Proceedings of the Joint Conference of COST Actions FP1402 & FP1404 KTH Building Materials, 10.3.2016

Cross Laminated Timber – A competitive wood product for visionary and fire safe buildings

Editors:
Andreas Falk, Philipp Dietsch & Joachim Schmid

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This book may be cited as:
Falk, A., Dietsch P. and Schmid, J. (eds) (2016)
Proceedings of the Joint Conference of COST Actions FP1402 & FP1404
"Cross Laminated Timber – A competitive wood product for visionary and fire safe buildings". KTH Royal Institute of Technology, Division of Building Materials.
ISBN: 978-91-7729-043-8. March 10-11, Stockholm, Sweden

ISBN: 978-91-7729-043-8
TRITA-BYMA 2016:2
ISSN: 0349-5752
KTH Royal Institute of Technology, School of Architecture and the Built Environment,
Department of Civil and Architectural Engineering
Division of Building Materials, SE-100 44 Stockholm

ISSN: 0284-5172
SP Report 2016:41
SP Technical Research Institute of Sweden, Box 857, SE-501 15 Borås

Printed in Sweden by USAB, Stockholm 2016

Acknowledgements:
The members of COST Actions FP1402 and FP1404 would like to thank the COST Association in Brussels for financially supporting this publication.
Stockholm City is recognized with gratitude for inviting the delegates of the conference to the grand reception at the Stockholm City Hall on March 10. 2016.

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The delegates of the Joint Conference of COST Actions FP1402 & FP1404 Cross Laminated Timber – a competitive wood product for visionary and fire safe buildings. KTH Royal Institute of Technology, Stockholm, 10.3.2016

Foreword

Cross Laminated Timber uses the anisotropy of wood to its advantage by placing laminations across the grain. This elegant solution has led to Cross Laminated Timber (CLT or XLAM) being among the most significant recent innovations in timber engineering. It has grown to an economically significant area of R&D in wood sciences and has the ability to replace many traditional building products with a sustainable solution capable to answer the requests of our contemporary society, e.g. regional production, innovative and flexible designing possibilities, sustainability as well as safe design with respect to the building process and the final product.

Invented in Mid Europe some decades ago, the most influential research and development of this product has also been based in Europe for a long time. This is challenged today, as other continents – having started work in this field later but as one joint effort – tend to overtake the European success story. The reasons for this are the known challenges within a unified Europe, very long standardization processes necessitating individual national approaches as well as competition within the timber construction sector rather than joint approaching of new markets.

This conference has compiled knowledge about the material and its use, focusing on the design of CLT structures in ambient and fire situations. To this aim, two COST Actions – *FP1402 Basis of structural timber design – from research to standards*, and *FP1404 Fire Safe Use of Bio-Based Building Products* – joined efforts to provide contributions and presentations from world leading experts and to bring together expert communities to realize a scientific discourse on these important and interdisciplinary topics, leading to further joint harmonisation progression in research and development.

This Joint Conference was meant to contribute to a high-quality and open scientific and technical dialogue within the timber community. The programme therefore included time for debate after the presentations as well as the formation of Think Tanks in which all participants, guided by essential questions, discussed the future challenges and development of CLT.

During the Joint Conference, over 200 participants from the worldwide CLT community (ca. 50% coming from industry) showed their will to initiate joint work, concentrating strengths with respect to simplification, harmonisation and, most of all, understanding different points of views. This book contains not only the State-of-the-Art in research and practice, it also documents the valuable thoughts of experts on challenges and necessary developments of CLT within the next years.

The Chairs of both COST Actions and the local organiser and host of the event would like to thank the authors for contributing with technical papers and presentations and all participants for their trust and collaboration in this Joint Conference as well as their outstanding energy in research & development, design, construction and standardisation of CLT, the leading product for future timber construction.

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KTH Building Materials

For updates and further information please visit:

www.costfp1402.tum.de

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COST Action FP1402 – FP1404
"Joint Conference of COST Action FP1402 and FP1404 – Cross Laminated Timber – a competitive wood product for visionary and fire safe buildings"

Thursday 10th - Friday 11th March 2016, KTH Stockholm, Sweden

The scientific papers and minutes of the technical presentations

Introduction to CLT, Product Properties, Strength Classes

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Summary

We introduce briefly the product cross laminated timber (CLT) and outline the development and basics of this European engineered timber product, innovated to be used as large-sized stand-alone structural element for load-bearing purposes. The main focus of our contribution is to present and discuss properties of side face bonded homogeneous CLT made of Norway spruce. A strength class system and corresponding characteristic properties of CLT exposed to loads in- and out-of-plane are presented. Background documents are referenced. In view of a concerted standardization system the characteristic properties are linked with adequate test configurations and reference dimensions of the base material and the product itself. The concertation of product properties, reference dimensions & conditions with adequate test configurations is a prerequisite and the basis for harmonization and international standardization of this innovative timber product of meanwhile global interest. Final remarks and an outlook conclude this contribution.

1. Introduction

In timber engineering more and more engineered wood products (EWPs) are used. These products allow using timber in dimensions multiple-times larger than typically known from sawn timber with the additional advantage of higher homogeneity in terms of lower dispersing properties and lower variation between

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products from different production batches and producers. These aspects are achieved by production processes which interlink the main steps (i) material classification, (ii) separation and (iii) innovative assembling to rigid composite structures via adhesive bonding.

Within the last decades various EWPs for load-bearing purposes entered the market; primary linear members, like glued laminated timber (glulam; GLT), (finger jointed) construction timber or duo- and trio-beams, but also two-dimensional products like laminated veneer lumber (LVL) and oriented strand boards (OSB). By tradition timber engineering mainly based on linear members; large hall structures with truss systems or solid-web girders, timber bridges and single- and multi-story houses, office and school buildings erected as light-frame timber structures with OSB or LVL as diaphragm component. The market share of timber structures, compared to that of mineral-based solid construction materials like masonry and reinforced concrete, was only a view percentage, at least in Europe.

In Central Europe and more than two decades ago a new EWP for load-bearing purposes was invented, called cross-laminated timber (CLT). It constitutes a stand-alone laminar and large-sized plate-like structural element, which is commonly composed of an uneven number of layers (usually three, five or seven), each made of boards placed side-by-side, which are arranged crosswise to each other, usually at an angle of 90°. These elements, which can be used as whole floor and wall elements with and without openings, are capable of bearing loads in- and out-of-plane. Common dimensions of CLT elements are in length up to 18 or even 30 m, in width up to 3.0 or even 4.8 m and in thickness seldom above 300 to 400 mm; see e.g. [1,2].

The idea behind CLT is in principle not new and its basic structure comparable to common joinery and carpentry products with the same major advantage of high dimensional stability in-plane due to cross-wise layering. The innovative aspect of CLT is its thickness which allows using it as a stand-alone structural element with outstanding strength and stiffness properties. The large dimensions, its easy handling and versatile applicability opens new markets for timber engineering and allows architecture and engineering to realize (super)structures and monolithic buildings in timber. In fact, CLT is also a high-value alternative for reinforced concrete or other mineral-based solid construction materials whereby CLT acts as a serious competitor on the market [2]. Intensive research activities on CLT started 1990 in Graz / Austria and first residential buildings in CLT reflecting the current state-of-the-art were realized by Moser (1995) [3]. Meanwhile, CLT as innovative Central European product with about 500 TSD m³ production volume per year in Europe, has become a product of global interest. Not only has it initiated research activities internationally but also the global establishment of production sites and activities in regard to standardization and harmonization, in particular in countries like Canada, United States, Japan, China and New Zealand. The solid structure of CLT allows also using timber species with lower mechanical properties than

Norway spruce (*Picea abies*), the species typically used in Europe. Global interest and activities in conjunction with the ability to use local timber species lead to new CLT products with high local or regional added value, an important aspect when considering CLT as sustainable, CO_2-active construction material with a great chance to boost its international relevance further. Meanwhile great efforts are made in establishing the Solid Timber Construction Technique in Cross Laminated Timber, a building construction system which allows demonstration of the potential use as well as economic and competitive advantages of CLT.

Within this contribution we aim on presenting product properties in conjunction with a strength class system, providing the essential input parameters for the design process, as well as corresponding test configurations. With focus on Europe and the state-of-the-art we further concentrate on CLT of Norway spruce with homogeneous layup (all boards correspond to the same strength class) as a quasi-rigid composite structure with side face bonded layers.

2. Characteristic Properties of CLT and Test Configurations

2.1 General Comments and Definitions

2.1.1 Reference Dimensions for CLT Lamellas and CLT Elements

The European product standard for CLT, EN 16351 [4], allows layer thicknesses in the range of $t_\ell = 6$ to 45 mm and board or lamella widths within $w_\ell = 40$ and 300 mm. In view of standardization and construction tenders the widely accepted standard layer thicknesses in Central Europe are $t_\ell = 20$, 30 and 40 mm. Due to rolling shear stresses in layers of CLT loaded out-of-plane, a minimum width of $w_\ell \geq 4\,t_\ell$ is advised, otherwise a reduced rolling shear resistance has to be considered, see e.g. [5] and Section 2.3.2.

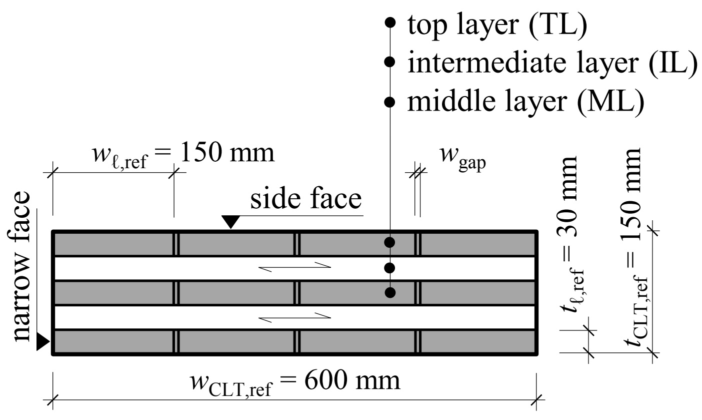


Fig. 1 Reference cross-section of a CLT element and relevant terms.

The heterogeneity of the raw material timber and potential influences during testing necessitate the standardization of reference dimensions and test conditions with the aim to allow for traceable and reliable product properties on a reference and comparable basis. In respect to the standard width of solid timber, $w_{\ell,\mathrm{ref}} = 150$ mm

(EN 384 [6]), and the range of common CLT layer thicknesses, for lamellas of CLT a reference cross section of $w_{\ell,\text{ref}} \times t_{\ell,\text{ref}} = 150 \times 30\ \text{mm}^2$ is proposed. To account for the laminar, plate-like structure of CLT and homogenization effects due to mutual (inter)action of lamellas and layers within the CLT structure, for a CLT element we propose a reference width equal to four-times the width of the reference lamella, with $w_{\text{CLT,ref}} = 4\ w_{\ell,\text{ref}} = 600$ mm. In view of a common number of layers between three and seven, for the reference thickness of a CLT element a five-layer element with homogeneous layup and layers in reference thickness are suggested, with $t_{\text{CLT,ref}} = 5\ t_{\ell,\text{ref}} = 150$ mm, see Fig. 1. Consequently, in the main axis of such a reference CLT element there are $N = 12$ lamellas (elements) running in the same direction which is roughly equal to the reference cross section of GLT where we have 15 lamellas, with $t_{\ell,\text{GLT,ref}} = 40$ mm and $w_{\text{GLT,ref}} \times d_{\text{GLT,ref}} = 150 \times 600\ \text{mm}^2$.

2.1.2 Proposed CLT Strength Class System and Background Information

Mechanical properties as well as density of CLT show significantly lower variability than corresponding properties of the base material, board or lamella. This circumstance is not new and one major advantage of EWPs in general. The reason therefore is the homogenization of base material's (element's) properties due to their common (inter)action in the serial, sub-parallel laminar structure of CLT where homogenization takes place within and between the individual layers. Although this (inter)action influences also the mean strengths and elastic and shear moduli, the major influence is on the variability which decreases remarkably. Consequently, with increasing number of interacting elements, product properties' distributions concentrate more and more around their mean values and the lower quantiles, e.g. the 5 %-quantiles, rise. In fact, the higher the variability in base material's properties, the higher the possible gain caused by homogenization [7]. Beside these homogenization effects, also known as stochastic system effects, there exist also mechanical system effects which may have also an influence on EWPs' properties in relation to the base material's properties. However, numerous investigations in the past conclude a dominance of stochastic system effects in the overall description of system properties based on element's properties; a compilation of these investigations and modelling approaches can be found e.g. in [7].

These principal considerations are taken into account when discussing a possible strength class system for CLT. We suggest, in-line with the strength class system for GLT (see e.g. EN 14080 [8]), CLT strength classes which are related to the physical potential of the base material. Therefore, so called load-bearing models and models for elastic and shear moduli as well as density are required which describe CLT properties at reference conditions and dimensions in relation to reference base material's properties. Furthermore, the name of a strength class should reflect the product, the reference product's property(ies) and the principal layup or composition. In regard to GLT and [8] we suggest e.g. CL 28h or CL 28c, with "CL" as acronym for CLT, "28" as the corresponding characteristic (5 %-quantile) bending strength of CLT out-of-plane and "h" or "c" for a homogeneous (all layers of equal strength class) or combined (heterogeneous) layup of CLT, respectively, see e.g. [2,9]. Although CLT is capable bearing loads in- and out-of-plane we chose the characteristic bending strength out-of-plane as reference strength property (i) because it is one relevant product property, (ii) in reference to GLT and [8], and (iii) because the load-bearing model for CLT in bending out-of-plane was the first one for CLT and meanwhile has been multiple times confirmed (see [10]; Section 2.3.1). As outlined in [2] it is also meaningful to relate properties of CLT to that of GLT. This is a consequence of comparable reference cross section dimensions and number of elements in main direction, see Section 2.1.1. With the characteristic bending strength of CLT out-of-plane as reference strength class property, for the corresponding load-bearing model the tensile strength parallel to grain of the base material as main model parameter is required. The reason is that the edge bending stresses in CLT loaded out-of-plane primary cause tensile stresses parallel to grain in top layer's lamellas.

Back to the dominant role of stochastic system effects in EWP's properties' description: comprehensive investigations on tensile strength parallel to grain of lamellas from Norway spruce have shown that the variability in strength properties depends on the grading method and the number of classes the material was graded in. For visually graded sawn timber, irrespective if grading was performed in one or two strength classes plus rejection, the coefficient of variation usually found was $CV[f_{t,0}] = 35 \pm 5$ %. Boards graded mechanically in just one common grading class plus rejection, so that the boards with higher properties remained in the grading class, showed a comparable value for $CV[f_{t,0}]$. However, mechanical grading in more than one common grading class plus rejection reduces the variability due to higher accuracy in the mechanical grading process compared to visual grading and the outcome was found to be within $CV[f_{t,0}] = 25 \pm 5$ %, see [11]. Board material graded to a specific strength class showing high strength variability has a higher mean value than board material of the same strength class, with the same characteristic (5 %-quantile) strength value but with lower variability. Consequently, higher system effects and gain in resistance on the 5 %-quantile level can be achieved in EWPs by using base material of higher variability. We take this circumstance into account by defining a strength class system for CLT exemplarily based on a base material strength class T14 (see e.g. EN 14080 [8]), with $f_{t,0,\ell,k} = 14.0$ N/mm² and $E_{0,\ell,mean} = 11{,}000$ N/mm², as characteristic (5 %-quantile) tensile strength parallel to grain and mean elastic modulus parallel to grain, respectively. In general, strength properties are directly influenced by local growth characteristics, e.g. knots, knot clusters and grain deviation. This leads to a dependency of EWPs strength properties on the variability of strength properties of the base material, e.g. $CV[f_{t,0}]$. Known exceptions are e.g. shear as well as tension and compression strength perpendicular to grain; see e.g. [8]. In contrast, elastic EWP properties and density constitute average base material properties; a differentiation in respect to $CV[f_{t,0}]$ is not required.

Within the following sub-chapters and sections properties of CLT exposed to loads in- and out-of-plane are discussed. The presented values correspond to CLT made of Norway spruce, with side face bonded layers, homogenous layup and elements according to the specified reference dimensions (see Section 2.1.1) as well as the reference test conditions as specified e.g. in EN 408 [12], e.g. reference conditions with 20 °C and 65 % relative humidity which corresponds to an equilibrium reference moisture content of $u_{\text{ref}} = 12$ %.

2.2 Density of CLT

The mean density of CLT, $\rho_{\text{CLT,mean}}$, is the same as of the base material, with $\rho_{\text{CLT,mean}} = \rho_{\ell,\text{mean}}$, as far as the layer thicknesses are not too small so that the amount of adhesive used in production, usually with significantly higher density than the base material, has not any relevant influence. An influence of a few percentages can be expected when the layer thicknesses are below 10 mm.

Due to averaging effects in a CLT element the variability in density is much lower than in the base material. According to the Central Limit Theorem of probability theory, for a sufficiently large amount of boards or lamellas in a CLT element, N, the reduction in variability can be described by $\text{CV}[\rho_{\text{CLT}}] \approx \text{CV}[\rho_\ell] / \sqrt{N}$. Tests have shown that there is sufficient agreement between theoretically calculated and experimentally observed values already in CLT elements in reference dimensions but also for three-layer CLT elements.

As consequence of these statements and with $\text{CV}[\rho_\ell] = 8$ % (in general, 6 to 10 %; see [7]), the characteristic (5 %-quantile) product density of CLT in terms of base material's density and via normal approximation is given as

$$\rho_{\text{CLT,k}} = \frac{1 - 1.645\,\text{CV}\left[\rho_l\right]/\sqrt{N}}{1 - 1.645\,\text{CV}\left[\rho_l\right]}\,\rho_{l,\text{k}} \overset{N \geq 10}{\rightarrow} 1.10\,\rho_{l,\text{k}}. \tag{1}$$

These regulations are in-line with [8] for GLT. However and as outlined in [13,14], in designing joints in side or narrow face of CLT, and considering density as the only material property indicating the embedment and withdrawal capacity of fasteners, in case of fasteners penetrating only one layer or lamella of CLT the density of the base material, $\rho_{\ell,\text{k}}$, shall be used.

2.3 CLT out-of-plane: Strength Values, Moduli of Elasticity and Shear

2.3.1 Properties in Bending

In view of the load-bearing model for CLT in bending [10], the characteristic (5 %-quantile) bending strength out-of-plane, $f_{\text{m,CLT,k}}$, as reference value for the proposed CLT strength class system, is regulated in dependency of the characteristic (5 %-quantile) tensile strength parallel to grain of the base material (boards), $f_{\text{t},0,\ell,\text{k}}$, and the corresponding coefficient of variation, $\text{CV}[f_{\text{t},0,\ell}]$, see

$$f_{\text{m,CLT,k}} = k_{\text{m,CLT}}\, f_{\text{t},0,l,\text{k}}^{0.8}, \text{ with } k_{\text{m,CLT}} = k_{sys,\text{m}}\, k_{\text{CLT/GLT}}\, k_{\text{h,CLT}}\, k_{\text{CV\_t}}, \tag{2}$$

with $k_{sys,m}$ as system effect due to mutually interacting lamellas in CLT element's main direction [15], k_{CV_t} as factor which considers differences in CV[$f_{t,0,\ell}$] [11], $k_{h,CLT}$ as depth factor equal to GLT according to [8] and $k_{CLT/GLT}$ as factor which considers empirically determined differences in homogenization effects between CLT and GLT.

Table 1: CLT strength classes; characteristic values of CLT out-of-plane.

Base material T14; CV[$f_{t,0,\ell}$] =		25 ± 5 %	35 ± 5 %
Property [–]	**Symbol [–]**	**CL 24h**	**CL 28h**
Bending strength	$f_{m,CLT,k}$ [N/mm²]	24.0	28.0
Tensile strength perpendicular to grain	$f_{t,90,CLT,k}$ [N/mm²]	0.5	
Compression strength perpendicular to grain	$f_{c,90,CLT,k}$ [N/mm²]	3.0	
Shear strength	$f_{v,CLT,k}$ [N/mm²]	3.5	
Rolling shear strength	$f_{r,CLT,k}$ [N/mm²] $f_{r,lay,k}$ [N/mm²]	1.40, for $w_\ell / t_\ell \geq 4$ 0.80, for $w_\ell / t_\ell < 4$	
Modulus of elasticity parallel to grain	$E_{0,CLT,mean}$ [N/mm²] $E_{0,lay,mean}$ [N/mm²]	11,600	
Modulus of elasticity perpendicular to grain	$E_{90,CLT,mean}$ [N/mm²] $E_{90,lay,mean}$ [N/mm²]	300	
Modulus of elasticity in compression perp. to grain	$E_{c,90,CLT,mean}$ [N/mm²]	450	
Shear modulus	$G_{0,lay,mean}$ [N/mm²]	650	
Rolling shear modulus	$G_{r,lay,mean}$ [N/mm²]	100, for $w_\ell / t_\ell \geq 4$ 65, for $w_\ell / t_\ell < 4$	
Elastic & shear properties' 5 %-quantiles	$E_{CLT,05}$ [N/mm²] $E_{lay,05}$ [N/mm²] $G_{CLT,05}$ [N/mm²] $G_{lay,05}$ [N/mm²]	$E_{05} = 5/6\ E_{mean}$ $G_{05} = 5/6\ G_{mean}$	

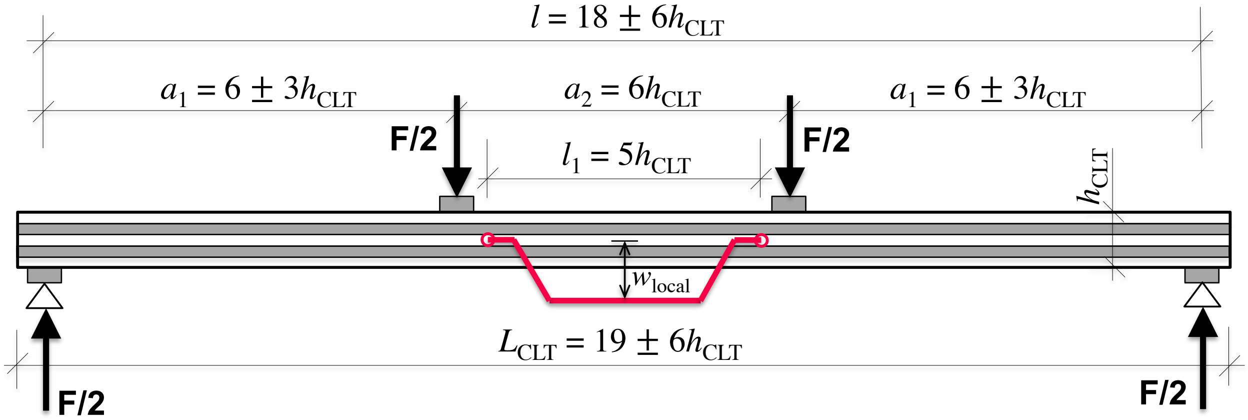
By using board material of strength class T14 but with different CV[$f_{t,0,\ell}$] the CLT strength classes, CL 24h and CL 28h, can be achieved, see Table 1.

For the modulus of elasticity parallel to grain the relationship

$$E_{0,CLT,mean} = E_{0,lay,mean} = 1.05\ E_{0,l,mean} \tag{3}$$

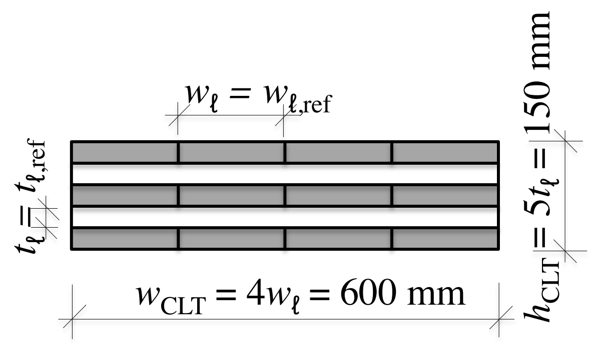
is proposed which is conform with regulations for GLT in [8], see Table 1. Recent investigations have shown that the amount of system action on mean values in GLT in comparison to the elastic properties of the base material, as consequence of parallel and serial acting springs, is negligible [16]; the relationship $E_{0,\mathrm{CLT,mean}} = E_{0,\ell,\mathrm{mean}}$ would be more appropriate.

The determination of the bending properties of CLT, strength and modulus of elasticity, is carried out according to the four-point-bending test setup in [12]; see Fig. 2. To prevent rolling shear failures in this orthogonal laminar structure, it is suggested to increase the length a_1, as distance between support and load introduction, to $a_1 = 6 \pm 3\ h_{\mathrm{CLT}}$. Thus, a_1 depends on the CLT layup, the ratio w_ℓ / t_ℓ and the basic material properties. Based on experience with CLT made of Norway spruce an optimum length of $a_1 = 7.5\ h_{\mathrm{CLT}}$ was found. In-line with [12] it is proposed to measure the local deformations, w_{local}, at both narrow faces in the neutral axis and within the shear free area.



reference cross section for bending with

$t_{\ell,\mathrm{ref}}$ = 30 mm; $w_{\ell,\mathrm{ref}}$ = 150 mm



with:

h_{CLT}	… thickness of the specimen
l	… span of the specimen
l_1	… gauge length for measuring w_{local}
L_{CLT}	… length of the specimen
$w_{\ell,\mathrm{ref}}/t_{\ell,\mathrm{ref}}$	… reference width/thickness of single lamella

Fig. 2 Test setup for bending tests for loads out-of-plane.

The calculation of the bending properties is based on the Timoshenko beam, assuming $E_0 = E_{0,\ell,\mathrm{mean}}$ and $E_{90} = 0$. Eqs. (4–6) show the calculation of the bending stiffness, K_{CLT}, the bending strength, $f_{\mathrm{m,CLT}}$, and the modulus of elasticity in bending, $E_{0,\mathrm{CLT}}$. Therein, $\Delta F / \Delta w$ is the relationship between changes in load and deformation, determined within the linear elastic range between 0.1 and 0.4 $F_{\max}$.

$$K_{\mathrm{CLT}} = \sum (E_{\mathrm{i}}\, I_{\mathrm{i}}) + \sum (E_{\mathrm{i}}\, A_{\mathrm{i}}\, e_{\mathrm{i}}^2) \tag{4}$$

$$f_{\mathrm{m,CLT}} = \frac{F_{\max}/2\, a_1}{K_{\mathrm{CLT}}}\, z\, E_0 \tag{5}$$

$$E_{0,\mathrm{CLT}} = \frac{a_1\, l_1^2}{16\, \frac{K_{\mathrm{CLT}}}{E_0}}\, \frac{\Delta F}{\Delta w} \tag{6}$$

2.3.2 Shear and Rolling Shear Properties

The shear strength of CLT against loads out-of-plane is regulated in compliance with GLT and [8], with $f_{\mathrm{v,CLT,k}} = 3.5$ N/mm².

For the shear modulus of CLT layers, $G_{0,\mathrm{lay,mean}}$, and in analogy to the modulus of elasticity in bending out-of-plane (see Section 2.3.1) we propose the same value as for the base material, with $G_{0,\mathrm{lay,mean}} = G_{0,\ell,\mathrm{mean}}$, see Table 1. This suggestion is also in-line with [8].

Due to the orthogonal layering, the transverse layers in CLT are exposed to rolling shear. The resistance against these stresses depends on the ratio w_ℓ / t_ℓ; the increasing amount of tension perpendicular to grain stresses combined with rolling shear stresses lead to a remarkable decrease in resistance; e.g. [5]. Other investigations on rolling shear strength were made e.g. by [17–20]. Recently, Ehrhart et al. [21] suggest a bi-linear approach for the characteristic (5 %-quantile) rolling shear strength, dependent on the ratio w_ℓ / t_ℓ, see

$$f_{\mathrm{r,CLT,k}} = \min\left\{0.2 + 0.3\, \frac{w_l}{t_l}; 1.40\right\}. \tag{7}$$

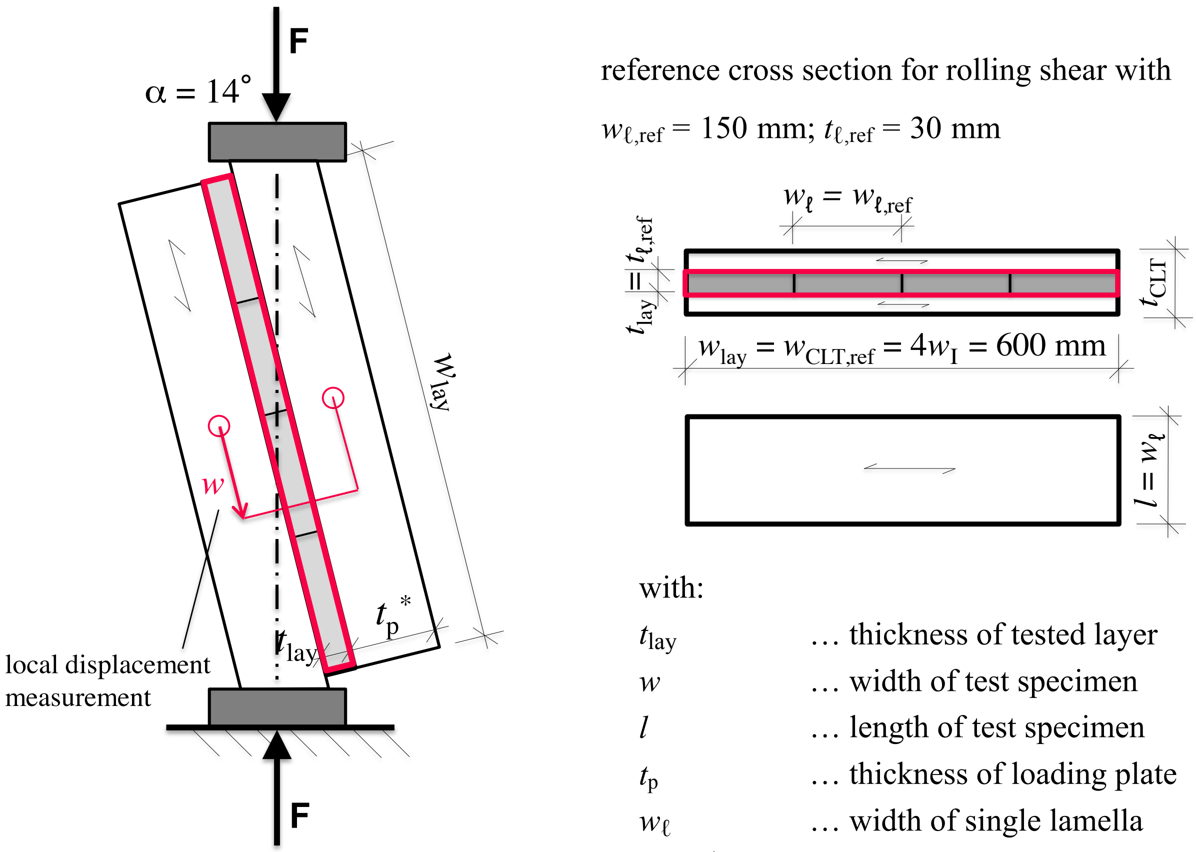
For the ease of use the regulation of only two values, without bi-linear interaction, is recommended, with $f_{\mathrm{r,CLT,k}} = f_{\mathrm{r,lay,k}} = 1.40$ and 0.80 for $w_\ell / t_\ell \geq 4$ and $w_\ell / t_\ell < 4$, respectively.

The rolling shear modulus, $G_{\mathrm{r,lay}}$, also depends on the ratio w_ℓ / t_ℓ. Current technical assessment documents and past investigations outline a ratio $G_{\mathrm{r,lay,mean}} / G_{0,\mathrm{lay,mean}} = 1 / 10$ for $w_\ell / t_\ell \geq 4$. Recent investigations in [21] conclude a much higher rolling shear modulus, with $G_{\mathrm{r,lay,mean}} = 100$ and 65 N/mm² for $w_\ell / t_\ell \geq 4$ and $w_\ell / t_\ell = 2$, respectively, and a bi-linear approach, with

$$G_{\mathrm{r,CLT,mean}} = \min\left\{30 + 17.5\, \frac{w_l}{t_l}; 100\right\}. \tag{8}$$

The reason for these higher values is that current European CLT products show an increasing amount of base material taken closer to the pith whereas in the past primary side-boards had been used. The significant relationship between rolling shear modulus and the annual ring pattern, i.e. the relative position of the board's center to the pith (e.g. [19,21,22]) leads in that case to higher values of rolling shear modulus. However, in respect to the ease of use we propose two fixed values instead of a bi-linear relationship, see Table 1.

Determination of the rolling shear properties is based on the shear test setup in [12]; see e.g. [20,21] and Fig. 3. The load is introduced at an angle of 14° to the loading plates which are made of steel or wood and rigidly glued on the test specimen. The relative displacement of the loading plates is measured on both sides and by displacement transducers. The advantage of this test configuration is that either single board segments, segments of CLT layers or even segments of whole CLT elements can be tested.



with:

t_{lay}	… thickness of tested layer
w	… width of test specimen
l	… length of test specimen
t_p	… thickness of loading plate
w_ℓ	… width of single lamella
$w_{\ell,ref}/t_{\ell,ref}$	… reference width/thickness of single lamella

Fig. 3 Test setup for rolling shear tests.

The calculation of the rolling shear properties, $f_{r,CLT(lay)}$ and $G_{r,CLT(lay)}$, can be done according to [12] and by means of Eqs. (9,10).

$$f_{r,lay} = \frac{F_{max}\cos 14^\circ}{l\,w} \tag{9}$$

$$G_{r,lay} = \frac{\Delta F\cos 14^\circ}{\Delta w}\,\frac{t_{lay}}{l\,w_{lay}} \tag{10}$$

2.3.3 Tension Perpendicular to Grain

The authors are not aware of any investigation on tensile properties of CLT elements perpendicular to grain. In some cases the strength property might be design relevant, e.g. in lap joints between floor elements. Based on an engineering judgement made by considering the laminar structure of CLT in analogy to GLT, for the product CLT we propose to use the same characteristic values as for GLT according to [8], see Table 1.

2.3.4 Compression Perpendicular to Grain

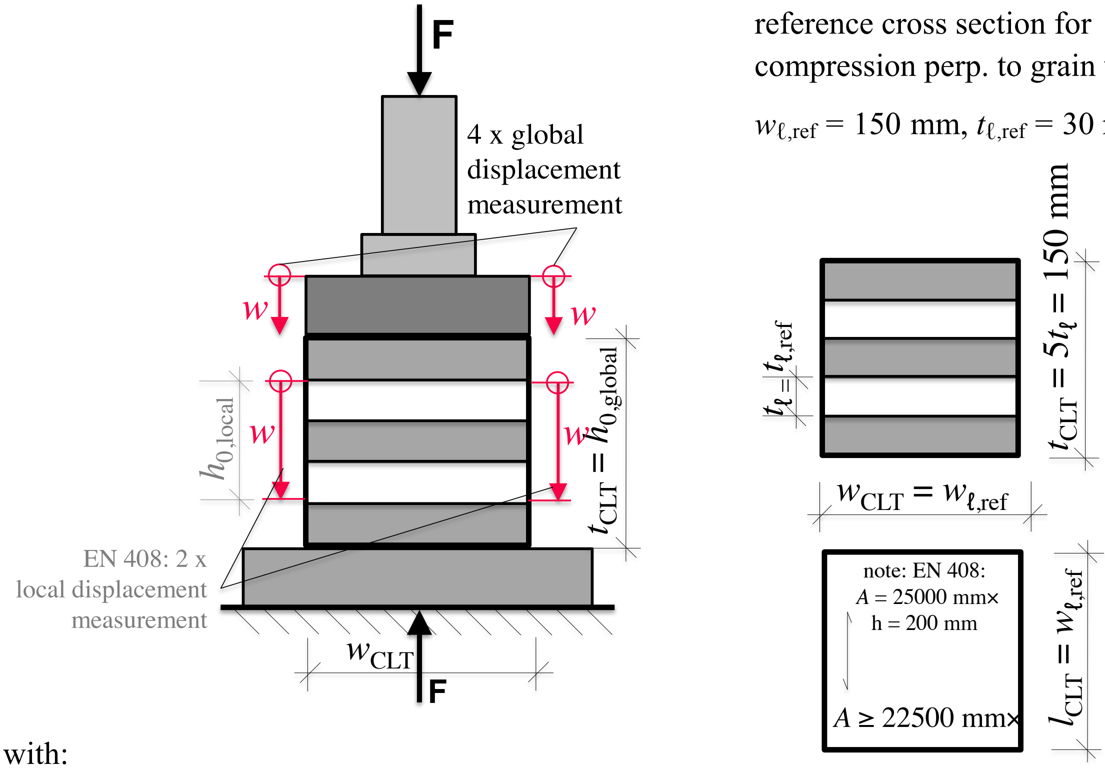
CLT as two-dimensional structural element opens new horizons in timber engineering. The dimensions and outstanding properties predestine CLT to be used in large-spanned and line or point-supported structures. Thus, properties in compression perpendicular to grain are of high relevance in designing CLT elements against loads out-of-plane. Several investigations concentrated on the basic product (single prism) and system properties (different load configurations) of CLT in compression perpendicular to grain, e.g. [23–28]. In comparison to GLT about 30 % higher elastic and strength properties were found for CLT [28]; see Table 1. The reason for this is the reinforcement by the transverse layers in the orthogonal structure of CLT reducing tensile stresses perpendicular to grain.

The determination of base properties of CLT in compression perpendicular to grain is suggested according to the test setup in [12]. Therein a centric load introduction on full-surface loaded prism is anchored. In contrast to the specifications in [12], a (global) displacement measurement is proposed via four displacement transducers arranged at the corners of the loading plate. Thus, the calculated modulus of elasticity is referenced to the whole thickness of the test specimen (gauge length $h_0 = t_{CLT}$), which allows determining a representative value for the given layup. Previous tests of numerous authors have shown that the base properties of CLT perpendicular to the grain are only to a small amount influenced by the layup. Of course, in respect to so far discussed reference dimensions for CLT test specimen a reference prism differing from specification in [12] is recommended, see Fig. 4.

