.1 (1)).

NOTE Suitable plate or shell elements can be based on Mindlin-Reissner plate theory [1].

(2) Multi-axial spanning slabs with not edge glued boards should be modelled including the joints in between the single boards as E_{90} cannot be set towards zero. Multi-axial spanning slabs with edge glued boards should be modelled without joints in between the single boards and $E_{90} > 0$.

(3) The CLT's cross-section should be modelled as one single cross-section without modelling the single layers. Effective stiffness values may be used which can be calculated according to Annex C. Alternatively, an effective plate thickness may be used.

(4) If plate or shell elements are used, only a direct numerical computation of deformations and internal forces is possible, but no direct numerical computation of the stress or the strain distribution across the cross-section height in the FE model. Stress and strain distributions over the cross-section height may be obtained analytically according to Annex C, if the application limits of the analytical formulas are met.

(5) The used software should account for E_0, E_{90}, G_0 and G_{90} in input, computation and post-processing.

5.2.6.4 Models using multilayer options

CLT - out of plane loading

(1) Clause 5.2.6.3 applies.

5.2.6.5 Models using solid elements

CLT - out of plane loading

(1) All single layers should be modelled and finally assembled to one cross-section, with local material coordinate systems depending on the single layers material orientations. Thus, no effective stiffness values should be used.

(2) Multi-axial spanning slabs with not edge glued boards should be modelled including the joints in between the single boards as E_{90} cannot be set towards zero. Multi-axial spanning slabs with edge glued boards should be modelled without joints in between the single boards and $E_{90} > 0$.

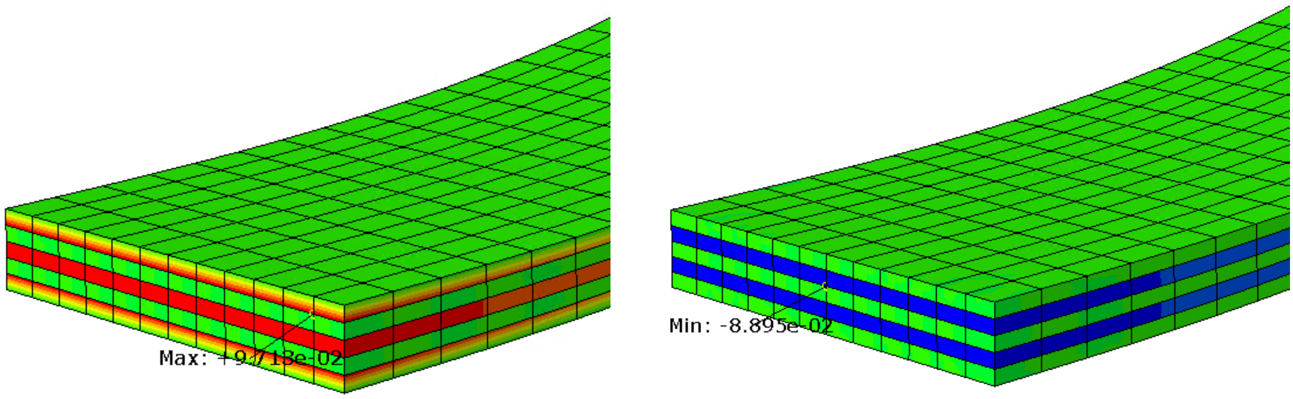
(3) For coupling the respective nodes of the surfaces of the different CLT layers rigid coupling (tie constraints) may be used. The surfaces of the layers oriented in span-direction should be chosen as master surfaces whereas the surfaces of the cross layers should be chosen as slave surfaces. The influence of the gluing of the individual layers may be neglected.

(4) Deformations, the stress and the strain distribution across the cross-sections height may be directly obtained numerically when using solid elements (see Fig. 5.1). Thus, solid elements may be used where spatial stress distributions do prevail.

(5) Requirements to the used software:

- For input, computation and post-processing E_1, E_2, E_3 and G_{12}, G_{13}, G_{23} should be considered.

- Working with local material coordinate systems should be possible.
- Constraints and/or rigid-coupling options should be offered.
- Multiple coupling conditions per node or using spring elements should be possible.



(a) Shear stresses τ_{xz} of longitudinal layers 1,3 and 5 in [N/mm^2]

(b) (Rolling) shear stresses $\tau_{xz,r}$ of cross layers 2 and 4 in [N/mm^2]

Fig. 5.1: Exemplary numerical results of a uniformly loaded CLT plate spanning in one direction.

5.3 Supports, connections, joints and load modelling

5.3.1 Definition of supports, connections and joints

(1) Numerical models should be chosen to reflect in a realistic or conservative manner the behaviour of the physical supports, connections or joints of the load-bearing structure. If relevant, the stiffness, the deformation and the load-bearing capacity of supports, connections or joints should be considered. [68]

(2) If structural behaviour is modelled on a reduced global, member or local level (clause 5.1 (6) and (7)), supports should be chosen to reflect in a realistic or conservative manner the behaviour of the adjacent structural systems. The chosen supports should consider the stiffness, the deformation and the load-bearing capacity of the adjacent structural systems. [68]

NOTE If modelling on a reduced global, member or local level adjacent structures can be modelled as effective members with effective stiffnesses. E.g. if modelling a horizontally braced glulam beam, the horizontal bracing can be modelled be means of an effective beam element with effective bending stiffness connected to the glulam beams with springs.

(3) The degrees of freedom of the boundary conditions should be chosen in a way that the structural system is at least statically determined.

(4) Support, connection and joint properties should be chosen based on the applied FE based design method according to clause 4.2 (7) and (8).

(5) Numerical models should account for support, connection and joint eccentricities. [68]

NOTE Eccentricities in the supports, connections or joints of structural members can be represented in the mechanical model as fictitious beam elements or rigid links/elements not having a material origin. [68]

(6) Where the stiffness of supports, connections or joints affects the SQRs, a sensitivity analysis may/should be carried out to identify unfavourable situation. [68] – This still has to be edited

NOTE To do so, higher, mean and lower values of stiffnesses can be assigned in the numerical model. [68]

(7) For joints with connectors with different stiffnesses see EN 1995-1-1. – This still has to be edited

(8) Special boundary conditions may be chosen in a compatible way with the buckling check. [68]

(9) If concentrated or point supports, connections or joints are used in plate or shell models, or concentrated, point or line supports, connections or joints are used in solid models, numerical stress and strain concentrations can occur near these, which should be investigated. Recommendations are given in Annex B. [68]

(10) Special care should be taken when defining supports, connections or joints to avoid undesired clamping effects (especially in case of plate, shell and solid elements). [68]

NOTE Supports that act as pinned supports in a linear analysis can produce undesired stiffening effects in a non-linear analysis. [68]

(11) Care should be taken if symmetry conditions are applied and symmetry plane passes through supports or loads. [68]

NOTE Symmetry can only be used where the expected structural behaviour (failure mode, buckling mode, deformed shape, loading and supporting conditions etc.) has been verified to be symmetrical. [68]

(12) If not stated differently in EN 1995-1-1, relevant product standards or technical approvals the positive influences of friction may be disregarded.– This still has to be edited

(13) Fictitious beam elements may be used to model eccentric connections, joints or supports. The orientation of fictitious beam elements should coincide as closely as possible with the actual connection, joint or support configuration. [68]

(14) Spring elements may be used to model the translational and/or rotational stiffness of connections, joints or supports. The location of spring elements should coincide as closely as possible with the actual connection, joint or support configuration. [68]

(15) If connections, joints and supports are modelled using fictitious beam elements or springs, the translational stiffnesses may be defined based on the slip modulus K_d of the fasteners and/or the rotational stiffness $I_p K_d$, with the polar moment of inertia I_p with respect to rotation pole. K_d should be either constant or load dependent. [68] – This still has to be edited

(16) Connections and joints may be modelled with linear elastic load-deformation behaviour (constant stiffness K_d). – This still has to be edited

(17) Connections and joints may be modelled with elastic-plastic load-deformation behaviour (load depending stiffness K_d), if the members and joints are able to develop sufficient deformation and/or rotation capacity. – This still has to be edited

(18) The surface roughness and pre-damage due to machining in the region of a compression contact between two members can practically lead to a significant reduction of the contact stiffness. This effect can occur especially when comparing experimentally and numerically determined displacements (and resulting stiffnesses) in the contact area of joints with cut surfaces approximately perpendicular to the fibre. This may be considered by a reduced stiffness of contact or interface elements or a material layer with reduced stiffnesses. – This still has to be edited

(19) Geometrical imperfections like e.g. hole clearance or geometry deviations and structural imperfections like e.g. knots can have a significant influence on the stiffness and load-bearing capacity of connections, joints or supports. This should be considered when the load-deformation behaviour of connections, joints or supports is investigated directly using solid elements and no explicit slip moduli K are used for the numerical modelling. – This still has to be edited

(20) Contact elements or other interface elements may be used for modelling force or displacement transfer in between different structural members made of solid elements. The mesh of such adjacent structural members should be compatible. For each node on the contact surface of a member there should be exactly one corresponding node on the contact surface of the adjacent member. Care should be taken of the penetration of members due to contact or interface element properties.

NOTE 1 Friction coefficients can be assigned to contact or interface elements according to relevant standards, literature, experience or experimental results.

NOTE 2 Crack opening in predefined planes can be modelled by means of cohesive zones (clause 5.4.3).

(21) For modelling supports, connections or joints especially with solid elements additional nodes

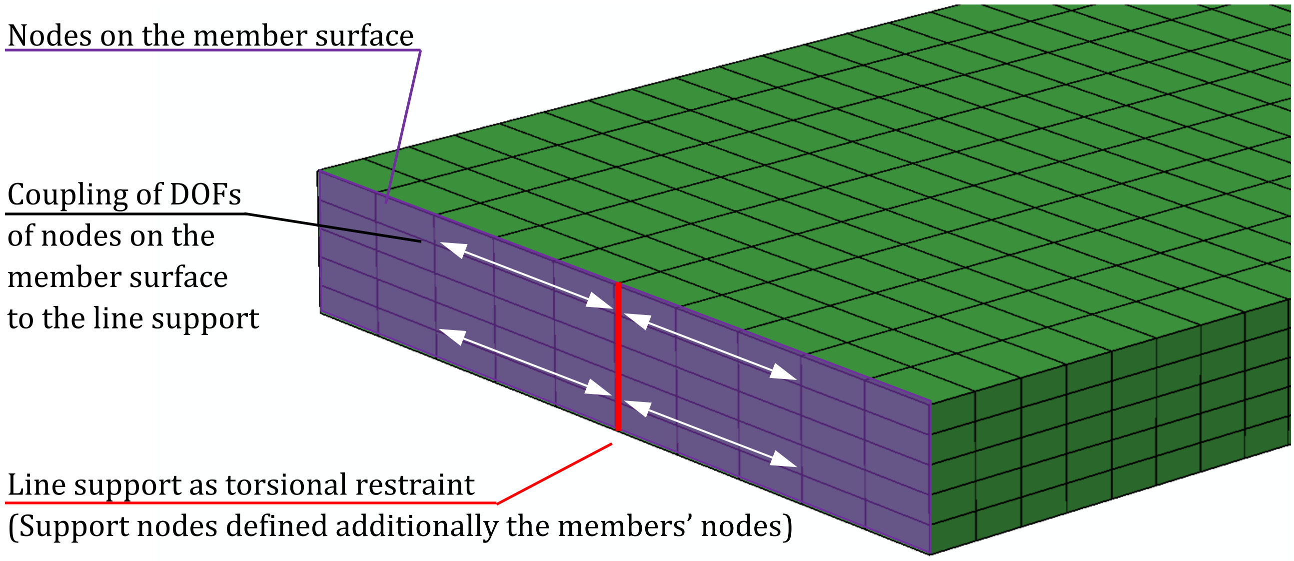


Fig. 5.2: Exemplary modelling of the torsional restraint of a fork support of a CLT plate

may be modelled and utilised for creating certain boundary conditions (see **Fig. 5.2**).

(22) Single dowel-type connectors and joints consisting of multiple dowel-type connectors which are loaded perpendicular to the longitudinal axis of the connectors may be modelled by means of beam-on-foundation models. With such models the elastic-plastic behaviour of single connectors and whole joints can be considered. The application of beam-on-foundation models is described in Annex D. – This still has to be edited

5.3.2 Definition of loading

(1) For structural design verification according to clause 8, actions, partial factors and load combinations should be chosen according to EN 1991 (all relevant parts) and EN 1995 (all relevant parts).

(2) In the case of analysis used for model verification and validation loading in deviation from (1) may be chosen (clause 7).

(3) Actions may be applied as forces, displacements, rotations, strains or temperatures (type of action).

(4) Where a non-linear analysis is used, the appropriateness of the chosen actions (forces or displacements) should be checked and verified according to clause 7. [68]

NOTE Loads applied by means of forces or displacements can lead to different results, arising from the load-deformation behaviour, different load transfer and load distribution modes in the numerical model. [68]

(5) Where geometrically non-linear analysis is performed, and follower load effects are possible, either they should be incorporated into the analysis, or it should be verified that their influence is negligible. [68]

(6) If structural behaviour is modelled on a reduced global, member or local level (clause 5.1 (6) and (7)), actions resulting from adjacent structural systems should be considered.

(7) The numerical model should account for load eccentricities. [68]

(8) If concentrated or point loads are used in plate or shell models, or concentrated, point or line

loads are used in solid models, numerical stress and strain concentrations can occur near these, which should be investigated. Recommendations are given Annex B.

(9) Special care should be taken when defining actions by means of line or surface displacements or rotations to avoid undesired clamping effects.

5.3.3 Additional rules for cross-laminated timber (CLT)

(1) Special care should be taken as the support modelling has a huge impact on the stress distribution when using solid elements, especially on shear- and rolling shear stresses. Depending on the type of modelling of the supports parasitic stresses and strains can appear.

(2) Hinged supports in z-direction may be modelled by coupling the deformations all nodes at the cross-section of the support in z-direction to one support node. The deformation of the support node in z-direction may be set to zero. Alternatively, hinged supports may be modelled by springs in z-direction at all nodes of the cross-section at the support. (see [37])

NOTE If using a spring for modelling a support in z-direction a stiffness of 10^{10} N/mm per m^2 of the cross-section at the support can be used.

5.4 Material modelling

5.4.1 General

(1) Material specific influence parameters that should be considered when modelling timber (products) are:

a. Product type (e.g. glulam) and material grade (e.g. GL 24h),

b. anisotropy (e.g. longitudinal, radial and tangential (L/R/T) or edgewise and flatwise) and fibre direction,

c. stress direction (tension, compression, shear) and stress interactions,

d. type of loading (static or dynamic),

e. material scattering and inhomogeneities (results e.g. in the size effect and structural imperfections),

f. size effect (also known as volume effect),

g. structural imperfections,

h. failure behaviour,

i. plasticising and magnitude of loading (proportionality limit),

j. moisture content (MC; service class SC),

k. time dependent behaviour such as creep, relaxation, swelling, shrinkage, reduction of strength / creep strength,

l. temperature,

m. stochastic effects depending on the applied safety concept (e.g. ULS or SLS on chosen material stiffnesses) and the FE based design method (clause 4.2).

NOTE 1 All these parameters can influence the stress-strain relationship and thus the material stiffness and strength of timber (products).

NOTE 2 Currently, no material model for timber (products) is known that sufficiently accurately represents the mechanical material behaviour under the influencing parameters mentioned. Material modelling of wood is therefore always carried out under simplifying assumptions.