The calculation of the properties for compression perpendicular to the grain, $f_{c,90,CLT}$ and $E_{90,CLT}$, can be done according to [12]; see Eqs. (11,12).

$$f_{c,90,CLT} = \frac{F_{c,90,max}}{l_{CLT}\ w_{CLT}} \tag{11}$$

$$E_{c,90,CLT} = \frac{h_0}{A}\,\frac{\Delta F}{\Delta w} \tag{12}$$



with:

t_{CLT}	… thickness of test specimen
w_{CLT}	… width of the test specimen
l_{CLT}	… length of the specimen
h_0	… gauge length: $h_{0,\mathrm{global}} = t_{\mathrm{CLT}}$ (global measurement); $h_{0,\mathrm{local}} = 0.6\ t_{\mathrm{CLT}}$ (local measurement according to EN 408 [12])
$w_{\ell,\mathrm{ref}}/t_{\ell,\mathrm{ref}}$	… reference width/thickness of single lamella

Fig. 4 Test setup for compression perpendicular to the grain.

2.4 CLT in-plane: Strength Values and Shear Moduli

2.4.1 Properties in Tension Parallel to Grain

Properties of CLT in tension parallel to grain have not been investigated so far, not theoretically and nor via experiments. In engineering practice and on a conservative basis the same value for CLT as for the base material is used and only the lamellas oriented in load direction, i.e. the net cross section, A_{net}, is considered. However, the quasi-rigid composite structure of CLT consequence a mutual interaction of all boards in the CLT cross section oriented in load direction as far as the load is applied homogeneously. In view of a reference CLT cross section with $N = 12$ lamellas (three layers times four lamellas) and investigations on the system factor in tension parallel to grain (e.g. [15,29]) the following relationships are proposed ([2,9]):

$$f_{t,0,CLT,net,k} = k_{sys,t,0}\; f_{t,0,l,k}, \tag{13}$$

with

$$k_{sys,t,0} = \begin{cases} \min\left\{0.075 \ln\left(N\right) + 1; 1.20\right\} & \ldots \text{ for CV}\left[f_{t,0,l}\right] = 25 \pm 5\% \\ \min\left\{0.130 \ln\left(N\right) + 1; 1.35\right\} & \ldots \text{ for CV}\left[f_{t,0,l}\right] = 35 \pm 5\% \end{cases}. \tag{14}$$

This approach bases again on A_{net} but additionally takes into account the homogenization effects within the parallel acting system as function of CV[$f_{t,0,\ell}$], see Table 2. The values are slightly lower than for GLT according to [8] which regulates the characteristic tensile strength parallel to grain to be equal to 80 % of the characteristics bending strength, but higher than for CLT according to the Austrian National regulations for Eurocode 5 (ÖNORM B 1995-1-1 [30]) which allows only to apply a k_{sys}-factor on a single layer, which is in the reference case of four parallel acting boards equal to k_{sys} = 1.09; compare: $k_{sys,t}$ for N = 4 according to Eq. (7) yields 1.10 and 1.18 for base material's strength classes with low and high variability, respectively.

Table 2: CLT strength classes; characteristic values of CLT in-plane.

	Base material T14; CV[$f_{t,0,\ell}$] =	25 ± 5 %	35 ± 5 %
Property [–]	**Symbol [–]**	**CL 24h**	**CL 28h**
Tensile strength parallel to grain	$f_{t,0,CLT,net,k}$ [N/mm²]	16.0	18.0
Compression strength parallel to grain	$f_{c,0,CLT,net,k}$ [N/mm²]	24.0	28.0
Shear strength in-plane (shear & torsion)	$f_{v,net,k,ref}$ [N/mm²] $f_{v,gross,k}$ [N/mm²] $f_{T,node,k}$ [N/mm²]	5.5 3.5 2.5	
Shear modulus in-plane	$G_{CLT,mean}$ [N/mm²]	450 a) 650 b)	
Shear properties' 5 %-quantiles	$G_{CLT,05}$ [N/mm²]	$G_{05} = 5/6\; G_{mean}$	

a) simplified value for CLT without narrow face bonding or with cracks or checks; more detailed approach provided by [31]

b) CLT with narrow face bonding; edge bonding has to be secured over the entire lifetime

2.4.2 Properties in Compression Parallel to Grain

Equal to the tensile properties of CLT parallel to grain, also for compression there are no investigations available so far. Current engineering practice is analog to the approach in tension; however, the mutual interacting layers and lamellas indicate again potentially higher characteristic (5 %-quantile) strength properties in CLT than in the base material, expressible by a system factor $k_{sys,c}(N) \geq 1.00$ as multiplier on the compression strength $f_{c,0,\ell,k}$, see e.g. [9]. For the ease of use, we propose to regulate compression strength parallel to grain of CLT, $f_{c,0,CLT,net,k}$, analog to GLT and according to [8], with $f_{c,0,CLT,net,k} = f_{m,CLT,k}$, see Table 2.

2.4.3 Shear Properties of CLT in-plane

The properties of CLT elements against loads in-plane have been subject of numerous investigations. According to [31–34], three different failure mechanisms have to be distinguished for CLT with and without adhesive bonding on the narrow face: (i) gross-shear failure of the CLT element by longitudinal shearing of all layers of CLT with narrow face bonding, and in CLT without narrow face bonded layers (ii) net-shear failure by exceeding the shear resistance in-plane in layers oriented in CLT's weak direction (e.g. [32,36–38]), and (iii) torsion failure in the gluing-interfaces between the orthogonal layers [17,39,40]. However, a reliable achievement of gross- and net-shear failures in CLT elements failed and the possibility to extrapolate the outcomes from single node testing was not verified so far.

In a comprehensive test campaign using the test configuration of [41] failures in gross- and net-shear could be reliably achieved; [42]. Based on a parameter study in [42] the following characteristic shear properties were derived (see Table 2):

- for CLT elements without narrow face bonding, usually failing in net-shear, a reference characteristic (5 %-quantile) net-shear strength of $f_{v,net,k,ref} = 5.5$ N/mm² applies as far as the thickness of layers in weak axis, $t_{\ell,fail}$, is smaller or equal to 40 mm and the width of gaps between adjacent lamellas within one layer, w_{gap}, is smaller or equal to 6 mm; this strength value is referenced to the net cross section of a CLT element, A_{net}, which is equal to the length of a diaphragm times the sum of layer thicknesses in weak direction, t_{net}, given as

  $$t_{net} = \min\left\{\sum t_{1,L}; \sum t_{1,T}\right\}, \tag{15}$$

  with $t_{\ell,L}$ and $t_{\ell,T}$ as layer thicknesses parallel and transverse the orientation of the top layers, respectively;

- for CLT elements without narrow face bonding with $t_{\ell,fail}$ below 40 mm but in the range of 20 to 40 mm higher shear strengths were observed. In rare cases of layup parameters $\sum t_{1,T} / \sum t_{1,L} \geq 0.8$ net-shear failures in top and middle layers instead in intermediate layers may occur. Due to boundary effects lower shear strength of top layers equal to a nominal 10 mm thicker layer was found.

- for simplification, in CLT elements without narrow face bonded layers an average shear modulus of $G_{CLT,mean} = 450$ N/mm² applies. For cases requiring a more detailed calculation the approach according to Bogensperger et al. [31] is proposed;
- for CLT elements with narrow face bonding, usually failing in gross-shear, a characteristic (5 %-quantile) gross-shear strength of $f_{v,gross,k} = 3.5$ N/mm², dedicated to the entire cross section, A_{gross}, and a mean shear modulus of $G_{CLT,mean} = 650$ N/mm² apply. Both values correspond to values for GLT according to [8] and can be used as long as the narrow face bonding is secured. The possibility of cracks due to swelling and shrinkage, at least in the top layers, has to be considered.

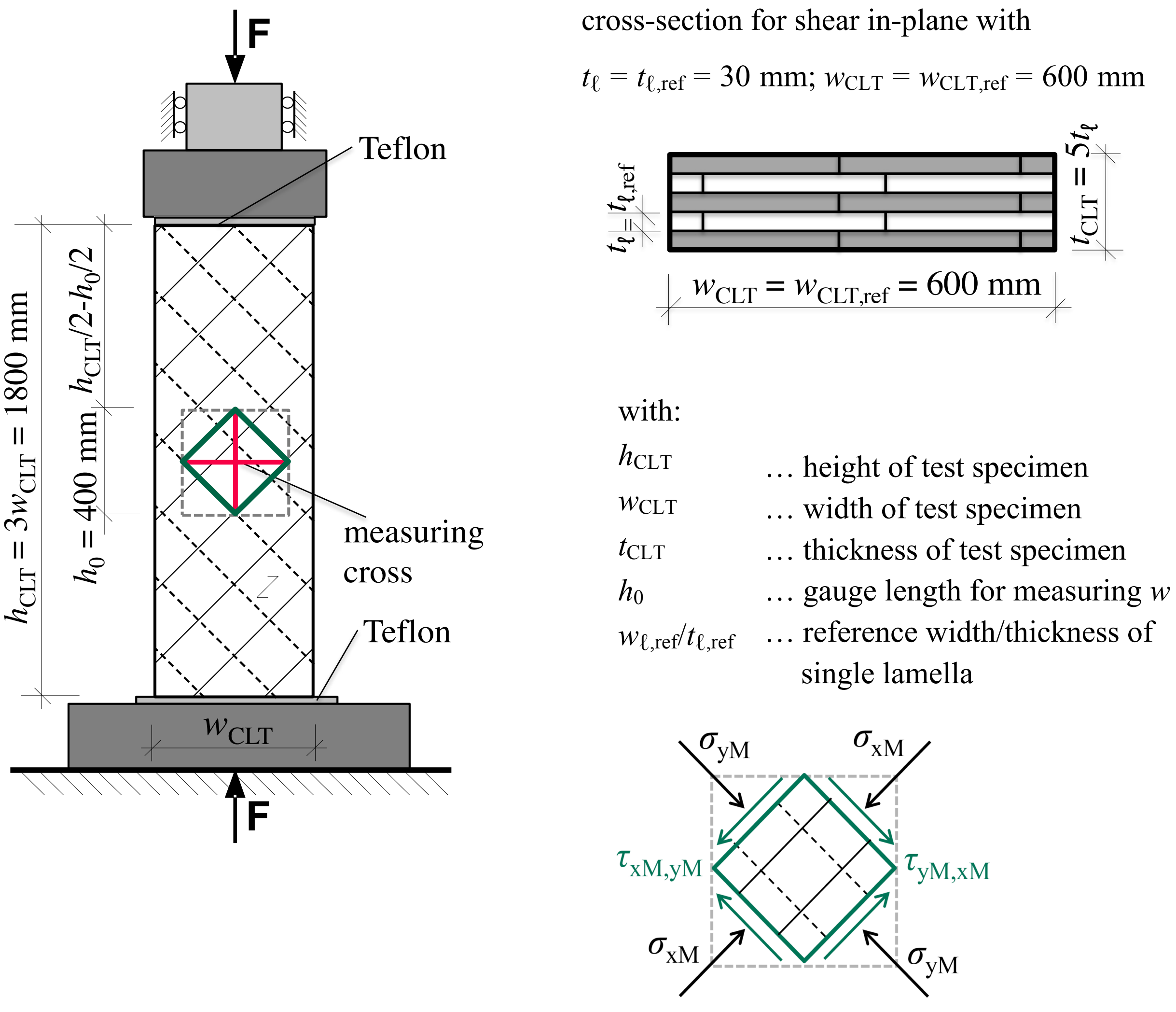


Fig. 5 Test setup for determination of the properties for shear in-plane.

The characteristic (5 %-quantile) shear strength against the torsion failure mechanism is based on single node tests and was found with $f_{T,node,k} = 2.5$ N/mm², see Table 2.

Recently, [42] the successful applicability of the test configuration according to [41] was demonstrated. This configuration allows determining shear properties of CLT diaphragms with specifications mirroring current product parameter's variety. The configuration is based on a simple compression test conducted at an angle of 45° to layer orientation, see Fig. 5. The use of specimen with a ratio $h_{CLT} = 3\ w_{CLT}$ is suggested; see [42]. To minimize possible influences by friction Teflon strips may be arranged between specimen and loading and support. The local deformation measurement can be done via four strain transducers situated centrically on both side faces, vertically and horizontally, with gauge lengths $h_0 = 400$ mm. In case of very slender test specimen, which are prone to fail in buckling the use of centrically placed horizontal supports is recommended.

The shear stress at maximum load, $\tau_{xM,yM}$, is calculated by

$$\tau_{xM,yM} = \frac{F_{max}}{2\ w_{CLT}\ t_{CLT}}. \tag{16}$$

In calculation of the shear strength, $f_{v,CLT}$, differentiation in CLT with and without narrow face bonding is required as the typical failure mechanism is either gross- or net-shear.

In case of CLT with narrow face bonding Eqs. (17,18) apply; with σ_{90} as compression stress in y_M-direction, E_{90} as modulus of elasticity perpendicular to the grain of the base material (top layer), E_{yM} as weighted modulus in elasticity in y_M-direction (intermediate layer), E_0 as modulus of elasticity parallel to the grain of the base material, and $t_{\ell,T}\ /\ t_{\ell,L}$ as ratio between the sum of layer thicknesses in weak and strong diaphragm direction.

$$f_{v,gross} = \tau_{xM,yM} + 1.15\,\sigma_{90} + 0.13\,\sigma_{90}^2, \text{ with } \sigma_{90} = \tau_{xM,yM}\,\frac{E_{90}}{E_{yM}} \tag{17}$$

$$E_{yM} = \frac{\sum t_{l,yM}\ E_0 + \sum t_{l,xM}\ E_{90}}{t_{CLT}};\ t_{CLT} = \sum t_{l,xM} + \sum t_{l,yM} \text{ and } \sum t_{l,L} \geq \sum t_{l,T} \tag{18}$$

In case of CLT without narrow face bonding with and without gaps, Eqs. (19,20) apply; with σ_{90} as compression stress in x_M-direction, E_{90} as modulus of elasticity perpendicular to the grain of the base material, E_{xM} as weighted modulus of elasticity in x_M-direction (top layer), E_0 as modulus of elasticity parallel to the grain of the base material and t_{net} as sum of the layer thicknesses in weak diaphragm direction.

$$f_{v,net} = \tau_{xM,yM}\,\frac{t_{CLT}}{t_{net}} + 1.15\,\sigma_{90} + 0.13\,\sigma_{90}^2, \text{ with } \sigma_{90} = \tau_{xM,yM}\,\frac{E_{90}}{E_{xM}} \tag{19}$$

$$E_{xM} = \frac{\sum t_{l,xM}\ E_0 + \sum t_{l,yM}\ E_{90}}{t_{CLT}};\ t_{net} = \sum t_{l,T} \tag{20}$$

For CLT diaphragms with or without narrow face bonding the shear modulus, G_{CLT}, can be determined according to Eq. (21); in case of specimen featuring a ratio $h_{\mathrm{CLT}} / w_{\mathrm{CLT}} > 3$ a shear correction factor $\alpha_{\mathrm{G}} = 1.0$ can be applied.

$$G_{\mathrm{CLT}} = \alpha_{\mathrm{G}} \frac{h_0}{2\, w_{\mathrm{CLT}}\, t_{\mathrm{CLT}}} \frac{\Delta F}{\Delta w_{\mathrm{G,mean}}} \tag{21}$$

3. Conclusions and Outlook

3.1 Topics for Research and Development

Due to appearance and mechanical reasons as well as aspects of building physics the trend in CLT is towards minimizing the gaps between lamellas within layers and optimization of appearance quality. Apart from standardized layups with standard layer thicknesses of $t_\ell = 20$, 30 and 40 mm, the demand on other layups which are optimized for CLT hybrid structural elements, e.g. ribbed floor elements and box-beams, will increase. In that respect also the utilization of diverse timber species, e.g. for rising the stiffness by target-oriented stiffness grading and the use of high-capacity timber species, e.g. birch, will make it easier to fulfill the requirements in serviceability limit state (SLS) design, i.e. limits in deflection and vibration.

Testing and design prerequisite correction factors for the adjustment of characteristic product properties, determined and applicable for CLT elements with reference dimensions and tested at reference conditions, to CLT elements of other dimensions and exposed to other conditions. Factors enabling these adjustments, e.g. as part of load-bearing models for CLT and in respect to size, system (homogenization), stress distribution, moisture content and temperature, are required; further research is needed.

3.2 Standardization

Although the European CLT product standard, EN 16351 [4], has just been released, it is recommended to commence the work for its revision. We propose to address in particular the following aspects:

- The establishment of a CLT strength class system with reference to the base material’s (board or lamella) potential (for example based on T-strength classes according to [8] and reference conditions (e.g. moisture content, cross section, layup, etc.) are proposed. Nomination of CLT strength classes CLxxh and CLxxc according to glulam strength classes GLxxh and GLxxc in [8] is suggested.

- The analyses of all test configurations currently anchored in [4] in respect to their applicability and adequacy for determination of characteristic properties is necessary. If required, the amendment of existing and supplement of to date missing test configurations together with adequate design approaches and with concerted examination procedures for determination of characteristic product properties are seen as relevant for the establishment of the CLT strength class system. At the end, implementation of these test configurations in EN 408 [12] is seen as meaningful.
- It is deemed to be important to consider all timber species with a relevant market potential for the production of CLT, with focus on European softwood species (e.g. Norway spruce, pine, fir, larch). However, also other species of international importance should be considered, e.g. Japanese cedar (Sugi), Radiata pine.

4. Acknowledgements

The research work was conducted at the Institute of Timber Engineering and Wood Technology at the Graz University of Technology as well as at the Competence Centre Holz.Bau Forschungs GmbH in the framework of the COMET K-Project "focus solid timber solutions (focus_sts)". We would like to express our thanks to all partners involved. The publication, as part of the proceedings from the Joint Conference of FP1402 & FP1404 "Cross Laminated Timber – a competitive wood product for visionary and fire safe buildings" held on the 10th March 2016 at KTH / Stockholm, was written in the framework of the COST Action FP1402 "Basis of Structural Timber Design – from research to standards", chaired by Philipp Dietsch, Technische Universität München (www.costfp1402.tum.de).

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Minutes of Presentation I:
Introduction to CLT, Product Properties, Strength Classes

Presentation by Gerhard Schickhofer

Summary

Gerhard Schickhofer presents an introduction into the topic CLT. This relatively new material stands together with e.g. self-tapping screws for the progress in timber engineering allowing for versatile applications and new and modern architecture. The new CLT standard EN 16351 facilitates further implementation into the market.

- The discussed challenges with regard to CLT are:
- Material properties and their standardization
- Rolling shear failure
- Compression perpendicular to grain strength

Further research and development will focus on:

- Optimization of the structure of CLT and minimization of gaps etc.
- Hybrid structures

With regard to on-going standardization, the following topics are of importance:

- Strength class system
- New and adjusted test setups: single layer testing vs. full size member testing
- Including new wood species from different regions like e.g. Japanese Cedar or Radiata Pine.

Discussion

Staffan Svensson asks about the grading process and the determination of strength classes. Do lamellas of bad quality lead to CLT of higher strength class?

Reinhard Brander replies that the homogenisation effect yields for lamellas of low quality with high variation to a strong increase in the 5% percentile strength properties of the CLT product. Hence, CLT allows to benefit from the homogenisation effect especially for low grade timber compared to high grade timber.

Jochen Köhler asks for the suggested partial safety factors for CLT.

Gerhard Schickhofer suggests 1.25 as for glulam.

Current Status of European Product and Design Standards for Cross-Laminated Timber

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Summary

During the last years cross-laminated timber (X-lam) has become a widely used structural timber product. Many research projects on the design of cross laminated timber, including fire design, and connections between cross laminated timber members have been finalized and results have been applied e.g. on the basis of national technical approvals or European technical assessments (ETAs). At the end of 2015 EN 16351, the harmonized product standard on X-lam has been published and is believed to be applicable in the first half of 2016.

The current version of EC 5 does not deal with the design of cross-laminated timber. Hence CEN/TC 250/SC 5, the technical committee responsible for drafting Eurocode 5, has decided to establish a Working Group (WG) on this item. Additionally one of the Project teams will deal with the subject und will support CEN/TC 250 SC 5 WG 1 in drafting the new section for Eurocode 5-1-1.

This paper gives a brief overview on the products covered by the European product standard EN 16351, the planned activities of CEN/TC 250 SC 5 WG 1 and also on related activities of CEN/TC 124 WG 3, the working group being responsible for the harmonized product standard EN 16351.

1. Introduction

Cross-laminated timber is typically used for walls, floors or roofs in housing. Cross-laminated timber members may be loaded in or perpendicular to the plane (diaphragm or plate), can be subjected to distributed loads and point-loads and may have openings. For the calculation of stresses and strains in diaphragms and plates different methods exists (an overview may e.g. be taken from [1]). Different manuals do not only offer information on the design but also on the construction and detailing, see e.g. [2] to [6]. Furthermore X-lam can also be used as a beam in which the cross layers act as an reinforcement for stresses perpendicular to the grain due to static loads, changes in humidity, notches and holes, see e.g. [7], [8].

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2. Cross-laminated timber according to EN 16351:2015

2.1 Development and applicability

During the last years EN 16351:2015 [9] has been developed on the basis of a so-called Mandate of the European Commission and is a harmonised product standard (hEN). It will lead to a CE-mark and will get compulsory in Europe.

In order to get applicable in Europe, EN 16351 has to be published in the Official Journal of the EU (OJEU). When a standard is published in the OJEU for the first time, a coexistence period is given. At the end of the coexistence period any existing national product rules have to be withdrawn.

When this paper is written, the publication of more than 60 hEns in the OJEU is announced for the beginning of April 2016 but it is still unclear, if EN 16351 will be one of those standards.

2.2 Cross-laminated timber covered by EN 16351

Within EN 16351: 2015 cross laminated timber is defined as "structural timber consisting of at least three layers of which a minimum of three are orthogonally bonded, which always comprise timber layers and may also comprise wood-based panel layers".

The Standard applies to cross-laminated timber:

— to be used in service class 1 or 2 according to EN 1995-1-1;

— made of coniferous species and poplar;

— built up of at least three orthogonally bonded layers (at least two of them timber layers);

— having, depending on the number of layers, adjacent layers which may be bonded parallel to the grain;

— made of timber layers which are made of strength graded timber according to EN 14081-1;

— made of timber layers having thicknesses between 6 mm and 60 mm (including) taking into account the layup requirements given in EN 16351;

— made of timber layers which may be edge bonded or which are not bonded and have spacing less than 6 mm between adjacent laminations;

— which may comprise wood based panel layers made of structural plywood, structural laminated veneer lumber (LVL) or structural solid wood panels having thicknesses between 6 mm and 45 mm (including) for which joints between wood based panels in a layer are disregarded;

— bonded with adhesives, fulfilling the requirements given in this European standard;

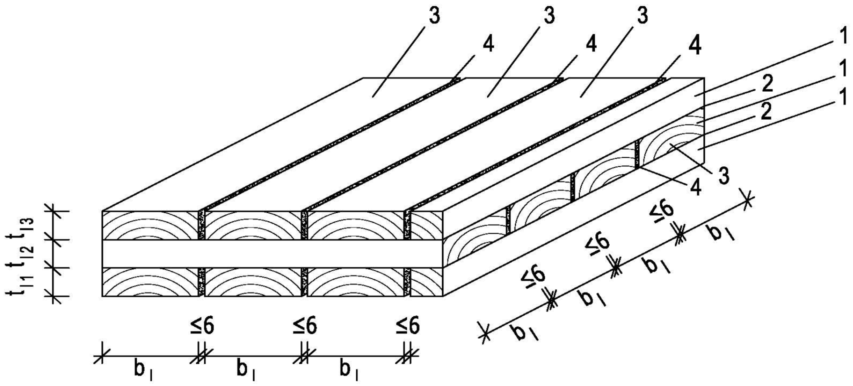
— having overall thicknesses up to 500 mm;

— which may comprise large finger joints as given in EN 16351;

— which may be curved;

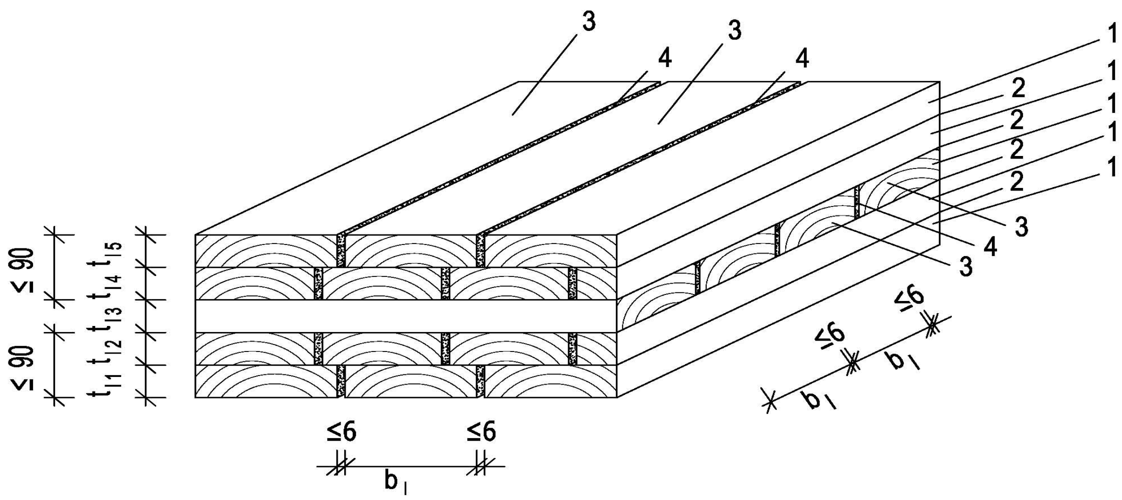
— which may be preservative treated against biological attack.

Figures 1 to 3 show examples of cross-laminated timber layups according to EN 16351:2015.



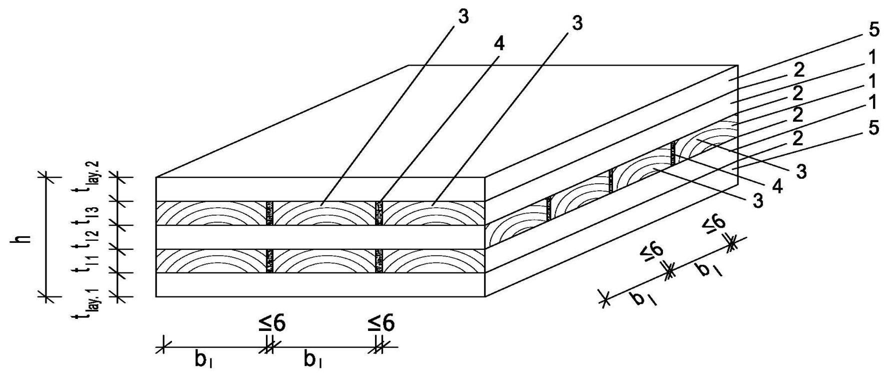
1 Timber layer with 6 mm ≤ t_t ≤ 45 (60) mm (depending on the position of the layer)
2 Bond line between layers
3 Lamination
4 Gap between lamination (if any)

Fig. 1 Example of a three-layered X-lam made of timber layers.



1 Timber layer with 6 mm ≤ t_t ≤ 45 mm
2 Bond line between layers
3 Lamination
4 Gap between lamination (if any)

Fig. 2 Example of a X-lam comprising adjacent timber layers being bonded parallel to the fibre direction.



1 Timber layer with 6 mm ≤ t_t ≤ 45 mm
2 Bond line between layers
3 Lamination
4 Gap between lamination (if any)
5 Wood based panel layers with t_l ≤ 45 mm (the sum of all wood based panel layer thicknesses shall not exceed 50% of the total thickness of the member

Fig. 3 Example of a X-lam comprising wood based panel layers.

2.3 Provision given in EN 16351

EN 16351 provides information on the determination of the performance characteristics (including provisions for tests), quality control (procedures for Assessment and Verification of Constancy of Performance) and marking.

Within EN 16351 neither a strength model for the determination of X-lam strength, stiffness and density properties from the respective layer properties nor X-lam strength classes are given. Strength, stiffness and density properties are declared by declaring the layer properties and the relevant geometrical data, e.g. cross sectional sizes, layup, layer thickness and orientations, presence of grooves, presence of edge bonds and ratio of lamination width to lamination thickness.

It has to be discussed, if a strength model and strength classes could be implemented in a future version of EN 16351.

In Figure 4 it is demonstrated how such a declaration could look like.

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EN 16351:2015

Cross laminated timber –Spruce layup ABC

For the manufacture of structural elements to be used in buildings and bridges

Modulus of elasticity, bending strength, compressive strength, tensile strength and shear strength	
– Geometrical data as	
– cross sectional size *(mm)*	2 450 x 211
– layup (mm/orientation)	42l – 42w – 43l – 42w – 42l
– edge bonds	No structural edge bonds
– grooves in laminations	No grooves
– ratio of b_l / t_l	>4
– strength, stiffness and characteristic density of layers	C30 – C24 – C24 – C24 – C30
– bending strength of finger joints in laminations as bending strength of timber	C30 – C24 – C24 – C24 – C30
Bonding strength as	

Fig. 4 Part of a CE-mark showing the declaration of X-lam strength, stiffness and density declared as layer properties and geometrical data (Source: CEN, Brussels).

3. New provisions for the design of cross laminated timber within a future Eurocode 5-1-1

3.1 Current situation

The existing EN 1995-1-1 [10] does not give specific design rules for cross-laminated timber as there had not been enough experience when the standard was finalized. Since then cross laminated timber has been widely used referring to design rules given in lots of scientific reports, national handbooks (see e.g. [2] to [6], [11]) or European technical approvals and some National Annexes to EN 1995-1-1, see e.g. [12] and [13].

3.2 Approach for the implementation of specific X-lam design rules in EN 1995-1-1

As the current edition of EN 1995-1-1 lacks approaches to design X-lam members the European standardization committee responsible for Eurocode 5, CEN/TC 250/SC 5, decided to form a Working Group 1 “Cross laminated timber” to develop proposals for such design rules. CEN/TC 250/SC 5/WG 1 will be supported by the Project Team PT SC5.T.1, which will draft the new sections on cross-laminated timber and reinforcement.

Both, CEN/TC 250/SC 5/WG 1 and PT SC5.T.1, will not only draft design rules but will also propose necessary changes on the above mentioned harmonized product standard EN 16351.

3.3 Work items

3.3.1 General

When this paper was written both, the working group and the project team, had only one meeting and no consultation with the convenor or the plenary meeting. Therefore the work items listed below might be subject to substantial changes.

3.3.2 Work items regarding product properties

As described above EN 16351:2015 covers a wide range of product variations. Not all of those variations are commonly used and not for all variations sufficient knowledge for standardization exists.

A possible prioritization of standardization needs with regard to the product variations is shown in Table 1.

Table 1: Product variations for X-lam according to EN 16351:2015 to be dealt with in a future EN 1995-1-1.

Cross laminated timber	**Current proposal for a prioritization of standardization needs for EN 1995-1-1**
— made from timber laminations made from specific coniferous species and poplar — having a symmetrical cross sectional layup — comprising laminations with or without grooves — comprising timber layers which are edge bonded or not edge bonded — with or without large finger joints	
— only comprising adjacent layers being crosswise oriented and — being assigned to one strength class	A
— only comprising adjacent layers being crosswise oriented and being assigned to more than one strength class	B
— comprising adjacent layers being parallel bonded and — being assigned to one or more strength class(es)	B
— being curved in one direction	C or not be taken into account
— also comprising wood based panel layers	Will not be taken into account

3.3.3 Work items regarding the design

CEN/TC 250/SC 5/WG 1 and PT SC5.T.1 intend to give specific design rules for

- diaphragms, plates and beams without or with large finger joints;
- design of cross sections including e.g. determination of $k_{c,90}$ values;
- combination of stresses for members subjected to loads in and perpendicular to the plane;
- the design of stress concentrations in different directions, e.g. point loads, supports of plates, load transfer through a diaphragm etc.;
- system factors, especially in the case of small elements comprising a limited number of laminations;

— the design of lintels (architraves);

— the determination of swelling and shrinkage values of X-lam depending on the layup.

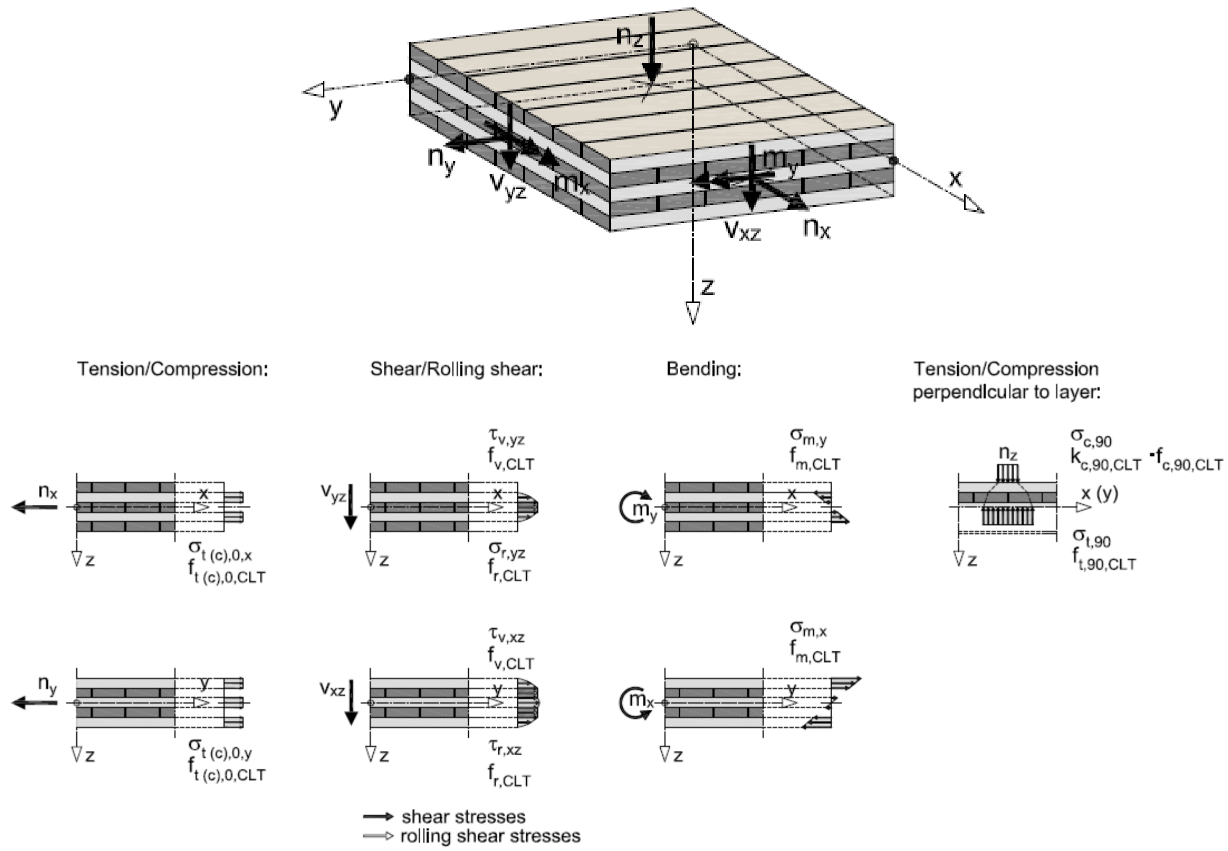

Fig. 5 X-lam plate and diaphragm with designation of forces, moments, stresses and strengths (Source: TU Graz).

Local reinforcements, e.g. using self-tapping screws will not be covered.

— The groups currently discuss, if specific rules for the arrangement of openings or the slenderness of structural diaphragms are needed.

— Regarding the calculation of stresses and strains in diaphragms and plates it is intended either to refer to [1] or to publish the content of this publication as a Technical Specification.

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Minutes of Presentation II:
CLT – Standardisation of the Product and its Design

Presentation by Tobias Wiegand

Summary:

Tobias Wiegand presents the on-going standardisation work for CLT. The new product standard EN 16351 has recently been published and covers the following topics:

- Definition of CLT, layer setup, dimensions, edge bonding, grooving
- Strength grading
- Adhesives according to EN 14080
- CLT only in Service classes 1 and 2
- Treated or untreated against biological attack
- Limitation to wood species according to EN 14080
- Inclusion of wood based panels
- Large finger joints in CLT
- Limited provision are given for curved CLT

Future developments of the standard will focus on:

- Developing a separate standard for the topics related to adhesives used in glued timber products
- Including test setups and test provisions for CLT into EN 408
- Standardizing the regulations regarding fire reaction

On-going development in standardization deals with the design of CLT elements. Future topics to be dealt with include:

- Connections, fire resistance, seismic behaviour of CLT

So far only CLT used as wall or floor elements will be included, in future CLT beams may be included as well.

The suggested partial safety factors and k_{mod} values are similar to glulam. The suggested statements on moisture influences and deformations are similar to plywood.

Additional conditions for vibration have to be evaluated as the current provisions in Eurocode 5 do not well cover typical CLT structures

The strength model for CLT has to be further developed and may be included in EN 16351.

Discussion:
Francois Colling asks about specifications of strength and stiffness properties for multi-layered panels.

Tobias Wiegand replies that information can be found in technical application documents since this also depends on national rules and technical approvals.

Eric Borgström asks for the correct naming of CLT in standards, since CLT has been made a trademark.

Tobias Wiegand replies that the name Xlam is used in the standards.

Basis of Design Principles – Application to CLT

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1. Introduction

A large proportion of the societal wealth is invested in the continuous development and maintenance of the built infrastructure. It is therefore essential that decisions in this regard are made on a rational basis. A structural design code should be such a rationale that facilitates design solutions that balance expected adverse consequences (e.g. in case of failure or deterioration) with investments into more safety (e.g. larger cross sections). Structural design codes are therefore calibrated on the basis of associated risks or, simplified, on the basis of associated failure probability. In this paper it is focused on the latter. Reliability based code calibration has been formulated by several researchers, see e.g. Ravindra and Galambos [1], Ellingwood et al. [2] and Rosenblueth and Esteva [3] and has been already implemented in several codes, see e.g. OHBDC [4], NBCC [5], and more recent the Eurocodes [6].