(2) Product type and material grade especially influence strength, stiffness, material scattering, anisotropy and cross-sectional composition. It should be considered according to product standards (e.g. EN 14080 [66] for glulam), technical approvals (e.g. ETA-14/0354 [67] for beech LVL beams) or experimental results.

(3) If relevant, the anisotropic material behaviour of wood should be considered. The anisotropic material behaviour may be simplified assuming:

a. cartesian coordinate system (common assumption; instead of polar coordinates due to growth characteristics),

b. orthotropy (common assumption; instead of anisotropy with non-perpendicular material axis)

c. transverse isotropy (common assumption for solid timber and glulam as radial and tangential direction can usually not be distinguished),

d. isotropy (e.g. required when using beam elements).

NOTE Wood is an anisotropic material with material properties depending on three material directions. These are the longitudinal, radial and tangential direction. For some timber products these are also referred to as the longitudinal, flatwise and edgewise direction. Processed timber products can have one, two, three or more material directions.

(4) For solid timber, glulam and CLT, radial and tangential direction should not be distinguished and a transverse isotropic or isotropic material behaviour with Cartesian coordinate system should be assumed.

(5) The number of independent material directions (clause (3)) and the element type (clause 5.2) should be chosen in dependence on each other.

(6) If applicable, dynamic effects should be considered according to the relevant parts of EN 1995.

(7) Material properties should be chosen based on the applied FE based design method according to clause 4.2 (7) and (8). In the case of numerical simulations, and if relevant, it should be chosen with care whether material scattering and inhomogeneities are modelled on a local or member level depending on the modelled phenomenon.

NOTE 1 Due to material scattering and inhomogeneities which result from the natural growth characteristics (knots, fibre deviations, …) of wood, material stiffness and strength of timber (products) of the same strength class can vary significantly within members and between different members.

NOTE 2 For modelling of members on global to member level and variation of material properties of glulam and LVL lamellas along their length, the *Karlsruher Rechenmodell* is widely accepted and also used for derivation of normative material properties (e.g. described in [5]). Similar approaches are described in e.g. [20], [48], [50]. These models usually can't explicitly account for material variation over the board width. This is still an open issue. For modelling on a local level only few modelling approaches like e.g. [35] exist.

(8) It may be assumed that elastic stiffnesses in tension, compression and bending in the same material direction (e.g. L or R or T) are equal up to the proportionality limit.

NOTE Due to the different behaviour of timber under tension, shear and compression and the complex interaction relations when timber is loaded at an angle to the grain, the stress direction influences the load-bearing behaviour.

(9) The size effect on the bending, tensile and shear strengths may be considered according to clause 5.4.2 and 5.4.3. The size effect on the compression strengths and material stiffnesses may be neglected.

NOTE Due to local material variability within timber members the strength is depending on the members' stressed volume. This is generally referred to as size or volume effect. With increased stressed volume and increased stresses, the probability of local errors leading to a component failure increases and thus the material strength decreases. This effect can be partly or completely counteracted by the influence of material grading. The size effect is especially relevant if (quasi) brittle failure behaviour (tensile or shear stress) is dominant.

(10) Structural imperfections should be accounted for according to clause 5.5.

NOTE Due to local material variability of timber (products), the centre of stiffness of members when considering scattering material properties and when assuming a homogeneous material does not coincide. The difference is referred to as structural imperfections.

(11) Timber (products) usually behaves ductile (plasticise) under compression and elastic / (quasi) brittle under tension and shear. This should be considered according to clause 5.4.2 and 5.4.3.

(12) The stress-strain proportionality level, defining the stress above which plasticising occurs, should be considered according to clause 5.4.2 and 5.4.3.

NOTE Plasticising in timber (products) can occur on strength levels significantly below the material strength due to local material failure on micro- and macroscopic level. Therefore, the material behaviour is linear elastic only up to specific load levels depending on the direction and type of stress (L/R/T, tension/compression/shear). This is referred to as proportionality level.

(13) Material stiffness and strength of timber (products) are directly related to the wood moisture content and usually decrease with increasing moisture content [6]. This should be considered according to clause 5.4.2 and 5.4.3.

NOTE The wood moisture content is specified in EN 1995-1-1 by means of the service classes (SC).

(14) If timber (products) are exposed to moisture content changes, swelling and shrinkage occur leading to additional deformations and strains. If these deformations and strains are prevented, restraint stresses can be caused which should be considered according to EN 1995-1-1 if relevant.

(15) If loading is applied over a period of time, creep deformations occur (depending on e.g. load direction, load level, duration of loading, moisture content, moisture content changes, …) and the material strength is reduced. This should be considered according to clause 5.4.2 and 5.4.3.

(16) Material stiffness and strength of timber (products) are directly related to the material temperature and usually decrease with increasing temperature [6]. This document only applies for the design of structures that are not exposed to temperatures above 60 °C. For temperatures below 60 °C the influence of temperature on material stiffness and strength may be neglected.

(17) In the case of analysis used for model verification and validation material properties in deviation from (18) may be chosen.

(18) For modelling of steel components see prEN 1993-1-14 [68].

(19) For modelling of concrete components see EN 1992-1-1.

(20) For hybrid structural systems consisting of materials with significantly different rheological behaviour (e.g. timber-concrete-composite structures), effects such as differential shrinkage, swelling and creep due to loading should be considered. [70]

(21) In the case of numerical simulations alternative material models to clause 5.4.2 to 5.4.4 may be used if they are verified and validated according to clause 7.

5.4.2 Modelling of material stiffness

5.4.2.1 General

(1) Structural behaviour on a (reduced) global level of modelling should be assessed by calculating the action effects with a linear elastic material model (clause 5.4.2.2) and LA, LBA, GNA or GNIA (clause 6.1.2). [70]

(2) For structural behaviour on a member and local level of modelling non-linear material behaviour (plasticising) of timber (products) may be considered for compression parallel or perpendicular to the grain (clause 5.4.2.3).

(3) In the case of numerical simulations and for structural behaviour on a member or local level of modelling and if accounting for material scattering on a local level, a post-failure decreases of tension and bending stiffness may be considered along experiments, literature or experience (clause 5.4.2.3).

5.4.2.2 Elastic material modelling

(1) The three-dimensional **orthotropic** elastic material stiffness matrix $\boldsymbol{C}$ of timber (products) may be written in Voigt's notation as follows:

$$\boldsymbol{C} = \begin{bmatrix} C_{11} & C_{12} & C_{13} & 0 & 0 & 0 \\ C_{21} & C_{22} & C_{23} & 0 & 0 & 0 \\ C_{31} & C_{32} & C_{33} & 0 & 0 & 0 \\ 0 & 0 & 0 & C_{44} & 0 & 0 \\ 0 & 0 & 0 & 0 & C_{55} & 0 \\ 0 & 0 & 0 & 0 & 0 & C_{66} \end{bmatrix} = \boldsymbol{S}^{-1} \tag{5.1}$$

The stiffness matrix **C** may be calculated from the inverse of the compliance matrix $\boldsymbol{S}$:

$$\boldsymbol{S} = \begin{bmatrix} S_{11} & S_{12} & S_{13} & 0 & 0 & 0 \\ S_{21} & S_{22} & S_{23} & 0 & 0 & 0 \\ S_{31} & S_{32} & S_{33} & 0 & 0 & 0 \\ 0 & 0 & 0 & S_{44} & 0 & 0 \\ 0 & 0 & 0 & 0 & S_{55} & 0 \\ 0 & 0 & 0 & 0 & 0 & S_{66} \end{bmatrix} \tag{5.2}$$

$$\boldsymbol{S} = \begin{bmatrix} 1/E_1 & -\nu_{21}/E_2 & -\nu_{31}/E_3 & 0 & 0 & 0 \\ -\nu_{12}/E_1 & 1/E_2 & -\nu_{32}/E_3 & 0 & 0 & 0 \\ -\nu_{13}/E_1 & -\nu_{23}/E_2 & 1/E_3 & 0 & 0 & 0 \\ 0 & 0 & 0 & 1/G_{23} & 0 & 0 \\ 0 & 0 & 0 & 0 & 1/G_{13} & 0 \\ 0 & 0 & 0 & 0 & 0 & 1/G_{12} \end{bmatrix} \tag{5.3}$$

with
$E_1 = E_L$ modulus of elasticity in longitudinal direction
$E_2 = E_R$ modulus of elasticity in radial direction
$E_3 = E_T$ modulus of elasticity in tangential direction
$G_{12} = G_{LR}$ shear modulus in longitudinal-radial plane
$G_{13} = G_{LT}$ shear modulus in longitudinal-tangential plane
$G_{23} = G_{RT}$ shear modulus in radial-tangential plane
$\nu_{12/LR/...}$ Poisson's ratios with first index as load and second index as deformation direction

NOTE Assumptions: Hooke's Law applies (linear elastic material behaviour, small deformations, temperature independency, moisture independency, homogenous material of uniform density, no coupling between stress components); 3 independent material directions; material directions perpendicular to each other; normal and shear stresses not coupled; shear stresses not coupled.

If symmetry is assumed, $\boldsymbol{C}$ may be directly determined as follows:

$$\boldsymbol{C}=\begin{bmatrix} (1-\nu_{23}\nu_{32})\Delta E_1 & (\nu_{12}+\nu_{13}\nu_{32})\Delta E_2 & (\nu_{13}+\nu_{12}\nu_{23})\Delta E_3 & 0 & 0 & 0 \\ (\nu_{12}+\nu_{13}\nu_{32})\Delta E_2 & (1-\nu_{13}\nu_{31})\Delta E_2 & (\nu_{23}+\nu_{21}\nu_{13})\Delta E_3 & 0 & 0 & 0 \\ (\nu_{13}+\nu_{12}\nu_{23})\Delta E_3 & (\nu_{23}+\nu_{21}\nu_{13})\Delta E_3 & (1-\nu_{12}\nu_{21})\Delta E_3 & 0 & 0 & 0 \\ 0 & 0 & 0 & G_{23} & 0 & 0 \\ 0 & 0 & 0 & 0 & G_{13} & 0 \\ 0 & 0 & 0 & 0 & 0 & G_{12} \end{bmatrix} \tag{5.4}$$

with $\Delta=\dfrac{1}{(1-\nu_{12}\nu_{21}-\nu_{13}\nu_{31}-\nu_{23}\nu_{32}-2\nu_{13}\nu_{21}\nu_{32})}$

$\dfrac{\nu_{12}}{E_1}=\dfrac{\nu_{21}}{E_2};\ \dfrac{\nu_{13}}{E_1}=\dfrac{\nu_{31}}{E_3};\ \dfrac{\nu_{23}}{E_2}=\dfrac{\nu_{32}}{E_3}$

(2) If **transverse isotropy** is assumed additionally to the assumptions in (1) the three-dimensional compliance matrix $\boldsymbol{S}$ may be written as follows:

$$\boldsymbol{S}=\begin{bmatrix} 1/E_1 & -\nu_{21}/E_2 & -\nu_{21}/E_2 & 0 & 0 & 0 \\ -\nu_{12}/E_1 & 1/E_2 & -\nu_{32}/E_2 & 0 & 0 & 0 \\ -\nu_{12}/E_1 & -\nu_{23}/E_2 & 1/E_2 & 0 & 0 & 0 \\ 0 & 0 & 0 & 1/G_{23} & 0 & 0 \\ 0 & 0 & 0 & 0 & 1/G_{12} & 0 \\ 0 & 0 & 0 & 0 & 0 & 1/G_{12} \end{bmatrix} \tag{5.5}$$

with

	$E_1=E_{\mathrm{L}}$	modulus of elasticity in longitudinal direction
	$E_2=E_{\mathrm{R=T}}$	modulus of elasticity perpendicular to the grain
	$G_{12}=G_{\mathrm{LR=LT}}$	shear modulus in longitudinal-radial/tangential
	$G_{23}=G_{\mathrm{RT}}$	shear modulus in radial-tangential plane
	$\nu_{12/\mathrm{LR=LT}/\dots}$	Poisson's ratios with first index as load and second index as deformation direction

The stiffness matrix $\boldsymbol{C}$ may be calculated according to Formulae (5.1). If assuming symmetry, a simplification of Formulae (5.4) may be used for calculating the stiffness matrix $\boldsymbol{C}$.

NOTE In Formulae (5.5) radial and tangential direction are not distinguished and thus the assumption is that material properties in both directions are equal (R = T). Formulae (5.5) can be rewritten for L = R or L = T.

(3) If **isotropy** is assumed additionally to the assumptions in (1) the three-dimensional compliance matrix $\boldsymbol{S}$ may be written as follows:

$$\boldsymbol{S}=\begin{bmatrix} 1/E & -\nu/E & -\nu/E & 0 & 0 & 0 \\ -\nu/E & 1/E & -\nu/E & 0 & 0 & 0 \\ -\nu/E & -\nu/E & 1/E & 0 & 0 & 0 \\ 0 & 0 & 0 & 1/G & 0 & 0 \\ 0 & 0 & 0 & 0 & 1/G & 0 \\ 0 & 0 & 0 & 0 & 0 & 1/G \end{bmatrix} \tag{5.6}$$

with

	E	modulus of elasticity
	G	shear modulus

$$\nu = \frac{E}{2G} - 1 \quad \text{Poisson's ratio, which should be calculated from known E and G} \tag{5.7}$$

(4) E and G in Formulae (5.1) to (5.7) should be chosen based on the applied FE based design method according to clause 4.2 (7) and (8).

NOTE 1 For modelling of members (on global to member level) with variation of material properties of glulam and LVL lamellas along their length the *Karlsruher Rechenmodell* is widely accepted and also used for derivation of normative material properties (e.g. described in [5]). Similar approaches are described in e.g. [20], [48], [50]. These models usually can't explicitly account for material variation over the board width. This is still an open issue. For modelling on a local level only few modelling approaches as e.g. [35] exist.

NOTE 2 *Probabilistic Model Code* [30] gives distribution functions and coefficients of variation (COV) for structural timber. More recent and specific values are published for glulam made of softwood in [23] and [47], for beech glulam in [21] and [54] and for beech LVL in [17], [32].

(5) For calculation of the critical bending stress/moment for lateral torsional buckling the product $E_{0,05}$ $G_{0,05}$ may be multiplied by 1,4 or both $E_{0,05}$ and $G_{0,05}$ separately each by 1,2.

(6) Formulae (5.3), (5.4), (5.5) or (5.6) should only be used, if all necessary material properties E and G are given in the relevant (product) standards or technical approvals.

NOTE If modelling glulam or solid timber, Formulae (5.3) and (5.4) cannot be used as it cannot be distinguished between radial and tangential direction for these timber products. Thus Formulae (5.5) or (5.6) can be used.

(7) The influence of the size effect on E, G and ν may be neglected. In the case of numerical simulations, the influence of the size effect on E and G may be considered with numerical models such as named in (6) or analytical models.

NOTE 1 Equations for the analytical consideration of the size effect on E_{L} and $G_{\mathrm{LT/LR}}$ of glulam are reported in [8] and [47].

NOTE 2 No data of the influence of the size effect on $E_{\mathrm{R/T}}$, G_{RT} and ν are known to the authors.

(8) For both numerical design calculations and numerical simulations Poisson's ratios ν in Formulae (5.1) to (5.7) should be chosen as mean values according to literature, experiments or assumed to be 0.3. Latter should only be assumed if effects of transverse strains are negligible (no local stress and strain concentrations in relevant areas and no lateral restraining effects) or if the relevant calculated SQR's deviate less than 10 % from computations with accurate Poisson's ratios taken from literature or experiments.

NOTE 1 No Poisson's ratios are given in current standards and regulations. Additionally, little is known about the distribution functions of Poisson's ratios.

NOTE 2 An overview of Poisson's ratios can be found in [43] and [57]. Additional values for soft- and hardwood can be found in [6], for beech LVL in [32] and for spruce in [40] or [41]. Poisson's ratios in literature are often determined on small defect-free test specimens and thus might not cover effects of material defects and variations.

(9) If using Formulae (5.6) for isotropic material, special care should be taken in choosing ν as Formulae (5.7) should hold.

(10) In the case of analysis used for model verification (clause 7.2), initial values of material prop-

erties should be chosen according to (4) to (9). If a sensitivity study is conducted at least two variations with smaller and two with higher values for the relevant material properties should be conducted. Variations should be within realistic scattering of material properties. For variation - 20 %, - 5 %, + 5 %, + 20 % starting from the initial value may be used for material properties E, G and ν.

(11) In the case of analysis used for model validation (clause 7.3), the material properties should be chosen according to the used accurate solution (benchmark) or mean values should be used.

(12) The material stiffness matrix $\boldsymbol{C}$ and the compliance matrix $\boldsymbol{S}$ should be chosen in accordance with the element type (clause 5.2).

(13) If plate, shell or beam elements (clause 5.2) are used, the material stiffness matrix $\boldsymbol{C}$ and the compliance matrix $\boldsymbol{S}$ in Formulae (5.5) and (5.6) may be reduced by superfluous rows and columns.

(14) In the case of numerical design calculations,

a. bar shaped members, for which the material behaviour perpendicular to the beam axis is not relevant or is accounted for separately and the criteria of clause 5.2.2 are met, should be modelled using beam elements (clause 5.2.2) with elastic material stiffness according to Formulae (5.6),

b. bar shaped members for which the criteria of (14) a. are not met, should be modelled using plate or shell elements (clause 5.2.3) with elastic material stiffness according to Formulae (5.5) or Formulae (5.6),

c. plate or shell shaped members, for which the material behaviour perpendicular to the surface is not relevant or is accounted for separately and the criteria of clause 5.2.3 are met, should be modelled either using plate or shell elements (clause 5.2.3) with elastic material stiffness according to Formulae (5.5) or Formulae (5.6) or using a girder grid (clause 5.2.6) with elastic material stiffness according to Formulae (5.6),

d. other members, should be modelled using solid elements (clause 5.2.4) with elastic material stiffness according to Formulae (5.3), Formulae (5.4), Formulae (5.5) or Formulae (5.6).

(15) In the case of numerical simulations, the elastic material stiffness $\boldsymbol{C}$ and compliance matrices $\boldsymbol{S}$ should be chosen according to the modelled phenomenon.

(16) The short term influence of the moisture content on E, G and ν in the case of

a. numerical design calculations may be neglected,

b. numerical simulations should be considered along experiments, literature or experience.

The influence of the moisture content on the modulus of elasticity parallel to the grain E_{L} of soft- and hardwood products may be considered according to EN 384 [62] by

$$E_{\mathrm{L}}(u) = \frac{E_{\mathrm{L}}}{1 + 0.01(u - 12)} \tag{5.8}$$

with $E_{\mathrm{L}}(u)$ modulus of elasticity for an arbitrary moisture content

E_L	modulus of elasticity for u = 12 %, e.g. according to EN 14080 [66] for glulam
u	moisture content in [%]

NOTE 1 Formulae (5.8) is applicable for common moisture contents in SC 1 to 3.

NOTE 2 Values for moisture content dependent reduction of E_L for different wood species can be found in [17]. Values for moisture dependent reduction of E, G and ν are reported for defect-free small specimen of spruce in [40], [41] and [57], of sycamore maple in [49] and of beech in [28], [42], [44] and [57]. Some values and recommendations can also be found in [6].

NOTE 3 The effects of temperature-moisture interaction on E and G are reported in [6].

(17) The influence of temperature on E, G and ν should be neglected (for temperatures below 60 °C for which this document is applicable). In the case of numerical simulations, the influence of temperature on E and G may be considered along experiments, literature or experience.

NOTE 1 Values for temperature dependent reduction of E and G are reported in [6].

NOTE 2 No data of the influence of temperature on ν are known to the authors.

NOTE 3 The effects of temperature-moisture interaction on E and G are reported in [6].

5.4.2.3 Plastic material modelling

5.4.2.3.1 General

(1) Non-linear material behaviour (plasticising) of timber (products) may be considered for compression parallel or perpendicular to the grain. Non-linear material behaviour for tension either may be considered according to (3) or should be neglected.

(2) If non-linear material behaviour of timber (products) significantly influences the SQRs it should be considered. This applies, among other things, to buckling phenomena with relevant compression forces and/or $f_{c,2} < f_m$. For buckling the reduction of bending stiffness EI due to plasticising may alternatively be considered by the equivalent imperfections according to clause 5.5.5.

(3) In the case of numerical simulations and if accounting for material scattering on a local level a pre-failure decrease of tension and bending stiffness due to microscopic failure mechanisms may be considered along experiments, literature or experience.

(4) Following material models may be used for modelling materially non-linear behaviour for compression parallel to the grain, as shown in Fig. 5.3, if (2) does not apply:

a. bi- or trilinear elastic-plastic material behaviour (with a nominal plateau slope for numerical stability)

b. multilinear or curved elastic-plastic material behaviour without softening

c. multilinear or curved elastic-plastic material behaviour with softening

d. elastic-plastic material based on experimental tests

If (2) applies, only b., c. or d. may be used.

NOTE 1 As a reduction of bending stiffness EI significantly influences buckling, a bilinear material behaviour would yield non-conservative results.

NOTE 2 If modelling material softening it can be necessary to use explicit dynamic analysis, damping or the arc-length method for modelling strains exceeding $\varepsilon_{el+pl,2,0}$. (see clause 6.4).

(5) Following material models may be used for modelling materially non-linear behaviour for compression perpendicular to the grain, as shown in Fig. 5.4:

a. bi- or trilinear elastic-plastic material behaviour (with a nominal plateau slope for numerical stability)

b. multilinear or curved elastic-plastic material behaviour

c. elastic-plastic material based on experimental tests

(6) The relationship of stress and plastic strain in Fig. 5.3 and Fig. 5.4 may be represented by an ellipsoid with a vertical tangent at $\varepsilon_{pl,1}$ and a horizontal (or with a gradient of 1 or $\alpha_{pl,c,90}$) tangent at $\varepsilon_{pl,2}$ and radii $(f_{c,2} - f_{c,1})$ and $\varepsilon_{el+pl,2}$ (see Fig. 5.5).

(7) The influence of stress interactions/multi-axial stress states on the elastic-plastic stress-strain curves under compression (Fig. 5.3 and Fig. 5.4) in the case of

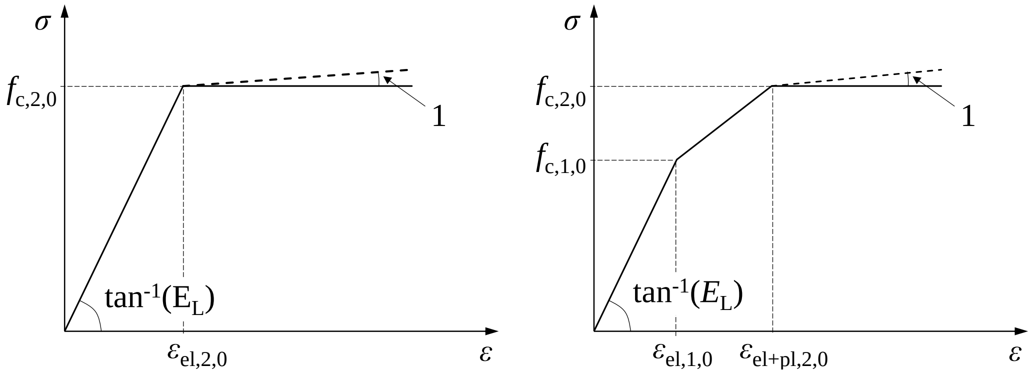
a. numerical design calculations may be neglected.

b. numerical simulations may be considered, which may be done by means of single-surface isotropic failure criteria (e.g. Hill, Hofmann, Tsai-Wu or Hashin criteria) [45] or multi-surface plasticity/yield criteria. Therefore, material parameters given in clause 5.4.2.3.2 to 5.4.2.3.4 should be adapted.

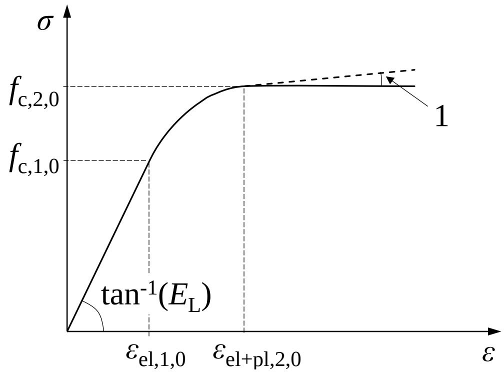
NOTE 1 Single-surface plasticity/yield criteria *

NOTE 2 Multi-surface plasticity/yield criteria are presented e.g. in [26], [27], [29].

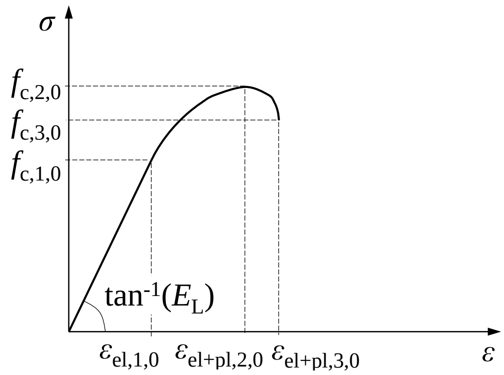
a. bi- or trilinear



b. multilinear or
curved
without softening



c. multilinear or
curved
with softening



d. experimentally
determined

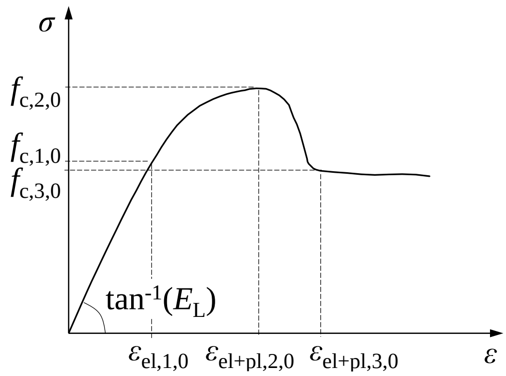


Fig. 5.3: Stress-strain curves for non-linear material modelling of timber (products) under compression parallel to the grain

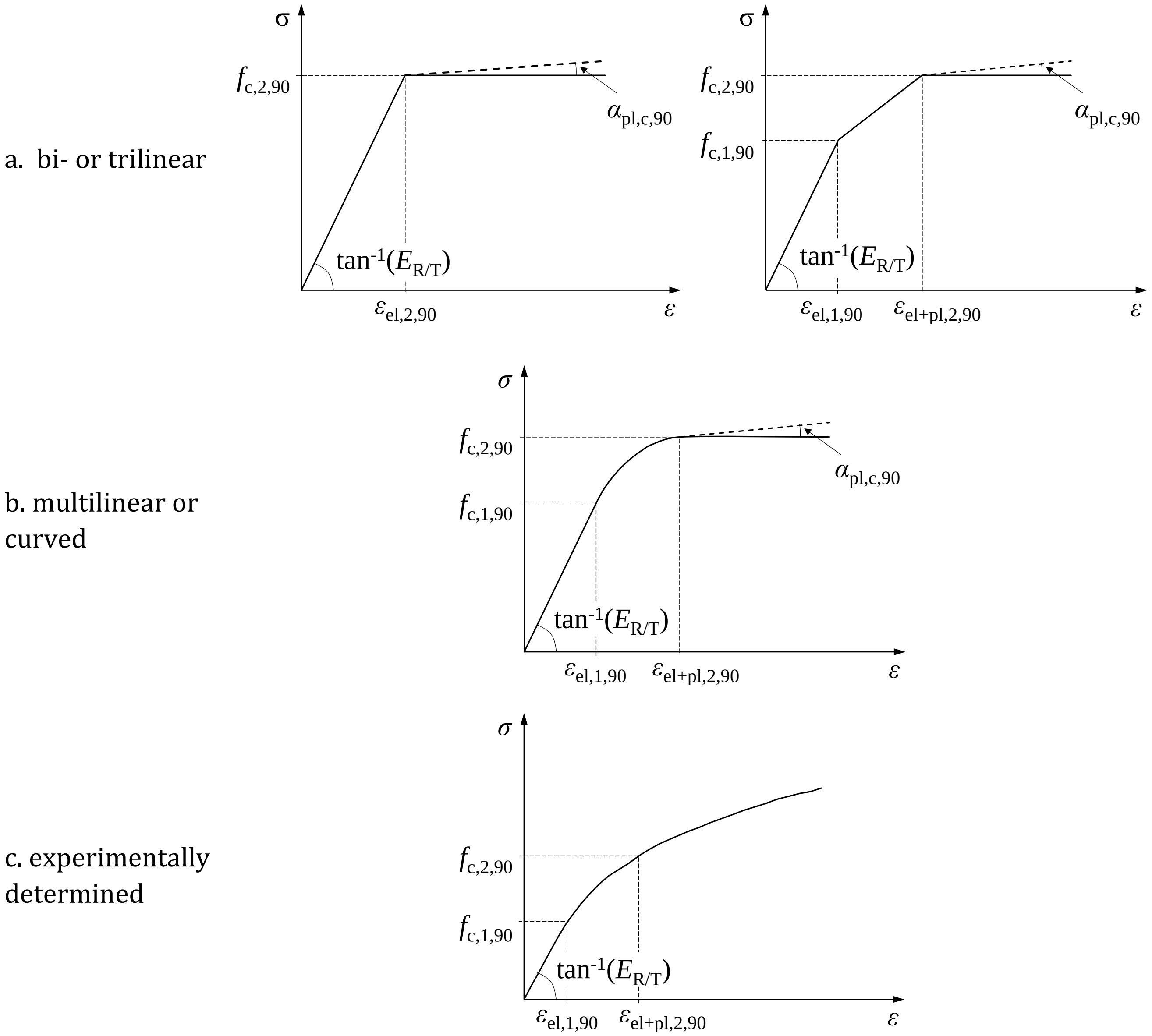


Fig. 5.4: Stress-strain curves for non -linear material modelling of timber (products) under compression perpendicular to the grain

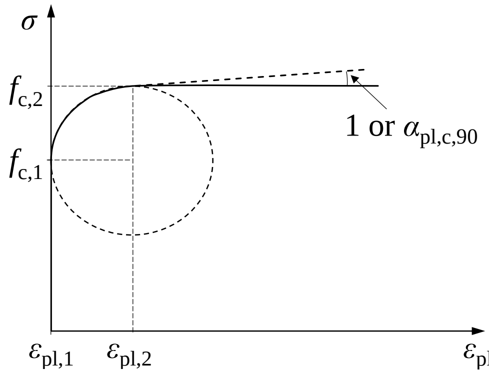


Fig. 5.5: Stress-plastic strain curve represented by an ellipsoid with vertical tangent at $\varepsilon_{pl,1}$ and horizontal (or with a gradient of 1 or $\alpha_{pl,c,90}$) tangent at $\varepsilon_{pl,2}$.