In the current version of EC 5 the design of cross-laminated timber (CLT) structures is not regulated. Due to the growing importance of CLT in the timber sector, it is one of the major goals of this COST Action[2] as well as for second generation of the Eurocodes to implement the design of CLT structures. In the present paper some general aspects, relevant for the implementation of CLT in European standards are summarized and discussed. Hereby it is particularly

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focused on the issues and challenges that are associated to the formulation of design equations that are consistent with the general philosophy of the Eurocodes as prescribed in EN 1990 [7]. At first a short introduction for the reliability based code calibration is presented. Here, the influence of the variability of the resistance of structural components on the partial safety factors is illustrated. In the second part of the paper, aspects relevant for the optimization of the partial safety factors (γm, γG and γQ) are discussed. That includes the influence of the material properties for different failure scenarios and the fabrication process. In the last part aspects essential for the development of a design concept for CLT are discussed.

2. Partial factor design and reliability based code calibration

2.1 Partial factor design concept

In the Eurocodes a semi-probabilistic design method is introduced through partial factor design that is generally applied to a wide range of design equations. The partial factor design format is formulated such that it facilitates the identification of acceptable and efficient design solutions concerning new structures. Basis is taken in generic information concerning load effects and material resistance in such a manner that the risk and the reliability of the structure are adequate and ensured in conformance to specified reliability requirements.

The partial factor design format comprises:

- consequence class categorization
- design situations
- design equations
- design values

Design equations are in general formulated for failure modes that may involve in principle both failure of individual cross sections of the structures, as well as failure modes involving the failures of several cross sections of the structures. The principle form of the design equation is given in Eq. (1) where z is a vector of design parameters (e.g. the cross sectional dimensions), $r_d(\mathbf{z})$ is the design value for the resistance, $s_d(\mathbf{z})$ is the design value for the action effect.

$$r_d(\mathbf{z}) - s_d(\mathbf{z}) > 0 \tag{1}$$

The design values for the various actions and material characteristics entering the design equations are determined with due consideration of the uncertainties associated with the relevant loads and resistances in such a way that design solutions that fulfil inequality (Eq. 1) are in consistency with the reliability requirements. The design value of a basic variable is generally defined as the multiplication (for loads and load effects) or division (for resistances) of the characteristic value taken as a specified fractile value from the statistical distribution that is chosen to represent the basic variable with a partial safety factor.

The identification and derivation of partial safety factors in such a way that the corresponding design solutions are consistent with the reliability requirements is performed by code authorities and in general termed code calibration.

2.2 Target reliability

Reliability requirements are generally expressed as the target reliability index or the target probability of failure, both referring to a reference period of one year.

Failure of a structural component is defined as a nonfulfillment of its associated performance requirements. These can in general refer to serviceability limit state requirements (e.g. excessive deformation, vibration) or ultimate limit state requirements (e.g. instability, rupture). The present paper only focuses on ultimate limit state requirements.

The customary approach to describe failure is by using the limit state function $g(x)$, according to Eq. (2). Here, $\mathbf{x}$ are realizations of the random variables $\mathbf{X}$, representing all uncertainties. For structural components the limit state function can be expressed through resistance R and load S.

$$\mathbf{F} = \{g(\mathbf{x}) \leq 0\} \quad \text{with} \quad g(\mathbf{x}) = r - s \tag{2}$$

In the case of a bending failure, which is a typical ultimate limit state failure of e.g. glued laminated timber (GLT; glulam) beams, the limit state function is defined as $g(\mathbf{x}) = f_\mathrm{m} - \sigma_\mathrm{m}$; where, f_m denotes the bending strength (resistance of the structural member), and σ_m denotes the bending stresses (as a function of the applied load). It is obvious that each realization, where the bending strength of the GLT beam is smaller than the applied bending stresses $f_\mathrm{m} \leq \sigma_\mathrm{m}$ leads to failure. Taking into account the entire range of the random variable $\mathbf{X}$, the probability of failure can be described using Eq. (3). Here $f_\mathbf{X}$ is the joint probability function of the variable $\mathbf{X}$.

$$P_f = P(g(\mathbf{X}) \leq 0) = \int_{g(\mathbf{x}) \leq 0} f_\mathbf{X}(\mathbf{x})\, d\mathbf{x} \tag{3}$$

The structural reliability is alternatively expressed with the so-called reliability index β (Eq. 4). A common value for the yearly target reliability index is $\beta \approx 4.2$, which corresponds to a yearly probability of failure $P_f \approx 10^{-5}$ (JCSS [8]). (Note that EN 1990 prescribes a yearly target reliability index of 4.7, design solutions according to existing codes, however, generally do not reach that high reliability.)

$$\beta = -\Phi^{-1}(P_f) \tag{4}$$

In general, both the applied load S and the resistance R are functions of time. In many cases the applied load shows a large variability over time, as they originate from environmental conditions (snow, wind) and the use of the structures. The resistance of a structural member is also a function of time; e.g. decreasing resistance over the time through deterioration processes.

For practical application, it is often possible to simplify problems such that they can be considered as time-independent reliability problems. If, e.g. the structural resistance is not affected by the time history of a variable load effect, it is sufficient to consider the variable load with its extreme realizations, whereas the extremes are always associated to a references time interval, e.g. one year.

However, if the resistance is affected by the time history of the load a so-called time dependent reliability analysis is necessary, see e.g. Melchers [9] and Faber [10]. In timber engineering the consideration of time history effects is necessary, e.g. for the assessment of duration of load effects.

2.3 Code calibration example

As an example a partial factor design equation (as generally introduced in (1) is given in Eq. (5). Here R_k, G_k and Q_k are the characteristic values of the resistance R, the permanent load G, and the time variable load Q. γ_m, γ_G and γ_Q are the corresponding partial safety factors. z is the so-called design variable, which is defined by the chosen dimensions of the structural component.

$$z\frac{R_k}{\gamma_m}-\gamma_G G_k-\gamma_Q Q_k=0 \tag{5}$$

The characteristic values for both load and resistance are in general defined as fractile values of the corresponding probability distributions. In the Eurocodes the following characteristic values are defined: R_k is the 5 % fractile value of a Lognormal distributed resistance, G_k is the 50 % fractile value (mean value) of the Normal distributed load (constant in time), and Q_k is the 98 % fractile value of the Gumbel distributed yearly maxima of the load (variable in time).

The corresponding partial safety factors can be calibrated to provide design solutions (z) with an acceptable failure probability P_f (Eq. 6). Here R, G, and Q are resistance and loads represented as random variables, $z^* = z(\gamma_m, \gamma_G, \gamma_Q)$ is the design solution identified with Eq. (5) as a function of the selected partial safety factors, and X is the model uncertainty.

$$\begin{aligned} &P_f=\mathrm{P}\left\{g\left(X,R,G,Q\right)<0\right\} \\ &\text{with} \\ &g\left(X,R,G,Q\right)=z^* XR-G-Q=0 \end{aligned} \tag{6}$$

In general, different design situations are relevant; i.e. different ratios between G and Q. This can be considered using a modification of Eqs. (5-6) into Eqs. (7-8). α_i might take values between 0 and 1, representing different ratios of G and Q. $\hat{R}$, $\hat{G}$, and $\hat{Q}$ are normalized to a mean value of 1. For each α_i one design equation exists, thus altogether n different design equations have to be considered.

$$z_i \frac{\hat{R}_k}{\gamma_m} - \gamma_G \alpha_i \hat{G}_k - \gamma_Q (1-\alpha_i) \hat{Q}_k = 0 \tag{7}$$

$$g_i\left(X, \hat{R}, \hat{G}, \hat{Q}\right) = z_i^* X \hat{R} - \alpha_i \hat{G} - \left(1-\alpha_i\right) \hat{Q} = 0 \tag{8}$$

The partial safety factors (γ_m, γ_G, and γ_Q) are calibrated by solving the optimisation problem given in Eq. (9).

$$\min_{\gamma} \left[\sum_{j=1}^{L} \left(\beta_{target} - \beta_j \right)^2 \right] \tag{9}$$

In this paper, the reliability based code calibration is just briefly introduced to illustrate the influence of uncertainties (load and resistance), in respect to codes. Please find more information, e.g. in JCSS [8] and Faber & Sørensen [11]. For examples of applications in the area of timber engineering see also Kohler et al. [12].

The design equation for a bending beam can be calibrated according to the procedure described above. For the calibration of the partial safety factors a range for the ratio between G and Q of $\alpha = \left[0.1, 0.2, ..., 0.8\right]$ is typical, i.e. the rather unrealistic design situations with less than 10 % and more than 80 % permanent load are excluded from the optimization.

One further aspect that has to be considered is the philosophy of the Eurocodes that the partial safety factors for the loads are material independent; therefore it is reasonable to fix γG and γQ and perform the optimization only subject to γm.

3. Variability of the material properties – influence on the partial safety factors

3.1 General

As mentioned above, the probability of failure and thus the reliability of a structure are directly related to the variability of the material used. In Fig. 2 and Fig. 3 the importance of the coefficient of variation (COV) is illustrated; one time in respect to the reliability index and one time in respect to the failure probability. In this example one constant load G (Normal distributed, COV = 0.10) and one variable load over time Q (Gumbel distributed, COV = 0.40) were used. The model uncertainty was assumed Lognormal distributed with COV = 0.10. The resistance (e.g. the bending strength) was assumed Lognormal distributed with different COVs. The partial safety factors were $\gamma_m = 1.25$, $\gamma_G = 1.35$ and $\gamma_Q = 1.50$. The calculation was performed with code cal [13].

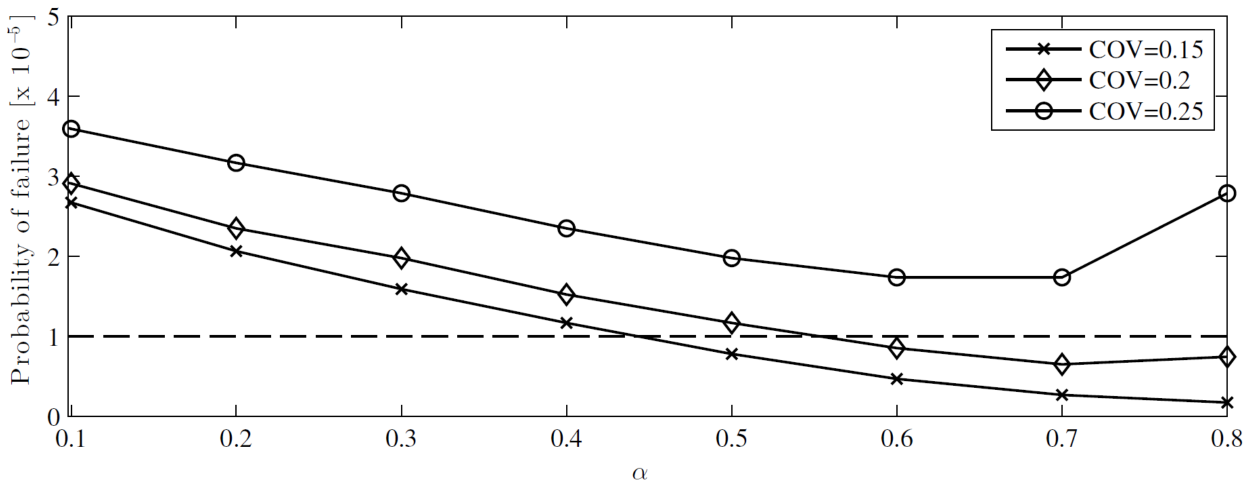

Fig. 2 Probability of failure for different COVs (γ_m = 1.25, γ_G = 1.35 and γ_Q = 1.50).

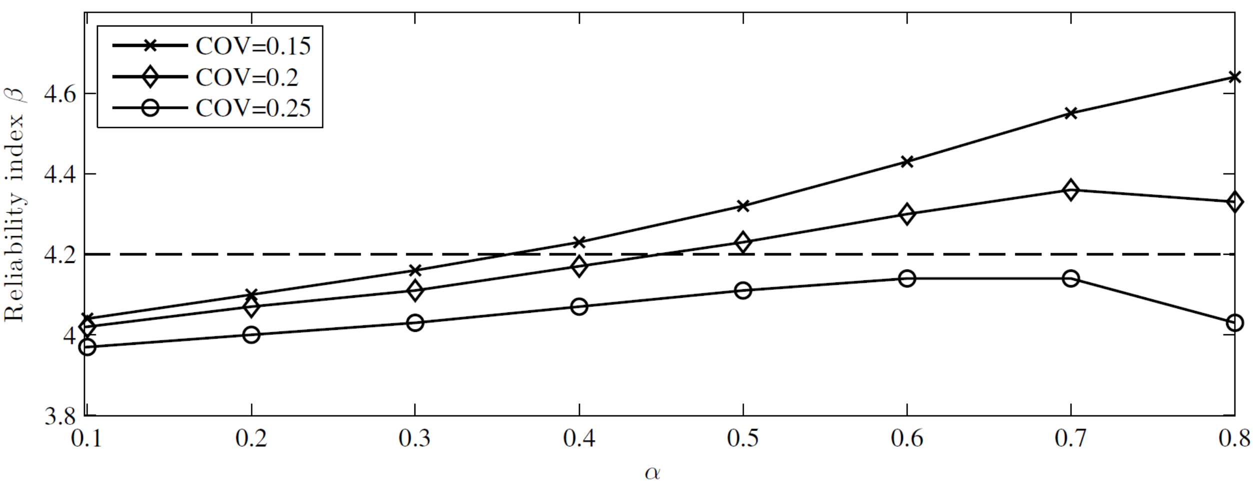

Fig. 3: Reliability index for different COVs (γ_m = 1.25, γ_G = 1.35 and γ_Q = 1.50).

The influence of the coefficient of variation COV of the resistance (material properties) on the failure probability (Fig. 2) and on the reliability index (Fig. 3) is illustrated for different ratios between *G* and *Q*. It is obvious that the reliability index is smaller for larger COVs and thus the probability of failure is larger. However, the differences are significant; e.g. the probability of failure for α = 0.6 are $P_f = 0.47 \cdot 10^{-5}$, $P_f = 0.85 \cdot 10^{-5}$ and $P_f = 1.75 \cdot 10^{-5}$ for COV = 0.15, COV = 0.20 and COV = 0.25, respectively. In other words, for this load situation the probability of failure is 3.7 times higher when the variability of the material properties is COV = 0.25 instead of COV = 0.15.

Taking into account the target reliability (dashed line) it is obvious that the chosen partial safety factor γ_m = 1.25 is too small for material with a large variability of the material properties and too conservative for material with a small variability. However, using the approach described above the partial safety factor γ_m can be optimised according to Eq. (9). The results are summarized in Table 1; optimised for a range $\alpha = \left[0.1, 0.2, ..., 0.8\right]$.

Table 1: Calibrated partial safety factors for the resistance for different COVs (for constant γ_G = 1.35 and γ_Q = 1.50).

COV of the resistance (Lognormal distributed)	γ_m
0.15	1.20
0.20	1.24
0.25	1.33

3.2 Strength and stiffness related behavior of CLT standard test specimen vs. structural components

In the above example, a relationship between the material resistance coefficient of variation (COV) and the reliability and corresponding partial safety factor is established. It is important to consider not only the COV of a typical material test sample in this regard. The variability has to represent all uncertainties associated with the prediction of the load bearing capacity of the CLT product in the structure as this property is addressed by the variable R above.

When modelling timber material properties in a structure, i.e. at any generic point, in time and in space, several issues related to timber grading, size effects and duration of load effects have to be taken into account, see also Köhler [15]. For engineered timber products the situation is even more complex as the joint behavior of the assembled timber material has to be represented. Furthermore, does the production process of engineered timber products, i.e. the several ways how production can be performed, affect the variability and uncertainty of the properties of the product. In Fig. 4 the various aspects that influence the load bearing capacity of CLT at a generic point in the structure are illustrated. The base material for the production of CLT is graded structural timber. Graded structural timber is available in form of strength classes, i.e. classes of structural timber with specified target reference properties as timber density, timber bending MOE and timber bending MOR. The targets for the reference properties are expressed as fractile values from the corresponding anticipated probability distribution functions; 5 % fractile for the density and the bending MOR and 50 % for the bending MOE. All other strength and stiffness related properties of the graded structural timber are estimated based on the classification made based on the reference properties. Different base material strength classes can be used for the production of CLT and different production techniques exist to produce a classified and specified CLT product. The classified CLT product has assigned values for the strength and stiffness properties associated to different possible failure modes. These failure modes relate to standardized test set ups that are specified in order to imitate the loading and failure modes in real structures as close as possible. Test data from these standardized tests are taken to verify the strength and stiffness properties of the CLT product and to quantify the coefficient of variation of the measured properties. Together with the analysis of model calculations for these failure modes the entire production and classification process is calibrated and validated. However, the production and classification process is not perfect and under full control, so, beyond the uncertainty that is associated to the variability of measured test data the uncertainty due to the imperfect production and classification process has to be considered.

Let's assume the production process and the classification to the CLT product is perfect and under full control, leading to strength and stiffness related properties as specified and expressed with 5 % and 50 % fractile values together with known probability distribution functions and coefficients of variation (as measured in the verification experiments). Then the properties that are related to standardized tests still would have to be related to the strength and stiffness related properties in a generic point in the structure. Scale effects, duration of load and moisture effects and a possible combination of different loading modes also affect the relevant property here and have to be considered.

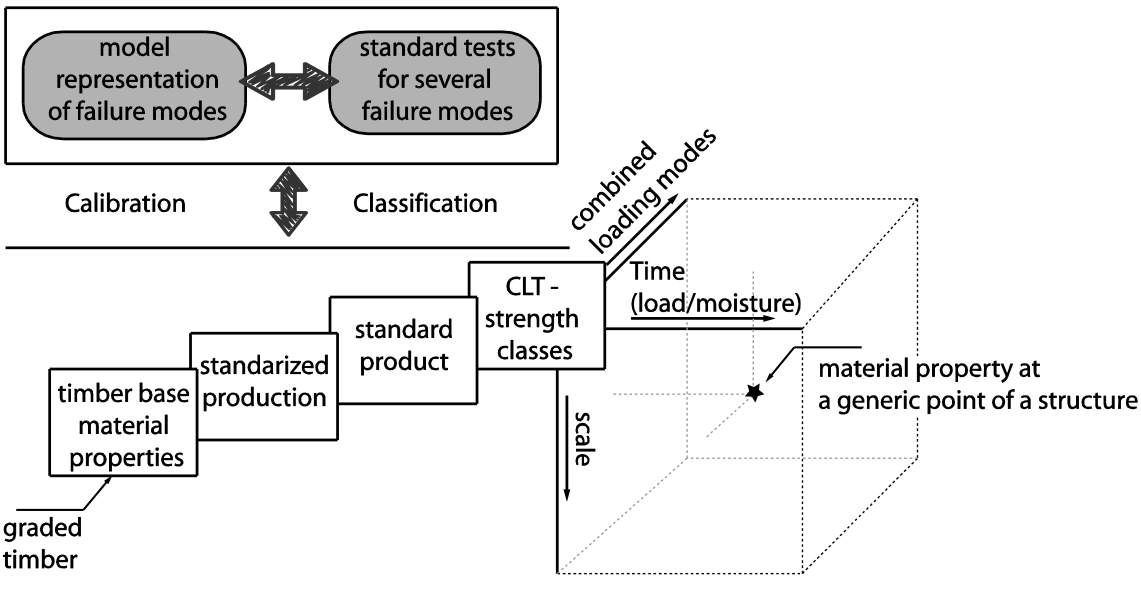

Fig. 4 Strength and stiffness related properties are relevant to represent in structural design assessment. However, this includes the consideration of various aspects.

3.3 Material properties of CLT for different failure scenarios (ULS)

In the current version of EC 5 one partial safety factor γ_m for each material is given. However, it is well established, that the variability as well as the distribution function of the different failure scenarios are rather different (see e.g. [14], [15]).

For solid timber the influence was already investigated by [16] and indicate a significant over- and underestimation for different failure modes.

However, following the principle of the EC a target reliability has to be met for every failure mode. In this chapter current design approaches for different failure scenarios (different material properties) are illustrated and discussed; it is referenced to the example presented in the BSPhandbuch [17]. It is only focused on selected material properties of CLT elements in respect to ultimate limit state design. Aspects regarding stability, connections, serviceability and so on are not considered in this paper.

3.3.1 Bending strength

One possible approach to estimate the characteristic value of the bending capacity of CLT is by using the analogies to GLT. As the bending strength of GLT and CLT are both directly related to the tensile strength of the lowest lamella(s) the approach seems to be appropriate.

For structures loaded in parallel a so-called system strength factor k_{sys}, commonly defined as the ratio between quantiles of system and element load bearing capacity, is allowed according to EC 5 [6]. The reason therefore is that very low realizations of an element capacity will not automatically lead to failure of the structure, as the weak element acts together with the adjacent elements. The effects of reinforcing due to mutual action between adjacent lamellas lead to a decrease of the variability of the system properties compared to that of the single elements. The most appropriate approach to consider the additional safety due to a reduced variability would be the reduction of the partial safety factor. However, a similar effect can be achieved by increasing the design value with $k_{sys} > 1$. At this point it has to be mentioned that, to be consistent with solid timber and other engineered wood products such as GLT, CLT elements should be treated as individual structural components. Thus it is of particular importance to identify the actual variability of the material properties, here the bending strength.

Looking at the experimental investigations performed by Jöbstl [17] it seems that the variability of bending capacity experiments, expressed as COV, is about $\mathrm{COV}[f_{m,CLT}] = 0.08$ to 0.16, thus similar as for GLT. In a closer view, a regressive course of the variability is found with decreasing values at an increasing number of parallel acting lamellas in the top layers outlining a typical behavior of parallel systems. The mentioned investigations were made on especially produced CLT elements featuring a homogeneous layup, i.e. all lamellas originated from the same strength class. However, common European technical approvals and assessment documents currently regulating CLT products on the market frequently allow a certain share of lamellas featuring lower properties, i.e. 10 % of lamellas within one layer of one strength class below the majority of the base material. Since these investigations important developments in the CLT production process have taken place. This has the potential of stabilizing the production and the final product, i.e. expressed by lower variability of the product properties. Recent investigations have shown that a higher variability in the base material (elements) leads to higher homogenization, i.e. reduced partial safety factors and increased k_{sys}, but approximately similar variabilities in parallel systems if compared to systems of elements with lower variability; e.g. Brandner [19].

3.3.2 Shear strength

The shear strength is needed either for the design of floor elements (CLT plates loaded perpendicular to the plane direction, i.e. out-of-plane) and for wall elements, i.e. CLT plates loaded in-plane direction. In CLT elements exposed to shear in-plane three different failure scenarios have to be distinguished: gross-shear, net-shear and torsion failure; see e.g. [20], [21], [22], [23]. Consequently, five different shear properties are required; two for CLT out-of-plane (shear and rolling shear) and three for in-plane. According to the BSPhandbuch [17], for floor elements a characteristic shear strength $f_{v,CLT,k} = 3.0$ MPa is recommended. More recently, in [24] a value of $f_{v,CLT,k} = 3.5$ MPa, in-line with regulations for GLT according to EN 14080 [25], is proposed. For CLT elements loaded in-plane differentiation in products featuring narrow-face bonded lamellas within layers and without narrow-face bonding is made. Corresponding values are $f_{v,gross,k} = 3.5$ MPa, taking into account the gross cross section, and $f_{v,net,k,ref} = 5.5$ MPa, considering the layers only in the weak plane direction. In the latter case, verification of a potential torsion failure in the gluing interfaces between the layers has to be made; a value of $f_{T,node,k} = 2.5$ MPa is proposed; see e.g. [24], [26]. Following the experimental investigations in [26] for net-shear, shear modulus and density variability band-widths of $\mathrm{COV}[f_{v,net}] = 0.02$ to 0.08 (average COV = 0.04), $\mathrm{COV}[G_{\mathrm{CLT}}] = 0.03$ to 0.12 (average COV = 0.075), and $\mathrm{COV}[\rho_{CLT}] = 0.007$ to 0.026 (average COV = 0.016), were found by testing six to seven specimen taken from the same CLT element at each parameter setting. Considering this aspect the variability in net-shear is estimated by $\mathrm{COV}[f_{v,net}] \approx 0.06$.

3.3.3 Rolling shear

The rolling shear properties were investigated e.g. in Ehrhart et al. [29] by testing Norway spruce and other wood species. The results indicate a large influence of the width to thickness ratio, w_ℓ / t_ℓ. In particular for timber boards with a small ratio very low rolling shear properties were identified, which might be a result of the test configuration, i.e. increasing stress peaks with decreasing ratio w_ℓ / t_ℓ. However, performing standard test according to EN 408 [30] the characteristic strength value is about $f_{r,CLT,k} = 1.4$ MPa with $\mathrm{COV}[f_{r,CLT}] = 0.13$ to 0.22.

3.3.4 Compression strength perpendicular to grain

In Bogensberger et al. [31] an experimental investigation for compression perpendicular to grain is presented. The test campaign included the investigation of the location of the applied load (e.g. center or edge) as well as the gauge length. The outcome of the investigation was a recommendation of a characteristic value 2.85 MPa, thus about 14 % larger as for GLT. More recently, Brandner and Schickhofer [32] report on a comprehensive test campaign conducted by Ciampitti

[33]. Considering these and previous test results on CLT elements found in literature, in comparison to GLT overall 30 % higher strength and modulus of elasticity in compression perpendicular to grain were concluded and a characteristic value of $f_{c,90,CLT,k}$ = 3.0 MPa together with COV[$f_{c,90,CLT}$] = 0.08 for the basic value is proposed.

When considering compression strength perpendicular to grain (test according to EN 408 [30]) it has be mentioned that the failure criteria is actually not an ULS; it is only an exceeding of a defined deformation. In this respect the calibration of the partial safety factors cannot be performed as introduced in Section 2. For this an additional parameter has to be considered: The probability of a structural failure given that the deformation exceeds or not exceed the threshold of the deformation.

Table 2: Compilation of some material properties' COVs of CLT.

Material property	COV
Bending strength (elements)	0.08-0.16
Net-shear strength (in-plane; elements)	0.06
Rolling shear (board segments)	0.13-0.22
Compression strength perpendicular to grain (basic value; prism)	0.08

3.4 Fabrication procedures

For the calibration of partial safety factors it is essential to know the product properties of CLT. Although the fabrication of CLT is meanwhile regulated on European level (EN 16351 [34]) current CLT producers still follow their individual approvals, i.e. European Technical Approval or Assessment documents. Thus the material properties of CLT as well as their variability cannot be described uniformly.

Due to the differences of the regulations combined with the rather low experience of at least some CLT producers (CLT is a rather young engineered wood product, thus the fabrication might be less optimized compared to e.g. GLT) the variability of the material properties identified within one individual campaign has to be treated carefully. In other words, it might be realistic that the results of the same experimental investigation might be rather different when using specimens fabricated by another producer. This is for example indicated in Brandner et al. [26], [27] presenting data from in-plane shear tests. In this study, samples of CLT elements from three different producers are shown outlining significant different mean densities although CLT with equal base material of nominal strength class C24 was requested. As tested shear properties are influenced by product parameters others than density a comparable conclusion for these properties cannot be made.

However it is apparent that the variability in shear properties as well as density seems not to be significantly influenced by the producer. With focus on properties of the population and by assuming comparable variabilities between samples, differences in mean values may result in variabilities of the basic population significantly larger than in individual studies; e.g. the variability of the bending strength of all CLT plates (of one strength class) fabricated in Germany might be larger than the variability from one study (one fabricant, one growing region, etc.). As already outlined in Section 3.3.1, the possibility to use a certain share of base material with lower properties than the majority may additionally cause higher variabilities than in CLT of a homogeneous base material, although this aspect is judged to be only of minor relevance if the amount of elements acting in parallel in this laminar, two-dimensional product is large enough.

Taking this into account larger COVs have to be used when performing the reliability analysis.

3.5 Fire resistance

For the design of structural timber members at normal temperature the 5% fractile values are used for the material properties (e.g. strength properties); according to EC 5 [6]. In contrast, for the fire design of structural timber members, EC 5 (part 2) [34], gives conversion factors to enable design with 20 % fractile values. That reflects the results of traditional fire codes in Europe (for a detailed description see König [36], [37]).

The approach for fire design of structural timber members is different to other materials; e.g. concrete still use 5 % fractile values in the fire situation. This contradiction has been recognized in the scientific community. Motivated by this, a research project titled "Reliability based design of timber in fire", with the objective to analyze the current approach for the fire design of timber members based on EC 5 and the determination of the required safety factors in case of fire based on reliability analysis, is currently performed at ETH Zurich within the framework of COST FP 1404 activities [38].

For the implementation of CLT for fire design into the new version of EC 5 this issue should be covered first before optimizing design solutions for fire exposure.

3.6 Modification factor k_{mod}

One of the distinctive characteristics of timber is that its strength is influenced by the intensity and the duration of the applied stresses. Although this phenomenon is similar to that of fatigue in metals, strength degradation in timber is observed even under static (permanent) loading. This effect is referred to as the duration of load (DOL) effect. Numerous experimental programs have focused on the investigation of the DOL effects in clear wood specimen and later on also in full size timber components [39]. A variety of models have been proposed to describe the phenomenon. Hereby, it has been mainly focused on the duration of load effect of bending specimen. Some of the proposed models have a physical hypothesis of the phenomena as a basis; however, they all consist of variable model parameters, which can be calibrated to observed experimental data. The domain of experimental evidence is thus rather limited and it is always the question of proper extrapolation to other applications in timber engineering. In absence of experimental investigations of the DOL effect for CLT it seems appropriate to assume a rather similar behavior for bending and tension (as it is generally done for glulam). However, this might be not true for other failure modes where also the long-term stress-strain behavior of the glue line is relevant.

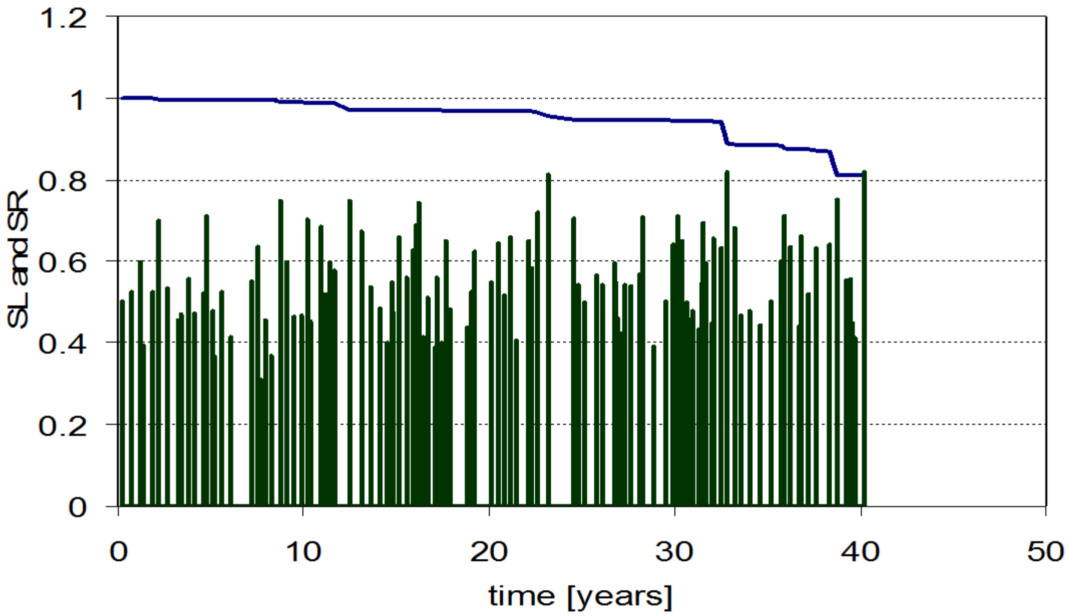

Fig. 5 Load and resistance time history. Results from simulation. [15]

The effect of duration of load was already mentioned in Section 2. A typical realization of $R(t)$ and $S(t)$ is illustrated in Fig. 5. It is obvious that the resistance of the structural component decreases over time, and thus probability of failure P_f increases over time.

In the present code formats this effect is represented by a modification factor k_{mod}; i.e. Eq. (5) modifies to:

$$k_{\text{mod}} z \frac{R_{k,t=0}}{\gamma_m} - \gamma_G G_k - \gamma_Q Q_k = 0 \tag{10}$$

The modification factor k_{mod} in Eurocode 5 can be chosen depending on the classification of characteristic climate and a classification of the characteristic load duration conditions of the structural component relevant in the design situation.

4. Conclusions & Outlook

The design of CLT structures is not regulated in the current version of EC 5. Due to the importance of CLT for timber structures, in particular for multi-story buildings, it is one goal of this COST to implement the design of CLT structures.

In the present paper some general aspects, relevant for the implementation of CLT in European standards were summarized and discussed.

5. Acknowledgements

The publication, as part of the proceedings from the Joint Conference of FP1402 & FP1404 "Cross Laminated Timber – a competitive wood product for visionary and fire safe buildings" held on the 10th March 2016 at KTH / Stockholm, was written in the framework of the COST Action FP1402 “Basis of Structural Timber Design – from research to standards”, (www.costfp1402.tum.de). The support from COST is highly acknowledged.

The comments and suggestions of Dr. Philipp Dietsch during the preparation of the manuscript are also highly acknowledged.

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Minutes of Presentation III:
Basis of Design Principles – Application to CLT

Presentation by Jochen Köhler

Summary:

Jochen Köhler explains the safety concept relevant for the implementation of CLT in design standards. It is important to also account for cost effectiveness. Code calibration is based on experience and theory.

The impact of new loading modes and other load domains for CLT compared to glulam has to be evaluated. Timber structures are often light structures with variable loads governing the design.

The variation of material properties has an impact on the design solution in order to achieve the target reliability index as suggested in EC0 and by JCSS. Decreasing COV due to the homogenization effects of CLT lead to different optimal partial safety factors.

A major challenge is the transfer of test results to the behaviour of the structural members in practice: sound sampling is necessary in order to be able to give safe predictions for e.g. samples of other origin, different layups or production lines or situation with different loading types.

The characteristic values used in the design equations have to represent the actual situation in practice with differences in scale, time, duration of load, moisture, combined loading, etc.

The low variation in test results from the laboratory may lead to an underestimation of the variability of the structural behaviour in practice. Hence, very strict product and material standards are required.

Discussion:

Joachim Schmid comments on individual CE products strength classes outside of harmonized product standards hEN’s; are deviating partial safety factors expected from general strength classes where maybe higher safety is achieved.

Jochen Köhler mentions that, from a safety perspective very strict and prescribed regulations for the production are desirable.

Pierre Quenneville asks whether the production process or the natural variability of the material would be easier to control?

Jochen Köhler answers that we can benefit from the homogenization effect for the material properties of CLT.

ULS Design of CLT Elements – Basics and Some Special Topics

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Summary

Within the last decade cross-laminated timber (CLT) has become a very well known engineered timber product. The orthogonal, laminar structure allows its application as two-dimensional plate element (e.g. as wall or floor) as well as beam element, able to bear loads in- and out-of-plane. Our contribution related to the ultimate limit state (ULS) design of these elements is separated in two parts: the first part concentrates on the main procedures for common design situations of CLT elements used as wall and floor elements. Within the second part some special topics, e.g. the design of CLT used within ribbed floors, the local load introduction in walls and floors and buckling of CLT by including two-dimensional load carrying behaviour are addressed.

1. Introduction

Apart from some national application documents for Eurocode 5 (EC 5), for example [1,2], design regulations for CLT elements are still missing in European design standards. Meanwhile, CLT has to be designed according to numerous, product-specific technical approvals. Harmonisation of these currently available design procedures is imperative and envisaged for the revision of EC 5. Currently, some guidelines are available that are based on the design concept anchored in the European design codes and summarise findings of numerous research projects, for example [3-5]. Furthermore, there are also guidelines based on other design concepts and written for nations outside of Europe, for example the CLT Handbook for the Canadian market [6] and for the US market [7], which are based on North American standards.

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Within the following sections, peculiarities of CLT in the ultimate limit state (ULS) design process with respect to the classification in elements exposed to loads in-plane and out-of-plane as well as some special topics (e.g. ribbed floor elements and local load introduction) are discussed.

2. General Comments

Therein presented verifications are restricted to CLT elements with homogeneous layup, i.e. equal base material's strength class in all layers. This is because of the circumstance that so far published load-bearing models and corresponding strength values of CLT elements are in general referred to this homogeneous layup.

Furthermore, we only concentrate on the "cold design" of CLT elements. Although required, accidental load cases like fire and seismic actions are not discussed. A state-of-the-art summary on fire design can be found e.g. in [8].

3. Loads out-of-plane

Within the following sub-sections the most common procedures for ultimate limit state (ULS) design of CLT, e.g. if used as floor and roof elements, are presented. Due to this limitation serviceability limit state (SLS) design of CLT floor elements, i.e. design in respect to deflections and vibrations, which frequently determines the overall design of these elements, is not part of this contribution. The interested reader is kindly referred e.g. to [9,10].

One specific aspect of CLT, as an orthogonal laminar structure in comparison to unidirectional timber elements, e.g. glued laminated timber (glulam; GLT), is its shear flexibility due to rolling shear in the cross layers. Thus, consideration of the influence of shear is mandatory. Because of constraints in the beam theory by Euler-Bernoulli with respect to shear other theories are required, e.g. the γ-method [11-14], the shear analogy method [15-17] and the transverse shear-flexible beam according to Timoshenko [18,19]. A comparison of these theories, outlining their individual advantages and disadvantages, is given in [20]. In summary, within a practical relevant range of $l_{CLT} / t_{CLT} \geq 15$, with l_{CLT} and t_{CLT} as length (span) and thickness of a CLT element, respectively, all these approaches are comparable and

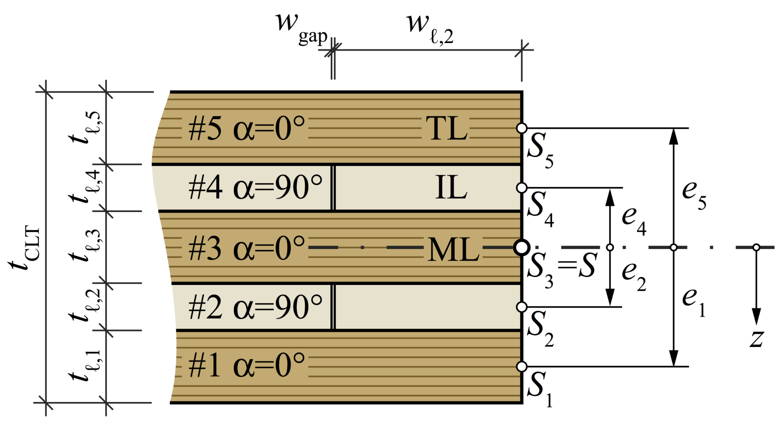


Fig. 1 Definitions of dimensions and distances, exemplarily on a five-layer CLT element; TL, IL and ML as top, intermediate and middle layer.

applicable. However, consistency between the design method used for calculation of stiffness and stress and the method applied for examination of experimentally determined strength and stiffness properties has to be ensured.

3.1 Bending

In CLT elements loaded out-of-plane, the maximum design bending stress on the edge, $\sigma_{max,d}$, has to be less than or equal to the design value of the bending strength for CLT, $f_{m,CLT,d}$; see Eq. (1).

$$\frac{\sigma_{max,d}}{f_{m,CLT,d}} \leq 1.0 \tag{1}$$

The individual layer orientation and corresponding material parameters have to be taken into account when calculating stresses (e.g. bending stresses, $\sigma(z)$; see Eq. (2)) and stiffness values (e.g. bending stiffness, K_{CLT}; see Eq. (3)); with M, z, $E(z)$, E_i, I_i, A_i and e_i as bending moment, coordinate, modulus of elasticity for given z, modulus of elasticity, moment of inertia and cross section related to layer i, and distance between the centres of gravity of the i^{th} layer, S_i, and of the CLT element, S, respectively; see Fig. 1.

$$\sigma(z) = \frac{M}{K_{CLT}} \cdot z \cdot E(z) \tag{2}$$

$$K_{CLT} = \sum(E_i \cdot I_i) + \sum(E_i \cdot A_i \cdot e_i^2) \tag{3}$$

Common CLT elements show a limited possibility to transfer normal stresses (tension and compression perpendicular to grain) in the cross layers. The reasons are product specific regular or irregular gaps in CLT produced without narrow face bonded lamellas, as well as cracks, at least in the top layers, in CLT with narrow face bonded lamellas as consequence of swelling and shrinkage in use. Thus, the calculation of bending stresses can be simplified by $E_{90,lay} = 0$. In doing so, the bending stresses in longitudinal layers increase and are therefore on the safe side.

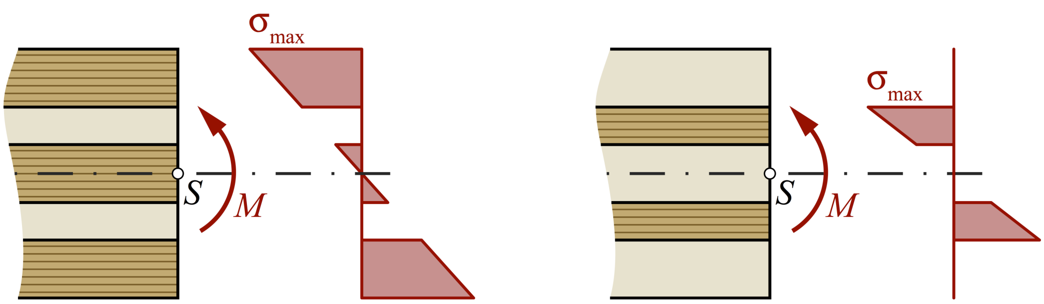


Fig. 2 Normal stress distribution over the cross section of a CLT element loaded in bending out-of-plane assuming $E_{90,lay}$ = 0: longitudinal layers as top layers (left); cross layers as top layers (right).

3.2 Shear

The distribution of shear stresses over the cross section of CLT elements loaded out-of-plane can be calculated according to Eq. (4). The assumption of $E_{90,\mathrm{lay}} = 0$ leads to constant instead of parabolic rolling shear stresses in cross layers. The shear stresses are maximal at the CLT element's centre of gravity. However, due to the orthogonal structure of CLT the verification of shear and rolling shear is required; see Eq. (5). In longitudinal layers, a proof of shear stress, $\tau_{\mathrm{max,d}}$, vs. shear strength, $f_{\mathrm{v,lay,d}}$, and in cross layers a proof of rolling shear stress, $\tau_{\mathrm{r,max,d}}$, vs. rolling shear strength, $f_{\mathrm{r,lay,d}}$, has to be done; see Fig. 3.

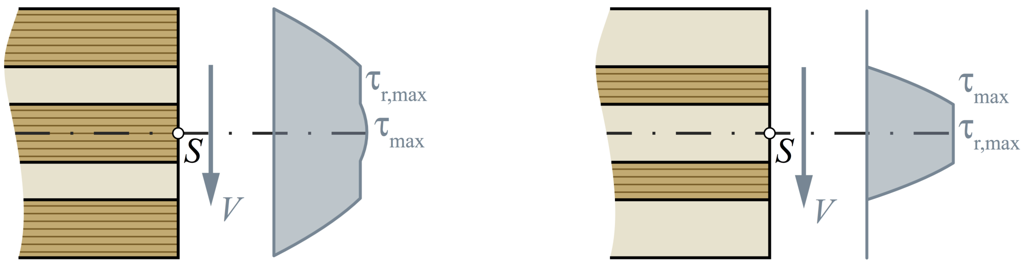


Fig. 3 Shear stress distribution over the cross section of a CLT element loaded out-of-plane assuming $E_{90,\mathrm{lay}}$ = 0: longitudinal layers as top layers (left); cross layers as top layers (right).

$$\tau(z_0) = \frac{V \cdot \int_{A_0} E(z) \cdot z \cdot \mathrm{d}A}{K_{\mathrm{CLT}} \cdot w(z_0)} \tag{4}$$

$$\frac{\tau_{\mathrm{max,d}}}{f_{\mathrm{v,lay,d}}} \leq 1.0 \quad \text{and} \quad \frac{\tau_{\mathrm{r,max,d}}}{f_{\mathrm{r,lay,d}}} \leq 1.0 \tag{5}$$

3.3 Compression Perpendicular to Grain

The characteristic properties of compression perpendicular to grain, i.e. modulus of elasticity, $E_{\mathrm{c},90}$, and strength, $f_{\mathrm{c},90}$, are based on tests conducted on small prismatic specimen whose surfaces are homogeneously and completely stressed in compression perpendicular to grain. However, for the design of structural elements, which are typically loaded only on a partial surface area, other and in general higher resistances and stiffness values can be applied. For compression perpendicular to grain strength this circumstance is considered by the coefficient $k_{\mathrm{c},90}$; see Eurocode 5 [21]. This coefficient is currently regulated for linear members and as function of the timber product and the load configuration. Apart from the Austrian National regulations for EC 5 [2], which contains $k_{\mathrm{c,90,CLT}}$ values for CLT for various load configurations, European regulations are missing.

Several attempts have been made to identify $k_{\mathrm{c,90,CLT}}$ values for CLT at various load configurations; see e.g. [22-26]. Recently, [27] summarized previous test results and analysed the applicability of van der Put's stress dispersion model; e.g. [28].

This model, originally developed for linear members, was adapted to be usable also for planar structural elements featuring a laminar structure with an orthogonal layup, which is considered by using separate load dispersion angles for longitudinal and cross layers with 45° and 15°, respectively.

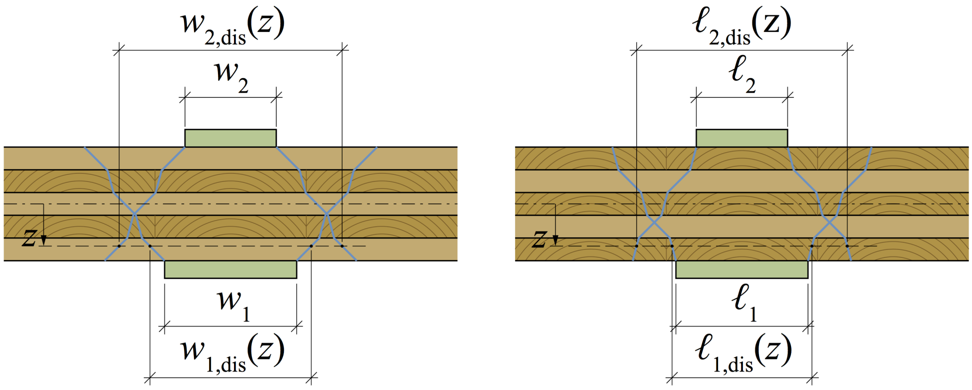


Fig. 4 Notations of the load distribution model for compression perp. to grain.

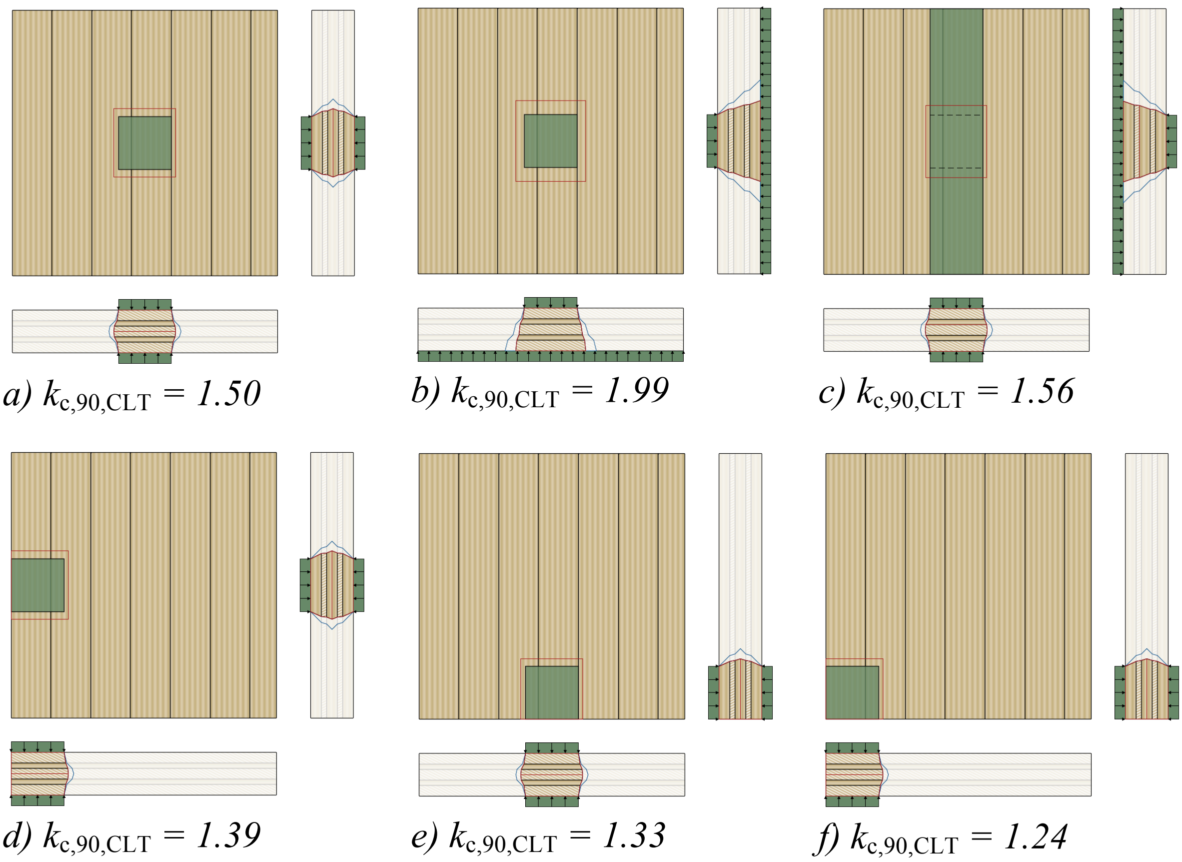


Fig. 5 $k_{c,90,CLT}$ e.g. for a five-layer CLT element with layup 40-20-40-20-40 (layer thicknesses from top to bottom, in [mm]) and loaded area 20 × 20 cm^2: a) central load transmission; b) central load introduction; c) wall on column; d) load trans. on edge, top layers parallel to edge; e) load trans. on edge, top layers perpendicular to edge; f) load trans. at corner.

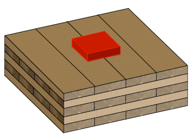
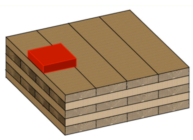
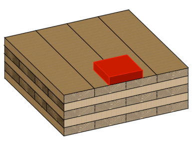
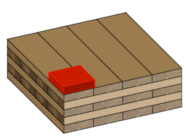
Consequently, although strength and modulus of elasticity determined on the basic cubic CLT specimen are approximately 30 % higher than for GLT, the reduced amount of stress dispersion in the cross layers results in lower $k_{c,90}$ values than found for GLT in comparable dimensions. Thus, $k_{c,90,CLT}$ depends on the layup and in particular on the share of cross layers in the CLT element as well as the loaded area. However, by using this new model approach a very good agreement between model and test results was found. Furthermore it was shown that the same $k_{c,90,CLT}$ values determined for strength can be also applied as multiplier for the modulus of elasticity in compression perpendicular to grain.

$$k_{c,90,CLT} = \frac{A_{c,ef}}{A_{c,sec}} \tag{6}$$

$$A_{c,ef} = \max\left[\min\left(\sqrt{w_{1,dis}(z)\cdot w_1};\sqrt{w_{2,dis}(z)\cdot w_2}\right)\cdot \min\left(\sqrt{\ell_{1,dis}(z)\cdot \ell_1};\sqrt{\ell_{2,dis}(z)\cdot \ell_2}\right)\right] \tag{7}$$

$$A_{c,sec} = \min(w_1;w_2)\cdot\min(\ell_1;\ell_2) \tag{8}$$

Table 1: Proposed band-widths of $k_{c,90,CLT}$ for narrow face bonded CLT elements point-loaded out-of-plane (loaded area 20 × 20 cm²) based on a base characteristic compression strength perpendicular to grain of $f_{c,90,CLT,k}$ = 3.0 N/mm², determined on a CLT prism.

load configuration		number of layers	$k_{c,90,CLT}$ transmission [1)]	$k_{c,90,CLT}$ introduction [1)]
	central	3	1.14 – 1.37	1.27 – 1.76
		5	1.29 – 1.63	1.49 – 2.26
		7	1.52 – 1.88	2.04 – 2.77
	edge, parallel to grain of top layer	3	1.11 – 1.29	1.23 – 1.57
		5	1.19 – 1.47	1.38 – 1.91
		7	1.41 – 1.64	1.79 – 2.24
	edge, perpendicular to grain of top layer	3	1.09 – 1.26	1.18 – 1.51
		5	1.17 – 1.44	1.33 – 1.86
		7	1.35 – 1.61	1.68 – 2.20
	corner	3	1.07 – 1.18	1.13 – 1.35
		5	1.12 – 1.30	1.23 – 1.57
		7	1.25 – 1.41	1.48 – 1.78

[1)] see Fig. (5)

All these investigations were conducted only on locally loaded but completely full-surface supported specimen (load configurations termed as “introduction”) or on locally, point or line loaded and supported specimen (load configurations termed as “transmission”) featuring equal load and support areas vertically aligned to each other. A more general formulation for introduction and transmission load configurations is outlined in Eqs. (6–8) and illustrated in Fig. 4; the application of this approach is exemplarily demonstrated in Fig. 5.

Table 2: Proposed band-widths of $k_{c,90,CLT}$ for narrow face bonded CLT elements line-loaded out-of-plane (load width 20 cm) based on a base characteristic compression strength perpendicular to grain of $f_{c,90,CLT,k}$ = 3.0 N/mm², determined on a CLT prism; figures acc. to [24].

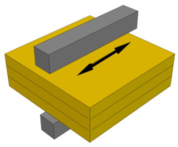
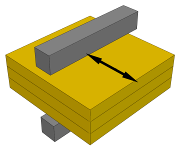
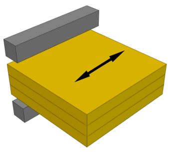
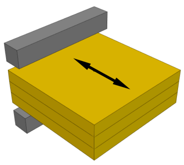
load configuration		number of layers	$k_{c,90,CLT}$	
			transmission	introduction
	central, parallel to grain of top layer	3	1.04 – 1.14	1.08 – 1.27
		5	1.09 – 1.25	1.18 – 1.46
		7	1.17 – 1.35	1.33 – 1.62
	central, perpendicular to grain of top layer	3	1.09 – 1.21	1.18 – 1.38
		5	1.14 – 1.30	1.27 – 1.55
		7	1.25 – 1.40	1.46 – 1.71
	edge, parallel to grain of top layer	3	1.02 – 1.07	1.04 – 1.14
		5	1.05 – 1.13	1.09 – 1.23
		7	1.09 – 1.17	1.16 – 1.31
	edge, perpendicular to grain of top layer	3	1.05 – 1.10	1.09 – 1.19
		5	1.07 – 1.15	1.13 – 1.28
		7	1.13 – 1.20	1.23 – 1.36

Table 1 and Table 2 show band-widths of the coefficient $k_{c,90,CLT}$ for some transmission and introduction load configurations considering all European CLT products with narrow face bonding currently on the market. The calculation was made by means of the CLTdesigner [29]. According to these results standardisation of fixed values for $k_{c,90,CLT}$ requires differentiation in load configuration, including also splitting in transmission and introduction, as well as the number of layers in the CLT element. Even than an universal applicability requires regulations on a very conservative basis. For wide-spanned, line- or point-supported timber structures where compression perpendicular to grain may govern the design the implementation of the adapted stress dispersion model according to Eqs. (6–8), additional to tabulated values on a conservative basis, is recommended. Thereby regulations in respect to the gap execution, i.e. CLT with and without narrow face bonding and with and without gaps, are required.

4. Loads in-plane

In the following sub-sections, the procedures required for ULS design for the most common load situations of CLT elements loaded in-plane, e.g. CLT walls, are presented.

4.1 Compression

For members loaded concentrically and axially in compression Eq. (9) shall be fulfilled; with N_d as the design normal force, $A_{net,ef}$ as the effective net cross section (cross section of the effective share of layers oriented parallel to N_d; for homogenous layup and $E_{90} = 0$: $A_{net,ef} = A_{net}$, the cross section of layers oriented parallel to N_d), and $f_{c,0,CLT,net,d}$ as the design strength parallel to grain of CLT-element's net cross section.

$$\frac{N_d}{A_{net,ef} \cdot f_{c,0,CLT,net,d}} \leq 1.0 \tag{9}$$

However, in case of slender members loaded in compression, the possibility of a lateral buckling failure has to be considered. For this case, two design methods are available:

- verification according to the equivalent beam method;
- verification according to the theory of 2nd order.

Applying the equivalent beam method, Eq. (10) has to be fulfilled.

$$\frac{N_d}{k_c \cdot A_{net,ef} \cdot f_{c,0,CLT,net,d}} \leq 1.0 \tag{10}$$

Under Eq. (10) the compressive strength is reduced by the instability factor kc ≤ 1.0 which is calculated according to Eq. (12). This instability factor is a function of the relative slenderness, λrel, the shape of the cross section and the quality of manufacturing (straightness factor βc; see Eq. (14)). The relative slenderness, λrel, according to Eq. (15) is dependent on the ideal elastic buckling load, ncr; see Eq. (11). This equation also considers the shear flexibility, relevant in particular for CLT. The 5 %-quantile of the bending stiffness, KCLT,05, and of the shear stiffness, SCLT,05, is calculated by means of E0,lay,05, G0,lay,05 and Gr,lay,05, the 5 %-quantiles of the modulus of elasticity in grain direction, the shear and rolling shear modulus, respectively.

$$n_{\mathrm{cr}} = \frac{K_{\mathrm{CLT,05}} \cdot \pi^2}{l_{\mathrm{k}}^2 \cdot \left(1 + \frac{K_{\mathrm{CLT,05}} \cdot \pi^2}{S_{\mathrm{CLT,05}} \cdot l_{\mathrm{k}}^2}\right)} \tag{11}$$

$$k_{\mathrm{c}} = \min\begin{bmatrix} 1.0 \\ \dfrac{1}{k + \sqrt{k^2 - \lambda_{\mathrm{rel}}^2}} \end{bmatrix} \tag{12}$$

$$k = 0.5 \cdot \left(1 + \beta_{\mathrm{c}} \cdot \left(\lambda_{\mathrm{rel}} - 0.3\right) + \lambda_{\mathrm{rel}}^2\right) \tag{13}$$

$$\beta_{\mathrm{c}} = 0.1 \tag{14}$$

$$\lambda_{\mathrm{rel}} = \sqrt{\frac{A_{\mathrm{net,ef}} \cdot f_{\mathrm{c,0,CLT,net,k}}}{n_{\mathrm{cr}}}} \tag{15}$$

When doing the verification according to the theory of 2nd order, which is based on the equilibrium of the deformed system, then the effects of induced deflection on internal forces and moments are considered explicitly. This is expressed by a combined load situation where the normal force, N_{d}, interacts with the bending moment, M_{d}. In that case Eq. (16) shall be verified.

$$\left(\frac{N_{\mathrm{d}}}{A_{\mathrm{net,ef}} \cdot f_{\mathrm{c,0,CLT,net,d}}}\right)^2 + \frac{M_{\mathrm{d}}^2}{W_{\mathrm{ef}} \cdot f_{\mathrm{m,CLT,d}}} \le 1.0 \tag{16}$$

4.2 Shear

In CLT diaphragms exposed to shear in-plane in principle three different failure mechanisms have to be distinguished: (i) gross-shear failure of CLT elements featuring narrow face bonding by longitudinal shearing of all layers, and in CLT without narrow face bonding (ii) net-shear failure by exceeding the shear resistance in layers oriented in CLT's weak direction and (iii) torsion failure in the gluing-interfaces between the orthogonal layers; see Fig. 6 and [30-32].

Within a recent comprehensive experimental investigation, where several product parameters potentially influencing the shear properties of CLT in-plane were varied in a range typical for current CLT products, reliable failures in gross- and net-shear were achieved; see [33]. In conclusion, CLT elements featuring narrow face bonding may fail in gross-shear whereas elements without narrow face bonding may fail either in net-shear or in torsion. The main parameters influencing the shear properties are the gap execution (narrow face bonded, without narrow face bonding and without / with gaps, with decreasing properties in mentioned order) and in case of CLT without narrow face bonding the layer thickness (decreasing properties with increasing thickness). Following these aspects, for the design differentiation in CLT diaphragms without and with narrow face bonding has to be made. In the following a design concept for CLT diaphragms, as proposed in [33], is presented. Verification procedures for linear CLT elements loaded in-plane, e.g. CLT elements used as girders or lintel beams, can be found e.g. in [31].

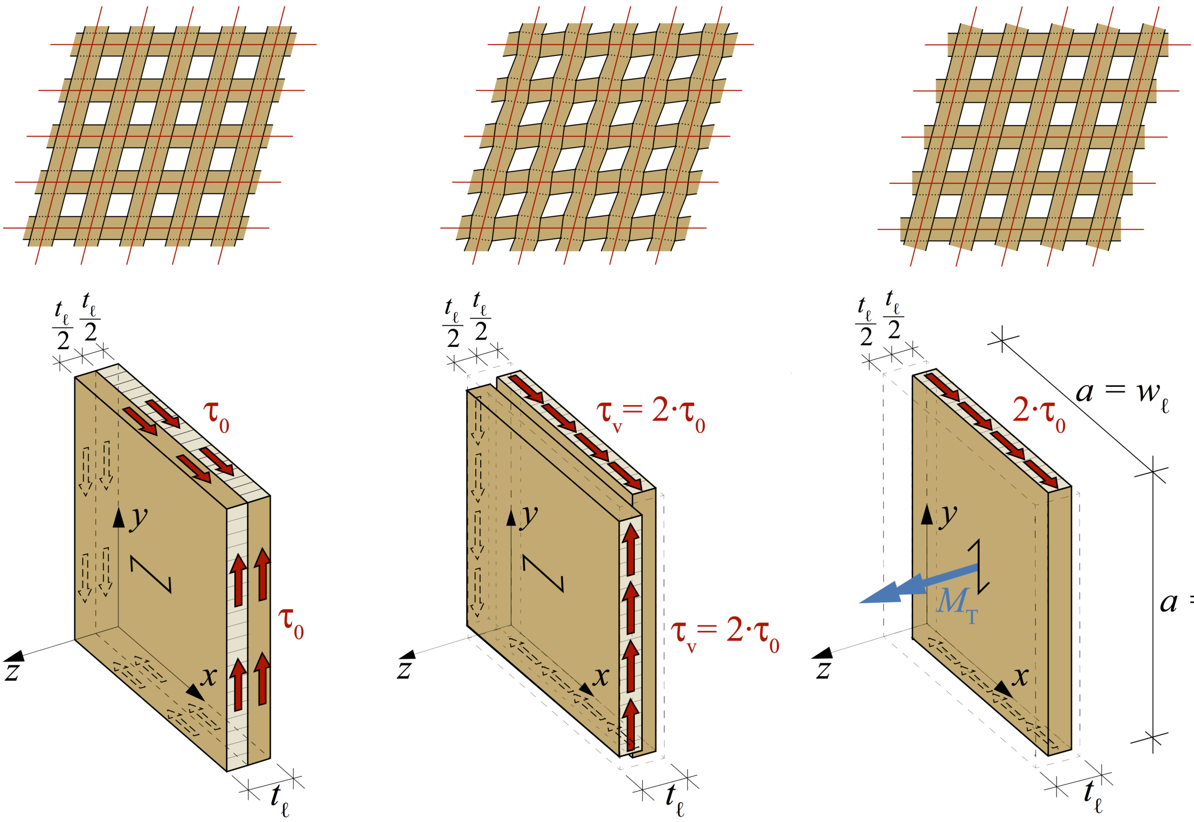


Fig. 6 Shear stresses in CLT diaphragms – failure mechanisms: gross-shear (left); net-shear (middle); torsion (right); figures below based on a RVSE according to Bogensperger et al. (2010).

4.2.1 Verification of CLT Diaphragms without Narrow Face Bonding

The distinct relation between layer thickness and net-shear strength in CLT elements without narrow face bonding consequence a dependency of the shear strength on the layup parameter (ratio between sums of layer thicknesses in weak and strong direction) when calculated based on the gross cross section. Consequently, a verification concept based on the net-shear strength and the associated layers prone to fail with thicknesses $t_{\ell,\mathrm{fail}}$ and thus independent on the layup parameter was derived. This concept mirrors the approach applied for the verification of longitudinal stresses (tension & compression) in CLT diaphragms which is also done in reference to the net cross section; see e.g. Section 4.1. The design concept is based on the layers in the weaker diaphragm direction in combination with a reference characteristic net-shear strength of

$f_{v,net,k,ref} = 5.5$ N/mm^2. For simplification this strength value is applicable for layer thicknesses $t_{\ell,fail} \leq 40$ mm and gap widths of $w_{gap} \leq 6$ mm. However, in case of thinner layers with 20 mm $\leq t_{\ell,fail} < 40$ mm and without gaps or reliefs higher net-shear strength values according to

$$f_{v,net,k} = f_{v,net,k,ref} \min\left\{\left(\frac{40}{t_{\ell,fail}}\right)^{0.3}; 1.20\right\} \tag{17}$$

can be used; see [33]. In case of CLT elements with a layup parameter ≥ 0.8, indicating a potential failure of the top and middle layer(s), verification of net-shear has to be met for both diaphragm directions. In doing so, reduced shear strength of the top layers according to nominal 10 mm thicker layers shall be considered.

In addition to the verification of net-shear the verification of torsional stresses, i.e. the potential failure between two layers in the vicinity of the glued bond, has to be met. Following [3], i.e. considering a characteristic torsional strength $f_{T,node,k} = 2.5$ N/mm² in combination with the values for $f_{v,net,k} \leq 6.6$ N/mm^2 according to Eq. (17), it can be concluded that the torsional failure mechanism can potentially govern only in cases of CLT diaphragms featuring a ratio between board thickness to board width or average distance of reliefs, t_ℓ / a or t_ℓ / w_ℓ, exceeding 0.25. In those cases the verification of torsional stresses according to [34] is proposed.

The shear modulus can be also determined according to the approach in [34]. For simplification a layup independent value of $G_{CLT,mean} = 450$ N/mm^2 is proposed.

4.2.2 Verification of CLT Diaphragms with Narrow Face Bonding

In testing narrow face bonded CLT elements a gross-shear failure, followed by net-shear failure was observed together with significantly higher resistances and shear moduli. For such elements, the shear properties known from glulam (e.g. [35]), $f_{v,gross,k} = 3.5$ N/mm^2 and $G_{CLT,mean} = 650$ N/mm^2, and the verification considering the gross cross section of the diaphragm are proposed. This implies that the narrow face bonding is preserved throughout the lifetime of the CLT element. Cracks due to climatic changes, at least in the top layers, are to be expected. Based on an engineering judgement and in view of the reduction factor k_{cr}, which takes into account shrinkage cracks in linear timber members (see [36]), the utilization of a reduced cross section, e.g. by considering only 30 to 50 % of the top layer thickness, $t_{\ell,TL}$, is suggested. However, additional investigations to better quantify this approach are required. In any case and also in respect to a potential delamination, the certified application of the utilized glue and the correct execution of the bond shall be ensured and controlled.

5. Special Topics

5.1 Shear and Bending Verification of Point-Supported CLT Floor Elements

Due to the orthogonal, laminar structure common layups of CLT floor elements have a different stiffness in their major (oriented parallel to top layer) and minor (oriented perpendicular to top layer) axis. As these elements are typically spanned in longitudinal direction, i.e. with a layup optimised in major direction, and due to static and economical reasons spanned over the smaller side of rooms, the load transfer in CLT roof and floor elements is usually uniaxial; verification on simple plate strips are sufficient.

In some circumstances and constructions a two-dimensional examination may be required. In such cases the internal forces and deformations can be calculated after the shear-flexible Reissner-Mindlin plate theory. In point-supported CLT elements (e.g. by columns) locally very high shear forces can occur which make a shear failure in vicinity of the supporting area possible. This failure mechanism is also known as “punching”; however, this kind of punching in CLT is rather a local shear and/or bending failure and thus not comparable with punching known from reinforced concrete structures. Investigations about this failure mechanism were made e.g. by Mestek [37], who focused on centrically loaded small-scaled CLT elements of approximately 1.1 × 1.5 m^2 reinforced against shear failures at their support.

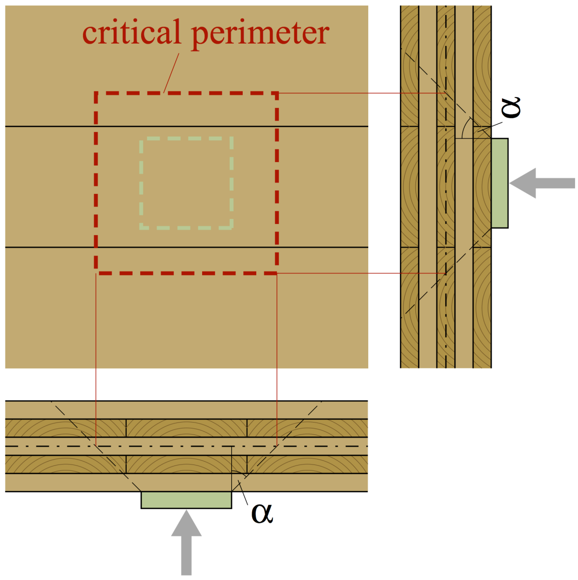


Fig. 7 Critical perimeter for determining the controlling rolling shear stress at point support areas according to [37].

Tests on elements with comparable dimensions but without shear reinforcements but reinforcements against compression perp. to grain were conducted at the competence centre holz.bau forschungs gmbh in 2014. Thereby the findings in [37], e.g. in respect to the critical perimeter (see Fig. 7) for determining the controlling rolling shear stress and the locally applicable (virtual) shear strength, could be confirmed; see [38,39]. Further bending tests performed on CLT elements in larger dimensions (2.5×4.0 m^2) but with a comparable test setup basically failed in bending; see [38,39]. It is obvious that the local type of failure (shear and/or bending) of CLT elements exposed to concentrated loads out-of-plane is amongst others highly dependent on the geometrical dimensions. A reliable and practical design concept for point-supported CLT elements is still missing and further investigations required; see also [40,41] for further information.

5.2 Ribbed Floors as Composite of CLT and GLT

Ribbed floor and roof elements, commonly realized as a quasi-rigid composite structure of CLT plate elements and GLT ribs, constitute an important alternative to other structural systems and structural materials, in particular where large spans and/or high stiffness are required. As consequence of shear deformation in the CLT the normal stresses in the flanges of the ribbed elements (T-beam cross section) are non-uniformly distributed along the width, w, of the T-beam. Thus, for a simple structural analysis of beams the effective width, w_{ef}, is required. This w_{ef} corresponds to a reduced width of the flange satisfying uniform normal stress and plane strain distribution; see Fig. 8. The resultant normal force in the flange according to w_{ef} is thereby equal to the resultant of the real stress distribution integrated over the entire width w.

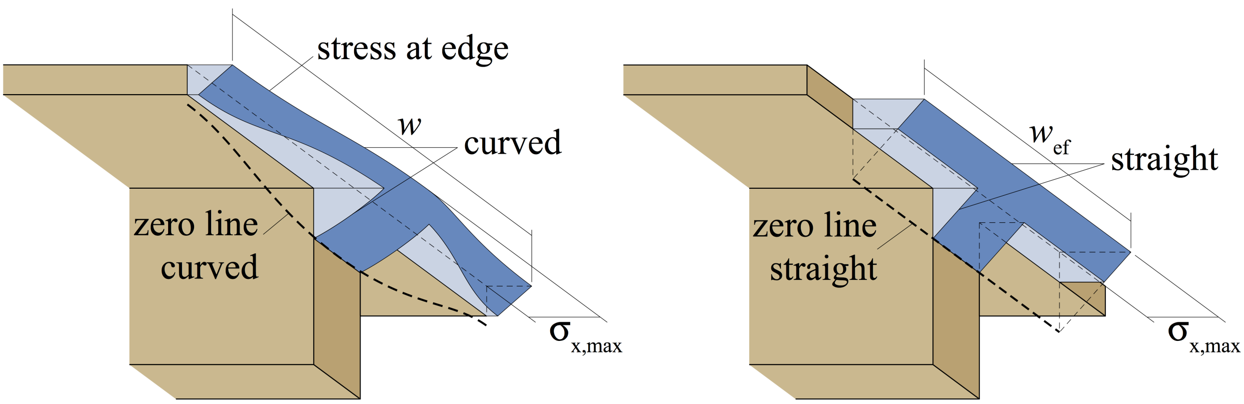


Fig. 8 Real (left) and idealised (right) stress distribution in a T-beam.

For determination of the effective width, the rib, modelled as beam, and the CLT element, modelled as a plate loaded in-plane, are coupled by a spring; see [42]. This spring considers the shear flexibility of the CLT element in the local area of force transmission as well as the shear flexibility of the connection itself. By solving the resulting differential equation system and equating the maximum bending stress of the beam-plate-model with those of the beam-model the effective width, w_{ef}, is determined.

The parameter w_{ef} is a function of the longitudinal position along the T-beam. The maximum occurs in the middle of the span; it declines in the direction of supports.

First and foremost the effective width depends on the ratio of span, L, to in-between distance of ribs, w. Other main influencing parameters are (i) the type of loading (uniformly distributed vs. concentrated load), (ii) the type of verification (ULS vs. SLS), (iii) the system (single-span vs. continuous beam), and (iv) the stiffness values and shear flexibility of the CLT element. The effect of reduced area in case of concentrated loads is only relevant for calculation of stresses in ULS but not in SLS design; see Fig. 9 left. Deflections can be well approximated independent of the type of loading by applying w_{ef} from a uniformly distributed ULS load case.

Continuous beams under distributed loads show also reduced areas in the effective width distribution at their supports. Petersen [43] suggested to use the effective width $w_{ef,S}$ of a single-span beam with a concentrated load and a span of $L = L_S$ (distance of points with bending moment equals to zero) at support areas (bending moment distribution almost triangle-shaped) and the effective width $w_{ef,F}$ of a single-span beam with distributed load and a span of $L = L_F$ in between; see Fig. 9 right.

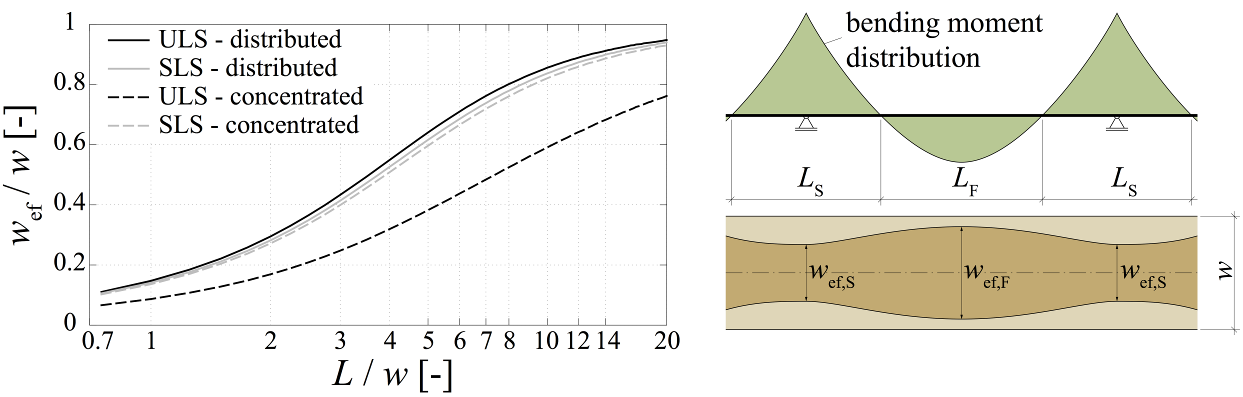


Fig. 9 Differences in effective width, depending on the type of loading (distributed vs. concentrated load) and the type of verification (ULS vs. SLS; left); distribution of the effective width along the length of a continuous beam (right).