5.4.2.3.2 Input values - general

(1) Values for $f_{c,2,0} = f_{c,0,k}$, $f_{c,2,0} = f_{c,0,k}$ and $E_{L/R/T}$ in Fig. 5.3 and Fig. 5.4 should be chosen based on the applied FE based design method according to clause 4.2 (7) and (8). Other relevant material parameters should be chosen according to clause 5.4.2.3.3 and 5.4.2.3.4.

NOTE 1 For modelling of members (on global to member level) with variation of material properties of glulam and LVL lamellas along their length, the *Karlsruher Rechenmodell* is widely accepted and also used for derivation of normative material properties (e.g. described in [5]). Similar approaches are described in e.g. [20], [48], [50]. These models usually can't explicitly account for material variation over the board width. This is still an open issue. For modelling on a local level only few modelling approaches as e.g. [35] exist.

NOTE 2 $E_{L/R/T}$: *Probabilistic Model Code* [30] gives distribution functions and coefficients of variation (COV) for structural timber. More recent and specific values are published for glulam made of softwood in [23] and [47], for beech glulam in [21] and [54] and for beech LVL in [32]

NOTE 3 $f_{c,2,0}$: *Probabilistic Model Code* [30] gives distribution functions and coefficients of variation (COV) for structural timber. More recent and specific values are published for glulam made of softwood in [17], [23] and [47], for beech glulam in [17] and [54] and for beech LVL in [16] and [17].

NOTE 4 $f_{c,2,90}$: *Probabilistic Model Code* [30] gives distribution functions and coefficients of variation (COV) for structural timber. More recent and specific values are published for glulam made of softwood in [47], for beech LVL in [55] and CLT in [9].

(2) In the case of analysis used for model verification (clause 7.2), initial values of material properties should be chosen according to (1), clause 5.4.2.3.3 and 5.4.2.3.4. If a sensitivity study is conducted at least two variations with smaller and two with higher values for the relevant material properties should be conducted. Variations should be within realistic scattering of material properties. For variation - 20 %, - 5 %, + 5 %, + 20 % starting from the initial value may be used for material properties E, $f_{c,2,0}$, $f_{c,2,90}$. $\alpha_{pl,c,90}$.$f_{c,1,90}$, $f_{c,1,90}$, $\varepsilon_{pl,2}$ and $\varepsilon_{pl,2,90}$ should be varied in the range of values given in clause 5.4.2.3.3 and 5.4.2.3.4.

(3) In the case of analysis used for model validation (clause 7.3), the material properties should be chosen according to the used accurate solutions (benchmark) or mean values should be used.

(4) The influence of the size effect under compression in the case of

a. numerical design calculations may be neglected. For laminated timber decks or floors see NOTE 3.

b. numerical simulations may be considered with numerical models such as named in (1) NOTE 1 or analytical models.

NOTE 1 Equations for the analytical consideration of the size effect on $f_{c,2,0}$ for glulam made of softwood are reported in [2] and for beech LVL are reported in [14].

NOTE 2 No data of the influence of the size effect on $f_{c,1,0}$, $f_{c,1,90}$, $f_{c,3,0}$, $\varepsilon_{pl,2,0}$. $\varepsilon_{pl,2,90}$. $\varepsilon_{pl,3,0}$ and $\alpha_{pl,c,90}$ are known to the authors.

NOTE 3 For laminated timber decks or floors the values of k_{sys} for increasing $f_{c,2,0}$ according to EN 1995-1-1 can be used.

(5) The short term influence of the moisture content on E should be considered according to clause 5.4.2.2 (16).

The short term influence of the moisture content on $f_{c,2,0}$ and $f_{c,2,90}$, in the case of

a. numerical design calculations should be considered using k_{mod} according to EN 1995-1-1,

b. numerical simulations should be considered using k_{mod} according to EN 1995-1-1, experiments, literature or experience.

NOTE 1 Values for moisture content dependent reduction of $f_{c,2,0}$ for different wood species can be found in [17]. Values for moisture dependent reduction of $f_{c,2,0}$ and $f_{c,2,90}$ are reported for defect-free small specimen of sycamore maple in [49] and of beech in [29], [42] and [44]. Some values and recommendations can also be found in [6], [43].

NOTE 2 Little data of the influence of the moisture content on $f_{c,1,0}$, $f_{c,1,90}$, $f_{c,3,0}$, ε_{pl} and $\alpha_{pl,c,90}$ are known to the authors. Values and experimental data for beech LVL are reported in [18], [53].

NOTE 3 The effects of temperature-moisture interaction on E and f are reported in [6].

(6) The influence of temperature on E, f_c and ε_{pl} may be neglected (for temperatures below 60 °C for which this document is applicable). In the case of numerical simulations, the influence of temperature on E and f_c may be considered along experiments, literature or experience.

NOTE 1 Values for temperature dependent reduction of E and $f_{c,2}$ are reported in [6], [43].

NOTE 2 No data of the influence of temperature on $f_{c,1,0}$, $f_{c,1,90}$, $f_{c,3,0}$, ε_{pl} and $\alpha_{pl,c,90}$ are known to the authors.

NOTE 3 The effects of temperature-moisture interaction on E and $f_{c,2}$ are reported in [6].

5.4.2.3.3 Further input values - compression parallel to the grain

(1) The proportionality limit for compression parallel to the grain $f_{c,1,0}$ should be chosen according to Formulae (5.9).

$$f_{c,1,0} = k_{lin,c,0} f_{c,2,0} \tag{5.9}$$

with $k_{lin,c,0}$ factor describing the proportionality limit for compression parallel to the grain exceeding which plasticising occurs.
softwood: Values in literature range between 0,70 and 0,85 [26]. 0,75 may be chosen.
beech LVL: Values between 0,60 and 0,85 are reported in [16], [18], [53]. 0,65 may be chosen [53].

$f_{c,2,0}$ compressive strength parallel to the grain.

(2) The total strain when reaching the compressive strength parallel to the grain $\varepsilon_{el+pl,2,0}$ should be chosen according to Formulae (5.10).

$$\varepsilon_{el+pl,2,0} = \varepsilon_{el,2,0} + \varepsilon_{pl,2,0} \tag{5.10}$$

with $\varepsilon_{el,2,0} = \dfrac{f_{c,2,0}}{E_L}$ elastic strain when the compressive strength parallel to the grain $f_{c,2,0}$ is reached.

$\varepsilon_{pl,2,0} = k_{pl,2,0} \varepsilon_{el,2,0}$ plastic strain when the compressive strength parallel to the grain $f_{c,2,0}$ is reached.
softwood: Values in literature for $k_{pl,2,0}$ range between 0,10 and 0,35 with a mean of approx.

0,25 [22]. 0,25 may be assumed [4].
beech LVL: Values between 1,00 and 2,00 are reported in [16], [18], [53]. 1,25 may be assumed [53].

(3) If using material models from Fig. 5.3 a. and b. a minimal slope after exceeding $f_{c,0}$ and $\varepsilon_{el+pl,2,0}$ may be assumed for numerical stability.

(4) The compressive strength parallel to the grain after softening $f_{c,3,0}$ may be chosen according to Formulae (5.11).

$$f_{c,3,0} = k_{end,3,0} f_{c,2,0} \tag{5.11}$$

with $k_{end,3,0}$ factor describing the compressive strength parallel to the grain after softening.
softwood: Values in literature for $k_{end,3,0}$ range between 0,85 and 1,00 [26]. 0,85 may be assumed [26].
beech LVL: 0,85 may be assumed [51].

(5) The total strain parallel to the grain after softening $\varepsilon_{el+pl,3,0}$ may be chosen according to Formulae (5.12).

$$\varepsilon_{el+pl,3,0} = \varepsilon_{el,3,0} + \varepsilon_{pl,3,0} \tag{5.12}$$

with $\varepsilon_{el,3,0} = \frac{f_{3,0}}{E_L}$ elastic strain when the compressive strength parallel to the grain after softening $f_{c,3,0}$ is reached

$\varepsilon_{pl,3,0} = k_{pl,3,0} \varepsilon_{el,3,0}$ plastic strain when the compressive strength parallel to the grain after softening $f_{c,3,0}$ is reached
$k_{pl,3,0} = 2\ k_{pl,2,0}$ may be assumed.

5.4.2.3.4 Further input values - compression perpendicular to the grain

(1) The proportionality limit for compression perpendicular to the grain $f_{c,1,90}$ may be chosen according to Formulae (5.13).

$$f_{c,1,90} = k_{lin,c,90} f_{c,2,90} \tag{5.13}$$

with $k_{lin,c,90}$ factor describing the proportionality limit for compression perpendicular to the grain exceeding which plasticising occurs.
softwood: 0,75 may be assumed.

$f_{c,2,90}$ compressive strength perpendicular to the grain.

(2) The total strain when reaching the compressive strength perpendicular to the grain $\varepsilon_{el+pl,2,90}$ should be chosen according to Formulae (5.14).

$$\varepsilon_{el+pl,2,90} = \varepsilon_{el,2,90} + \varepsilon_{pl,2,90} \tag{5.14}$$

with $\varepsilon_{el,2,90} = \frac{f_{c,2,90}}{E_{R/T}}$ elastic strain when the compressive strength perpendicular to the grain $f_{c,2,90}$ is

	reached.
$\varepsilon_{pl,2,90} = k_{pl,2,90}\varepsilon_{el,2,90}$	plastic strain when the compressive strength perpendicular to the grain $f_{c,2,90}$ is reached. Softwood: 3,00 may be assumed.

(3) The slope of the stress-strain curve perpendicular to the grain after reaching the compressive strength $\alpha_{pl,c,90}$ may be assumed to

$$\alpha_{\text{pl,c,90}} = 5{,}0 \tag{5.15}$$

5.4.2.4 Modelling of long-term behaviour

(1) In the case of numerical design calculations creep deformations should be considered by adjusting E_L using k_{def} according to EN 1995-1-1.

NOTE 1 Creep factors k_{def} in EN 1995-1-1 were derived based on bending tests [33]. Thus, k_{def} is accurate for bending (in RL and RT plane) and compression parallel to the grain, but overestimating deformations for tension parallel to the grain and underestimating deformations for compression and tension perpendicular to the grain, shear and torsion [25].

NOTE 2 Although a large number of experimental studies on creep at different stress directions exist [24], it is difficult to define specific creep factors. According to [24] and [43] it can be assumed that creep for

- compression perpendicular to the grain is 5 to 8 times higher,
- tension parallel to the grain is 4 times lower,
- tension perpendicular to the grain is 1.25 to 2 times higher,
- shear (in LR and LT plane) is 6 times higher,
- torsion or shear in RT plane is 3 times higher,

than creep for bending and compression parallel to the grain. Due to the reported large scatter of creep deformations [24], [43], [46], the creep factors are only a rough approximation.

(2) In the case of numerical simulations creep deformations may be considered using the same values as for numerical design calculations or more sophisticated models specifically accounting for the influence of moisture content (variations), load duration, load level, etc..

NOTE 1 An overview over different rheological models is given in [46] and an implementation of such a model in commercial FE software described in [39].

NOTE 2 Values for the scattering of creep factors for controlled climate are e.g. reported in [24] and [43] and for SC2 in general in [46].

(3) In the case of numerical design calculations, the effects of the duration of load on the compressive strength (creep strength) should be considered by adjusting $f_{c,2}$ using k_{mod} according to EN 1995-1-1.

NOTE 1 For which case was k_{mod} derived?

NOTE 2 Add values for other loading situations.

(4) In the case of numerical simulations, the effects of the duration of load on the compressive strength (creep strength) should be considered using the same values as for numerical design cal-

culations or more sophisticated models specifically accounting for the influence of moisture content (variations), load duration, load level, etc..

NOTE 1 An overview over different rheological models is given in [46] and an implementation of such a model in commercial FE software described in [39].

NOTE 2 Add values for load duration and moisture content dependent creep factors.

(5) It should be ensured that creep deformations over the time approach a limit value. Thus, no secondary or tertiary creep should occur (explanations see e.g. [43]). This should be done by means of limiting the applicable stresses (clause 5.4.3).

NOTE The limiting of applicable stresses can lead to a reduction of strength for e.g. solid timber by 30 to 84 % [24]. Due to the safety factors, design stresses are usually below this limit [25].

(6) Stress changes due to relaxation may be considered in a simplified way analogous to creep deformations (clause (2) and (3)) at constant stresses at time *t = 0* according to Formulae (5.16) or with more sophisticated analytical models like e.g. mentioned in (3).

$$\sigma_{\mathrm{fin}} = \frac{\sigma_0}{1 + k_{def}} \tag{5.16}$$

with σ_0 stress at time t = 0

σ_{fin} stress at time t = 50 a due to relaxation

k_{def} factor for the evaluation of creep deformation according to EN 1995-1-1

NOTE 1 Relaxation is based on the same phenomena as creep and thus influenced by the same parameters.

NOTE 2 EN 1995-1-1 gives no hints for stress reduction due to relaxation. Values from literature. In [43] it is stated that generally a stress reduction of around 60 % occur in SC1 (Formulae (5.16) yields a reduction of 40 % for solid timber and glulam in SC1).

(6) Swelling and shrinkage may be considered according to EN 1995-1-1.

5.4.3 Modelling of material failure and strength

(1) Material strength and failure may be modelled by means of limiting stresses and strains. If relevant stress or strain concentrations / singularities occur or post failure behaviour is investigated, more sophisticated models should be applied according to (3) and (4). Thus, different failure criteria may be applied within different areas of a model depending on which stresses (tension, compression, shear in L/R/T direction) are relevant and if local stress or strain concentrations / singularities occur.

(2) If neither stress or strain concentrations / singularities, nor post failure behaviour are relevant and/or are investigated, material failure and strength should be modelled applying ultimate stresses and strains according to (5) to (6) and (9) to (13).

(3) Where relevant stress or strain concentrations / singularities occur, it should first be checked according to Annex B whether these are geometrical or numerical stress and strain concentrations / singularities and if these should be considered in design. For relevant geometrical stress and strain concentrations under

a. compression, stresses and strains should not be limited (clause (5) to (6) and (9) to (13) does not apply) and strains larger than $\varepsilon_{el+pl,2,0}$ (Fig. 5.3 and Fig. 5.4) be allowed.

b. tension perpendicular to the grain or shear in RL and RT plane may be modelled by means of fracture mechanic approaches, cohesive zones or continuum damage mechanics according to (17) to (19).

c. tension parallel to the grain or shear in RT plane may be modelled by means of fracture mechanic approaches or continuum damage mechanics according to (17) and (19).

NOTE Generally, only geometrical stress and strain concentrations are relevant for design. Numerical singularities may be neglected in the design as they result from errors of numerical approximation of the physical stresses or strains.

(4) In the case of numerical simulations post failure behaviour for

a. tension perpendicular to the grain or shear in RL and RT plane may be modelled by means of fracture mechanic approaches, cohesive zones or continuum damage mechanics according to (17) to (19).

b. tension parallel to the grain or shear in RT plane may be modelled by means of fracture mechanic approaches or continuum damage mechanics according to (17) and (19).

(5) Material strengths / abort criteria should be chosen based on the applied FE based design method according to clause 4.2 (7) and (8).

NOTE 1 For numerical design calculations, the maximum compressive strain is limited to $\varepsilon_{\mathrm{el+pl,2,0}}$ (Fig. 5.3 and Fig. 5.4) and the maximum tensile and shear strain to $\varepsilon_{\mathrm{el,2}} = f_{\mathrm{k}} / E$.

NOTE 2 For numerical simulations, compressive strains larger than $\varepsilon_{\mathrm{el+pl,2,0}}$ (Fig. 5.3 and Fig. 5.4) and tensile and shear strains larger than $\varepsilon_{\mathrm{el,2}} = f_{\mathrm{k}} / E$ can be allowed.

NOTE 3 For modelling of members (on global to member level) with variation of material properties of glulam and LVL lamellas along their length, the *Karlsruher Rechenmodell* is widely accepted and also used for derivation of normative material properties (e.g. described in [5]). Similar approaches are described in e.g. [20], [48], [50]. These models usually can't explicitly account for material variation over the board width. This is still an open issue. For modelling on a local level only few modelling approaches as e.g. [35] exist.

NOTE 4 *Probabilistic Model Code* [30] gives distribution functions and coefficients of variation (COV) for structural timber. More recent and specific values are published for glulam made of softwood in [23] and [47], for beech glulam in [21] and [54] and for beech LVL in [14], [15], [16] and [17].

(6) For pure one-axis bending, bending strengths f_{m} should be used for design verification of tension parallel to the grain instead of tensile strengths $f_{t,0}$.

NOTE Physically the bending strengths in EN 1995-1-1 are actually tensile strength accounting for the size effect due to bending.

(7) In the case of analysis used for model verification (clause 7.2), initial values of material properties should be chosen according to (1) to (5). If a sensitivity study is conducted at least two variations with smaller and two with higher values for the relevant material properties should be conducted. Variations should be within realistic scattering of material properties. For variation of

a. f_c and $f_{t,0}$ - 20 %, - 5 %, + 5 %, + 20 %,

b. f_m and $f_{v,0}$ - 30 %, - 7 %, + 7 %, + 30 %,

c. $f_{t,90}$ - 40 %, - 10 %, + 10 %, + 40 %,

starting from the initial value may be used.

(8) In the case of analysis used for model validation (clause 7.3), the material properties should be chosen according to the accurate solutions (benchmark) or mean values should be used.

(9) Strength values for stress interactions/multi-axial stress states in the case of

a. numerical design calculations should be determined according to EN 1995-1-1. Where no interaction rules are given in EN 1995-1-1, linear interaction may be assumed.

b. numerical simulations may be adjusted by the same factors as given by the interaction rules according to EN 1995-1-1. Where no interaction rules are given in EN 1995-1-1, linear interaction or interaction along experiments, literature or experience may be assumed.

NOTE Add literature

It may be assumed that coefficients of variation are not influenced by stress interactions.

(10) The influence of the size effect

a. in the case of numerical design calculations may be considered according to EN 1995-1-1, relevant product standards or technical approvals.

b. in the case of numerical simulations may be considered according a., experiments, literature or experience.

NOTE 1 In case of numerical simulations, numerical models such as in (5) NOTE 3 can be used.

NOTE 2 In case of numerical simulations, equations for the analytical consideration of the size effect on

- $f_{c,2,0}$ for glulam made of softwood are reported in [2] and for beech LVL in [14],
- $f_{c,2,90}$ no data of the influence of the size effect are known to the authors,
- $f_{m,0}$ for glulam made of softwood are reported in [19] and [23], solid timber made of softwood in [13], glulam made of hardwood *, solid timber made of hardwood * and for beech LVL in [14],
- $f_{t,0}$ for glulam made of softwood are reported in [23], solid timber made of softwood in [11], glulam made of hardwood *, solid timber made of hardwood * and for beech LVL in [14],
- $f_{t,90}$
- f_v for glulam made of softwood are reported in *, solid timber made of softwood in [7], glulam made of hardwood *, solid timber made of hardwood * and for beech LVL in [14],
- $f_{v,90}$ *

can be used.

(11) The short term influence of the moisture content on strengths in the case of

a. numerical design calculations should be considered using k_{mod} according to EN 1995-1-1,

b. numerical simulations should be considered along EN 1995-1-1, experiments, literature or experience.

NOTE 1 Values for moisture content dependent reduction of
- $f_{c,2,0}$ for different wood species can be found in [17],
- $f_{c,2,0}$ and $f_{c,2,90}$ are reported for defect-free small specimen of sycamore maple in [49] and of beech in [29], [42] and [44].
- Some values and recommendations can also be found in [6], [43].

NOTE 2 The effects of temperature-moisture interaction on f are reported in [6].

(12) The effects of load duration and the moisture content on strengths in the case of

a. numerical design calculations should be considered using k_{mod} according to EN 1995-1-1,

b. numerical simulations should be considered using the same values as for numerical design calculations or more sophisticated models specifically accounting for the influence of moisture content (variations), load duration, load level, etc.

NOTE 1 An overview over different rheological models is given in [46] and an implementation of such a model in commercial FE software described in [39].

NOTE 2 Values for load duration and moisture content dependent reduction of ??? are reported in [???] *

(13) The influence of temperature on strengths should be neglected (for temperatures below 60 °C for which this document is applicable). In the case of numerical simulations, the influence of temperature on strengths may be considered along experiments, literature or experience.

NOTE 1 Values for temperature dependent reduction of strengths are reported in [6], [43].

NOTE 2 The effects of temperature-moisture interaction on strengths are reported in [6].

(14) See clause 5.2.1 (14).

(15) Solver *

(16) Highly deformed elements *

(17) Fracture mechanic / fracture energy *

(18) Cohesive zone method / elements (CZM) may be used for modelling splitting / cracks in predefined planes. The material properties until the strength is reached should be modelled according to clause 5.4.2 and 5.4.3. A linear material softening (crack opening) after exceeding $\varepsilon_{el,2}$ may be chosen with $\varepsilon_{el+pl,3}$ according to Formulae (5.17). For numerical stability a residual strength of $f_{t/v,3} = 0.1\ N/mm^2$ may be chosen. Fig. 5.6 illustrates an exemplary material behaviour of CZM.

$$\varepsilon_{\mathrm{el+pl},3} = 5 \cdot \varepsilon_{\mathrm{el},2} = 5 \cdot \frac{f_{\mathrm{t/v},2}}{E} \tag{5.17}$$

with E modulus of elasticity according to clause 5.4.2

$f_{\mathrm{t/v},2}$ factor for the evaluation of creep deformation according to EN 1995-1-1

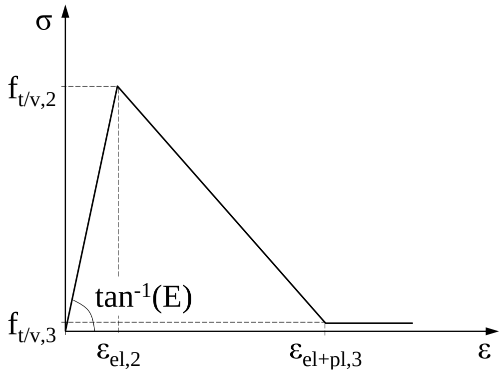


Fig. 5.6: Defining the material behaviour of a cohesive zone model (CZM)

(19) Continuum damage mechanics *

5.4.4 Additional rules for cross laminated timber (CLT)

(1) The mechanical properties of CLT should be taken from manufacturer's specifications (ETAs or National Technical Approvals). Alternatively, mechanical properties may be taken from EN 16351 or from physical experiments according to EN 408.

(2) For modelling of CLT elements, which are not edge glued and if the joints between the single boards are not modelled, the modulus of elasticity perpendicular to the grain E_{90} of the single layers should conservatively be set to zero [61]. If solid elements are used this may numerically be approximated with $E_{90} = 1\ N/mm^2$ to obtain a meaningful stiffness matrix.

(3) For modelling of CLT elements with edge glued boards the modulus of elasticity perpendicular to the grain E_{90} should be considered according to (1).

(4) For the modelling of CLT where the layers are not arranged at 90° to each other monoclinic anisotropic material models or laminate theory may be used.

5.5 Imperfections

5.5.1 General

(1) Where imperfections are included in the FE model, they should account for the effects of geometric deviations from the ideally planned geometry, material scattering (leading to deviations of the position of the shear centre and the cross-section centroid) and boundary condition defects (e.g. uneven foundation, etc.). [68]

NOTE 1 Geometric imperfections are closely linked with production and assembly tolerances. See EN 1995-3: Execution Rules. [70]

NOTE 2 Load eccentricities, flexibility of connections and supports, shear deformations or long term effects such as e.g. creep are not included in the given imperfection values. [70]

NOTE 3 Imperfections do not need to fulfil the boundary conditions of the calculation model. [70]

(2) Equivalent imperfections may also account for the reduction of the bending stiffness EI due to plasticising under compression parallel to grain (see clause 5.4.2.3 (2)).

(3) Values of imperfections should be chosen based on the applied FE based design method according to clause 4.2 (7) and (8). [68]

(4) Further rules may be specified in relevant parts of EN 1995.[68]

5.5.2 Types of imperfections

(1) In case of numerical simulations one of the following imperfection types may be applied:

a. geometric imperfections (clause 5.5.3), structural imperfections due to material scattering (clause 5.5.4) and plasticising (clause 5.4.2.3),

b. equivalent geometric imperfections (clause 5.5.5). These are covering the effects of both the geometric imperfections, structural imperfections and eventually plasticising.

(2) In case of numerical design calculations equivalent geometric imperfections (clause 5.5.5) should be applied.

(3) The shape of the geometric imperfections or equivalent geometric imperfections may be defined in the FE model in the following ways:

a. measured imperfection shape of the structural element or system (only permitted for geometric imperfections),

b. imperfection shape based on sinusoidal or parabolic functions along experience, taking into account alternative eigenmodes (permitted for both geometric and equivalent geometric imperfections),

c. imperfection shape based on linear bifurcation analysis (LBA) corresponding to the eigenmode (shape) associated with the expected failure mode or with a combination of eigenmodes (permitted for both geometric and equivalent geometric imperfections).

NOTE There can be more than one relevant eigenmode, so the lowest eigenvalue is not always the source of the most important imperfection. [68]

(4) Imperfections on three different levels may be considered:

a. system imperfections; translational or rotational deviations of the position of entire members, joints, actions, ... from the ideal planned state, which influence the load-bearing behaviour at (reduced) global level of modelling (for geometric and equivalent geometric imperfections),

b. member imperfections; material scattering and/or translational and/or rotational deviations of the geometry of single members, joints, actions, ... from the ideal planned state, which influence the load-bearing behaviour at member level (for geometric, structural and equivalent geometric imperfections),

c. local connection imperfections; translational or rotational deviations of the position or geometry of single fasteners from the ideal planned state (like hole tolerances of dowels), which influence the load-bearing behaviour at local level of modelling (for geometric imperfections),

NOTE Local connection imperfections can be used for numerical simulation of joints and connections with scattering input parameters.

(5) If geometric or equivalent geometric imperfections are used in a non-linear analysis, imperfections corresponding to each investigated buckling mode should be adopted.

(6) The most detrimental imperfection (that could realistically occur) should be chosen in calculating each potential failure mode. If the choice of this mode is not clearly evident, several imperfection shapes and combinations should be investigated.

(7) If more than one geometric or equivalent geometric imperfection form is used, combinations of these forms should additionally be considered. Rules are defined in clause 5.5.6.

(8) The direction of the chosen imperfection(s) (imperfection combinations) should be chosen to identify the lowest resistance. If the relevant imperfection direction is not evident or defined by other rules, imperfections with different directions should be investigated, where physically possible.

5.5.3 Geometric imperfections

(1) Geometric imperfections may be chosen by considering manufacturing and erection processes and the associated manufacturing and erection tolerances. The imperfection shapes may be chosen according to 5.5.2 (3).

(2) Within numerical simulations scattering geometric imperfections of single members may be chosen on the basis of a normal distribution with mean value μ = 0 and standard deviation σ according to clause (3) to (7).

NOTE The standard deviation was determined by assuming the 95 % quantile value according to EN 1995-1-1.

(3) The standard deviation σ of the sway of columns i_{sway} in radians may be taken as:

$$\sigma(i_{sway}) = \begin{cases} \dfrac{1}{400} & \text{for } h \leq 5000 \text{ mm} \\ \dfrac{1}{400}\sqrt{\dfrac{5000}{h}} & \text{for } h > 5000\ mm \end{cases} \tag{5.18}$$

with h Column height in [mm]

NOTE The column sway has been defined across all materials based on the investigations by Lindner and Giezelt [34].

(4) The standard deviation σ of the sway of beams i_{sway} at the fork supports in radians may be taken as:

$$\sigma(i_{sway}) = \begin{cases} \dfrac{1}{400} & \text{for fork bearings with high tolerances} \\ \dfrac{1}{200} & \text{for fork bearings with low tolerances} \end{cases} \tag{5.19}$$

Fork bearings with low tolerances are lateral timber members or comparable constructions. Fork bearings with high tolerances are concrete pockets or comparable constructions.

NOTE For background information see [52].

(5) The standard deviation σ of the bow of members i_{bow} may be taken as:

$$\sigma(i_{\text{bow}}) = \frac{L}{2000} \tag{5.20}$$

with L Span of the member [mm]

NOTE For background information see [4] and [52].

(6) The standard deviation σ of the twist of beams at midspan i_{twist} in radians may be taken as:

$$\sigma(i_{twist}) = \frac{L}{3000 H_{\text{mid}}} \tag{5.21}$$

with L Span of the beam [mm]

H_{mid} Beam height at midspan [mm]

NOTE For background information see [52].

(7) Values for nail plate trusses may be chosen according to Kessel et al. [31].

5.5.4 Structural imperfections

(1) Structural imperfections should consider the effects of material scattering in direction of the buckling behaviour.

(2) Structural imperfections may be modelled by directly implementing scattering material properties (see 5.4.1 (9) Note).

5.5.5 Equivalent geometric imperfections

(1) Equivalent system and member imperfections may be chosen in accordance with EN 1995-1-1.

(2) The pattern of the equivalent geometric imperfections should reflect the constructional detailing and the boundary conditions in a realistic and safe manner. [68]

(3) If linear elastic or bilinear elasto-plastic (see Fig. 5.3 a.) material behaviour is assumed for compression parallel to the grain and relevant compression forces occur and/or $f_{c,2} < f_m$, the effects of the reduction of bending stiffness *EI* due to plasticising should be considered in the equivalent imperfections.

(4) Within numerical simulations scattering equivalent geometric imperfections of single members may be chosen according to 5.5.3 (2) to (7).

(5) Equivalent geometric imperfections may be substituted by appropriate fictitious forces acting on the member according to EN 1995-1-1.

5.5.6 Imperfection combinations

(1) The final imperfect shape of the analysed structure should be obtained by superposition of the imperfections (geometric imperfections and material scattering or equivalent geometric imperfections) on the perfect structure covering all possible failure modes and geometric deviations. [68]

(2) Amongst all the explored imperfection combinations (where physically relevant), the one that gives the smallest resistance value should be used to determine the resistance of the structure. [68]

(3) Where both geometric and structural imperfections are used, all the geometric and structural imperfections should be applied to the model at the same time. If using scattering values for both this may lead to a (partial) neutralisation of the imperfection influences for some of the many models.