With these determined effective widths the verification in ULS and SLS can be carried out apart from local shear stresses in the CLT element above the rib. The local load introduction leads to shear stresses in the CLT element, which are higher than calculated according to the classical shear formula. Thiel et al. [44] proposed to use an effective width, $w_{ef,\tau}$, for determining the maximum rolling shear stress in the CLT element, $\tau_{r,max}$, which is equal to the width of the rib plus a dispersal width as function of the dispersion angle and the thickness of the first layer at the bottom of the CLT element if running parallel to the rib; otherwise $w_{ef,\tau}$ is equal to the width of the rib, w_{rib}. For the distribution angle 45° are assumed; see Fig. 10. The proposed calculation method was verified via Finite-Element (FE) calculations showing acceptable deviations.

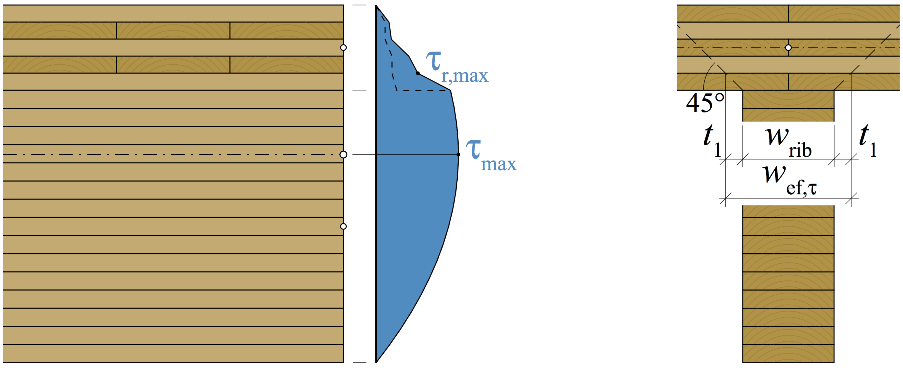


Fig. 10 Shear stress distribution over a T-beam cross section (left); proposal in [44] *for determining the effective width for calculation of (rolling) shear stresses in the CLT floor element above the rib (right).*

5.3 Concentrated Loads in-plane on CLT Diaphragms

We now focus on the distribution of stresses caused by concentrated loads in-plane on CLT diaphragms, e.g. on CLT walls. Corresponding investigations are published in [45]. An effective load distribution width, w_{ef}, was introduced, which is defined such that the mechanically correct stresses along the axis of symmetry in a certain depth of such a diaphragm can be calculated; see Fig. 11.

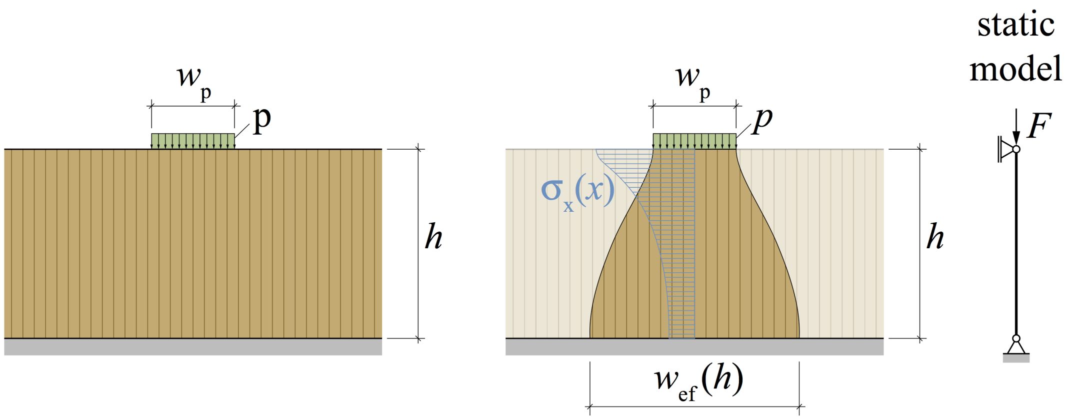


Fig. 11 CLT wall with local load p uniformly distributed over the length w_p, the effective load distribution width $w_{ef}(x)$, the stress distribution $\sigma_x(x)$ along the axis of symmetry and over the diaphragm's height h as well as the static model.

The effective width, w_{ef}, is strongly influenced by the material properties (axial stiffness c_x and c_y as well as shear stiffness c_{xy}), the cross section layup as well as the support conditions. Solving the differential equation system for this concentrated load case without software is very time consuming. An approximation for w_{ef} via $w_{ef,approx}$, see Eq. (18), is based on the analytical solution of an orthotropic semi-infinite plate, $w_{ef,ortho,inf}$, see Eq. (19), and presented in [45] (approximations for discrete points) and [46] (approximations for continuous course).

$$w_{\text{ef,approx}}(x) = \min\begin{cases} w_{\text{ef,ortho,inf}}(h)\cdot\left(\dfrac{2}{3}+\beta\cdot\dfrac{w_{\text{p}}}{2h}\right) & \text{valid for } \dfrac{w_{\text{p}}}{h}\le 0.5 \text{ and } L\ge h \\ 0.9\cdot w_{\text{ef,ortho,inf}}(x) & \end{cases} \qquad (18)$$

with: β = –0.35 for vertical top layers and β = –0.28 for horizontal top layers

$$w_{\text{ef,ortho,inf}}(x) = \frac{\dfrac{w_{\text{p}}}{2}\cdot\pi\cdot(\lambda_1-\lambda_2)}{\lambda_1\cdot\arctan\left(\dfrac{w_{\text{p}}}{2\lambda_2\cdot x}\right)-\lambda_2\cdot\arctan\left(\dfrac{w_{\text{p}}}{2\lambda_1\cdot x}\right)} \qquad (19)$$

with: $\lambda_1 = \sqrt{p^2+\sqrt{p^4-q^4}}$; $\lambda_2 = \sqrt{p^2-\sqrt{p^4-q^4}}$; $p = \sqrt{\frac{1}{2}\cdot\frac{c_{\text{x}}}{c_{\text{xy}}}}$; $q = \sqrt[4]{\frac{c_{\text{x}}}{c_{\text{y}}}}$

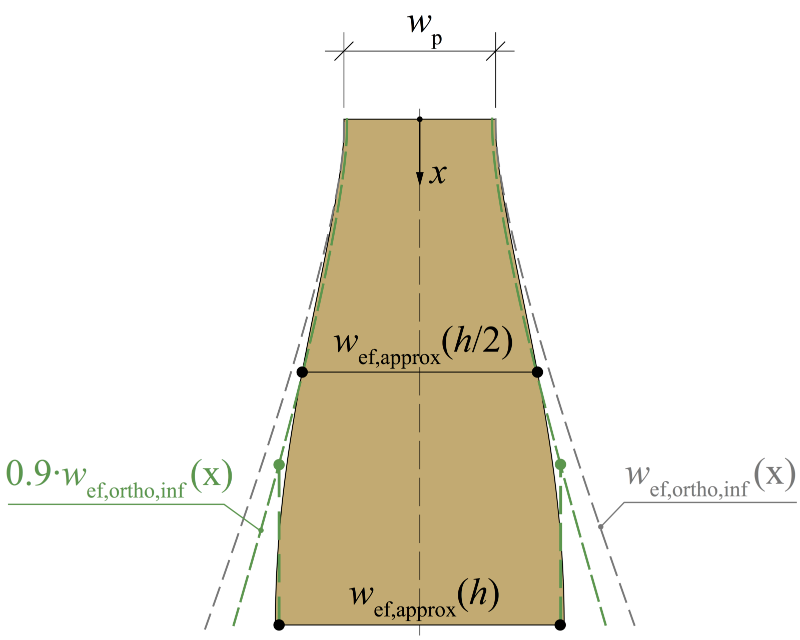


Fig. 12 Approximate effective load distribution width, $w_{\text{ef,approx.}}$.

For concentrated loads with w_{p} / h > 0.5 the effective load distribution width, w_{ef}, is approximately equal to the width w_{p} of the concentrated load itself.

Cases where the bottom edge of the wall is not continuously (load introduction cases) but discretely supported via columns are termed load transmission cases. In case that the load introduction width is equal to the width of the supports and the resultant forces are acting on the same line, symmetry can be assumed in the middle of the wall. In that case and having orthotropic material properties w_{ef} can be well approximated via the load introduction case and by considering only the half height of the CLT diaphragm. Deviations from the analytical solution by using this approximation are less than 1.5 % and therefore negligible, considering CLT products which are currently on the market.

For a concentrated load at the edge of a CLT diaphragm currently neither an analytical solution nor an approximation is available. In that case and meanwhile w_{ef} can be calculated numerically, e.g. by means of Finite Element Method (FEM).

To which extent this effective width in the middle of the wall can be used for a reliable but economic proof of stability (buckling) is currently part of a research project at the competence centre holz.bau forschungs gmbh. However, apart from the stability analyses of the diaphragm therein calculated effective widths can be applied for verifying compression perpendicular to grain at a potential floor element situated at the bottom of the CLT wall and also the connections there.

5.4 Buckling by Including Two-Dimensional Load Carrying Behaviour

CLT diaphragms, in particular walls, are mainly stressed in compression in-plane. Consequently, buckling has to be considered in the design procedure. According to current practice the verification against buckling is done on the basis of CLT columns; see Section 4.1. Within a recent study the critical buckling loads of CLT under uniaxial in-plane compression were investigated by analysing different support conditions, the influence of the orthotropic material behaviour and the transverse shear flexibility. Therein, curves for the buckling coefficient for CLT elements including the two-dimensional load carrying behaviour are presented [47]. The critical buckling load is always higher than that from the beam solution, but the benefit highly depends on the support conditions as well as the orthotropic parameters and dimensions of the wall.

6. Summary and Conclusions

In this contribution the main design procedures at ultimate limit state (ULS) for CLT used as plate elements loaded in- and out-of-plane are presented. Furthermore, a design process for CLT elements used as flange of a ribbed floor is proposed and some local load applications on wall and floor elements are discussed. Last but not least the critical buckling loads by including two-dimensional load carrying behaviour are approached.

Although numerous research activities in the past, which addressed a majority of relevant design issues related to CLT structures, there are still some remaining issues requiring further investigations, for example CLT walls and floors with large openings, design approaches for CLT elements with heterogeneous layups, regulations for the interaction of stresses as well as the application of CLT as folded plate.

7. Acknowledgements

The research work within the project 'focus_sts' is financed by the competence centre holz.bau forschungs gmbh and performed in collaboration with the Institute of Timber Engineering and Wood Technology of the Graz University of Technology.

The project is fostered through the funds of the Federal Ministry of Science, Research and Economy, the Austrian Ministry for Transport, Innovation and Technology, the Styrian Business Promotion Agency Association and the province of Styria (A12), the Carinthian Economic Promotion Fund (KWF), the province of Lower Austria department economy, tourism and technology as well as the Business Location Tirol.

The publication, as part of the proceedings from the Joint Conference of FP1402 & FP1404 "Cross Laminated Timber – a competitive wood product for visionary and fire safe buildings" held on the 10th March 2016 at KTH / Stockholm, was written in the framework of the COST Action FP1402 "Basis of Structural Timber Design – from research to standards", chaired by Philipp Dietsch, Technische Universität München (www.costfp1402.tum.de).

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9. Main Symbols

9.1 Latin upper case letters

A	area	L	span
E	modulus of elasticity	M	moment
F	force	N	normal force
G	shear modulus	S	shear stiffness; centre of gravity
I	moment of inertia	V	shear force
K	bending stiffness	W	section modulus

9.2 Latin lower case letters

a	mean distance of cracks	q	factor
c	stiffness value per unit length	t	thickness
e	distance	w	width
f	strength value	x	global axis parallel to grain direction of the top layers (major direction of the CLT)
h	height	y	global axis orthogonal to grain direction of the top layers (minor axis of the CLT)
k	coefficient	z	global axis perpendicular to the plane of the cross laminated timber
ℓ	length		
n	normal force per unit length		
p	pressure; factor		

9.3 Greek lower case letters

α	angle
β	factor (straightness factor with subscript c)
λ	slenderness
σ	normal stress
τ	shear stress

9.4 Subscripts

CLT	properties of cross laminated timber
IL	intermediate layer
ML	middle layer
T	torsion
TL	top layer
approx	approximate value
cr	critical; crack
d	design value
dis	dispersion
ef	effective
fail	failure
gap	gap
gross	gross
i	index number
inf	(semi-)infinite
k	characteristic value
lay	properties of layers
ℓ	properties of lamella
m	bending
max	maximum
mean	mean value
net	net
node	node
ortho	orthotropic
p	pressure
r	rolling shear
ref	reference
rel	relative
rib	rib
sec	intersection
t	tension
v	shear
x	global axis parallel to grain direction of the top layers (major direction of the CLT)
y	global axis orthogonal to grain direction of the top layers (minor axis of the CLT)
z	global axis perpendicular to the plane of the cross laminated timber
τ	related to shear stress
0	local axis parallel to the grain
05	5 %-quantile
90	local axis perpendicular to the grain (both tangential and radial)

Minutes of Presentation IV:
ULS Design of CLT Elements – Basics and Some Special Topics

Presentation by Alexandra Thiel

Summary:

Alexandra Thiel discusses various aspects of the design of CLT:

- Loading in and out of plane
- Tension-, compression-, bending-, shear-stresses
- Special applications

More specific, Alexandra Thiel discusses the following issues:

- Determination of internal forces can be made according to Timoshenko beam theory, Shear analogy, γ-method and Finite-element method. The latter can better account for point loads. However, design is often governed by SLS.
- Models for the load distribution in compression perp. to grain where derived from the van der Put (1988) model by Brandner and Schickhofer. These account for e.g. the size of the point load, length of the support or edge distance.
- The design of shear in CLT diaphragms was discussed by Brandner, Dietsch et al. (2015). The net shear strength is dependent on the layup, cracks, gaps, thickness of layers etc. The torsional strength needs only to be calculated for specific ratios of thickness/width > 0,25.
- The effective width of ribbed floors with CLT depends amongst others on the loading type (concentrated vs. distributed) and ULS or SLS design. Proposals for a simplified design can e.g. be derived from Petersen (1988).
- Sophisticated models are currently under development for the determination of the load distribution angle of concentrated loads on wall diaphragms.

Further efforts are necessary for the development of rules for

- Design for point supported floor elements
- Buckling
- SLS design
- Regulation for interaction of stresses

Alexandra Thiel introduced the software CLT designer (www.cltdesigner.at).

Discussion:

Fernando Perez asks how to account for small cracks due to varying or small moisture contents and especially for the more or less random distribution of cracks for elements with narrow face bonding?

Reinhard Brandner replies that net shear design can be used in these cases, it is assumed that the distance between cracks fulfill the boundaries for the effective width.

Fernando Perez asks if it is possible to achieve gross shear failure for elements with narrow face bonding?

Reinhard Brandner replies that in conditions in laboratory gross shear failure was observed. In practice the thickness of the surface layers might be reduced due to cracks. After initial gross shear failure, net shear failure will happen and is expected to be relevant in the situation in practice.

Introduction to Structural Fire Design with a Focus on Timber and CLT structures

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Edinburgh, United Kingdom

Summary

A key consideration in the design of all buildings is their ability to resist the effects of possible unwanted fires, and to provide an acceptable level of fire protection against loss of life, property protection, and environmental protection. The building design professions have, over many years, developed frameworks and design procedures for achieving the relevant fire safety objectives. However, surprisingly few members of this community are aware of the fundamental functional objectives for structures in fire, the true meaning of the term 'fire resistance', and the means by which this is achieved in practice. This paper presents the concept of fire resistance, the manners in which it might be determined, and the ways that it can be used to design fire safe buildings. The goal is to set the stage for fruitful discussions and collaboration between COST Actions FP1404 and F1402.

1. Introduction

The purpose of this brief paper is to introduce readers, who may not be familiar with the specialist discipline of structural fire engineering, or with concepts such as 'fire resistance', to the necessary fundamental concepts to understand and interrogate fire safety engineering considerations associated with timber, and in particular cross-laminated timber (CLT), structures. Both traditional (sometimes called prescriptive) and performance-based structural fire design methodologies are briefly discussed, as are concepts of fire resistance, fire resistance testing, and fire resistance calculation for heavy timber and CLT elements.

2. Traditional Structural Fire Engineering Design

Structural design for fire conditions generally follows the same approach as for structural design under ambient conditions, however because a severe fire in most buildings is a statistically 'rare' event, the load and resistance factors specified in building codes for the fire limit state change to reflect this fact. Structural elements

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are required to demonstrate satisfactory performance in resisting 'failure' when exposed to a 'standard fire' for a prescribed duration of heating (in the case of prescriptive design) or for full burnout of the available fuel in a building compartment (in performance based design).

2.1 Philosophy and Goals

According to [1], the governing structural design equation during fire can be expressed in general as:

$$\alpha_\theta E_\theta \leq \varphi_\theta R_\theta \tag{1}$$

where the subscript θ denotes the effects of elevated temperature, and which may have an effect on each of the terms in Equation 1. For instance:

$E_\theta =$ The specified effect of loads acting on the structure at elevated temperature. That thermal expansion of structural elements may introduce new loads into the structure due to restraint to thermal expansion.

$\alpha_\theta =$ Load factors applied to the specified loads for the elevated temperature condition. These are typically reduced as compared with the ambient temperature values to reflect the most likely load condition during a fire.

$R_\theta =$ The calculated resistance of a member at elevated temperature, based on material properties that have been reduced due to the damaging effects of heating (and in the case of timber reduced cross-sectional dimensions).

$\varphi_\theta =$ The resistance factor applied to the calculated resistance or to specified properties and dimensions, workmanship, type of failure, and uncertainty in the prediction of resistance at high temperature. These are typically set to 1.0 to reflect the nominal member or material strength at elevated temperature.

Or, according to the Structural Eurocodes [2]:

$$E_{fi,d,t} \leq R_{fi,d,t} \tag{2}$$

where:

$E_{fi,d,t} =$ The design value of the relevant effects of actions in the fire situation at time t (which is calculated using appropriate load and combination factors).

$R_{fi,d,t} =$ The design value of the resistance of the member in the fire situation at time t (which is determined using appropriate material reduction (partial safety) factors).

Structural design for fire has historically considered three distinct failure modes that must be prevented in satisfying Equation 1. Noting that designs should be implemented so that fire cannot spread beyond the compartment of origin for the requisite period, these three failure modes are:

a. loss of load bearing capacity (i.e. structural collapse, criterion load bearing R);

b. passage of flame or hot gas through a building element (e.g. wall or floor), which would represent a breach of fire compartmentation (criterion integrity E); and

c. excessive temperature rise at the exposed face of the structural element, which may also represent a breach of fire compartmentation (criterion insulation I).

2.2 Loads and Load Combinations

Load combinations for use in Limit States Design for ultimate capacity at ambient conditions are widely available in national design codes. In the case of structural fire design, load combinations are altered to reflect the low probability of occurrence of a severe fire, as well as the fact that the loads acting on a structure on a day-to-day basis are less than those used for ultimate strength design. Various jurisdictions apply slightly different load combinations for fire, but one straightforward example might be [3]:

$$\textit{Fire Load Demand} = 1.2D + A_k + (0.5L \text{ or } 0.2S) \tag{3}$$

In the above equation, D is the 'dead' or permanent load, L is the 'live' or non-permanent load, S is the snow load, and A_k is the load associated with the fire itself (for example the load resulting from differential or restrained thermal expansion).

Other building codes may also include the effects of snow and wind loads during fire, however again at reduced levels as compared with ambient temperature design. The Structural Eurocodes [2] apply a similar philosophy, however using somewhat different terminology:

$$\textit{Fire Load Demand} = \sum_{j\geq 1} G_{k,j} + A_d + \left(\Psi_{1,1} \, or \, \Psi_{2,1}\right) Q_{k,1} + \sum_{i>1} \Psi_{2,i} Q_{k,i} \tag{4}$$

where:

$G_{k,j}$ = The characteristic value of permanent (i.e. dead load) action j.

A_d = The design value of an accidental action (e.g. loads due to heating).

$Q_{k,1}$ = Characteristic value of the leading variable (i.e. live load) action 1.

$Q_{k,i}$ = Characteristic value of the accompanying variable action i.

$\Psi_{1,1}$ = Factor for frequent value of a variable action 1.

$\Psi_{2,1}$ = Factor for quasi-permanent value of a variable action 1.

$\Psi_{2,i}$ = Factor for quasi-permanent value of an accompanying variable action i.

The important result of assuming these reduced loads during fire is that most structures are subjected to loads during fire of less than 50 % of their ambient design capacity [1]. It is also noteworthy that the value of the load or load effects resulting from the fire itself (e.g. due to thermal deformations) should be included; as denoted by A_k in Equation 3.

3. Fire Resistance

For better of worse, structural fire resistance is currently defined as the duration of standard heating during which elements are able to resist 'failure', as defined above (e.g. 1 hour, 2 hour, etc). Fire resistance has historically been determined via standard fire tests, however advances in fire science and engineering have more recently enabled a wide range of experimental and computation methods to perform predictive calculations of fire resistance for various types of structural materials and elements, including timber and CLT beams, columns, slabs, and walls.

3.1 Fire Testing Determination of Fire Resistance (Prescriptive)

Fire resistance has historically been evaluated via experiments in which a structural member is subjected to a standard temperature time curve in a fire testing furnace, under sustained load, for a specified duration. The standard temperature time curve is commonly referred to as the 'standard fire'. It is noteworthy that current standard fire testing procedures were formalised ~80 years ago, based on tests performed ~100 years ago [4]. Such tests are performed according to ISO 834 [5], or similar, and are typically based on 'full-scale' or 'model-scale' tests of members that are performed on 'representative specimens'. It is also noteworthy that the implication of the standard test is that the built assembly in a real structure will perform at least as well as the tested assembly did in the fire test [1]. However, this is not universally true [4].

To perform a standard fire test, a test specimen is constructed to accurately represent as-built construction. The specimen is then placed in a rigid loading frame, which is positioned inside, next to, or on top of a standard testing furnace (depending on the member type). The likely service load is typically applied, and maintained as a constant load whilst the member is subjected to heating (typically by gas burners controlled using plate thermometers) following the standard time-temperature curve. The test is continued until the desired rating is achieved or one of the failure criteria is reached. The standard temperature versus time curve is shown in Figure 1, which is also the basis of the fire design of timber members according to Eurocode 5.

Such standard fire tests undertaken in furnaces have a number of noteworthy shortcomings that designers ought to be aware of so that they can better understand and navigate the current regulatory environment regarding fire safety in timber (and other) buildings. Fire tests are relatively expensive, specimen size is limited and may not be realistic in many cases, the effect of restraint or continuity are virtually impossible to apply realistically, redistribution of moments and system response features are typically poorly captured, data from proprietary tests is not generally available for research, lower load levels (while realistic) are not normally considered, critical failure modes may be overlooked, reproducibility is relatively poor (both between tests and between labs), the standard fire is not representative of a real fire in a real building, and there is in general no consideration of post-fire actions and cooling effects [1, 4].

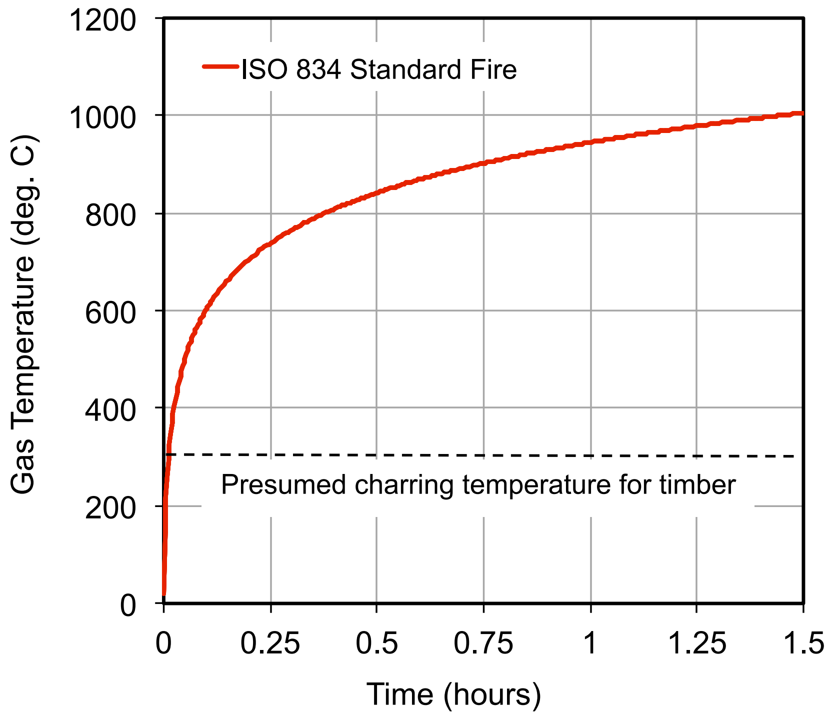

Fig. 1 The ISO 834 [5] *standard fire curve.*

Whilst based on the available historical evidence, standard fire tests appear to have served the fire safety community well, particularly for timber buildings, a full understanding of the realism of such tests is important for designers who wish innovate in building design under a performance-based mind set; such an understanding presents numerous opportunities to innovate whilst preserving, or even enhancing, public safety. Interested readers are encouraged to consult [4] for a full discussion of relevant factors in standard furnace testing.

3.2 Calculating the Fire Resistance of Timber and CLT Members

Because fire resistance testing is usually time and cost expensive, calculation methods have been developed to analyze structural designs in various materials for fire conditions. Such calculation methods have been formulated based on a range of analyses of data from standard tests, experimental programs, and theoretically based investigations.

Structural fire design calculations for mass timber elements are available in design codes internationally, and may take many forms. The most advanced and rational guidance is likely that set out in Eurocode 5 [6], which can be used to determine the fire resistance of timber elements based on a presumed charring rate and a residual cross section calculation methodology (described below). While CLT is not explicitly treated in the Eurocodes, current practice is to design CLT essentially as would be done for solid softwood timber; subject to suitable modifications to account for its crosswise lay-up. Such an approach takes advantage of sacrificial self-insulation of the timber by surface charring and loss of an acceptable depth of the surface timber, which protects and insulates the underlying timber.

Two specific simplified analysis methods are suggested in Eurocode 5 to determine the load bearing capacity of a mass timber (and, by extension, CLT) element during exposure to a standard fire: (1) the reduced cross section method; and (2) the reduced properties method. Eurocode 5 [6] also suggests a method by which so-called parametric (or 'natural' burnout) fires can be considered, however this method is not widely used and are not approved for use in many jurisdictions. Strictly speaking, the reduced properties method only applies to elements subject to fire from three or four sides, which is not typically applicable for CLT elements and is therefore not discussed herein (indeed, it is rarely used in practice even when applicable, and is slated for deletion from the upcoming revision to Eurocode 5).

The reduced cross section method assumes that softwood timber will char at a notional charring rate during exposure to a standard fire, and then uses this notional charring rate to predict the depth of charred timber during fire. The char, which is typically taken to be represented by the depth of the 300^{o}C isotherm within the heated timber, is assumed not to contribute to the element's load bearing capacity. To account for the presence of a zone of heated timber beneath the char, an additional 7mm layer of 'zero-strength' timber is also assumed to make no contribution to strength or stiffness. The capacity of the timber structural element is then determined based on its ambient temperature mechanical properties, accounting only for the 'residual' cross section and with the charred timber and zero-strength layer ignored. This simplified approach was originally derived during the 1980s, based on numerical simulations of the fire behaviour of a limited number glued-laminated timber beams exposed to fire on three-sides by Schaffer [7]. The wider applicability of this approach to CLT remains somewhat uncertain (and doubtful according to some authors) [8].

An additional analysis and design option which is available, but rarely used at present, for structural fire designers is the possibility to undertake 'advanced simulations' using the methods outlined in Annex B of Eurocode 5 [6].

4. Working Group 2 of COST Action FP1404 – Structural Elements made of Bio-Based Materials and Detailing

4.1 Performance-Based Structural Fire Engineering

Bio-based building products, e.g. such as timber or CLT structural members, have a long history in building design. Historical fires led to combustibility being the main reason why bio-based building materials could not be used in many applications, particularly in dense urban centres. When performance based design (PBD) for fire became possible during recent decades, many national building regulations effectively opened the market for bio-based building products, provided that suitable technical justifications could be made to show that such materials and systems achieved the functional objectives of the building regulations with respect to fire safety. However, differences between building regulations in different countries still exist, even within Europe [9], and the use of combustible bio-based building products remains relatively limited; particularly for structural framing in multi-storey building construction. Whether this is because the prescriptive guidance is irrational and/or out-dated, or because legitimate fire safety concerns exist, is not well known, particularly when large amounts of exposed timber are present in buildings.

So-called performance based fire safety engineering (FSE) is increasingly accepted as a means to develop performance-based structural fire engineering designs; it allows a PBD with customized, rational building solutions. However, knowledge of the performance of many bio-based building materials and systems, including exposed multi-storey CLT, under the non-standard fire scenarios, which might need to be considered during a rational PBD assessment of a building, is comparatively poorly developed and this hinders PBD of such systems.

4.2 Working Group Activities

Fire resistance classification systems for structural elements under so-called 'standard' fire curve exposures are well established and commonly used. However, a common database on structures fulfilling certain fire classes needs to be established. In addition, the use of the Eurocodes' parametric/natural fires may be very important in real building CLT applications for both safety and efficient/sustainable structural designs. For this reason, the related material property data and fire protection methods need to be gathered and reviewed. In addition, guidance and best practice on detailing in construction (e.g. penetrations, cavities, connections, etc.) are needed to ensure adequate fire safety.

From previous research projects on engineered timber products, there already exists a wealth of information from various sources. However, this information needs to be compiled and reviewed so that it can be modified and applied, potentially to other bio-based materials, including CLT.

Working Group 2 (WG2) of COST Action FP1404, *Structural elements made of bio-based materials and detailing*, deals with producing a database of available knowledge as well as new information on material properties, structural response, and fire protection schemes for bio-based materials and construction systems, including CLT, in different, credible fire scenarios, both in and around buildings. The main activities being undertaken include:

a. developing and perpetuating an improved understanding of the structural response, and hence fire resistance, of bio-based structural elements and materials (including CLT) exposed to a range of 'standard' and realistic (natural) fire scenarios;

b. characterizing the reactive, thermal, and mechanical material properties of relevant bio-based building materials and fire protection methods (including CLT) so as to enable, by use of fire engineering and rational engineering judgement, their use in building applications which might otherwise be prohibited by prescriptive fire safety regulations;

c. developing and disseminating databases of information on the performance of both bio-based structural elements and construction materials (including CLT) relevant to points (a) and (b) above; and
d. studying the effects of construction detailing and structural connections of various types for achieving fire safety in a bio-based built environment.

Working Group 2 of COST Action FP1404 is working together with the standardization body CEN TC 250/SC 5 “Eurocode 5 – Design of Timber Structures” who are currently working on a revision of Eurocode 5 (EN 1995-1-2) [6], expected in 2020. The outcome of WG2’s activities will serve as an important evidence base for the envisioned revision.

Furthermore, collaboration between the COST Actions FP1404 and COST Action FP1402 "Basis of Structural Timber Design - from research to standards” is critically important, since FP1402 can provide information on the future vision for the design and construction of CLT buildings; such information is essential in developing the requisite structural fire design guidance and information.

5. Acknowledgements

The authors would like to acknowledge the COST Action FP 1404 project partners, The University of Edinburgh, United Kingdom, and ETH Zurich, Switzerland, for supporting this contribution to the first joint event of COST Actions FP1402 and FP1404.

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Minutes of Presentation V:
Concepts for Fire Protection in Buildings from CLT in View of European Building Regulations

Presentation by Luke Bisby

Summary:

Luke Bisby gives an introduction to the background of fire design, ISO fire, resistance vs. failure and time resistance design. Future challenges are the incorporation of fuel load design compared to fire resistance timetables.

In addition further research is needed regarding the following questions:

- Does CLT change compartment fire dynamics?
- How about self-extinguishing structures from CLT?
- Is delamination a risk in case of fire?
- How about smouldering combustion?
- What is the mechanical and structural response of CLT elements to fire exposure?
- What opportunities would a more rational approach open up?

Discussion:

Delegate asks how openings effect the fuel load?

Luke Bisby replies that there are still many open questions regarding the fuel load. The idea behind the time resistance design of structures was mainly driven by the aim of insurance companies to avoid collapse of structures. In future the focus will be more on saving lives.

Delegate asks if there are any advantages or disadvantages of the design according to parametric fires?

Luke Bisby replies that parametric fire has only a minor impact on e.g. the charring rate in timber, whereas for steel structures it leads to a big advantage.

Fire Design of CLT

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Summary

This paper gives an overview of the main research conducted on the fire resistance of CLT and its design according to Eurocode 5 (EN 1995-1-2) [1]. Thereby, it is concentrated on the fire design of CLT using an effective cross-section method (introduced as reduced cross-section method in Eurocode 5) or the design using advanced calculation methods. The effective cross-section method needs information on two main parameters as input, namely the charring rate and the zero-strength layer thickness. It is shown that the model for protected timber members according to Eurocode 5 can be used to determine the residual cross-section of CLT considering possible local falling off of layers during fire. In contrast, the thickness of the zero-strength layer is still an open research topic. The paper presents an overview of conducted simulations to model the fire resistance of CLT and to determine the zero-strength layer thickness for various CLT compositions. It seems that the complex performance of CLT in fire cannot be captured with one single number for the zero-strength layer thickness independent, e.g. on the layer thickness and composition. It can be summarised that further research on the determination of the zero-strength layer needs to be conducted to ensure a safe and easy to use fire design method of CLT and equally provide an economically and ecologically worthwhile use of the product. The final section of this paper gives an overview of European fire safety regulations for timber buildings in general and with regard to the use of CLT.

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1. Introduction

Cross-laminated timber (CLT) panels are relatively new engineered wood products that can be used as load-bearing wall, floor and roof elements in innovative and high quality modern timber structures. Unlike light timber frame constructions, where single timber studs are responsible for the transfer of the vertical loads, the use of large solid timber panels allow the transfer of high vertical loads and guarantee a high building stiffness and robustness. Other main advantages of CLT are an excellent thermal insulation and air tightness. The use of large solid timber panels is also favourable in case of fire, as the risk of fire spread through void cavities is reduced in comparison to light timber frame constructions. However, large solid timber panels may increase the fire load in the room.

CLT is composed of simple softwood boards, between 10 and 40 mm thick and 80 to 240 mm wide. The variety of cross-section layups is very large. The number of layers ranges from 3 to 7, or even 9 and the thicknesses of the different layers can be identical or varied. The layers are usually crosswise oriented. The result is a two dimensional structural system, which can carry loads in longitudinal and transversal direction. The size and form of CLT panels is limited by production, transportation and erection possibilities. In common practice, the bottom layer and the top layer are oriented parallel to the long axis of the panel with a symmetrical cross-sectional layup (see Figure 1).

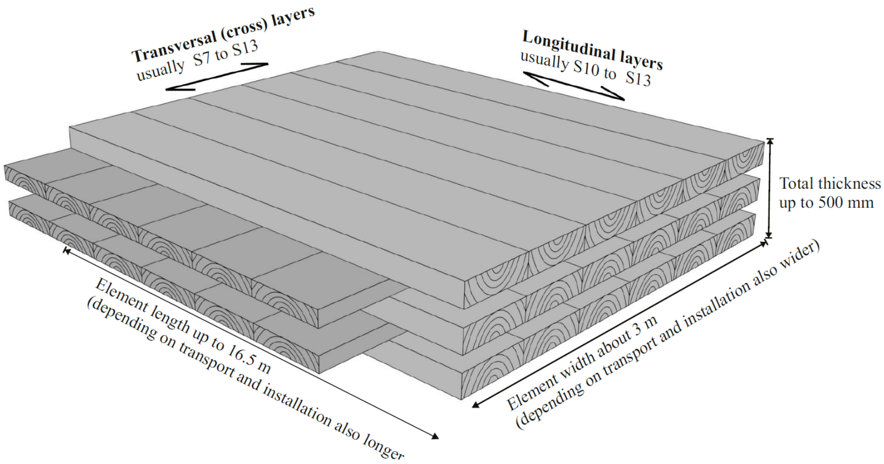


Fig. 1 Sketch of a cross-laminated timber panel. In this paper, longitudinal layers and load-bearing layers are used interchangeable.

Combustible building materials like timber burn on their surface, release energy and thus contribute to fire propagation and the development of smoke in case of fire. The main precondition for the use of wood for buildings is an adequate fire safety. Fire safety is an important contribution to feeling comfortable and an important criterion for the choice of material in particular for residential buildings.

This paper summarises the main research conducted on the fire behaviour of CLT and its fire design according to EN 1995-1-2 [1]. Thereby, it is concentrated to give an overview of relevant outcome of numerous fire tests with CLT and the consequences for the charring behaviour. Further, an overview of conducted simulations is presented to model the fire resistance of CLT and to determine the zero-strength layer thickness for various CLT compositions. Additionally, the paper gives up-to-date fire design recommendations for CLT and summarises European fire safety regulations.

2. Charring of timber

Timber is a combustible material and thus differs from most other commonly used structural building materials. When sufficient heat is applied to wood, a process of thermal degradation (pyrolysis) takes place producing combustible gases, accompanied by a loss in mass. A charred layer is then formed on the fire-exposed surfaces and the char layer grows in thickness as the fire progresses, reducing the cross-sectional dimensions of the timber member. The char layer is a good insulator and protects the remaining uncharred residual cross-section against heat. For timber surfaces unprotected throughout the time of fire exposure, the charring rate can be assumed constant with time for standard fire exposure [2].

As a basic value, the charring rate β_0 is usually taken as the value observed for one-dimensional heat transfer under ISO-fire exposure in a semi-infinite timber slab. Table 1 gives the basic design charring rate β_0 for different materials according to EN 1995-1-2 [1].

Table 1: Basic design charring rate β_0 according to EN 1995-1-2 [1].