(4) Equivalent bow and sway imperfections of columns may only be applied in one direction. All other equivalent imperfections (also from different levels 5.5.2 (3)) should be added linearly.

(5) Further rules may be specified in relevant parts of EN 1995.[68]

6. Analysis

6.1 Structural analysis

6.1.1 General

(1) The type of analysis should be chosen based on the non-linearities to be modelled and whether imperfections have to be considered.

(2) Imperfection sensitivity should be investigated according to clause 7.2 under consideration of the limit criteria given in EN 1995.

(3) Imperfection sensitivity of structural members with kink points in tension or compression (Fig. 6.1) should be investigated.

(4) Structural non-linearities arise from the following sources:

a. geometric non-linearity,

b. material non-linearity,

c. topological/contact non-linearity. [68]

NOTE 1 Non-linearities can also be caused by elastic structural elements in an assembly, where an abrupt change in stiffness occurs, such as a slender tension member passing into compression (or vice-versa). [68]

NOTE 2 Non-linear joint behaviour is another source of structural non-linearity. [68]

(5) Geometric non-linearity ((4) a.) is caused by a change in the geometry of the structure (moderate-to-large displacements relative to the geometry, and/or strains in parts of the structure) resulting in changes in the force distribution or stiffness conditions [68]. This can be due to e.g.

a. second order effects,

b. slender tension members passing into compression (snap-through, or vice-versa).

(6) Material non-linearity ((4) b.) arises from the non-linear stress-strain relationship of the material or springs with non-linear behaviour if the structure or part of it is loaded beyond the linear elastic part / proportionality limit of the material model or spring. Material models and springs should be distinguished in the analysis as linear and non-linear. [68]

(7) Topological non-linearity ((4) c.) arises from a change in the contact status during the analysis. [68]

(8) It should be distinguished between sensitivity to global and local imperfection according to Table 6.2.

Fig. 6.1: Examples for kink points k in tension and compression members of trusses [70]

6.1.2 Type of analysis

(1) The type of analysis should be chosen based on the problem to be solved and the non-linearities to be modelled according to Table 6.1. Additionally, the type of analysis should be chosen depending on the imperfection sensitivity according to Table 6.2. If clause 5.4.2.3 (2) applies for buckling phenomena, material non-linearity may be modelled by means of equivalent imperfections (see clause 5.5.5 (3)).

(2) Several types of analysis may be combined within one model for covering different phenomena and on different levels of modelling.

(3) Linear elastic analysis (LA)

An analysis of the perfect structure using the assumptions of small displacements, small strains and a linear elastic material law. The linearity of the theory results from the assumptions of a linearization of all the physical (linear elastic stress-strain relationship), geometrical (small strains) and equilibrium equations (small displacements). In LA analysis geometrical and material non-linearities and imperfections are neglected. [68]

Linear elastic analysis (LA) may be used if none of the non-linearities in clause 6.1.1 (4) is required for the FE model (see clause 6.1.1 (2)) or in the case of numerical design requiring a subsequent design check where the non-linearities are covered by the design checks. [68]

(4) Linear bifurcation (eigenvalue) analysis (LBA)

A linear bifurcation analysis (LBA) predicts the eigenvalues and eigenmodes of the structure at which the structure may buckle into different deformed shapes, assuming no change of geometry before bifurcation, and linear elastic material model. Imperfections of all kinds are ignored. The analysis provides the elastic critical bifurcation stress of the structure, defined by σ_{cr}. [68]

NOTE It is recommended to use the initial stiffness if modelling non-linear joints in a LBA analysis. [68]

(5) Materially non-linear analysis (MNA)

An analysis of the perfect structure using the assumptions of small displacements, small strains

Table 6.1: Type of analysis [68]

Type of analysis	deformations	material law	geometry
Linear elastic analysis (LA)	linear	linear elastic	perfect
Linear bifurcation (eigenvalue) analysis (LBA)	bifurcation	linear elastic	perfect
Materially non-linear analysis (MNA)	linear	elastic-plastic	perfect
Geometrically non-linear analysis (GNA)	non-linear	linear elastic	perfect
Geometrically and materially non-linear analysis (GMNA)	non-linear	elastic-plastic	perfect
Geometrically non-linear elastic analysis with imperfections (GNIA)	non-linear	linear elastic	imperfect
Geometrically and materially non-linear analysis with imperfections (GMNIA)	non-linear	elastic-plastic	imperfect

Table 6.2: Type of analysis depending on the sensitivity to global or local imperfections

Structure is sensitive to	**Analysis of structural behaviour on**	
	(reduced) global level	local level
global & local imperfections	GNIA, GNMIA	GNIA, GNMIA, LA+EN 1995-1-1[1]
global imperfections	GNIA, GNMIA	LA, GNA, MNA, GNMA
local imperfections	LA, GNA, MNA, GNMA	GNIA, GNMIA, LA+EN 1995-1-1[1]
no imperfections	LA, GNA, MNA, GNMA	

[1] *Using LA and the effective length methods or second order theory calculations described in EN 1995-1-1 (effective lengths and critical stresses may be calculated using LBA, see clause 8.1.3.2), if the relevant criteria are met.*

and an elastic-plastic material law. In MNA analysis geometrical non-linearities and imperfections are neglected. [68]

MNA may be used where non-linearities of type clause 6.1.1 (4) b. occur.

(6) Geometrically non-linear analysis (GNA)

An analysis of the perfect structure using a linear elastic material law and including geometric non-linearity. In GNA analysis material non-linearities and imperfections are neglected. [68]

GNA or GNIA should be used where non-linearities of type clause 6.1.1 (4) a. or c. have a relevant influence on the SQRs and lead to a reduction of load-bearing capacity. Criteria when geometrically non-linear behaviour should be considered are given in EN 1995-1-1.

(7) Geometrically and materially non-linear analysis (GMNA)

An analysis of the perfect structure using an elastic-plastic material law and including geometric non-linearity. In GMNA analysis imperfections are neglected. [68]

GNMA may be used where non-linearities of type clause 6.1.1 (4) a., b. and c. have a relevant influence on the SQRs and lead to a reduction of load-bearing capacity.

(8) Geometrically non-linear elastic analysis with imperfections (GNIA)

An analysis of the imperfect structure using a linear elastic material law and including the geometric non-linearity. In GNIA analysis material non-linearities are neglected. [68]

GNA or GNIA should be used where non-linearities of type (4) a. or c. have a relevant influence on the SQRs and lead to a reduction of load-bearing capacity. Criteria when geometrically non-linear behaviour should be considered are given in EN 1995-1-1.

(9) Geometrically and materially non-linear analysis with imperfections (GMNIA)

An analysis of the imperfect structure using an elastic-plastic material law and including geometric non-linearity. In GMNIA analysis all possible effects are considered. [68]

GNMIA may be used where non-linearities of type clause 6.1.1 (4) a., b. and c. have a relevant influence on the SQRs and lead to a reduction of load-bearing capacity.

(10) The expected results and the graphical explanation of the different type of analysis are presented in Figure 6.1. [68]

NOTE LBA does not always provide the highest load factor amongst these different analyses, as suggested by Figure 6.1, since the post-buckling behaviour of the structure may lead to higher loads before

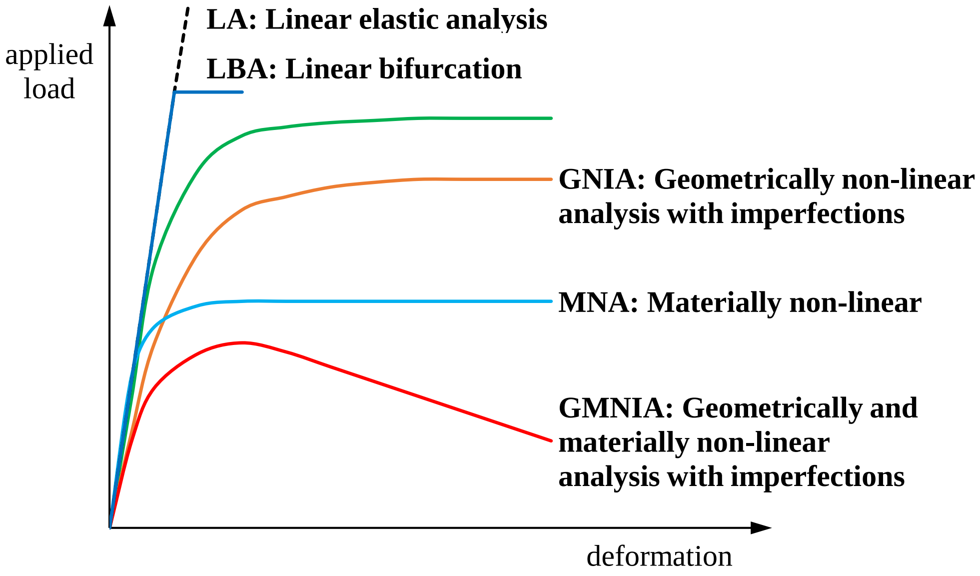


Fig. 6.2: Graphical explanation of different type of analysis [68]

a fully non-linear failure criterion is reached (stable secondary loading path). [68]

(11) If linear analysis (LA) is used, the principle of the superposition is applicable and the solution is independent of the loading history. [68]

(12) If non-linear analysis is used, a separate analysis of each load case or load combination should be performed; superposition cannot be applied. [68]

6.2 Thermal analysis *

This clause still has to be edited.

6.3 Additional rules for cross laminated timber (CLT)

(1) If CLT elements are not modelled using solid elements, stress and strain distributions over the cross-section height (see Fig. 6.3 and Fig. 6.4) may be obtained analytically according to Annex C, if the application limits of the analytical formulas are met.

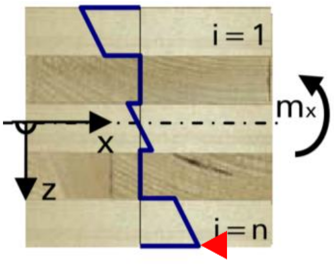


Fig. 6.3: Bending stress distribution over the cross-section's height and maximum bending stress (red arrow) of a uniformly loaded CLT plate [56]

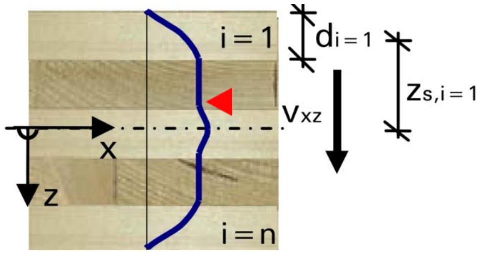


Fig. 6.4: Shear stress distribution over the cross-sections' height, maximum shear stress (green) and maximum rolling shear stress (red) of a uniformly loaded CLT plate [56]

(2) For an analytical calculation of stresses and strains there are currently the following three methods available:

a. Linear elastic stress determination
b. Gamma method
c. Shear analogy method

For choice and application limits see Annex C (4) and (5).

6.4 Solver *

(1) The convergence of the solution procedure should be checked according to clause 7.2. [70]

This clause still has to be edited.

7. Validation and verification

7.1 General

(1) Verification and validation should prove that the model is appropriate. This should include checking of SQRs and failure mechanisms. [68]

(2) Verification and validation may be executed according to clauses 7.2 and 7.3. Restrictions and exceptions are given in 7.1(6) and (7). [68]

(3) Verification demonstrates that the numerical model and analysis is properly implemented, understood and applied (clause 7.2 (2) to (6)). Additionally, that the used numerical solution is a good approximation of exact mathematical solutions / mechanical models or benchmarks (clause 7.2 (2)). [68]

(4) Validation is the comparison of numerical results to known accurate solutions (benchmarks) to demonstrate that the model correctly or conservatively captures the physical phenomena to be modelled. The numerical model should be validated for each phenomenon to be analysed. [68]

NOTE Benchmarks can consist of accurate analytical, numerical or experimental results (see e.g. Annex E).

(5) Verification and validation processes may be partially or fully overlapping. At first, the accuracy of the numerical model, the discretization of the mathematical model and the chosen type of analysis should be demonstrated by verification. Validation should be the second step comparing the physical behaviour and the results of the chosen numerical model or modelling technique. [68]

(6) If the numerical model is used for numerical design calculation for standard design cases (check of failure modes with existing Eurocode-based design resistance model) or analysis requiring a subsequent design check, the verification and validation process may be made on the basis of previous experience on similar models (see clause 7.2 (7) and 7.3 (2)). [68]

(7) If application of Finite Element based design methods brings significant resistance increases or decreases of SQRs compared to well-established traditional design methods, (6), clause 7.2 (7) and 7.3 (2) should not be applied. [68]

(8) A graphical interpretation of a validation and verification process is presented in Fig. 7.1. [68]

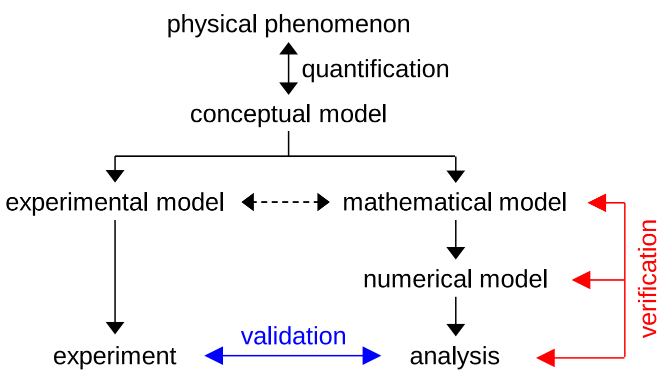


Fig. 7.1: Interpretation of a validation and verification process [68]

7.2 Verification

(1) The verification process should include the following checks:

a. engineering judgement of the calculation results,

b. discretization check (mesh study),

c. solver convergence study (including check of solver settings),

d. sensitivity check of input parameters,

e. imperfection sensitivity analysis (if relevant). [68]

(2) Engineering judgement should be applied to the results of a calculation. The relevant SQRs (deformations, load-displacement paths, internal force diagrams, etc.) should be checked. Such checks may use exact mathematical solution / mechanical models, benchmarks or previous experience. [68]

(3) Within the discretization check, a mesh study should be used to show that the chosen element type and size are accurate for the analysed problem and the calculation results are not significantly influenced by the discretization. A mesh convergence study should be executed to check if the relevant SRQs converge if the mesh is refined. The chosen mesh size should satisfy the 5 % test. If computation times are low, a 1 % test may be applied. (see Fig. 7.2) [68]

NOTE A relatively secure prediction of the correct value of a SRQ can be obtained by plotting its numerically computed value (y-axis) against the inverse of the total number of DOFs in the mesh (x-axis). Extrapolation of the resulting curve to x = 0 provides a clear indication of the best estimate of the correct solution and permits the 5% test (5% difference with the converged value of the SRQ). An example is given in Fig. 7.2. [68]

(4) In a solver convergence study, it should be ensured that the numerical results have converged

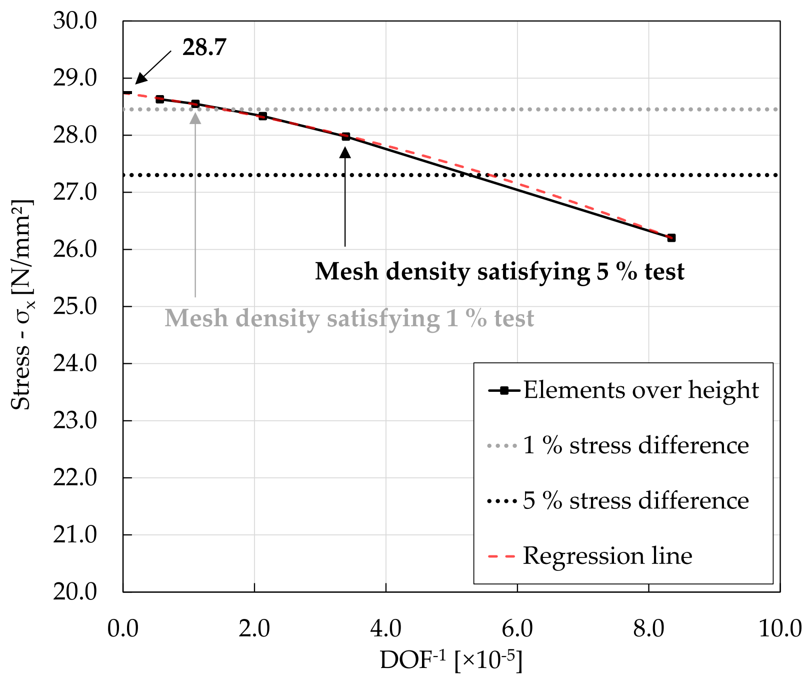


Fig. 7.2: Exemplary 5 % and 1 % test for the maximum bending stresses σ_x of a timber beam plotted over the inverse of the degrees of freedom DOF^{-1}

when the computation of a load or displacement step is finished. This is closely related to the chosen solver settings (clause 6.4) and may be done by means of software-integrated criteria.

(5) The sensitivity check is a variation of the relevant input parameters that determines which parameters are crucial to the relevant SRQ and whether these parameters should be defined with higher precision or not [68]. Geometrical and material properties and the size of time/load steps should be checked. Values for the variation of material parameters are given in clause 5.4.2.2 (10), 5.4.2.3.2 (2) and 5.4.3 (7). Other variations should be chosen along experience.

(6) An imperfection sensitivity analysis is a check whether the results of the numerical solution (SRQs) are sensitive to imperfections in general and the chosen imperfection type, shape and magnitude. Global and local imperfection sensitivity should be investigated if relevant. [68]

(7) If the numerical model is used for numerical design calculation for standard design cases (check of failure modes with existing Eurocode-based design resistance model) or analysis requiring a subsequent design check, the verification steps described in (1) b. to e. may be omitted on the basis of experience on similar models.

(8) Geometrical and material properties should be chosen according to clause 5.

7.3 Validation

(1) A benchmark case should be adopted as a reference in the validation process to check the numerical model and its application (type of analysis, solver settings, limit state criteria, failure mechanism, etc.) in the particular application field. If a similar benchmark case (1st level validation) is not available, one case closest to the analysed problem should be used (2nd level validation). Within a comparison, a numerical model having identical parameters as the benchmark case should be developed and recalculated. The difference between the numerical calculation results and the benchmark should be evaluated according to (2) to (6). [68]

NOTE In the validation no upper limit is set for the model uncertainty, but a factor for considering the model uncertainty in design according to clause 8 is determined.

(2) In case of numerical design calculations requiring a subsequent design check, the model uncertainty of the structural resistance is covered by the partial factor according to EN 1990 and all parts of EN 1995 and the validation may be eliminated.

(3) In the case of numerical design calculations with direct resistance check, the numerical model should be validated by using the partial factor for modelling γ_{FE}. [68]

NOTE 1 A minimum value of the partial factor for modelling of $\gamma_{\mathrm{FE,min}} = 1{,}1$ is recommended. The calculation method of γ_{FE} given in Annex A is recommended.

NOTE 2 If a numerical model is used in numerical design calculation with direct resistance check, the validation can be eliminated on the basis of experience of previous satisfactory performance in similar cases and can be γ_{FE} chosen based on experience, unless the National Annex gives different application rules. [68]

(4) The partial factor for modelling γ_{FE} covers the uncertainties of the numerical model and the executed type of analysis and does not override the application of any other partial factors given in EN 1990 and all parts of EN 1995. The application of the model factor is given in Fig. 7.3. [68]

(5) In the case of numerical simulations the model should be validated

a. according to (3), or

b. a statistical evaluation according to EN 1990 may be performed to determine the test-based resistance (i.e. actual partial factor). The uncertainty of design is treated in the same way as physical experiments according to EN 1990. [68]

(6) If the numerical simulation is performed using imperfections, an imperfection sensitivity study should be carried out within the validation process using different imperfection amplitudes (including imperfections in opposite directions, where physically possible) and imperfection combinations, if relevant. [68]

(7) Geometrical and material properties should be chosen based on the used benchmark.

(8) The failure mode of the numerical model and the benchmark should be consistent.

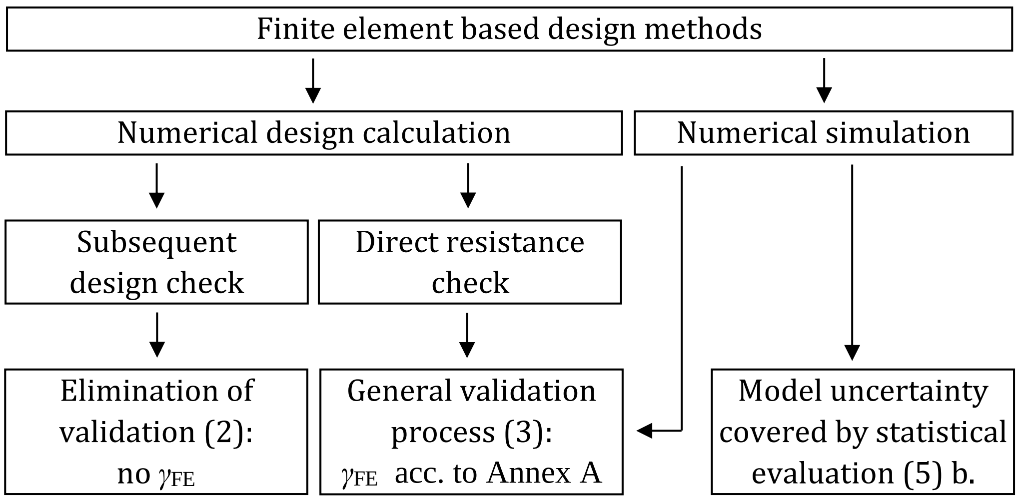


Fig. 7.3: Application of the partial factor for modelling γ_{FE} in validation.

8. Design methodology

8.1 Ultimate limit state

8.1.1 General

(1) The ultimate limit state check (excluding fatigue) using a FE model should be performed in one of the following ways:

a. elastic and plastic resistance check, where stability problems are not relevant (clause 8.1.2),

b. buckling resistance check (clause 8.1.3). [68]

(2) This standard gives design rules to predict the characteristic values of resistances calculated by numerical analysis. [68]

(3) Additional design criteria may be also specified in the relevant standard parts. [68]

(4) In the case of numerical design calculations requiring a subsequent design check, the numerically determined SQRs may be used for design verification according to relevant parts of EN 1995.

NOTE Uncertainties of the numerical model and the executed type of analysis are covered by the partial factors according to EN 1990 and all parts of EN 1995.

(5) In the case of numerical design calculations with direct resistance check, the characteristic strength f_k should be determined according to clause 8.1.2 to 8.1.4 and Formulae (8.1).

$$f_{\mathrm{k}} = \frac{f_{\mathrm{FE}}}{\gamma_{\mathrm{FE}}} \tag{8.1}$$

with f_{FE} numerically calculated strength based on nominal values according to 4.2 (7) considering the limit state criteria given in clause 8.1.2 to 8.1.4

γ_{FE} partial factor for modelling according to clause 7.3 and Annex A

(6) In the case of numerical simulations, the characteristic strength f_k may be determined based on a statistical evaluation according to EN 1990 under consideration of the modelling uncertainty according to clause 7.3 (5).

(7) The design strength f_d may be determined on the basis of the characteristic strength f_k divided by partial factors for the relevant failure modes according to EN 1995 (all parts). Depending on the input parameters of the numerical model the modification factors k_i and k_{mod} according to EN 1995 (all parts) should be considered. Additional design rules are given in the relevant standard parts.

(8) See clause 4.1 (2).

8.1.2 Elastic and plastic resistance check without buckling

(1) In the case of LA the principle of superposition is applicable and the solution is independent of the loading history. [68]

(2) In the case of GNA, MNA or GNMA the structure should be subjected to design values of the applied load combinations multiplied by the load amplification factor (starting from zero) until the limit state is reached. A separate analysis of each load case or load combination should be performed; superposition cannot be applied. [68]

(3) The numerical model can lead to stress and strain concentrations at locations of junctions, joints, changes of cross-section, load introduction places and in the region of supports or at locations where the FE mesh has changes in its regularity. Stress concentrations can have different origins which are classified as:

a. geometrical (physical) stress and strain concentrations,

b. numerical stress and strain concentrations (singularities). [68]

NOTE 1 Annex B gives rules for separation of geometric and numerical stress and strain concentrations. [68]

NOTE 2 Stress concentrations typically occur with elastic material behaviour.

(4) Numerical stress and strain concentrations (singularities) may be neglected in the design as they result from errors of numerical approximation of the physical stresses or strains. [68]

(5) Geometrical stress and strain concentrations should be considered or neglected in the design according to Annex B depending on the chosen FE based design method and limit state criteria. [68]

(6) The resistance of the structure should be determined in the case of LA by limiting the material stresses and strains according to clause 5.4.3 and in the case of GNA, MNA or GMNA according to clause 8.1.4. [68]

(7) Additional design criteria may be also specified in the relevant standard parts. [68]

8.1.3 Buckling resistance check

8.1.3.1 Design methods

(1) For the buckling resistance check the following alternative methods may be used (see also clause 6.1.1 (2)):

a. design using critical stresses from LBA analysis (clause 8.1.3.2) and EN 1995-1-1 (effective length method or second order theory calculations),

b. design by GNIA or GNMIA analysis (clause 8.1.3.3),

(2) Other design methods may be used in accordance with the relevant parts of EN 1995. [68]

(3) Special boundary conditions may be chosen in a compatible way with the buckling check. [68]

8.1.3.2 Design using critical stresses from LBA analysis

(1) Linear elastic bifurcation analysis (LBA) may be used to determine the critical stresses σ_{cr} of structures or members, that are related to the particular load case combination and loading situation. The critical stresses σ_{cr} may serve as input values for design verification according to EN 1995-1-1.

(2) The lowest eigenvalue which corresponds to the investigated failure mode should be considered in the analysis. [68]

8.1.3.3 Design by GNIA or GNMIA analysis

(1) In the case of numerical design calculations requiring a subsequent design check, GNIA may be used to determine geometrically non-linear internal forces, stresses, deformations and the relative slenderness ratio of the analysed structure related to the relevant load case combination. [68]

(2) In the case of numerical design calculations with direct resistance check and numerical simulations GNIA or GNMIA may be used to determine the behaviour of the structure represented by a load-displacement path related to the chosen boundary conditions and analysed load case combination. [68]

(3) Imperfections should be considered according to clause 5.5.

(4) All the relevant load case combinations causing compressive stresses in structural elements prone to buckling should be accounted for checking the buckling resistance. [68]

(5) For structural members with kink points (see clause 6.1.1 (3)) all the relevant load case combinations should be accounted for checking the buckling resistance.

(6) The resistance of the structure should be determined according to clause 8.1.4. [68]

8.1.4 Evaluation method of a non-linear analysis

(1) To evaluate the ultimate resistance of a structure, the design values of the applied load case combinations should be multiplied by an increasing load amplification factor (starting from zero) to determine the relevant load-deformation curve representing the structural behaviour of the analysed structure (see Fig. 8.1). A separate analysis of each load case or load combination should be performed. Superposition cannot be applied. [68]

NOTE The calculated resistance, derived from the maximum load amplification when the criterion of failure of that analysis is reached, depends on the specific assumptions of the analysis. [68]

(2) The structural resistance f_{FE} should be determined by the evaluation of the calculated load-deformation path by taking the lowest resistance obtained from the following three criteria C1, C2 and C3 (see Fig. 8.1).

a. Criterion C1: the ultimate stress according to clause 5.4.3,

b. Criterion C2: the maximum load level of the computed load-deformation path,

c. Criterion C3: the largest tolerable deformation (or strain). Limit strains are given in clause 5.4.3. Limit deformations are given in EN 1995-1-1. [68]

(3) The structural resistance f_{FE} should be sufficient to achieve the required reliability. The reliability should be evaluated in accordance with the principles set out in EN 1990. [68]

(4) In the case of a numerical design calculations with direct resistance check the computed structural resistance f_{FE} should be adjusted by the partial factor for modelling γ_{FE} according to clauses 7.3 and 8.1.1 for covering the uncertainties of the numerical model and the type of analysis. [68]

(5) In the case of a numerical simulations the computed structural resistance f_{FE} should be adjusted according to clauses 8.1.1 (6) for covering the uncertainties of the numerical model and the type of analysis.

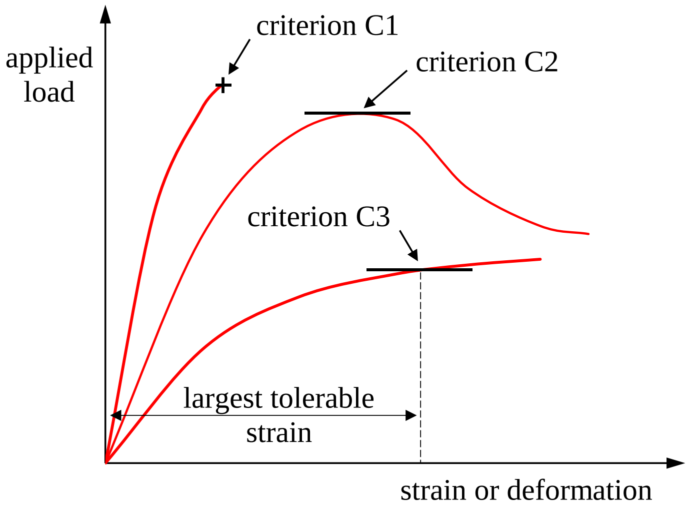


Fig. 8.1: Determination of structural resistance by material non-linear analysis

8.2 Fire Design *

This clause still has to be edited.

8.3 Serviceability limit state

8.3.1 General

(1) The numerical model may be used to check all the relevant serviceability criteria given in the EN 1990 and in all parts of EN 1995 with the provision that the design rules of the EN 1990 clauses 3.4 and 6.5 are considered in the numerical model. [68]

(2) Where FE is used in support of the serviceability limit state check, the geometrical properties of the model should be taken as nominal values for predicting the relevant stresses, deformations and vibrations (or eigen-frequencies) of the investigated structure. [68]

(3) The same type of analysis (given in clause 6.1.2) may be used to check serviceability limit state criteria than used to ultimate limit state check considering additional design rules may be given in all parts of EN 1995. [68]

(4) Deformations should be calculated assuming linear elastic material behaviour using LA, GNA or GNIA analysis considering the design rules of the relevant parts of EN 1995. [68]

8.3.2 Vibrations *

This clause still has to be edited.

8.4 Seismic design *

This clause still has to be edited.