Material	**β_0 [mm/min]**
Softwood and beech	
Glued laminated timber with a characteristic density ≥ 290 kg/m^3	0.65
Solid timber with a characteristic density ≥ 290 kg/m^3	0.65
Panels	
Wood panelling	0.9
Plywood	1.0
Wood-based panels other than plywood	0.9

Depending on the timber element and application, different coefficients can increase this basic design charring rate β_0. These coefficients are summarised in Table 2. For example, at the corners of the cross-section increased charring occurs, leading to corner rounding. Another example for increased charring is when timber joists or studs are protected by cavity insulation on their wide sides. In order to simplify the calculation of cross-sectional properties (area, section modulus and second moment of area) by assuming an equivalent rectangular residual cross-section, a notional charring rate β_n can be calculated so that it implicitly includes the effect of corner rounding and increased charring sections leading to approximately the same results.

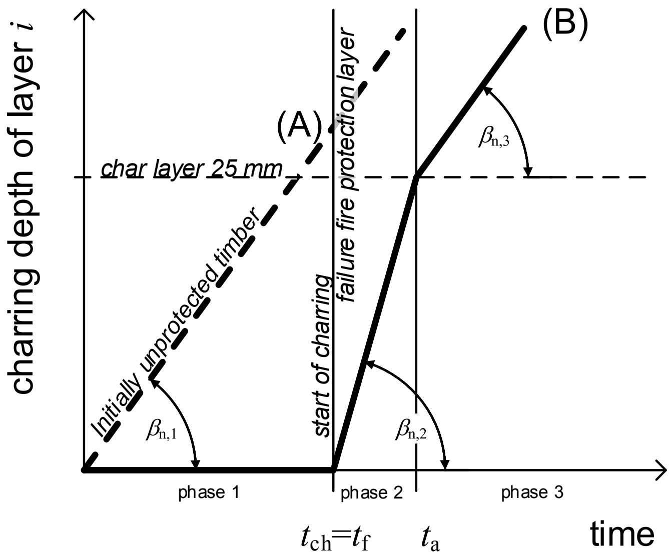
Equation 1 expresses a newly proposed relation between the basic design charring rate β_0 and the notional charring rate β_n. This new approach to determine the notional charring rate β_n with coefficients that influence the charring behaviour of timber members is very flexible and can easily be adapted and extended for different applications. Thus, it is worth mentioning that the overview of the k-factors is not complete. Further, the future fire design of timber members should always be performed using the notional charring rate β_n and hence considering the coefficients in Table 2.

$$\beta_n = k_s \cdot k_{pr} \cdot k_n \cdot k_g \cdot k_{cr} \cdot k_j \cdot k_{co} \cdot \beta_0 \tag{1}$$

Table 2: Coefficients k to determine the notional charring rate β_n.

Coefficient	**Description**	**Explanation**	**Reference**
k_s	Section coefficient	The section coefficient k_s considers the influence of the width of the timber member. This parameter is only significant for the charring rate on the narrow side. For charring on the wide side, the coefficient k_s can be neglected (k_s = 1.0). $k_s = \begin{cases} 1,2 & \text{for } 40mm \leq b \leq 60\ mm \\ 1,3 - 0,00167 \cdot b & \text{for } 60mm \leq b \leq 180\ mm \\ 1,0 & \text{for } b \geq 180\ mm \end{cases}$ With b: width of the narrow side of the cross-section in mm.	[3]
k_{pr}	Protection coefficient	The protection coefficient addresses the behaviour of protected timber surfaces, for which different charring rates should be applied during different phases of fire exposure.	[3]
k_n	Corner rounding	Since charring is greater near *cross-section corners*, *gaps* and *fissures*, notional charring rates β_n should be used to transform the irregular shape of residual cross-sections into simple rectangular cross-sections, or cross-sections composed of several rectangular parts.	[4,5]
k_g	Gaps between boards		[6]
k_{cr}	Cracks and char fissures		
k_j	Joint coefficient	The joint coefficient k_j considers the influence of joints in panels not backed by battens or structural members or panels and their influence on the protection and insulation time of these layers. Usually, k_j =1.0 for ultimate limit state design.	[7]
k_{co}	Connection coefficient	The connection coefficient k_{co} considers increased charring for connections with metal fasteners, which conduct heat into the core of the cross-section.	[8]

For protected timber surfaces, different charring rates should be applied during different phases of fire exposure [3]. Figure 2 gives the simplified model adopted by EN 1995-1-2 [1] when start of charring t_{ch} occurs at the same time as the failure t_f of the cladding. Phase 2 describes the increased charring of timber after the claddings have fallen off. EN 1995-1-2 [1] assumes that charring takes place at double the rate of initially unprotected surfaces. The main physical reasons for the increased charring rate observed after failure of the cladding is that, at that time, the fire temperature is already at a high level while no protective char layer exists to reduce the effect of the temperature. The protection provided by the char layer is assumed to grow progressively until its thickness has reached 25 mm. Then the charring rate decreases to the value for initially unprotected surfaces. The simplified model can be used for protective claddings made of wood-based panels or wood panelling and common regular gypsum plasterboards [9].



Key:

$\beta_{n,1}$	notional charring in phase 1
$\beta_{n,2}$	notional charring in phase 2
$\beta_{n,3}$	notional charring in phase 3
t_{ch}	start of charring
t_f	fall-off time (end of stickability) of the fire protection system (fire protection layer)
t_a	time when a fully developed char layer is reached

Note: charring of solid timber or the first layer of an assembly (A), and (B) for layers $i > 1$

Fig. 2 General description of charring for CLT products with initially unprotected protected surfaces (A) and initially protected surfaces (B) according to EN 1995-1-2 [1].

3. Charring of CLT

The charring behaviour of CLT is different to charring of homogenous timber panels due to the layered, glued composition and joints between the timber boards that can lead locally to increased charring.

An enormous amount of fire tests on single CLT wall and floor elements have been performed in recent years, see Table 3. In these tests, the layer thickness, the number of plies, the type of adhesive, the type of encapsulation, and the support conditions have been investigated among other factors. Further, full-scale compartment fire tests and ad-hoc testing with a radiant heat panel have been performed to analyse protected and unprotected CLT elements. Based on the performed experimental investigations, the following conclusions can be drawn for the charring behaviour of CLT elements:

- The calculation of the residual cross-section should consider the orientation of the CLT panel, being horizontal or vertical oriented. To determine the thickness of the char layer of floor elements, the following two boundary situations should be considered:

1. If the individual charred layers of the CLT panel do not fall off (also referred to stickability, see standard series EN 13381-X [10]), the forming charcoal layer protects the remaining CLT cross-section against heat. In this case, the CLT panel has a similar fire behaviour as solid wood.
2. If local falling off of the char layer occurs (also referred to loss of stickability), the fire protective function of the charcoal is lost. After the charred layers have fallen off, an increased charring is expected due to the increased fire temperature. This phenomenon is similar to the increased charring observed for protected timber surfaces after failure of the fire protective cladding, see Figure 2. This phenomenon can be considered using a double charring rate for the second layer (and the subsequent layers) for the first 25 mm of depth when falling off of the first layer occurs.

- For wall elements, the effect of falling off of charred layers was less pronounced in the performed experiments. However, load-bearing and unprotected wall elements should be carried out with at least five layer CLT elements to ensure a robust solution; further a minimum residual thickness of layers in span direction of 3 mm should be achieved [11]. With regard to the fire resistance, a thicker outer layer is generally beneficial so that a possible local falling off of charred layers occurs after about 45-60 minutes exposure to fire.

Table 3: Overview documented fire tests on CLT elements.

Ref.	Author	No. of tests	Investigated parameters
[12]	Frangi et al. (2008)	10	Layer thickness, number of plies, wall and floor elements
[13]	Frangi et al. (2009)	11	Layer thickness, number of plies, adhesive
[14]	Teibinger und Matzinger (2010)	12	Layer thickness, number of plies, adhesive, wall and floor elements, with and without encapsulation, encapsulation
[15]	Wilinder (2010)	27	Layer thickness, adhesive, with and without encapsulation
[16]	Craft et al. (2011)	6	Adhesive, with and without encapsulation, encapsulation
[17]	Friquin et al. (2011)	6	Layer thickness, number of plies, fire curves
[18]	Gustafsson (2011)	2	Encapsulation
[19]	Osborne et al. (2012)	8	Layer thickness, number of plies, wall and floor elements, with and without encapsulation, encapsulation
[20]	Menis (2012)	7	With and without encapsulation
[21]	Aguanno (2013)	8	Number of plies, with and without encapsulation, encapsulation
[11,22]	Schmid et al. (2013, 2015)	16	Layer thickness, number of plies, with and without encapsulation, encapsulation
[23]	Klippel et al. (2014)	10	Layer thickness, number of plies, support conditions

An increased charring due to the layered composition of CLT can be considered by using a greater notional charring rate β_n than the basic charring rate β_0 for one-dimensional charring. The fire safety in timber buildings handbook [9] defines the relation between the one-dimensional charring rate β_0 and the notional charring rate β_n using a coefficient k. In case a CLT layer consists of boards bonded together along their edges or the gap width between two boards is not greater than 2 mm, the basic design charring rate β_0 can be applied, meaning that the coefficient k_g =1.0. In case the gap is between 2 mm and 6 mm wide, the basic design charring rate β_0 should be multiplied by a coefficient k_g =1.2 to determine the notional charring rate. In case the gap width is greater than 6 mm, a fire exposure from three sides should be regarded in the calculation. It should also be noted that a load-bearing layer most likely has no gaps and thus k_g =1.0.

The approach given in the fire safety in timber buildings handbook should further be extended in the future, as described in Equation (1). Figure 3 shows a summary of the charring rates for typically used applications of CLT. The determination of the notional charring rate β_n for a typical CLT product needs to consider the following two coefficients (the other coefficient k given in Table 1 are set to 1.0 for typical CLT products):

- k_{pr} (protection coefficient)
- k_g (coefficient to account for gaps between boards)

It should be noted that for each layer and charring phase an individual notional charring rate $\beta_{n,i}$ can be calculated.

Figure 3 shows two possible approaches to calculate the residual cross-section for CLT floor elements taking into account falling off of layers: (1) The double charring rate model or (2) a simplified model using a mean notional charring rate $\beta_{n,mean}$ throughout and until 90 minutes of fire exposure. For a fire resistance equal or smaller than 90 minutes, a mean notional charring rate $\beta_{n,mean}$ = 1.1 mm/min (panel without gaps) was determined taking into account falling off of the layers and double charring for layers $i \geq 2$. This value constitutes to be a conservative value when using this simplified model. Further, it should be noted that this value is significantly higher than the charring rate specified in the current version of prDIN 20000-8 (in here $\beta_{n,mean}$ = 0.8 mm/min is used).

Whether a falling off of charred layers occurs, depends on the adhesive in the glue line between the boards and the composition of the CLT element (number and thickness of layers). For a fire resistance of 30 minutes there will be no influence of falling off of charred layers when the outer layer has a minimum depth of 25 mm, as only the first layer will be charred. For a fire rating of 60 and more minutes a clear difference in the residual cross-section is expected. However, it has to be noted that the fire resistance of a CLT element is not linearly related to the charring rate, as the charring of a perpendicular layer with low stiffness and strength properties, has nearly no effect on the overall load-carrying capacity.

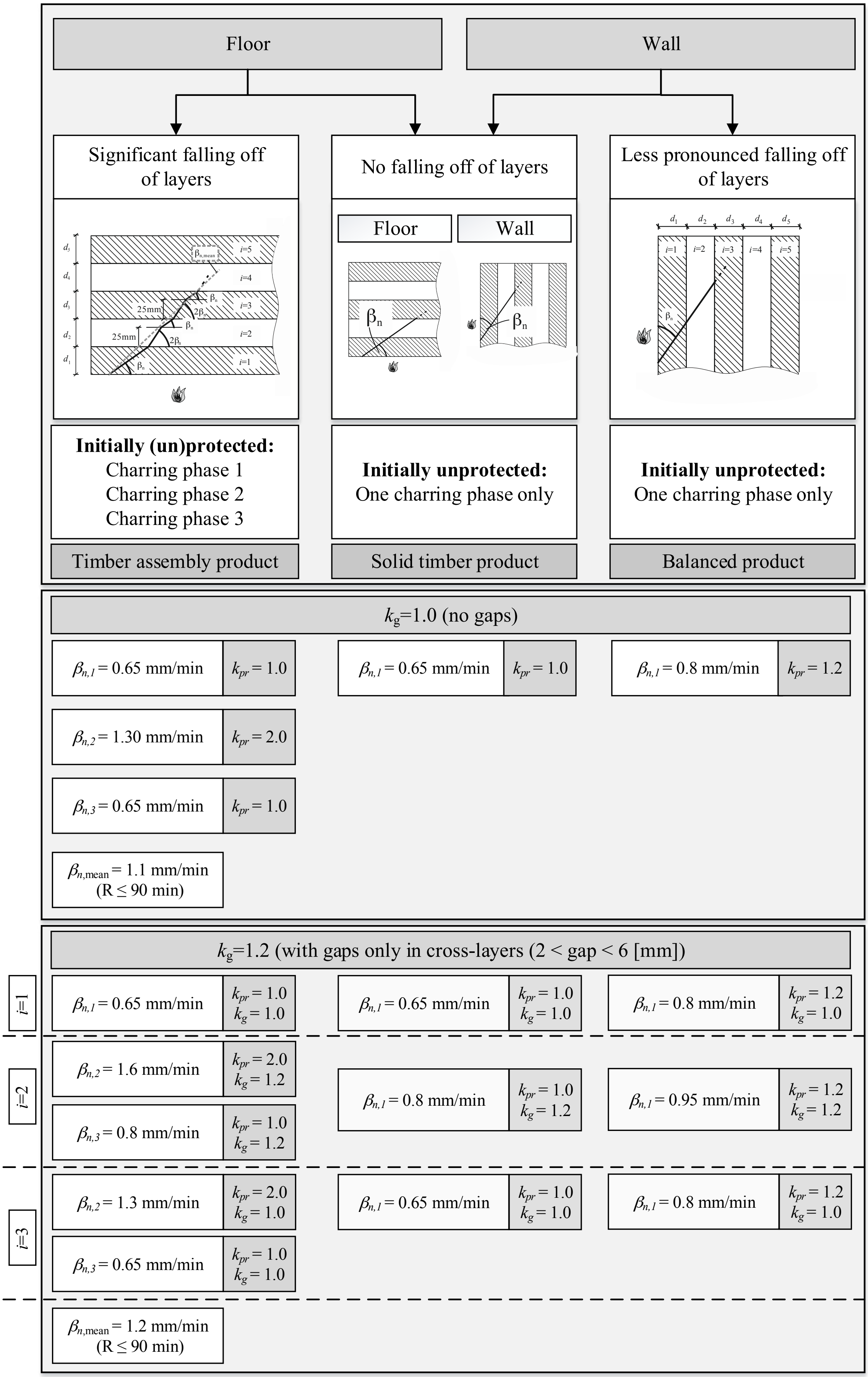


Fig. 3 Charring rates for different CLT applications (without any fire protection layer).

Further, it should also be noted that examples for fire design of CLT floor elements used in practice [23] showed that falling off of charred layers for common CLT panels and typical fire design situations has no influence on the design of the panel configuration. As a consequence, the fire design should not govern the design of a CLT element and thus no change of the layered structure is expected (regardless of the adhesive). The thickness and number of layers is rather given by the design at normal temperature, such as vibration, deflection, etc. (SLS).

4. European design methods for CLT exposed to fire

Fire reduces the cross-section as well as the stiffness and strength of the heated timber beyond the char layer. The current version of EN 1995-1-2 [1] does not provide specific information on the fire design of CLT panels. Nowadays, there are basically two options to calculate the fire design of timber elements following EN 1995-1-2 [1]:

(1) use of effective cross-section method or

(2) by means of advanced calculations using Annex B of EN 1995-1-2 [1].

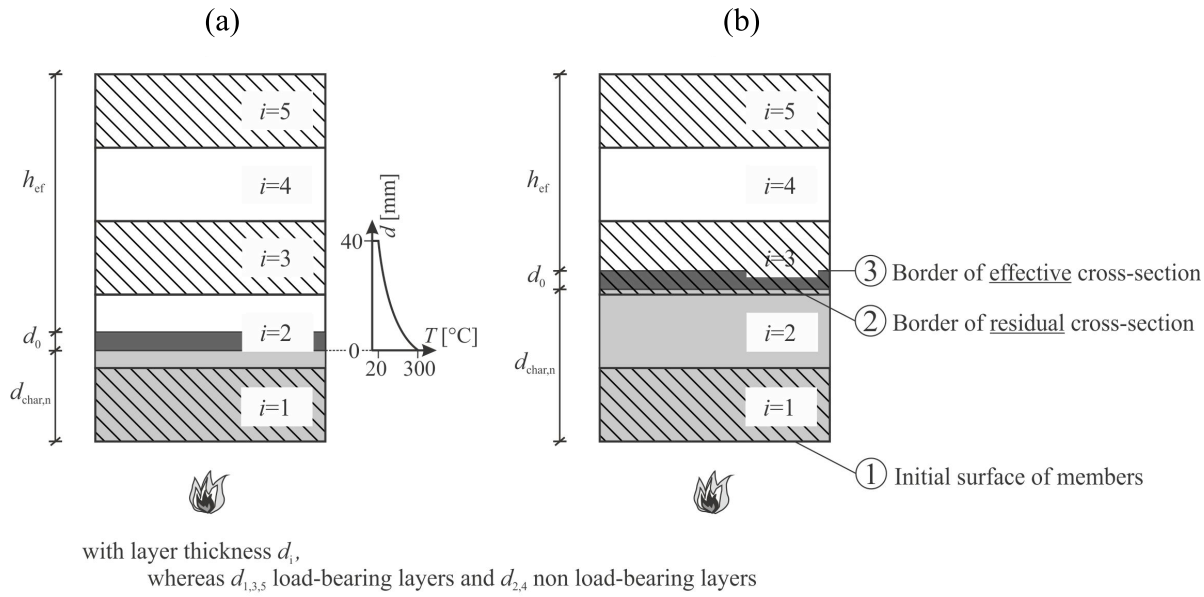


Fig. 4 Definition of residual cross-section and effective cross-section: a) d_0 layer is in cross-layer and thus no load-bearing layer, b) d_0 is in load-bearing layer.

The effective cross-section method according to EN 1995-1-2 [1] is – although not specifically assigned – also used for the fire design of CLT. This method considers the strength and stiffness reduction of the residual cross-section by adding an additional depth d_0 (called zero-strength layer) to the charred layer $d_{\text{char,n}}$ (see Figure 4). Thus, the method assumes an effective cross-section that is slightly smaller than the residual cross-section and has the same material properties at normal temperature at all points of this effective section. The effective cross-section can be calculated by reducing the residual cross-section at a specific time of fire exposure by the zero-strength layer, which is assumed to have neither strength nor stiffness. EN 1995-1-2 [1] gives a constant value of 7 mm for the zero-strength layer, independent on the application. However, it has previously been shown on the basis of advanced calculation methods [24,25] that the use of a constant value of 7 mm might lead to a non-conservative fire design of CLT elements. Moreover, a constant value might be not appropriate for the zero-strength layer independent of the element (glued-laminated timber, solid wood, CLT with different laminae thickness, etc.) and the state of stress [24,25].

Due to its structure with longitudinal and transversal layers CLT has a complex performance when exposed to fire. The mechanical properties of CLT depend on the grain orientation with respect to loading. When subjected to one-way bending, the load share of the orthogonal cross-layers may be neglected, as done in [26]. The different properties of the layers have consequences for the determination of the zero-strength layer thickness. Figure 4 shows that basically two limiting cases can be defined:

a) The border of the residual cross-section lies in a cross-layer and the zero-strength layer is only part of the non-load-bearing cross-layer
b) The border of the residual cross-section lies in a longitudinal layer and the zero-strength layer is only part of the load-bearing cross-layer

In case of (a), the zero-strength layer reduces only the part of the cross-section, which does not contribute to the bending resistance although the heat affected zone reaches already the next longitudinal layer as indicated by the temperature gradient in Figure 1 (a). As the intention of the effective cross-section method is to consider strength and stiffness reduction by heat, the zero-strength layer is not sufficient in this case and has to be increased accordingly. In case of (b), the zero-strength layer is fully accounting for a reduction of the longitudinal and thus load-bearing layer due to elevated temperatures. A mixture of case (a) and (b), of course, does also exist. As the current design according to EN 1995-1-2 [1] does not account for either of these cases for the effective cross-section method of CLT elements further rules are needed.

It should also be noted that the heat affected zone behind the char layer is greater than the thickness of d_0 = 7 mm. The heated zone (zone in which the temperature drops from 300 to 20°C) is constant after about 20 minutes of fire exposure and has an approximate thickness of 35 to 40 mm for initially unprotected members (see Figure 4).

The fire safety in timber buildings handbook [9] introduced a so-called compensating layer s_0 (equivalent to d_0 used in EN 1995-1-2 [1]) to account for strength and stiffness reduction of the heat affected timber below the char layer. Further, the thickness of this compensating layer is given in the handbook for three, five and seven layered CLT floor and wall elements for a fire exposure time smaller than 120 minutes. Compensating layers are given for protected and unprotected elements as well as for fire exposure on tension or compression side.

The compensating layer s_0 is given either as a constant value or depending on the height of the CLT panel. The values given in this handbook are based on the work performed in [24], which is described in the next section on "Simulation of CLT in fire".

The European CLT product standard EN 16351 [27] regulates the performance characteristics for CLT. Requirements for both the resistance to fire and the reaction to fire are given in this standard.

It should be noted that the actual version of EN 16351:2015 [27] contains two major error with respect to the fire design. Contrary to the standard it is not possible to use the reaction to fire classes (Euroclasses) according to EN 13501-2 [28] of single lamella to describe the reaction to fire class of the product. By definition, the Euroclass of a product has to be assessed by the end product and not parts of the product. A classification without further testing verification (CWFT) should be aimed for as it was done for glulam products to minimize testing costs for the producers; as long this is not available, the end product has to be tested to define the European reaction to fire classes. Further, the density of the lamella should not be the bases for the calculation of the charring rate. There are many studies available showing that the variation of charring of one wood product (with about one specific density) exposed in the same fire resistance test is about in the same order as the variation of charring depending on the density [7,29,30,31]. The authors conclude that for European wood one basic design charring rate should be used. Following this, EN 1995-1-2 [1] does not give rules for charring depending on the density, only two classes are given, see Table 1. For softwood products of characteristic density not lower than 290 kg/m^3 a one-dimensional basic design charring rate of $\beta_0 = 0.65$ mm/min shall be used.

5. Simulation of CLT in fire

Fire tests are time consuming and costly and can only be performed in certified fire laboratories. Moreover, only a few investigations combine the performance of fire tests with reference investigations at normal temperature. To develop and verify a design procedure, reference tests at normal temperature are required where the material properties of the specimens can be determined. Further, the zero-strength layer thickness d_0 can only be determined with adequate information of the material properties at normal temperature. Recently a transparent procedure was published to determine the zero-strength layer for any structural timber member [32].

In the past decade, numerical modelling of structural members became an effective and inexpensive alternative to predict and enlarge the information gained from fire tests. Once a model has been validated using experimental data, numerical models can be utilized to further evaluate and get an in-depth understanding of the fire performance of structural elements. Additionally, a model can be used to perform parametric studies by varying the input data, i.e. geometry, material properties and boundary conditions.

However, simulating timber elements is still challenging as the complexity of the material is difficult to consider. In the fire situation further challenges limit the use of commercial software. Reasons are the limitations of the software to take into account wood specific properties, e.g. moisture and mass transfer through the fire exposed cross-section. Further, the use of effective thermal and mechanical properties limit the application range to very narrow field of fire curves.

Software code capable to describe the load-bearing resistance of timber members shall be capable to take into account wood specific mechanics in fire characteristics. This is (i) the creation of plastic zones in compression while this is not possible in tension and (ii) the reduction of the strength and stiffness properties are different in tension and compression, see Figure 5 and 6. Further the software should be (iii) capable to allow the failure of a limited number (depth) of fibres at the side in tension: Failure may not cause immediate collapse of the member but allow load distribution within the member.

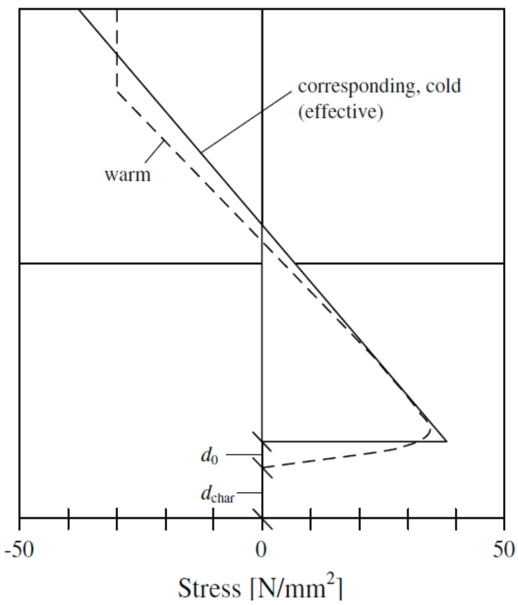


Fig. 5 Stress distribution along the centre line of a beam exposed to fire on three sides and subjected to bending (dashed curve), and the corresponding linear stress distribution with normal material properties and the same bending resistance (solid line) [9].

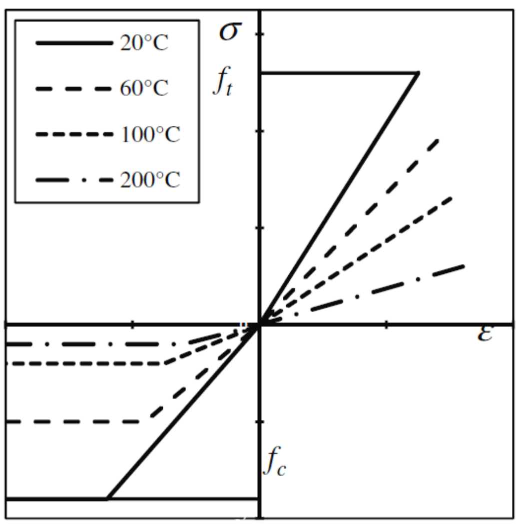


Fig. 6 Temperature-dependent stress–strain relationships parallel to the grain for wood at different temperatures with plasticity in compression only [9].

The above-mentioned wood mechanics in fire have been applied to CLT structural members by König and Schmid [24,25]. Results of the software code CSTFire were used to developed an easy-to-use design model in terms of using the effective cross-section method according to EN 1995-1-2 [1] in general and the zero-strength layer in specific.

The thermal and thermal-mechanical properties of timber according to Annex B of EN 1995-1-2 [1] were used in [33] to determine charring depths and the reduction of bending resistance of CLT when exposed to standard fire. Further, only the failure mode “failure of tensile lamella” was considered in the simulations since a shear failure was not expected in practice due to the high slenderness of the product.

Following the adaption of the software CSTFire to CLT products results were compared to actual design rules determined for glulam beams. It was found in [24,26] that for the most relevant stage of relative resistances from 0,2 to 0,4, $d_0 = 7$ mm according to EN 1995-1-2 [1] would lead to a non-conservative design of CLT in fire.

In a next step in [33], 27 model-scale tests with specimens of different structures (series MF and SF) were performed and results compared to the model. The tests also comprise reference tests at normal temperature to be able to predict the load-bearing capacity at normal temperature and to load the CLT members in the fire tests accordingly. This step is crucial for the determination of the zero-strength layer and is neglected in many studies or when tests are performed to achieve a fire resistance rating. Results of the model-scale tests showed that the model fitted well, especially for protected members and members with exposed side in compression (walls, continuous supported floors) a significant improvement was achieved compared to a constant zero-strength layer of 7 mm. Figure 7 shows selected results, details are available in [24,33]. Further, full-scale fire tests were performed to compare the performance of the same type of CLT in model-scale (fire exposed length with constant moment 1000 mm) with a full-scale test ((fire exposed length with constant moment 1800 mm). Results showed that model-scale tests are capable to predict the failure time in full-scale [33].

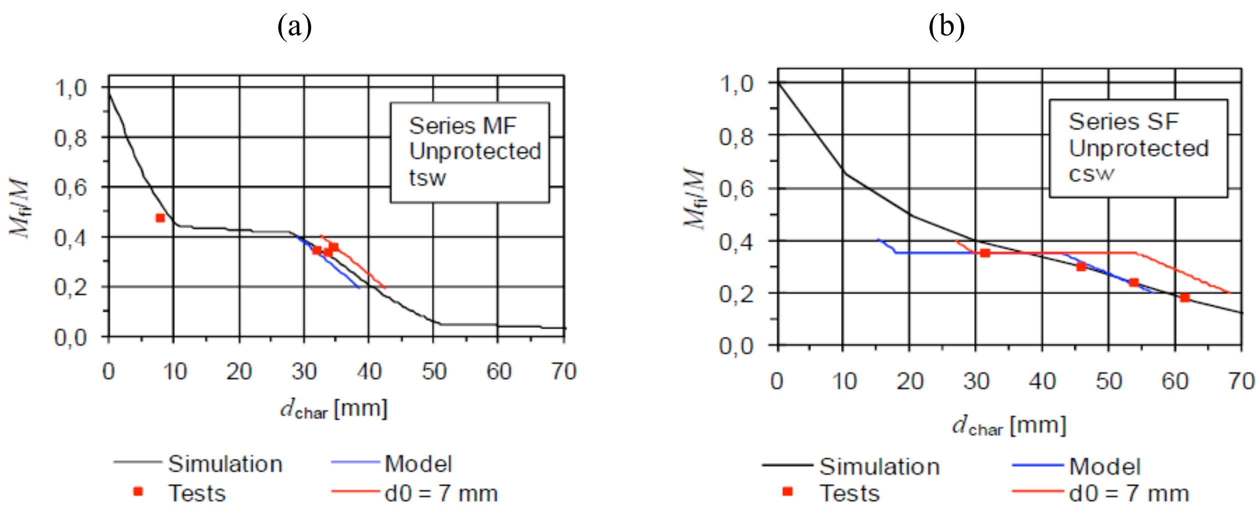


Fig. 7 Comparison of Test results with simulation and the easy-to-use design model for a) series MF, unprotected, with the fire exposed side in tension (tsw) and b) for series SF, unprotected with the fire exposed side in compression (csw) [33].

The following products and structures were considered and are covered by the simulations in [24]:

- Depths of CLT from 45 to 315 mm
- Layer thicknesses of CLT from 15 to 45 mm
- Layer numbers from 3 to 7
- Initially unprotected and initially protected CLT elements
- Fire-exposed side in tension or compression
- Symmetrical structure
- Standard fire exposure of 120 min.
- Floors are considered as single span beams as typical CLT joints are not capable of transferring bending moments.

Based on a market analysis, the large product portfolio was simulated and so called "regular" and "irregular" structures investigated for five layer products. It was concluded that it is sufficient to investigate products only with load-bearing layers of equal depth. As the values for the zero-strength layer vary considerable for all products it was not possible to describe all CLT products in all applications, i.e. as (a) floor or (b) wall element, (c) initially protected or (d) unprotected element and (e) with its exposed side in tension or (f) compression, with one constant value; such a procedure would lead to very uneconomic design in most of the cases. Instead it was decided to introduce simple functions considering the applications (a) to (f) for three, five and seven layer CLT products. For all cases linear or bilinear trend lines were developed describing mean values of the zero-strength layer.

As a main outcome of this investigation [33], the zero-strength layer thickness d_0 adopted for a maximum fire resistance time of 120 min for different CLT applications was presented. Further improvement, i.e. the accuracy of the zero-strength layer values, would be performed when the product portfolio would be limited or the fire resistance would be limited to 30, 60 or 90 minutes.

The experimental and numerical behaviour of CLT floor elements was recently investigated by other studies [34,35]. In this investigation, eight CLT panels were tested at normal temperature in a 4-point-bending configuration until failure. Further, two CLT floor panels produced from the same batch were tested in large-scale fire resistance tests under 4-point bending until failure of the specimens. Tests at normal temperature and in the fire situation make it possible to calculate the zero-strength layer d_0 and further are the basis of solid numerical analysis. The experimental investigation was accompanied by finite element simulations using Abaqus, to model the thermal-mechanical behaviour of the CLT panels. A comparison of test results with simulations concerning the mid-span deflection, the temperature along the cross-section height and the fire resistance showed good agreement. Additional evaluations on the stress distribution of CLT during fire are presented.

6. European fire safety regulations

Safety in case of fire is one of the seven Basic Requirements of Construction Works given in the current European Construction Products Regulation (CPR), which replaced the previous Construction Products Directive (CPD). The CPR was published in the Official Journal of the European Union on 4 April 2011 and became a binding legal requirement for all member states on 1 July 2013. According to the CPR the construction works must be designed and built in such a way that in the event of an outbreak of fire:

(a) the load-bearing capacity of the construction can be assumed for a specific period of time;

(b) the generation and spread of fire and smoke within the construction works are limited;

(c) the spread of fire to neighbouring construction works is limited;

(d) occupants can leave the construction works or be rescued by other means;

(e) the safety of rescue teams is taken into consideration.

In order to assure these essential requirements on fire safety, a European system including product standards, performance classes in case of fire, testing and calculation standards for fire performance has been introduced. The European Committee for Standardisation (CEN) brings together the National Standardization Bodies of 33 European countries and provides a platform for the development of European standards and other technical documents on various types of products, materials, services, and processes. While CEN TC127 (fire safety in buildings) is responsible for the development of standards for testing and classification, the Eurocodes for the design of timber structures, including EN 1995-1-2 [1] (Structural fire design) are developed by CEN TC250 SC5.

Although European standards are used on the technical level and the European harmonisation of standards for fire testing, classification and calculation has continuously progressed, regulatory requirements applicable to building types and end users, including the use of combustible materials in buildings, remain on national level. This means that fire safety is governed by national legislation and thus is defined on the political level, leading to major differences between European countries for the use of wood products in buildings. In the following, the fire safety regulations of Switzerland, Italy and Germany are briefly summarised.

The fire safety regulations in Switzerland are revised every ten years and the current fire safety regulations are valid since 1st of January 2015. The new regulations bring significant benefits in several areas with regard to the use of wood products in buildings. For example, it is now possible to use timber as a construction material in all categories of buildings, including high-rise buildings following specific rules in their design. Further, the simplification of the categories of buildings (low-rise, medium-rise and high-rise buildings) and uses makes a clear allocation of fire safety requirements. With regard to requirements of fire resistance, there is no difference between combustible and non-combustible structural elements. Further, the use of structural timber elements is possible in all fire resistance classes. By introducing sprinkler systems, the requirements of fire resistance are reduced by 30 minutes. With regard to requirements of reaction to fire, residential, office and school buildings as well as industrial and commercial buildings can be realised with structural timber elements with visible wood surfaces up to a total height of 30 m (low-rise and medium-rise buildings), with the exception of escape routes. For high-rise buildings, the application of structural timber elements is still now possible under certain conditions (full encapsulation). For CLT, no specific rules are mentioned in the Swiss fire safety regulations. For the fire design of timber buildings, Lignum has published an extensive documentation that is recognised by the fire authorise as state of the art.

In Italy, since the introduction of the new technical regulation for construction in 2008, it is possible to use structural timber for buildings of any height, whereas beforehand a special permit issued by the Upper Public Work Council was needed for multi-storey timber buildings of more than 4 stories. This technical regulation concerns the structural design (including seismic and fire). Timber can be used in any building category. Specific regulations for fire safety were issued for different building categories (e.g. blocks of flats, office buildings, schools, etc.) irrespective of the type of structural material used. These regulations prescribe limitations on the fire resistance classes of the structural members, and on the reaction to fire classes of the surfaces (e.g. at least 50% of the surface of the egress system must be made by non-combustible materials). For buildings with higher fire hazards (e.g. block of flats taller than 24 m), irrespective of the structural material used, a special permit issued by the Fire Brigade, the Fire Prevention Certificate, is required. For Cross-laminated timber (CLT) no specific rule applies. CLT is not mentioned in any national Italian regulation. Structural fire design of timber member is carried out using the Eurocode 5-Part 2 with the National Application Document.

In Scotland, since the introduction of new building regulations in 1964 it is possible to use structural timber in certain high-rise buildings. Building categories (low-rise, medium-rise and high-rise buildings) and usage categories result in well-defined fire safety requirements. The fire safety regulations intend to protect persons and property from the dangers and effects of fire. Timber as a construction material can be used in all categories of buildings. For high-rise buildings (above 18 m) the application of structural elements in wood is possible under certain conditions. Cross-laminated timber (CLT) is treated no differently than any other construction product, although any innovative construction product or technique may require greater scrutiny.

In Sweden, the main change in the building regulations occurred in 1994, when a performance-based approach was introduced. The new regulations open up for the use of timber structures without any limit in building height, if the performance requirements were met. Before that, maximum two storeys in timber had been permitted during more than 100 years. A major further revision occurred in 2012. The requirements on verification of the fire safety design have increased, and a clear distinction is made between simplified and analytical design. New occupancy classes have been introduced. There are two ways to meet the requirements: (1) simplified design and (2) analytical design. Simplified design consists of acceptable solutions listed in the general recommendations. Analytical design must be applied for deviating from those. Analytical design is mandatory for buildings with more than 16 floors. Boverket, the Swedish National Board for Housing, Building and Planning, is responsible for the building regulations for new construction, modification and remodelling. The developer and building owner are responsible for the compliance. Timber can be used as the load-bearing structure provided that the requirements for fire resistance and separation capacity are met. No specific rules apply for CLT.

According to French regulations, timber can be used in load-bearing structures for all types of buildings (dwelling, public and high-rise). The requirements related to fire resistance are the same for all construction materials. The highest CLT building (residential building) constructed in France has eight storeys (total height of about 25 m). Buildings currently under construction using CLT are designed on the basis of fire laboratory assessments using the standard ISO fire curve. Additionally, requirements given in EN 1995-1-2 [1] and the Fire safety in Timber building (European guideline) handbook [9] are followed.

In Germany, the existing model building code (MBO), which was introduced in 2002 considers five classes of buildings with residential and office occupancy and buildings for special purposes and defines fire safety requirements in terms of these classes. The classification into a building class depends on the height and total area of a building. All requirements are on prescriptive basis. The building code also includes information about the general fire safety requirements to allow for advanced engineering with performance-based design. Currently, timber structures can be erected up to five storey in height (up to a height of 13 m of the upper floor level) if the structural and separating timber elements are lined with a non-combustible encapsulation cladding ($K_2 60$). In addition to the regulations of the building code, the guideline of quasi fire-proof timber structures must be considered. However, this guideline excludes the use of CLT. Using CLT only becomes possible by applying alternative solutions in a fire safety concept. However, the replacement of timber frame structures with a CLT structure is common and generally accepted by the authorities. The accepted level of safety must be reached by other means, like the consideration of sprinklers or additional escape routes, if further deviations from the prescriptive requirements, such as visible CLT elements in multi storey buildings are aimed for. A more timber friendly building code was recently introduced in the state of Baden-Württemberg to allow timber structures including CLT up to the high-rise level.

Table 4: Regulatory limitations and possibilities for D and D_{FL} reaction to fire class products according to [36].

Country	Allowed number of storeys (or height of building in meters) for D class products					D/D_{FL} class products allowed in buildings with at least 3 storeys					
	Load-bearing structures			External cladding		Internal walls/ceilings				Floorings	
						Escape routes Sprinklers		Within apartments Sprinklers			
	Prescribed rules	Performance based (PB)	*Protection required*	No sprinklers	With sprinklers	No	Yes	No	Yes	Escape routes	Within apartments
Austria	6	No limit	*No*	6	6	-	-	+	+	-	+
Belgium	See PB	No limit	*No*	3 (10 m)	3 (10 m)	-	+[a]	+	+	-	+
Czech Republic	3-4 (12 m)			3-4 (12 m)	3-4 (12 m)	-	-	+	+	-	+
Denmark	3-4	No limit		3-4	3-4	-	-	-	-	+	+
Estonia	4	No limit	*No*	8	8	-	+	-	+	+	+
Finland	2 / 8[b]	No limit	*$K_2 10/K_2 30$*	2/4	8	-	-	+	+	+	+
France	No limit	No limit	*No*	4 or 50 m[c]		-	-	+	+	+	+
Germany	4-5	> 5	*$K_2 60$*	3 (7 m)	3 (7 m)	-	-	+	+	+	+
Greece	No limit	No limit	*No*	No limit	No limit	-	-	+	+	+	+
Ireland	3 (10m)	No limit		≥ 5	≥ 5	-	-	+	+	+	+
Italy	See PB	No limit	*No*	(12 m)	(12 m)	-	-	-	-	+	+
Latvia	4	Not used	*B-s1,d0*	4	4	-	-	+	+		
Macedonia	2			2							
Netherlands	13 m	No limit		3-4	≥ 5	-	-	+	+	+	+
Norway	4	No limit	*EI30/EI60,$K_2 10$*	4	4	-	-	+	+	+	+
Poland	3-4 (12 m)		*B-s1, d0*	(25 m)	(25 m)	-	-	+	+	-	+
Portugal	(9 m/single family)			(28 m)	(28 m)	-		+		-	+
Slovakia	2-4	Not permitted	*EI*	(12 m)	(12 m)	-	-	+	+	+	+
Slovenia	3 / 5 [b]	No limit	*EI30/EI60*	3 (10 m)	3 (10 m)	-	-	-	+	+	+
Spain	See PB	No limit	*EI30-EI120*	6 (18 m)	6 (18 m)	-	-	+	+	-	+
Sweden	See PB	No limit	*No*	2	≥ 5	-	-	-	+	+	+
Switzerland	(30 m)	No limit	*No*	(30 m)	(30 m)	-	+	+	+	+	+
Turkey	3	No limit	*F30B2/F60AB*	3	3	-	-	-	-		
United Kingdom	See PB	No limit		≥ 5	≥ 5	-	-	-	+	-	+

[a] Fire detection is the required active means
[b] With sprinklers
[c] Applicable for dwellings; more than 4 storeys requires compliance with French façade test

In the frame of one task of COST ACTION FP1404 WG3, regulatory approaches in different countries have been analysed and compared in order to identify unsolved topics/obstacles for the use of bio-based materials in buildings. The main results of the analysis are compiled in Table 3. It should be noted that in many cases there may be specific conditions in the national requirements, which are not taken into account in the summarizing table. Table 4 is trying to give a simplified overall picture on the situation. As a conclusion of the comparisons of regulatory requirements it is clearly seen that despite of the existence of the CPR and the development of Eurocodes, there is a broad variety of criteria and requirements for buildings in the various European countries because fire safety in buildings is on political level governed by national legislation. Further, the analysis clearly shows that, if the national regulations allow only prescribed solutions, there are major limitations in using bio-based building products. Performance based regulations (or performance based options in regulations) are more flexible (being material independent).

7. Conclusion and research needs

The residual cross-section of CLT elements can reliably be determined using a notional charring rate β_n. This notional charring rate β_n is calculated by multiplying the basic design charring rate for one-dimensional charring β_0 with different coefficients k taking into account the structure of the CLT element.

In order to use the effective cross-section method according to EN 1995-1-2 [1], in addition to the charring rate a second parameter is of importance, namely the so-called "zero-strength layer d_0". This d_0 value considers the losses in strength and stiffness of the residual cross-section close to the char layer due to the elevated temperatures in this area. Nowadays, a general zero-strength layer thickness of d_0 = 7 mm is being used in Europe for the fire design of CLT. However, it has been shown that d_0 is not a constant value and the thickness of d_0 depends on different parameters, such as the CLT structure, the applied load and the fire resistance.

Further, the paper gives a short summary of European fire safety regulations for timber buildings. The use of CLT is usually not explicitly mentioned in any European regulations. However, replacement of timber frame structures with a CLT structure might be accepted by authorities in some countries.

In future research projects on the fire behaviour of CLT – given that it is decided to use the effective cross-section method for the fire design – the thickness of the zero-strength layer should be optimised. This optimisation should either performed for a defined fire resistance time, such as 30, 60 and 90 minutes, or for a limited product portfolio of CLT (definition of selected CLT compositions). Thereby, it is very important that the determination of d_0 should ensure a safe fire design of CLT and equally provide an economically and ecologically worthwhile use of the product.

8. Acknowledgments

Luke Bisby, Dhionis Dhima, Massimo Fragiacomo, Danny Hopkin, Esko Mikkola, Birgit Östman, and Norman Werther are thanked on their contributions to the chapter on Fire safety regulations.

9. References

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Minutes of Presentation VI and VII:

Fire Design of CLT Elements (General, Tests vs. Design)

Presentation by Michael Klippel

Fire Design of CLT – Comparison of Design Concepts

Presentation by Andrea Frangi

The presentations VI and VII were held in conjunction without a break.

Summary:

Michael Klippel gives an overview over the fire design of CLT. The key for the fire design is an appropriate charring rate describing the loss of material per minute. The charcoal layer protects the virgin wood and can be compared to an intumescent coating (reactive fire protection system) of steel members. Klippel presents a proposed charring model, which represents a straightforward approach to calculate the charring rate using a number of coefficients.

Klippel presents the charring model adopted for CLT depending on the ability of the adhesive to provide protection to the virgin wood when the char line exceeds the glue line. In the future this ability of the char layer to stick to the virgin wood or other charred layers should be called “stickability”. The term “delamination” should not be used when talking about fire. The charring model is in good agreement with tests (full-scale). Three different cases have to be considered depending on the adhesive and the orientation of the CLT. ETH has collected over 100 fire tests from multiple institutes, most important is the coordination of all results with further results owned by industry. Klippel presents the principles of the “Effective cross-section method” where a zero-strength layer is used. This value is – contrary to today’s rules in Eurocode - not constant. For CLT it may comprise crossing layers and therefore it might increase significantly depending on the structure of the CLT.

Andrea Frangi presents the advanced calculation rules of Eurocode 5.

Simulations for CLT were divided in thermal and mechanical simulations. Comparison between the simulations and fire tests in different scales show good agreement.

Frangi informs about the proposed structure of Eurocode 5-1-2 (fire part) with respect to the design: there will be an easy-to-use model (tabulated data), a simplified model with basic models and one part on advanced calculations. Frangi presents the proposals and gives an overview of on-going standardisation and building regulations with respect to fire safety. In most of the countries, CLT is not considered separately. Finally, Frangi addresses the research gaps where centralised planning and organisation is needed. Especially companies are invited to work together to take over market shares from other materials rather than not sharing information and counteracting moving forward together.

Discussion:

Magdalena Sterley asks if finger joints influence the time to fall-off?

Andrea Frangi replies No.

Delegate asks if fallen-off layers influence the fire load?

Andrea Frangi replies that no, not really, as it is a small amount of wood compared to the total fire load. But it will influence the fire dynamics, as it might lead to a re-ignition and a second flashover, as observed in some tests performed in Canada.

Danny Hopkin asks if the simplified methods are good enough for complex (i.e. tall) buildings?

Andrea Frangi replies that yes, they are conservative.

Jochen Köhler asks if it can be problematic to assure that the advanced methods (e.g. finite element simulations) meet the target reliability.

Detailing of CLT with Respect to Fire Resistance

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Summary

This publication summarizes the state of the art of detailing for cross laminated timber elements (CLT) and compiles available test data and findings on in-plane joints of CLT elements, joints in CLT component connections as well as the influence of penetrations and mounting parts in CLT, all with respect to the separation and load bearing function in the case of fire.

1. Introduction

Besides the structural stability, the separating function for wall and floor elements represents one of the most essential capacities in the case of fire. The evaluation of the fire resistance for such building elements normally occurs on the basis of standardised fire tests, such as listed in EN 13501-2 [1], as well as approved calculation methods, such as those presented in EN 1995-1-2 [2]. These methods normally do not, or just to a low extent, take into account any joints and junctions to neighbouring elements, mounting parts or typical penetrations of service installations. However, one of the main principles within the European fire safety regulations of buildings is the limitation of the spread of fire and smoke to other compartments and neighbouring buildings.

Element joints, junctions and penetrations of building services through separating elements are unavoidable and also have to fulfil the general requirements with respect to overall fire safety. There is a necessity to plan and approve these for each material and construction method from the beginning of a project to avoid complex and expensive solutions in the latter stages of construction.

However, inspections and surveys of new and existing buildings repeatedly report for all building materials and construction methods that the risk for an early fire spread from one fire cell to the next is mainly caused by inappropriately designed joints and service installations in walls and floors. At the same time, Stürmer (2006) e.g. found 50% of the service installations not installed properly and not

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able to perform correctly in the case of fire, resulting in significant limitations of usability for egress ways and the structural elements [3].

With respect to timber structures, this aspect becomes even more important. On the one hand, only a small amount of approved technical solutions are currently available on the marked. On the other hand, the combustibility of bio-based materials may contribute to a fire spread if hot gases infiltrate the structural elements. Within this context studies showed, that a flow of hot gasses through timber elements increase the charring behaviour due to additional thermal exposure and preheating of typically unexposed regions [4] [5]. In addition, an early failure of integrity may occur as soon as hot gases are passing through separating elements.

For massive timber structures including CLT three flame spread paths can be identified. These must be taken into account within the design process to ensure an overall fire safety for buildings using CLT:

- in-plane joints to neighbouring prefabricated elements
- joints in junctions of components and to other building parts
- joints resulting from service installations and penetrations

A schematic of these paths is given in Figure 1.

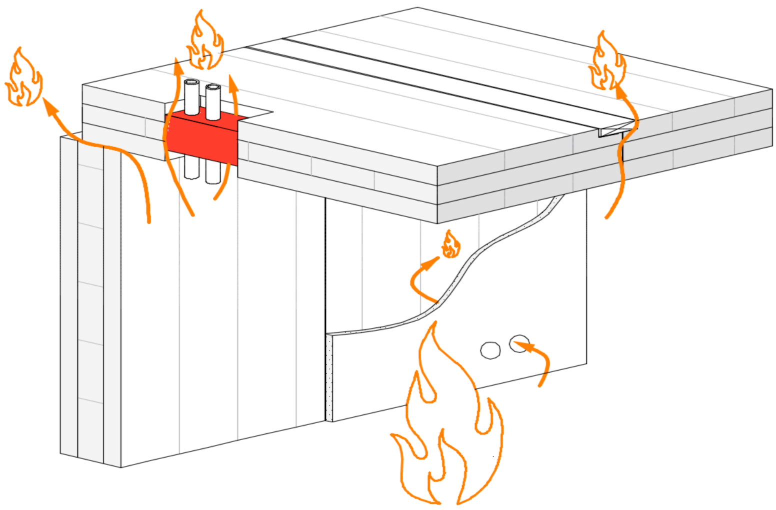


Fig. 1 Flame spread paths for buildings using CLT.

2. In-plane Joints of CLT Elements

In recent years many studies dealt with the evaluation of CLT elements for walls and floors with respect to load bearing or separating function in the case of fire. The main part of these research projects or industrial reports used standardised fire tests according to EN 1363 [6] and EN 1365 series [7]. Using these standards CLT wall and floor elements showed a fire resistance of up to 90 minutes. Beside the element itself these fire tests investigate the in-plane joints of the CLT elements. Moreover, full-scale natural fire tests were performed.

To evaluate the performance of in-plane connections current testing standards use the “EI” criterion according to EN 13501-2[1]. This approach ensures that the temperature does not increase more than 180°C in relation to ambient conditions and that hot gases do not ignite objects on the unexposed side. Some reports also investigate the smoke-tightness as a third criterion, which is not a standardized criterion so far. This leads to the situation that the results are hard to compare between different reports. The criterion of smoke tightness contributes to evaluate the overall fire performance as well as the efficiency of different measures for in-plane joints with CLT. Table 1 summarises fire tests with respect to in-plane joints from the last years.

Table 1: Overview of selected CLT fire tests including element joints.

Reference	**Description**
Frangi & Fontana 1999[8]	Small scale and full scale fire tests with hollow core CLT elements for 60 and 90 minutes; standard fire exposure including three different configurations of joints.
Polleres & Schober 2004 [9]	Fire tests to asses different element joints using an external single spline on the surface or an interior spline; 140 mm massive timber floor; standard fire exposure
Hosser & Kampmeier 2008 [10]; Kampmeier [11]	1) Small scale fire test to assess smoke tightness of massive timber elements including connecting joints, 160 mm thick unprotected elements, standard fire exposure to an area of 450 x 450mm², three different joint configuration
	2) Mid-sized scale test to assess smoke tightness and thermal integrity of massive timber elements for element joints and joints in wall-floor junctions,110 mm thick elements, standard fire exposure to an area 1200 x 1600 x 500mm³
	3) Full scale test to assess fire resistance and smoke tightness of three different connections, 120 mm thick massive timber elements including CLT; standard fire exposure
Winter & Stein 2007 [12]	Full scale tests with loaded massive timber element
Association for glued timber products 2013 [13,14]	Full scale tests of protected wall elements 0,08 x 2,98 x 3,28 m; 3-layered CLT elements; standard fire exposure over 90 minutes
Mc Gregor 2013 [15]	Full scale tests assembly 3,5 x 4,5 x 2,5 m³, 3 layered CLT elements; natural fire exposure

The outcome of all tests can be summarised as follows: Joints may lower the fire resistance and influence the smoke tightness in a negative way. Gaps resulting from fabrication inaccuracy or needed construction tolerances allow hot gases and smoke to pass through in the presence of over-pressure under fire conditions. Especially butt connections should be prevented or at least need additional actions.

In this context, McGregor (2013, [15]) found gases escaping from individual CLT elements as well as from the joints between neighbouring elements in many places in his first tests. Therefore, he used a fire rated silicon in all following tests to improve the performance of the element joints. This was effective, but still gases were observed escaping from the spatial elements to some degree. He reported an increase in temperature and a glowing combustion at the unexposed surface of an element joint. This burning-through occurred earlier than in the undisturbed panels (Fig. 2).

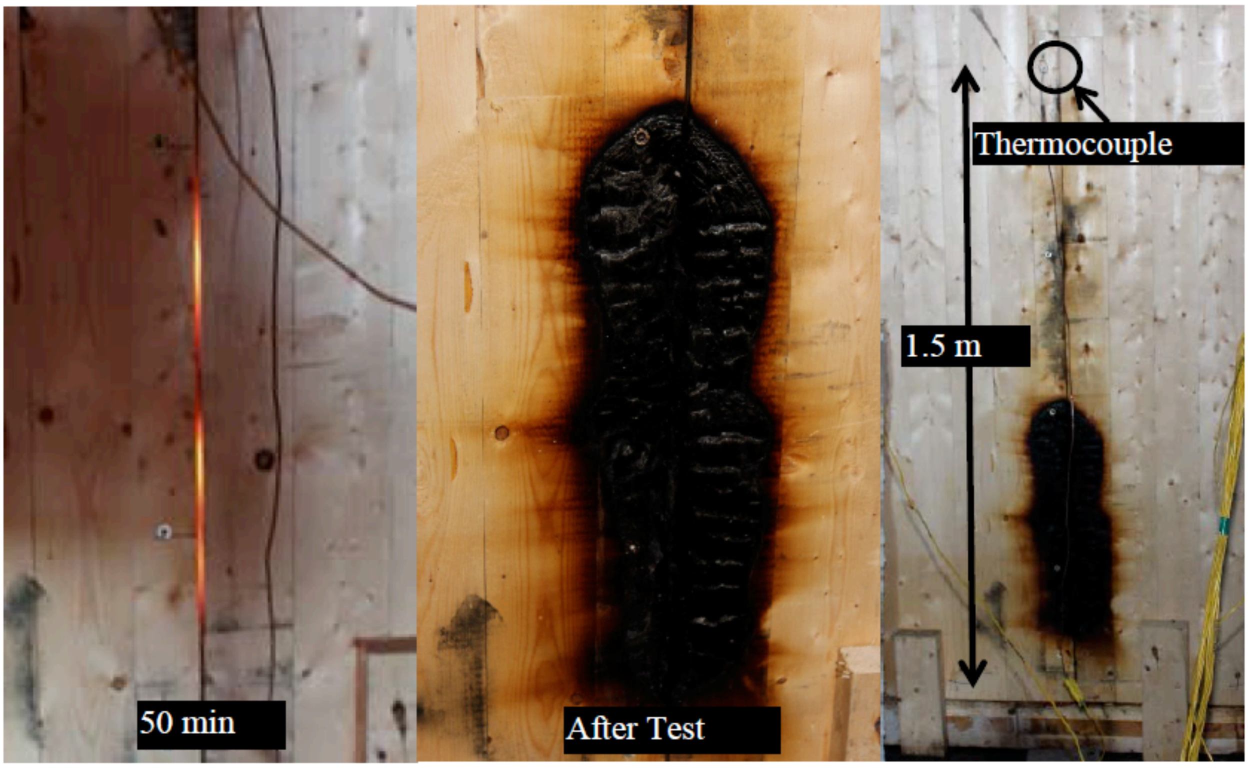


Fig. 2 Results from McGregor's work [15].

To avoid flow paths Hosser and Kampmeier (2008, [10]) examined the performance of compressed mineral wool implemented in simplified element joint configurations (Fig. 3). From the small-scale tests it was concluded that a 10 mm compressed mineral wool stripe is reasonable to achieve smoke-tightness in the in-plane element joints.

Further full-scale tests investigated the performance of realistic element joints using exterior splines and single or double tongue and groove joints. With respect to the smoke tightness, all in-plane element joints failed within 60 minutes in the tests although the double tongue and groove joint performed better as the single one. However, the separation function was fulfilled during the entire tests (Fig. 4). Therefore, the authors recommend using an elastic joint sealant on both sides of the connection if the element size does not allow an even compression of a mineral wool in the element gap due to structural purposes or fabrication inaccuracies.

Fig. 3. Variations of in-plane CLT element connections with 10 mm compressed mineral wool stripes (in green), Kampmeier (2008) [4], *Fig. 24.*
A) butt joint with mineral wool
B) step joint with mineral wool
C) interior double spline with mineral wool.

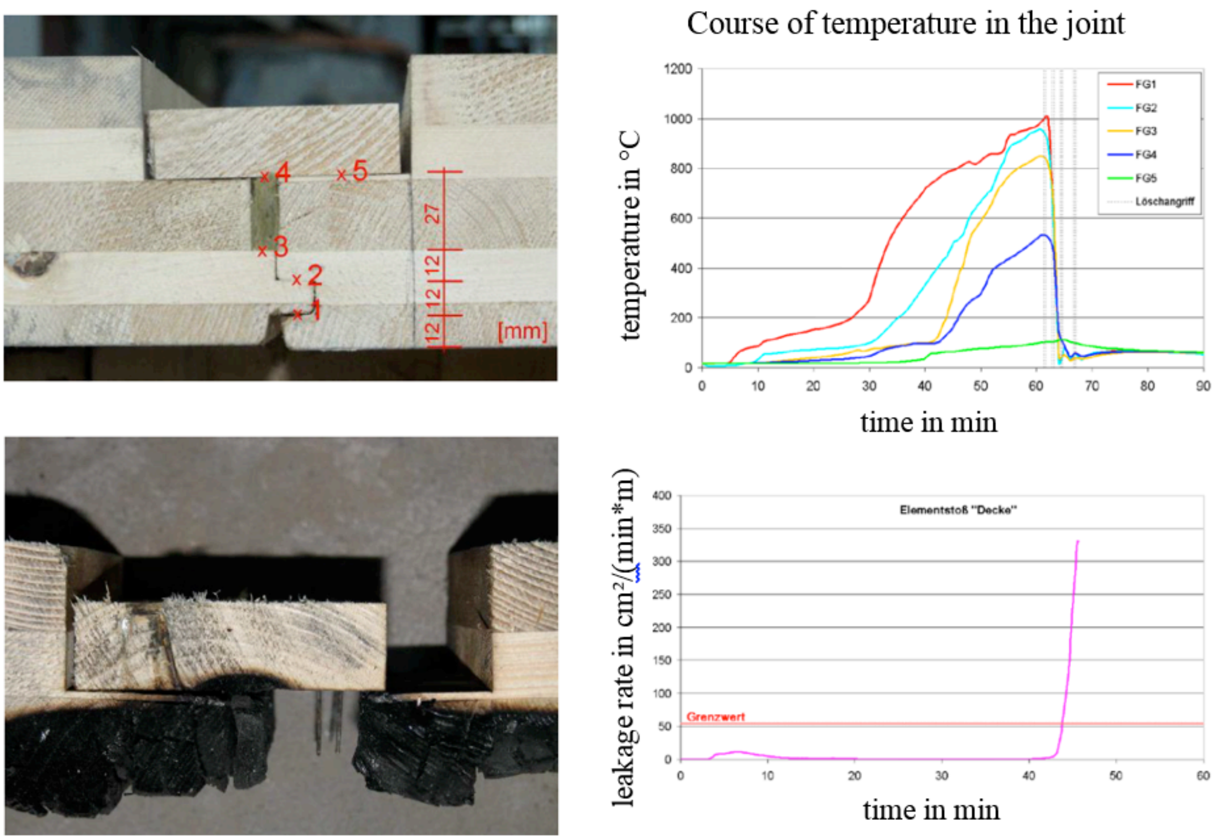


Fig. 4 Assessed element joint with location of thermocouples before and after the fire test and course of the temperatures and leakage rate during fire exposure, Hosser and Kampmeier (2008, [10]*).*

Nowadays, CLT element joints are normally based on exterior splines or step joints. These joints have been tested in Polleres and Schober (2004, [9]) or in tests of the Association for glued timber products (2013 [13], [14]). These covered and uncovered fire test show a fire resistance of the element joints of more than 90 minutes.

Teibinger (2012, [21]) derived from the Austrian tests that the fire safety will be reached if the remaining cross section covering an interior double spline, a step joint or an exterior spline is at least 2 cm. To avoid hot gases passing through additional sealing generally used for the purpose of air tightness were implementedin the tests of the Association for glued timber products ([13, 14]).

Frangi and Fontana (1999, [8]) confirm these statements with their investigation on hollow core CLT elements. The highest fire resistance was achieved using element joints where all cavities are filled with mineral wool in combination with a tongue and groove joint on the exposed as well as on the unexposed side. A big amount of smoke gases passed through joints using intumescent material as it takes a while to achieve the activating temperature of this material. A similar behavior was found in Winter and Stein (2007, [12]).

3. Corner Connections of CLT

Similar to element joints, joints in corner connections and joints to other building parts need an equivalent fire resistance. The aim is to prevent the spread of fire and smoke to other fire compartments. However, no standardised test method exists at the moment to assess the performance of fire exposed corner junctions. Therefore, existing test data and recommendations are based on tests following in general the EN 1365 series [7] procedures but also on small-scale tests or full-scale natural fire tests.

Teibinger (2011, [16]) tested two different corner connections with respect to fire performance. The CLT wall was lined with a 12.5 mm gypsum plasterboard and connected to a glulam floor element using a PUR elastomer vibration absorber to prevent sound transmission (Fig. 6a). In one test, the elastomer support was additionally sealed with an intumescent sealing compound at the exposed side, in another test, a simple non-fire rated acryl-sealing was used. The test with the intumescent sealing showed excellent results with no additional charring within the connection but also the second test with an acryl sealing reached 90 minutes without failure of integrity or escaping of smoke (Fig. 6a). Both setups met the same fire resistance as for the spatial elements.

Equivalent test results were reported by Merk et al. (2014, [17]) for mid-size scale fire tests with unprotected CLT floor elements and K_260 encapsulated walls. An elastomer vibration absorber was installed in the junction as typically used in practice (Fig. 6b). The vibration absorber was partly covered by the encapsulation cladding but no further sealing was applied at the fire exposed side. Within the tests only little penetratation of smoke occurred and no glowing combustion became evident. After testing, neither the connection nor the elastomere showed any fire impact (Fig. 6c).

Hosser and Kampmeier (2008, [10]) tested corner connections without any lining. They found that the connections easily resist a 60 minutes fire if the entire depth of the element is filled by a 10 mm mineral wool stripe, which is compressed to 5 mm when connecting the elements. All corner connections were secured with outside surface splines as well. The authors also pointed out, that the measured charring depths within the corner were less compared to the spatial elements and explained this fact with the lower heat flux density at inside corners.

(a) (b) (c)

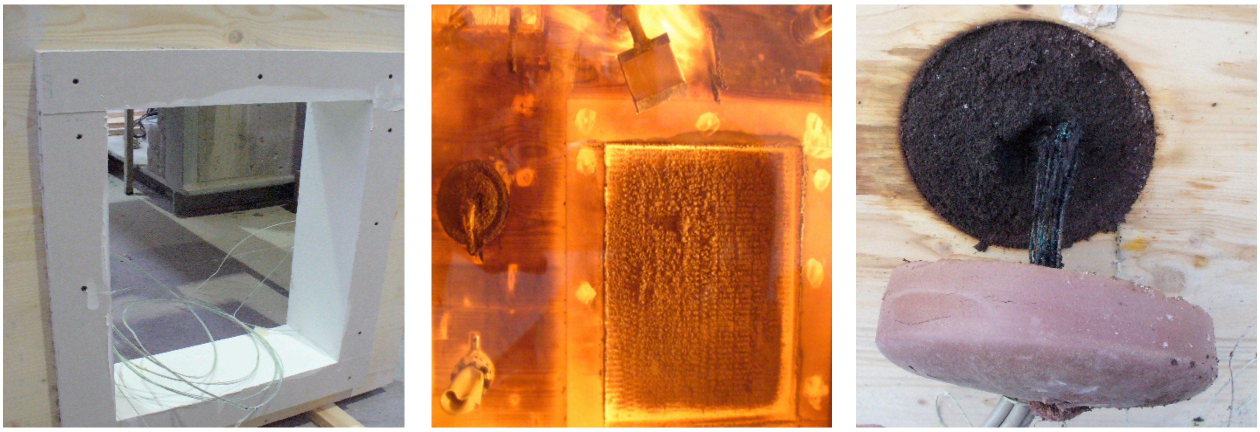
Fig. 6 Fire test with elastomer vibration absorber in wall to floor junction.
a) source [16], *b) source* [17], *c) source*[17].

When integrity requirements cannot be fulfilled by the CLT panel alone additional linings can be used to increase the failure time. These linings will not only contribute to an improved fire resistance of the CLT element but also to a reduction of the smoke permeability and a better thermal integrity of the element junction. These findings were derived by Winter and Stein (2007, [12]) from smoke tightness and fire resistance tests under ISO fire exposure with timber frame and CLT elements. Similar to in-plane element joints all examinations underline the need of an air tight sealing which is also required with regard to building physics such as for sound- and thermal insulation purposes.

4. Service Penetrations and Mounting Parts

In principle, penetration through fire rated assemblies should be limited. If they are essential for the use of a building or a unit by certified systems to maintain the assembly's fire rating. Until now approved sealing systems for service installations are typically only available for drywall or concrete constructions. Tested and approved solutions for timber structures are rare and slowly reaching the marked, even though they can be tested in accordance with EN 1366 series [18]. In general, fire tests and technical approvals show that every type of service installation passing through fire separating elements has its own specific characteristic, level of performance and, therefore, range of application. Hence, there is no single solution or product that will be used for all services and protects all elements in the same manner to avoid early fire spread. However, some research projects tried to provide general solutions in order to adapt existing and approved sealing systems for a fire safe use in timber structures like CLT (Fig. 7, Werther et al. 2012, [19]).

Investigations of Werther et al. (2012, [20]) comprised tests with penetrations of single wires, cable bundles, combustible service pipes, non-combustible service pipes and mixed penetrations. It was found, that systems with intumescent materials efficiently seal the gaps between the supply line and solid timber elements. For passive systems without capacity to expand under fire exposure a further sealant should be applied on both sides of the penetrated element. As a main concept to install multi penetration sealing systems, such as mineral wool boards, a non-combustible lining of the area over the entire thickness of the separating element is recommended.



(a) (b) (c)

Fig. 7 Fire test for various penetration sealings (Werther et al. 2012, [19]*).*

In addition, Teibinger and Matzinger (2012, [21]) tested sealing systems in CLT walls and floor elements for more than 90 min fire resistance. They also investigated potential joining details of service shafts and CLT floor elements. The tests showed that all sealing systems in the solid timber element fulfilled the requirements. However, the authors pointed out, that intumescent systems should be used preferably and the fastening means must be designed according to the aimed fire resistance.

With respect to mounting parts in CLT elements, like sockets and recessed electrical boxes that penetrate a fire rated lining or encapsulation cladding Merk et al. (2014, [17]) recommend an intumescent coating, to protect the timber behind the penetrated lining (Fig. 8). The intumescent coating was applied not only in the recession of the CLT elements but also at its surface circular around the penetration. This procedure prevented an early ignition and burning of the timber, because the protective lining always arched upwards during the fire exposure.

(a) (b) (c)

Fig. 8 Fire tests with recessed sockets in CLT treated with intumescent coating (the activation of the intumescent coating resulted in a black coloring at surface) (Merk et al. 2014, [17]*).*

5. Conclusion

To restrict the spread of fire and smoke and maintaining the integrity of fire seperating CLT elements several studies have been conducted. The focus lied on joining details, resulting from in-plane element joints, component connections and service installations.

All studies show that the prevention of flow paths is one of the essential measures to fulfil the fire safety requirements for the entire structure. For element joints and junctions, like wall to wall and wall to floor connections, the fire safety can easily be reached if the requirements for statics and building physics are fulfilled. The solid nature of CLT supports these characteristics. Several fire tests show that existing penetration sealing can be used in combination with CLT elements to assure fire safety.

Approved details for designing fire safe CLT structures can be taken from construction catalogues, such as published by the Holzforschung Austria (Teibinger and Matzinger 2013, [22]) or Technical University of Munich (Merk et al. 2014 [17]).

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Minutes of Presentation VIII:
Detailing of CLT with Respect to Fire Resistance

Presentation by Stefan Winter

Summary:

Stefan Winter introduces the audience to a very important, often neglected topic in fire design: detailing. This is often of low priority as many test standards and calculation rules cover “two-dimensional” views only, i.e. cross-section verifications whereby in reality connections and joints are sensitive points where fire or smoke spread may occur more often.

Tests of these sensitive locations, e.g. joints between wall and floor, are rare for all materials. SW presents research results where wall-floor joints are tested for CLT elements to investigate different joint assembly types (in-plane, L-, T-, X-type).

Any service installation (e.g. cable penetrations) normally decrease the fire resistance of the element (criteria E, I), thus special actions have to be taken here.

For CLT in-plane joints may be Z-shaped or using a fitting board which completes the lower or upper outer layer. Test results showed that many solutions included weaknesses; corner joints deform under loading, which is normally not considered by the designer.

Winter presents test results of conventional sealing systems applied in “gypsum frames”; results were satisfying.

Connections Between CLT Elements and Future Challenges For CLT in practice

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Summary

Since its launch over 15 years ago cross-laminated timber (CLT) has gained widespread use with production capacity in central Europe now reaching approximately 600,000 m^3 per year and worldwide production approximately 700,000 m^3 [1].

Use of CLT in the UK has seen a rapid increase over the last 10 years with several large school buildings of 10,000m^2 gross internal floor area and large-scale residential building buildings up to 8-storeys having been delivered. The signs are that this increased use of CLT is set to continue.

However, despite its widespread use, engineers in the UK still face several challenges when designing and delivering CLT buildings such as:

- inefficiencies in the CLT procurement process
- lack of detailed CLT design guidance
- insufficient or inconsistent manufacturers technical data for connections (screws & brackets)

The following paper seeks to highlight these challenges by describing typical UK practice used to deliver CLT buildings.

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1. Introduction

Cross-laminated timber is manufactured using generally Spruce planks that are glued together in alternating transverse and longitudinal layers. The resultant solid timber panels can be manufactured in thicknesses ranging from 60mm to 300mm and panel sizes up to 2.95 m wide x 16.50 m long. The panels are manufactured and pre-cut (forming all joints & openings) in the factory to provide an offsite/prefabricated building structure. The prefabricated panels are delivered to site flat-packed (to optimise transport efficiency) and erected by specially trained erection teams. All panel connections are formed on site, typically using nails and self-drilling screws. There is no pre-drilling of the panel connections unless the connection is to be left as exposed as a feature of the building finishes.

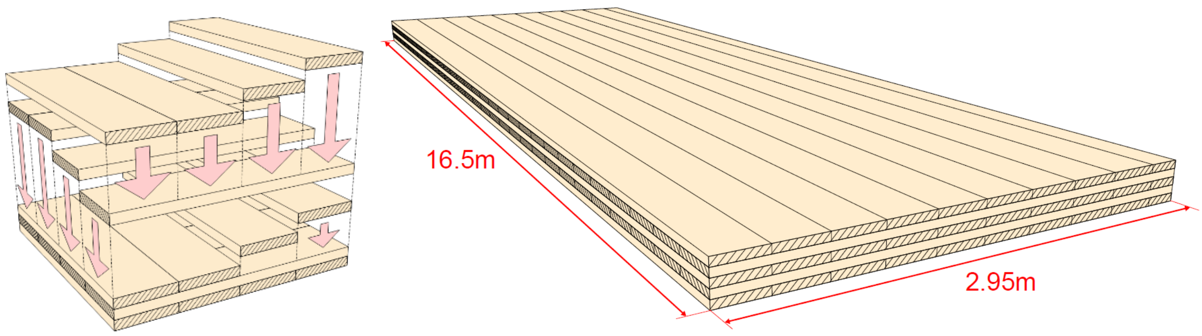


Fig. 1 Cross-Laminated Timber.

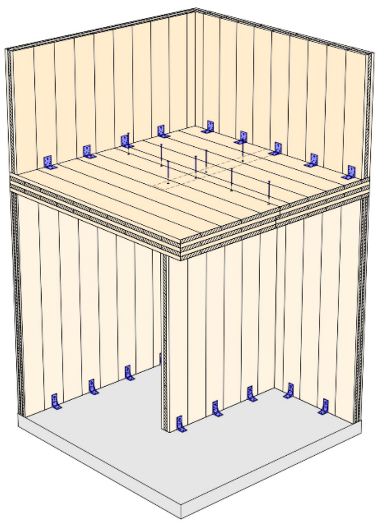
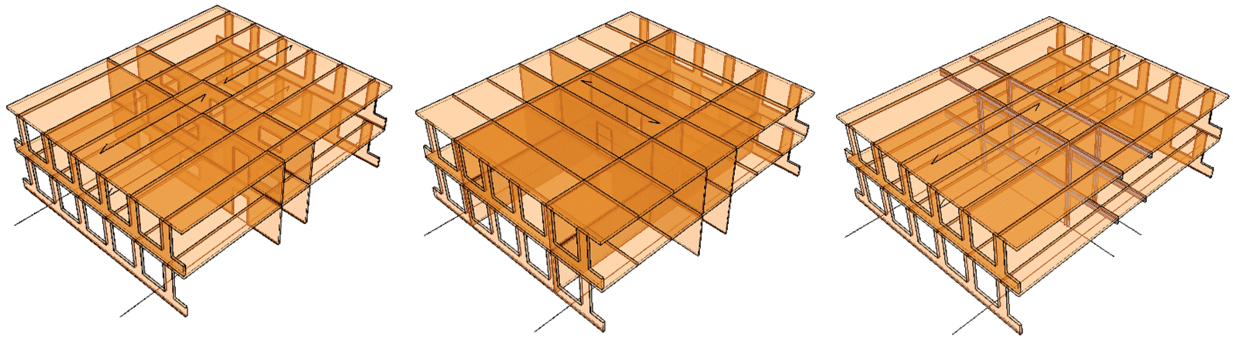


Fig. 2 Platform construction illustration.

Platform construction is the term used to describe the method of 'stacked' construction used in most modern multi-storey timber structures. Floor panels bear on top of the wall panels and subsequent wall panels are then erected directly on top of the floor panels. The panels are fixed together using a combination of screwed half lap joints and proprietary metal brackets (Fig. 2). These are described in more detail in the following sections.



(a) (b) (c)

Fig. 3 Illustration of 3 different CLT structure types.

CLT buildings can typically be sub-divided into three different structural forms:

a) Crosswall
b) Loadbearing façade/corridors
c) Hybrid (CLT/steel frame, CLT/glulam frame)

2. Design & procurement of a CLT building (typical UK practice) – the challenges

To begin the structural design of a CLT building requires the specific material and structural properties to be known. This is obtained from the product technical data produced by the CLT manufacturer (typically a European Technical Approval document). However, unlike precast concrete or structural steelwork, there is variation in the panel properties and structural performance across the different manufacturers. There has been some collaboration by some of the CLT manufacturers to 'standardise' overall panel thickness, which is helpful to the engineer.