9. Documentation

(1) The documentation of all FE analyses should include all relevant details of the modelled geometry, assumptions, chosen analyses and modelling steps. It should contain all the input data as well as the output documented in such a way that the calculations should be reproducible by third parties. [68]

(2) The documentation of the FE model, analysis and design may contain the following data:

a. name and version of the chosen FE program,

b. geometrical model (FE model geometry, element type, FE mesh, eccentricities, etc.),

c. material model (linear or non-linear, properties and characteristics),

d. support and load model (boundary conditions, prescribed displacements, loads with their combinations),

e. imperfections (geometrical imperfections, material scattering, if relevant),

f. type of analysis and convergence criteria (if relevant),

g. failure criteria,

h. results of the model validation and verification (if relevant),

i. results of the analysis (internal forces, stress distributions, displacements, deformed shapes, limit loads, bifurcation points, eigenvalues, buckling modes, where relevant),

j. limit state criteria to be checked (bases of the static check). [68]

Annex A

[informative]

Calculation of partial factor for modelling γ_{FE}

A.1 General

(1) This Informative Annex provides complementary guidance to clause 7.3 (2), (3), (4) and (6) and 8.1.1 (5) for defining the value of the partial factor for modelling. [68]

A.2 Scope and field of application

(1) This Informative Annex applies to numerical models used for numerical design calculations with direct resistance check and numerical simulations.

A.3 Calculation of partial factor for modelling γ_{FE}

(1) The partial factor for modelling may be determined based on the comparison of numerical calculations (f_{check}) to benchmarks such as test results ($f_{test,known}$) or known strengths obtained by well accepted calculation methods ($f_{k,known}$). [68]

$f_{k,known}$	is the calculated or known characteristic strength (benchmark),
$f_{test,known}$	is the known test result,
f_{check}	is the computed numerical strength for the checked structural resistance case. [68]

(2) All relevant input parameters of the numerical model (e.g. geometry and material properties, failure behaviour, type of analysis, …) should be chosen according to the used benchmark.

(3) The failure mode of the numerical model and the benchmark should be consistent.

(4) The partial factor for modelling may be determined according to (5) to (8).

(5) If numerous test results or characteristic strengths are available ($f_{test,known}$ or $f_{k,known}$) as benchmarks and numerical calculations are performed for each known case (f_{check}), the partial factor for modelling may be calculated based on statistical evaluation of the validation/application domain according to the rules given in EN 1990, Annex D. The ratio of the test results and the numerical simulation ($f_{k,known}/f_{check}$ or $f_{test,known}/f_{check}$) should be calculated at first for each sample (n) and the mean value (m_X) and the coefficient of variations (V_X) may be determined for the analysed validation/application domain. Based on the statistical evaluation the partial factor for modelling may be determined from Formula (A.1). [68]

$$\gamma_{FE} = \frac{1}{m_x(1 - k_n V_x)} \tag{A.1}$$

with

m_x	mean value of the ratio of the measured (or known) and the computed results ($f_{k,known}/f_{check}$ or $f_{test,known}/f_{check}$) for n samples,
k_n	characteristic fractile factor according to EN 1990, Annex D,

Table D.1 (data row corresponding to V_X unknown should be used),

V_x coefficient of variation of the ratio of the measured (or known) and the computed results ($R_{k,known}/R_{check}$ or $R_{test,known}/R_{check}$) for n samples.

(6) If only one known characteristic strength $f_{k,known}$ of one benchmark is available, the partial factor for modelling may be determined from Formula (A.2).

$$\gamma_{FE} = \frac{1,1}{m_x} \tag{A.2}$$

with m_x ratio of the known and the computed results ($R_{k,known}/R_{check}$).

(7) If a numerical model is used in numerical design calculation with direct resistance check, γ_{FE} may be chosen based on experience analogous to similar models with satisfactory performance.

NOTE The partial factor for modelling is related to the numerical model (each model can have different partial factor for modelling). If previously validated numerical models are used for problems with similar or slightly changed geometrical, loading or supporting conditions and there is no significant change in the analysed failure mode, the previously determined partial factor for modelling can be applied. [68]

(8) A minimum value of the partial factor for modelling of $\gamma_{FE,min}$ = 1,1 should be assumed.

(9) If a direct resistance check is performed to check failure modes where no relevant benchmarks exist and identification of similar structural form, loading and boundary conditions is difficult, the designer should use engineering judgement and seek expert advice to establish a suitable value. [68]

Annex B

[informative]

Stress concentrations

B.1 Use of this Annex

(1) This Informative Annex provides complementary guidance to clause 5.2.1 (6), 5.4.3 (1) to (3) and 8.1.2 (3) to (5) for defining the separation method of stress concentration and numerical singularities and considering the stress concentration in design. [68]

B.2 Scope and field of application

(1) This Informative Annex applies to plate, shell and solid Finite Element models where stress concentration occurs. [68]

B.3 Separation of stress concentration and numerical singularities

(1) One possible approach to separate the geometrical (physical) stress concentration and the numerical singularities is based on the determination of the mesh independent stresses or strains. The mesh independent stresses or strains are calculated values at integration points of the elements which are not affected by further FE mesh refinement, as shown in Fig. B.1. By mesh refinement an increased part of the geometrical stress concentration can be approached and the zone of the numerical singularities can be reduced. The mesh independent stresses can only be defined in the zones where calculation results using different FE mesh sizes are existing and the calculated values are identical (differences are smaller than 1% for all applied mesh size). If sharp edges are used in the numerical model the numerical singularities cannot be avoided. [68]

(2) Another optional way is to implement a rounding at the location of the sharp edges/corners. The size of the rounding radius has impact on the geometrical stress concentration. Special attention should be given to its value, where engineering judgement or real values may be used. [68]

(3) Special attention should be given to the separation of the geometrical (physical) stress concentration and the numerical singularities. The maximum computed stresses are sensitive to the

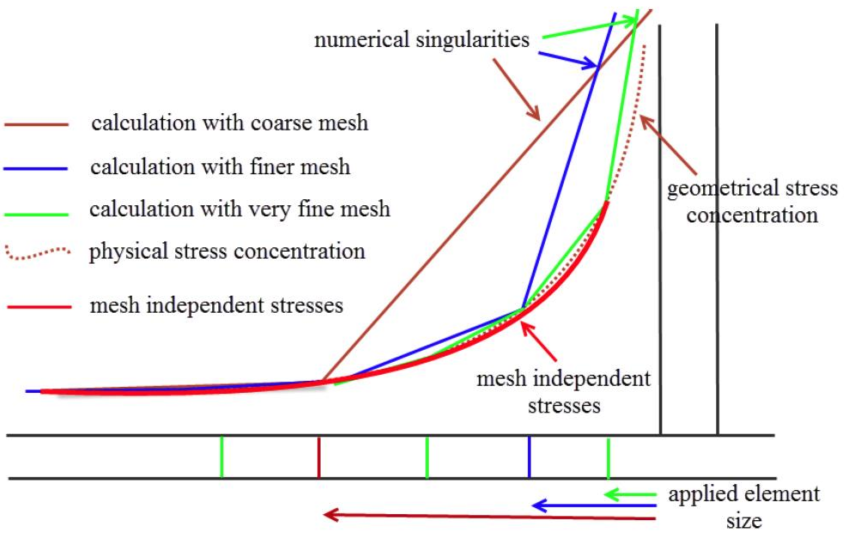


Fig. B.1: Determination of the mesh independent stresses [68] applied Finite Element type, element settings, shape and size of the mesh. Accuracy of the model should be checked by model verification. [68]

B.4 Consideration of stress concentration in design

(1) The need to consider stress concentration depends on the limit state criteria to be checked. [68]

(2) In the case of numerical design calculations requiring a subsequent design check, stress and strain concentrations may be neglected in the determination of stresses or internal forces used for further evaluation according to EN 1995. [68]

(3) In the case of numerical design calculations with direct resistance check and numerical simulations, stress concentration should be considered if brittle failure modes (tension and shear) or fatigue are checked. The effect of stress concentration is implicitly covered by the numerical model and the applied failure criteria according to 5.4.3 and 8.1. [68]

Annex C

(informative)

CLT elements

(1) Effective stiffness values (for non-edge glued boards and 1 m wide plate stripes)[56], [38]:

$$B_{\mathrm{x}} = E \cdot I_{\mathrm{n,x}} = \mathrm{k}_0 \cdot \left(\sum \frac{{d_{\mathrm{x,i}}}^3}{12} + \sum d_{\mathrm{x,i}} \cdot z_{\mathrm{s,i}}^2 \right) \tag{D.1}$$

$$\frac{1}{S_{\mathrm{xz}}} = \frac{1}{a^2} \left(\frac{d_1}{2 \cdot G_{\mathrm{xz,1}}} + \sum_{\mathrm{i}=2}^{\mathrm{n}-1} \frac{d_{\mathrm{i}}}{G_{\mathrm{xz,i}}} + \frac{d_{\mathrm{n}}}{2 \cdot G_{\mathrm{xz,n}}} \right) \tag{D.2}$$

$$S_{\mathrm{xz}} = \left(\frac{1}{S_{\mathrm{xz}}} \right)^{-1} \tag{D.3}$$

with:

E_0 Young's Modulus, in fibre direction, in N/mm²

$d_{\mathrm{x,i}}$ Depth of one single layer in x-direction (span direction), in mm

$z_{\mathrm{s,i}}$ Distance of single layers in x-direction from the centroidal axis, in mm

a Distance of the two outermost parallel layers (concerning their centroidal axes), in mm

G_{xz} Shear modulus, for parallel layers with $G_{\mathrm{xz}} = G_0$, for cross layers with $G_{\mathrm{xz}} = G_{90}$, in N/mm²

d_1 Depth of the first / upper outermost layer, in mm

d_{n} Depth of the last / under outermost layer, in mm

d_{i} Depth of layers between the outermost layers, in mm

NOTE Derivation for stiffnesses in y-direction for biaxial systems analogous).

(2) Equivalent Young's- and shear modulus (for software where no input of effective stiffness values is possible and plates should be modelled with their actual thickness):

$$E_{0,\mathrm{equ}} = \frac{B_{\mathrm{x}}}{I_{\mathrm{brutto}}} \tag{C.4}$$

$$E_{90,\mathrm{equ}} = \frac{B_{\mathrm{y}}}{I_{\mathrm{brutto}}} \tag{C.5}$$

$$G_{0,\mathrm{equ}} = \frac{S_{\mathrm{xz}}}{A_{\mathrm{brutto}}} \tag{C.6}$$

$$G_{90,\mathrm{equ}} = \frac{S_{\mathrm{yz}}}{A_{\mathrm{brutto}}} \tag{C.7}$$

(3) For an analytical calculation of stresses and strains there are currently the following three methods available:

a. Linear elastic stress determination

b. Gamma method
c. Shear analogy method

(4) The methods differ in the following points and may be executed according to following standards:

Linear elastic stress determination

- simplified approximation, taking into account the lower bending stiffness of the cross layers but not taking into account the impact of the rolling shear effect on bending stress increase (neglect of compliance)
- see below

Gamma Method

- taking into account the impact of the rolling shear effect on bending stress increase
- see EN 1995-1-1, Annex B [65] specified for CLT in EAD 130005-00-0304 [61]

Shear Analogy Method

- taking into account the impact of the rolling shear effect on bending stress increase
- see DIN EN 1995-1-1/NA, NCI NA 5.6.3 [59]

(5) The choice which method should be used is restricted by the following application limitations:

1. Linear elastic stress determination
   - If admitted by ETA
   - For simply supported beams / plates only
   - Length to depth ratio $l / d > 15$ [12]
   - Distributed loads only (no simultaneous occurrence of maximal internal bending moments and shear forces)
2. Gamma Method:
   - For a maximum of five single layers
   - For single-span and multi-span beams/plates (using a reduces span wide of *4/5 L*)
   - Distributed loads only
   - System-depended, (L_{ef})
3. Shear Analogy Method (Higher calculation effort / FE or frame work program required):
   - Not restricted to special load scenarios or boundary conditions
   - Thus, also point loads or point supported systems can be computed

(6) Linear elastic determination of stresses (simplified approach):

$$I_{\mathrm{n,x}} = \sum \frac{{d_{\mathrm{x,i}}}^3}{12} + \sum d_{\mathrm{x,i}} \cdot z_{\mathrm{s,i}}^2 \quad \text{(C.8)}$$

$$S_{\mathrm{n,x}} = \sum d_{\mathrm{x,i}} \cdot z_{\mathrm{s,i}} \quad \text{(C.9)}$$

Maximal bending stress:

$$\sigma_{\mathrm{Ed}} = \frac{M_{\mathrm{x,d}}}{I_{\mathrm{n,x}}} \cdot \frac{d}{2} \quad \text{(C.10)}$$

Shear stress:

$$\tau_{\mathrm{Ed}} = \frac{V_{\mathrm{xz,d}}}{I_{\mathrm{n,x}}} \cdot S_{\mathrm{n,x}} \quad \text{(C.11)}$$

with:

M_{x}	Moment for determination of bending stress in x-direction, in Nmm
V_{xz}	Shear force for determination of shear stresses in xz-plane, in N
$I_{\mathrm{n,x}}$	Moment of inertia, net cross-section in x-direction, in mm^4
$S_{\mathrm{n,x}}$	Static moment, net cross-section in x-direction, in mm^2
d	Depth of the CLT plate, in mm

NOTE Maximum rolling shear stress for $S_{\mathrm{n,x}}$ concerning the cross layer next to the middle layer.

Annex D *

[informative]

Beam-on-foundation models

This clause still has to be edited.

Annex E

[informative]

Benchmark cases

E.1 General

(1) This Informative Annex provides benchmarks for the verification and validation according to clause 7.2 and 7.3 and general information on the application of benchmarks.

(2) The documentation of a benchmark case should provide all relevant input parameters (structural system, geometry, boundary conditions, actions, material modelling and imperfections) and analysis settings for setting up a numerical model similar to the benchmark. The relevant results of the benchmark for verification and validation of a numerical model should be given. Additional information on possible modelling issues, graphics for illustration of inputs and results and further background information may be given.

(3) It should be checked whether a benchmark is appropriate for validation of a specific model.

(4) If using benchmarks for validation it should be distinguished between 1st level validation and 2nd level validation.

(5) 1st level validation describes the validation of a numerical model where a benchmark with the same structural system, type of actions (e.g. line loads), boundary conditions, material grade, failure behaviour and approximately the same geometry is used.

(6) 2nd level validation describes the validation of a numerical model where a benchmark with diverging structural system, geometry, type of actions (e.g. line loads), boundary conditions, material grade or failure behaviour is used. It should be described in documentation of the benchmark within which application limits a 2nd level validation is possible.

E.2 Simply supported CLT plate (non-edge glued) with uniformly distributed load

(1) See [37].

E.3 Simply supported beech LVL beam with point load at midspan

(1) See [36].

E.4 Single dowel-type connection tensile test with slotted-in steel plate

(1) See [10].

10. Miscellaneous

Author contributions

Conceptualization, Ulrike Kuhlmann (U.K.) and Janusch Töpler (J.T.); resources, U.K.; writing - original draft preparation :: general, J.T. :: connections, J.T. and Lea Buchholz (L.B.) :: CLT, Sabrina Machanek (S.M.) and J.T.; writing - review and editing, U.K. and J.T.; visualization, J.T.; supervision, U.K.; project administration, J.T.; funding acquisition, U.K., Matthias Euler (M.E.), J.T. and Julius Gauß (J.G.).
All authors have read and agreed to the published version of the manuscript.

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