Table 1: Panel thicknesses from main CLT manufacturers.

Manufacturer	Panel overall thicknesses (mm)													
KLH	60	78	--	90	95	100	108	117	120	125	140	145	--	162
Binderholz	60	--	80	90	--	100	--	--	120	--	140	--	160	--
MMK	60	--	80	90	--	100	--	--	120	--	140	--	160	--
Stora Enso	60	--	80	90	--	100	--	--	120	--	140	--	160	--

Manufacturer	Panel overall thicknesses (mm)													
KLH	180	182	200	201	208	--	226	230	--	248	260	280	300	320
Binderholz	180	--	200	--	--	--	--	--	--	--	--	--	--	--
MMK	180	--	200	--	--	220	--	--	240	--	260	280	--	--
Stora Enso	180	--	200	--	--	220	--	--	240	--	260	280	300	320

However, even considering this element of standardisation, there are still variations in the panel performances due to:

1. Variation in the overall panel thicknesses produced by a selection of different CLT manufacturers (Table 1).
2. Variation in the board thicknesses within the same panel thickness produced by two different CLT manufacturers (Fig. 4).
3. Variation in the material properties of a selection of different CLT manufacturers (from manufacturer's ETA documents) (Table 2).

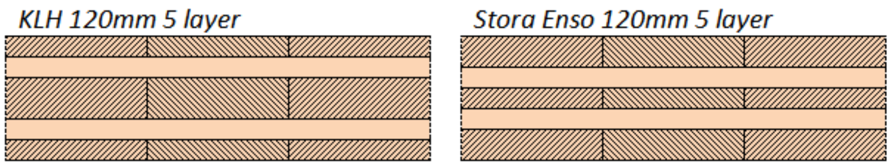


Fig. 4 Cross sections of two CLT panels with the same overall thickness from different manufacturers.

Table 2: Performance properties from main CLT manufacturers for panels made with C24 boards.

MANUFACTURER		Binderholz	KLH	MMK	Stora Enso
Density (kg/m^3)	ρ_{mean}	480	550	480	500
Actions perpendicular to the panel (N/mm^2)					
Modulus of Elasticity	$E_{0,mean}$	11000	12000	11600	12500
	$E_{0,05}$	7400	9500	7772	7400
	$E_{90,mean}$	370			
Shear Modulus	G_{mean}	690	690	650	690
	$G_{R,mean}$	50			
Bending Strength	$f_{m,k}$	24	24	24	26
Tensile Strength	$f_{t,90,k}$	0.4	0.12	0.12	0.12
Compressive Strength	$f_{c,90,k}$	2.5	2.7	2.5	2.5
Shear Strength	$f_{v,k}$	2.5	2.7	2.5	4
	$f_{R,v,k}$	0.7	1.5	1.1	1.25
Actions in plane of the panel (N/mm^2)					
Modulus of Elasticity	$E_{0,mean}$	11000	12000	11600	12500
	$E_{0,05}$	7400	9500	7772	7400
	$E_{90,mean}$	370			
Shear Modulus	G_{mean}	250	250	250	460
Bending Strength	$f_{m,k}$	24	23	24	24
Tensile Strength	$f_{t,0,k}$	14	16.5	14	14
Compressive Strength	$f_{c,0,k}$	21	24	21	21
Shear Strength	$f_{v,k}$	2.5	5.2	5	2.5

The recently published EN 16351:2015 [2], which sets-out minimum standards for CLT production may help to reduce the differences in the panel performance properties. However, currently this variation between CLT manufacturers creates a problem for the engineer designing a CLT building as it means a CLT manufacturer must first be selected in order to commence the design of the CLT structure and confirm the panel thicknesses.

This variation also creates inefficiencies and challenges in the procurement process for a CLT building which in the UK typically comprises fast track 'Design & Build' procurement approach whereby the main contractor will tender the CLT design (based on a preliminary design of the CLT structure) in order to select the CLT supplier. This means the engineer must either adopt a generic, 'loose fit', design approach to produce a preliminary design or produce three different CLT structure designs for three different CLT manufacturers. Following selection of the CLT manufacturer the final design (or re-design) of the CLT structure can be completed. Whichever method is adopted, the process is inefficient and puts pressure on the design process. This process is illustrated in the simplified flow diagram below (Fig. 5).

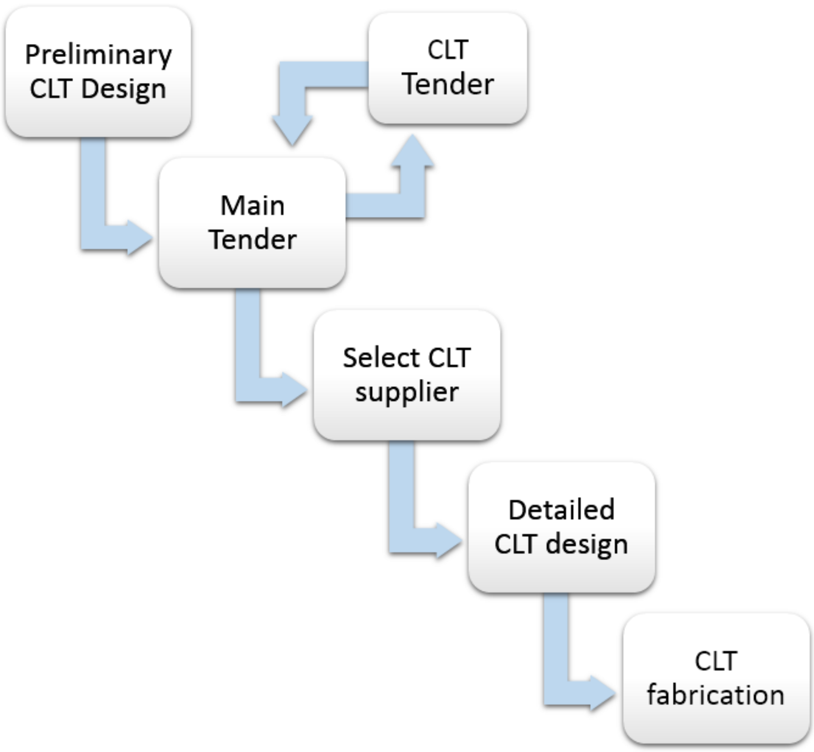


Fig. 5 CLT procurement and design process – typical UK project.

3. CLT Connection types – typical UK practice

3.1 Overview of typical CLT platform connector types

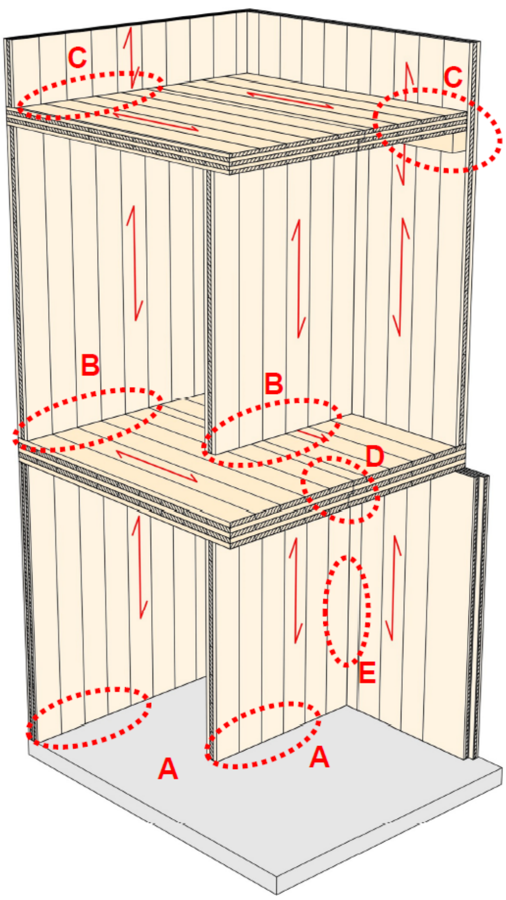
Typical CLT platform construction depends on the connections of all the individual 2-dimensional panels to form a 3-dimensional, stable structure. Figure 6 shows the main types of joints & connections used on a typical UK CLT project.

The typical connectors (Fig. 7) used in CLT construction can be categorised into two types:

1. Screws
2. Proprietary metal brackets/3-dimensional nail plates (3DNP)

Both types of connectors have many different sub-types specifically designed for different applications, which have to be taken into account for an efficient design.

The engineer has to commence the design of CLT connections by firstly selecting the connector type, and more specifically the manufacturer. The reason the latter is important is the fact that (similar to CLT manufacturers) there are differences in technical data and load capacities across the different connector manufacturers.



A. CLT wall to concrete connection
B. CLT wall to CLT floor connection
C. Roof parapet connection
D. Half lap
E. Wall junction

Fig. 6 Typical joints/connections of a CLT platform structure.

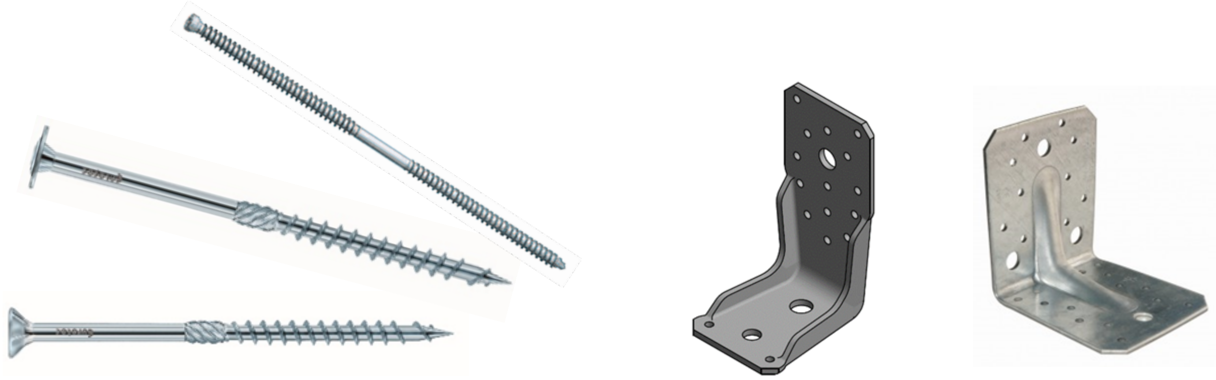


Fig. 7 Typical screw & bracket types.

3.2 CLT wall to Concrete connection (Type A)

CLT wall panels are connected to the concrete foundation using 3DNP. These are fixed down to the concrete foundation with a post-drilled mechanical anchor, and fixed to the base of the CLT wall panels using nails and/or screws. The wall panels are located on top of levelling shims to account for the slab level construction tolerances. Once panels are installed and fixed the gap between the bottoms of wall panels and the concrete slab are filled with non-shrink cementitious grout to ensure the structure loading is transferred to foundations as uniformly distributed line loads.

In this type of connection the main forces the connections are required to resist are:

- In plane shear loads (e.g. stability wall)
- Out of plane shear loads (e.g. external wall)
- Uplift tension (e.g. stability wall)
- In and out of plane shear loads (due to 'disproportionate collapse' load case)

(a) (b) (c)

Fig. 8 Typical CLT wall to concrete brackets: a) Hold down, b) TITAN, c) AKR135.

There is a range of different type of connectors to solve this connection (Fig. 8), each with different performance values. Three of the most typical 3DNP are:

1. Hold-downs (Fig. 8a) transfer high tension/uplift loads down to the foundations. Usually the limiting factor is the steel (bracket strength) and the anchor into the concrete (concrete pull-out failure). Due to the high load capacity a reduced number of connectors may be required to take the uplift load. However, this can result in concentrated loads/reactions causing problems for the design of the supporting foundations and a reduced degree of redundancy in the connection.
2. Shear brackets like the TITAN from Rothoblaas (Fig. 8b) provide a significant shear load capacity if they are fully nailed. The shear load transfer creates a tension load to be resisted by the anchor. This makes the steel and/or the concrete anchor the limiting factor. A thick washer can be added to resist tension loads. However the use of the washer creates a lever arm effect in the bottom flange thereby potentially increasing (by a factor of up to 2.0) the uplift load to be resisted by the anchor.
3. Post bases like the AKR135 from Simpson Strong-Tie (Fig. 8c) have stiffening flanges to transfer the uplift without increasing the load on the anchor and can transfer shear loads unlike the large hold-downs. The maximum capacities are lower than the previous two options increasing the number of connectors needed per wall length. Thus the loads are distributed along several brackets. This type of connector is sensitive to the edge spacing of the fixings in the bottom of the CLT wall panel (i.e. in the case of uplift). It is therefore important to carefully control construction tolerances of the concrete slab levels.

Fig. 9 Typical CLT wall to concrete connection with grouted gap at wall base.

Not all the connectors described can transfer all the load directions required. In such cases different connector types need to be mixed in one wall to solve the connection. There are two main options: The use of hold downs at either end of the wall combined with shear brackets at the centre, or the use of the smaller post base connectors which are able to take tension and shear loads distributed along the whole length of the wall. The latter is the usual solution with brackets typically spaced at 300-500mm centres as shown in the figure below.

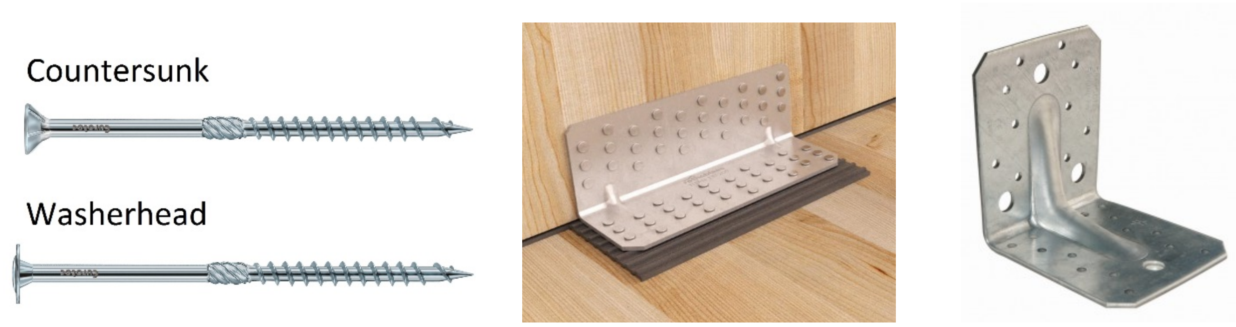
3.3 CLT wall to CLT floor to wall connection (Type B)

This is the typical connection in platform construction where CLT floor slabs are supported on top of the walls and the next wall is located on top of the slab. Some of the benefits of this method are the ease of installation, the accommodation of installation and fabrication tolerances and the direct load path to the horizontal shear loads between floor slabs and walls. One of the disadvantages is that in taller buildings, where vertical loads are higher, the compression perpendicular to the grain on the floor slab tends to be the limiting factor.

The range of loads this type of joint needs to transfer is the same as those detailed in the previous connection, type A.

Connection type B can be subdivided into two:

1. Slab to wall: CLT subcontractors prefer to fix the floor slabs down to the top of the wall below using structural washer head screws partially threaded (Fig. 10a). They provide shear capacity and clamp the panels together when the threaded part is on the point-side member. However if there is a need to transfer higher loads and/or the panel below has 5 layers with the screw going into the end grain it may be necessary to use countersunk screws installed at an angle to the grain. If that solution does not meet the requirements, a 3DNP can be fixed from the wall to the slab soffit. However, this creates difficulties and inefficiencies during installation and it should be avoided if possible.
2. Wall to slab: The most frequent solution for this connection is a 3DNP partially or fully nailed to both wall and floor panels. There are many different types of sizes and load directions that these can resist. However they can be divided in two main groups; i) larger brackets which only transfer shear loads (Fig. 10b) or ii) smaller brackets with shear and tension performance capacities (Fig. 10c). On the one hand, the latter type spread the load over a greater length of the wall as they are typically located at 300 mm to 500 mm centres (Fig. 11). If partially nailed brackets are specified, it should be clearly marked and checked later during installation which holes need to be nailed as it is critical to achieve the specified structural performance. On the other hand, if the wall is subjected to high loads and bigger shear brackets are needed, most likely a bespoke connection to take the uplift generated by that shear load will need to be designed.



(a) (b) (c)

Fig. 10 a) Structural timber screws (Eurotec), b) TITAN bracket (Rothoblaas) and c) ABR105 bracket (Simpson Strong-Tie).

Fig. 11 Typical CLT wall to slab connection showing ABR105 brackets.

There are other factors to take into account when designing this connection. For example, when the connection is on the external walls the brackets can only be fixed from the inside whereas when the connection is on internal walls the brackets can be staggered to either side of the wall (Fig. 12). Also it is important to account for the architectural finishes and whether or not they will cover the connector if the CLT wall is exposed.

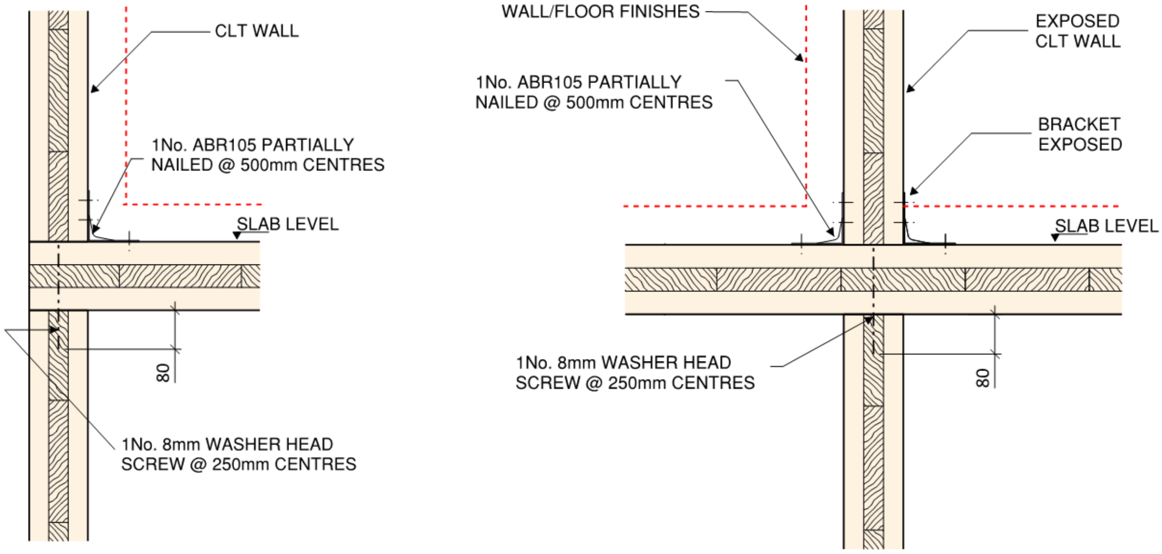


Fig. 12 Typical detail of internal and external CLT wall to CLT floor.

3.4 Half laps and wall junctions (Types D & E)

These connections are made to notionally tie all panels together and typically use washerhead-type screws installed through the half lap joint at 250 mm centres. They are also used in the design as horizontal ties for disproportionate collapse design to NA to BS EN 1991-1-7 [3], or to tie together the panels when using the notional removal approach.

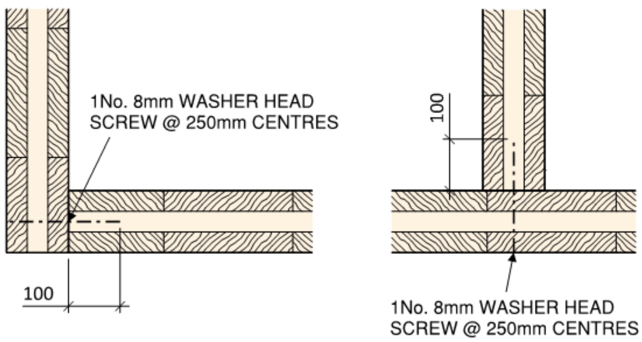


Fig. 13 Typical wall junction detail.

The wall-to-wall junctions are usually formed with washer head screws to simply clamp the panels together (Fig. 13). If there is a need to transfer axial loads by the screw, countersunk screws may be required installed at an angle to avoid going into end grain. However if they need to transfer a higher load the connection can be solved as the wall to slab connection using 3DNP.

When designing half laps to joint two floor slab panels (Fig. 14) there is a conflict between two design criteria: i) minimum timber wastage and ii) maximum edge distance. The width of the half lap should be kept to a minimum to optimise timber wastage. This is when an accurate minimum edge distance of a screwed connection on CLT (this is not well documented in ETAs or other relevant timber standards) plays a significant role. For example, a 50mm wide half lap (25mm edge distance) over the two long edges of a 12.5 m long x 2.4 m wide panel represents a timber wastage of more than 2 % in volume.

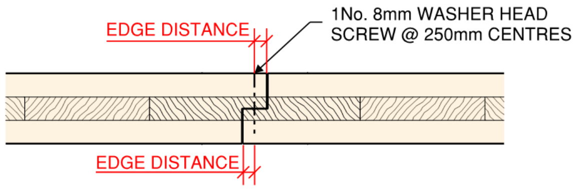


Fig. 14 Typical half lap detail.

If the half lap has been designed to transfer loads across the panel joint, it should be clearly specified in the drawings as a special connection detail. Also it is good practice to check it has been included in the production drawings and on-site checks during CLT erection should check the detail has been fixed as specified. A reduction of the half lap or the incorrect installation on site could lead to the failure of the connection.

3.5 Roof parapet connections (Type C)

Unless the roof is used as a terrace, the main action the parapet connection to the roof slab has to transfer is the horizontal wind load. Depending on the height and/or the wind pressures there are different approaches to solve the joint. From lower to higher demand they are:

1. If the parapet height is not significant and/or most of the parapet height is going to be covered by the roof finishes, the solution of a small ribbed 3DNP and a screw through the roof slab described above for the external wall to floor to wall connection may be enough to transfer the moment and shear actions (Fig. 15a).
2. The second option uses the same detail as the first one but adding a nail plate in the external face (Fig. 15b). This option has the disadvantages that the connection has to transfer a moment created by the wind load as option 1 and that the nail plate has to be fixed to the external face, which sometimes is not possible depending on the installation approach chosen by the CLT contractor.
3. The third option shows a change in the panel layout. The roof slab is not supported on top of the wall. Instead it has the wall running through with the slab supported by a CLT strip or timber member screwed in the wall (Fig. 15c). It can be beneficial when the parapet heights are significant or the wind causes a moment in the junction, which is too high for option 2 but can be taken by the wall as a cantilever member. The installation of the roof slab of this detail can be more complicated.

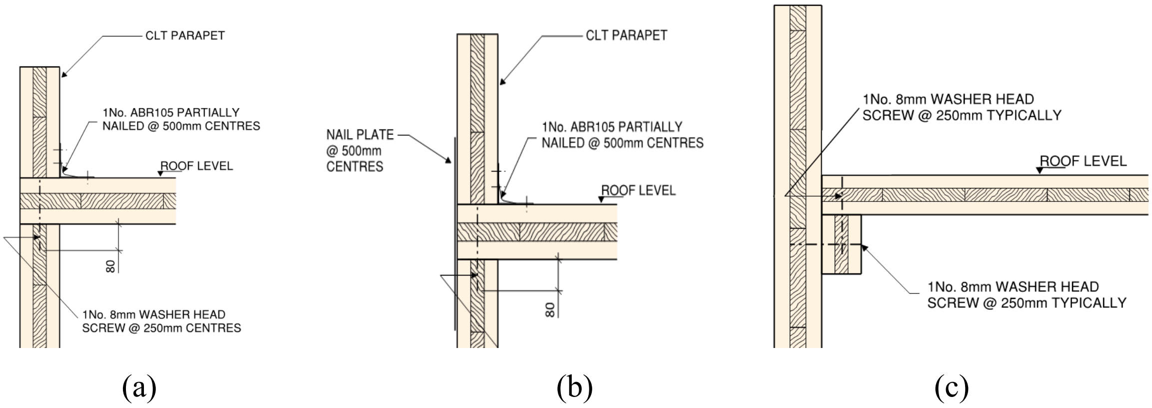


Fig. 15 Roof parapet connection options.

3.6 Special connections

There are some cases where a special connection detail is required to solve a localised high concentration of loads. Some typical examples of where special connections may be required are given below:

- High concentration of compression loads (e.g. reactions from stability walls or columns) can exceed the bearing capacity of the CLT. This is particularly likely if the wall or column is bearing on top of a CLT floor slab. There are three possible solutions: i) increase the wall thickness (or column section size), ii) locally reinforce the CLT floor panel with fully threaded screws (there are ETA's from screw manufacturers with methods to calculate buckling loads on self-tapping screws within timber) or iii) add a steel plate in the interface to spread the load over a larger area.
- A recent example encountered by the authors is a situation that required especial connections on cantilever stair half landings with steel angles (top and bottom) to form a moment connection (Fig 16).
- A common example of where non-standard connections are required is the fixing of localised steel elements such as lintel beams as shown in figure 17. This example shows how the bearing detail/connection can be pre-cut in the factory to help installation on site and minimise the number of connectors required.

Fig. 16 Special moment connection for a cantilever stair half landing slab.

Fig. 17 Special detail to support end bearing of a steel lintel beam.

4. Procurement of Connection Design – the challenges

It is typical UK practice for the detailed CLT connection design to be carried out as part of the CLT supplier/specialist contractor. This creates a problem for the CLT procurement process as discussed below.

In a typical CLT building the cost (supply only) of connections accounts for approximately 2-5 % of the total CLT package cost (supply & erect cost). Whilst this cost may not appear as particularly significant, the influence of the structural design (i.e. type of connection selected) and the installation (i.e. time required) of connections is of major significance. The first action when designing the CLT connections is selection of the manufacturer of the screws and brackets. However, this creates a challenge for the engineer due to:

1. Different manufacturers = different connector types
2. Different manufacturers = different load capacities

The technical data for the connectors is typically contained in the manufacturers ETA document. However, there is considerable variability across different manufacturers in the availability and consistency of the technical data required by the engineer for the connection design – thereby creating another challenge in the design process.

In addition to this the procurement process of the design of CLT connections creates challenges for the engineer in similar manner to the CLT procurement process outlined previously in this paper. This is illustrated in the flow diagram below but can be described as follows:

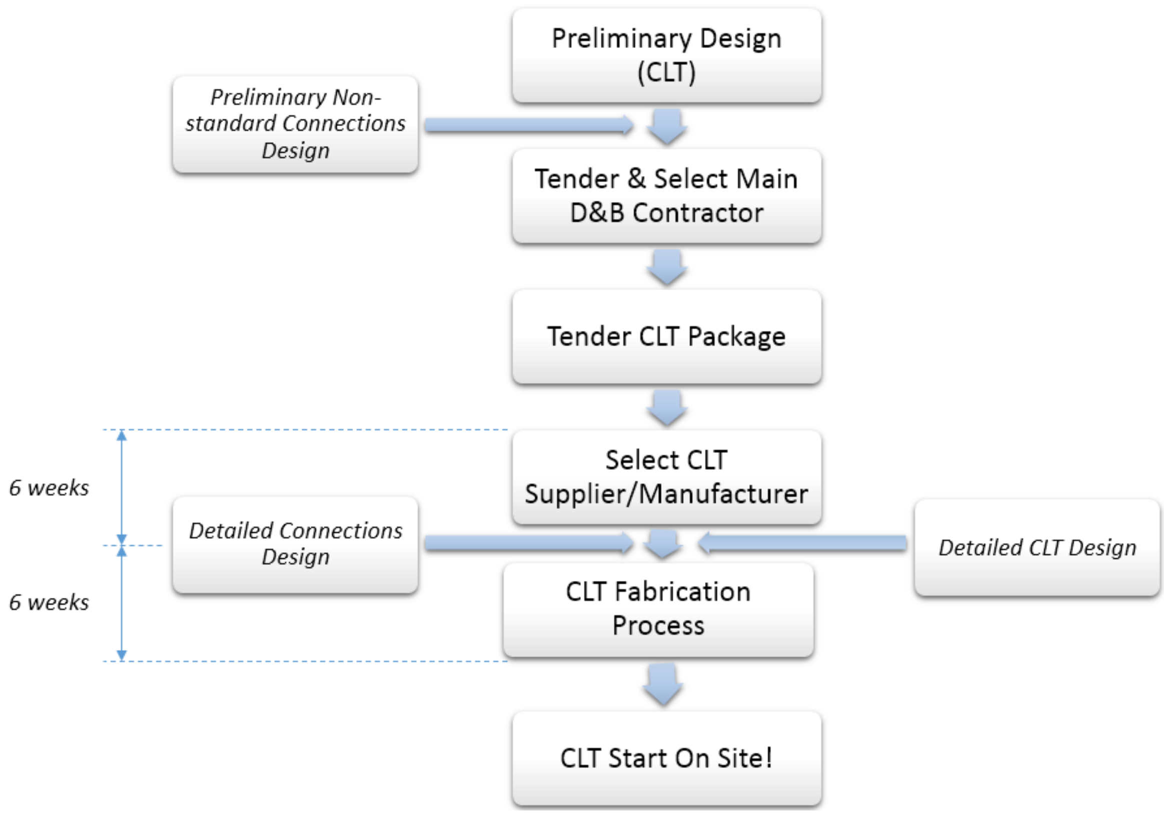


Fig. 18 Flow diagram illustrating the procurement process of a CLT building + connections.

The process of procuring connection design (Fig. 18) is in itself relatively logical. However, the main challenge to the engineer arises due to the typically fast track nature of modern construction programmes in the UK. Reference to the flow diagram shows that detailed connection design cannot commence until selection of the CLT supplier has been confirmed (and thereby the preferred connection manufacturer). Unfortunately (with fast track programmes) this tends to mark the point of commencement of the CLT fabrication process – typicaly 12-14 weeks in the UK. The result is that the engineer is put under immense pressure to deliver detailed connection design and detailed design of the CLT structure – all within a timeframe which can be as little as 6 weeks (i.e. before commencement of CLT factory production/processing.

5. Conclusion, future challenges and acknowledgements

The design process of a CLT building is fundamentally influenced by the procurement process, and in particular by the choice of CLT manufacturer and connector supplier. An early appointment of the CLT supplier can speed up the design phase and avoid inefficiencies of the design process and problems when changing from one supplier to another in the middle or end of the process.

There is a lack of standardised design guidance in Europe. Technical design data currently has to be obtained from a combination of sources such as: manufacturer's European Technical Approval documents, expert reports and some design guides from timber associations such as the Canadian CLT Handbook [4]. That lack of consistency and formal agreement in how to design with CLT makes the designer go back to first principles and refer to some clauses from the Eurocode 5 [5] and thereby not utilising the full benefits of CLT as a system. This last point is particularly important as material efficiency of CLT buildings becomes increasingly important when comparing costs of CLT buildings with steel or concrete frames.

Some of the future challenges the CLT industry will need to address are as follows:

- Streamlining of the procurement and design process to avoid having to redesign buildings caused by a change in the manufacturers. Alternatively a list of standard board types, which all manufacturers can make, i.e. overall thickness of the panel and thickness and orientation of each layer, should solve the need for any re-design.
- Affordable and easy to use structural analysis software for whole CLT structures based on multi-layer orthotropic panel elements. The current analysis software available either do not include those elements or are too expensive for a design office to purchase. There are free design software from institutions and some manufacturers. However they only cover the design of simple elements.
- Production of consistent and formal design guidance for CLT elements and connection details in CLT. One of the key areas where the lack of guidance is relevant is for vibration limit criteria for floor slabs and tie load requirements to avoid the disproportionate collapse (there are guidelines for tie loads for other construction materials but not for CLT). This might take the form of a European version of the Canadian & American 'CLT Handbook' produced by FP Innovations.
- Production of design guidance for CLT in seismic locations. It will not impact directly on the UK but it is relevant to many European countries.
- Address the timber engineering knowledge & skills shortage in the UK by introducing timber engineering as a core subject within university engineering courses.

- High rise CLT buildings: there have been several studies investigating the use of CLT in Super-tall buildings. These identify one of the main issues to solve is the connections where the high stress concentrations lead to significant vertical movements. The design of connections is therefore one of the keys to unlock the opportunities for building taller in CLT.

The authors would like to acknowledge the contribution from Eurotec, Rothoblaas and Simpson Strong-Tie in sharing their connector images with us.

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Minutes of Presentation IX:
Connections Between CLT Elements and Future Challenges for CLT in Practice

Presentation by Tristan Wallwork

Summary:

Tristan Wallwork presents some selected projects built in CLT. Most of the projects are schools or commercial buildings. The structures are often not timber-only structures but hybrid structures. TW presents a typical problem for structural engineers: two CLT products have the same dimension but the structure (layup) and thus the characteristics are different. This is valid for all manufactures and may lead to problems in case of change of the manufacturer by the building owner. He presents details and it seems that the producers are hunting N/mm2 in order to compete with each other.

Wallwork highlights the advantage of drilling techniques available for CLT products, including self-drilling screws and pre-drilled products. Depending on the connection type (wood-wood) and (wood-concrete), different possibilities are offered by the producers of connectors. The joint details may be sensitive with respect to their final application, e.g. sound transmission. Lack in the education of designers becomes most visible in the connection techniques applied. A major problem for the building industry is that different CLT product types may require different connectors. The conclusions mostly concern the limitations due to different CLT manufactures. This situation does not ease the material flow, the exchangeability and especially the competition with other materials.

COST Action FP1402 – FP1404
"Joint Conference of COST Action FP1402 and FP1404 – Cross Laminated Timber – a competitive wood product for visionary and fire safe buildings"

Thursday 10th - Friday 11th March 2016, KTH Stockholm, Sweden

Minutes of the Think Tanks

Think Tanks

The existing challenges to render CLT a commonly accepted structural building product all over the world can best be solved by a unified and concerted action of all stakeholders involved. The key to this is the inclusion of building practice and industry. The previous lack of active participation of these stakeholders is the main reason for the reluctance and problems encountered with the first set of Structural Eurocodes.

The Joint Conference of COST Actions FP1402 and FP1404 attracted participants from all over the world, representing the major stakeholders from industry & product developers, planners & consultants, research & development, policy makers & code writers. Hence it was of interest to not only convey the current state-of-the-art and recent results in R&D, but also to make use of the knowledge, experience and opinions of all competences present at the conference.

This was realized by dedicating one full session to discussions amongst the participants within five Think Tanks. The purpose of the Think Tanks was to support the abovementioned objective by defining joint aims and facilitating corresponding liaisons that tackle the actual problems. The discussions within the five Think Tanks were guided by the following questions:

1. What is essential to be included in a new Eurocode 5:2020?

2. Research vs. practice: what are the gaps between both and how can we close them?

3. Which "gaps" can realistically be solved until the end of 2017 (short-term needs)? (June 2018: end of Project Team SC5.1 "CLT")

4. Which questions have to be solved - if not until 2017 - in the near future (midterm needs)?

5. Individual question defined by the Chairs of each session.

The results of the Think Tanks were presented to the full audience in the closing session at the end of the conference. All discussions within the five Think Tanks were documented and the minutes are presented on the following pages. They represent an indispensable source for COST Actions FP1402 and FP1404. Hence they will be analyzed with respect to definition of objectives and work plans for the remaining time of both COST Actions.

Minutes of Think Tank Nr. 1 (white)

1. Welcome

The leaders of Think Tank 1 (white) – Gerhard Schickhofer & Reinhard Brandner – welcomed all participants and introduced the general idea of the Think Tank, asking all members to actively participate as this team can help bring forward CLT construction in future. The four main questions raised also in all other Think Tanks are presented. It follows a discussion addressing these four questions implicitly.

2. Questions to be Discussed

Question 1:
What is essential to be included in a new Eurocode 5:2020?

Question 2:
Research vs. practice: what are the gaps between both and how can we close them?

Question 3:
Which "gaps" can realistically be solved until the end of 2017?

Question 4:
Which questions have to be solved – if not until 2017 – in the near future?

Discussion related to question 1 to 4:

The importance of connections focusing on SLS / stiffness and when it comes to the design of tall buildings is addressed.

The general question is raised if there is a difference in the response of CLT structures exposed to wind or seismic actions. The common sense was "no".

Regarding the standardisation process of CLT it is mentioned that the Canadians elaborated a CLT design standard of 80 pages. It is responded that in Europe the aim is to include CLT in EC5 by adapting existing regulations and outlining specialities. The idea of a design standard, which provides information for designers by addressing different knowledge levels is raised; this is related also to the presentation of FRANGI regarding the fire design of CLT this evening. On one hand a code should be simple for the industry but on the other hand should provide more detailed information for experts.

The question is raised whether or not a connection system especially for CLT is needed; this in regard to the currently commonly used system of angle brackets and hold-downs, as presented by WALLWORK at this conference, a system developed for light-frame timber structures. It is argued that for CLT a stronger connection system is required as angle brackets are too weak. In addition, mounting of angle brackets takes too much time. Advantages of connections prefabricated already at production site are placed. However, these connections require a more detailed planning which increases the pressure on the designer in a phase were time is already lacking. It is outlined that building with CLT might require more than one connection system, as CLT can be used differently and versatile. It is added that in developing connection systems for CLT it is required to consider also connections between CLT and other (timber) products / elements.

It is mentioned that variabilities in CLT productions influence product properties. In practice determination of layups and properties is important. It is responded that a strength class system for CLT, as presented this morning by SCHICKHOFER, is currently under discussion within the Project Team on CLT for EC5 revision.

It is asked what other topics are currently under discussion, related to CLT and in respect to the revision of EC5. It is responded that the main focus is on the design of CLT elements in- and out-of-plane; it is planned to implement also other topics related to CLT. The needs for regulations in respect to (i) openings in wall & floor elements, and (ii) justification of joints between CLT elements, in particular in case of concentrated loads on floor elements, are outlined. It is stressed that methods allowing design via “hand calculations” should be implemented in EC5. Furthermore, EC5 should be more descriptive; the amount of pages of the code should not be an issue.

It is asked if there is a difference in designing a beam or a diaphragm in in-plane shear. It is responded that apart from different shear stress distributions there should be no difference.

Regarding regulations for connections it is suggested to regulate the high variety of different possibilities and products in technical specifications of EC5 and not in the main document. It is responded that connections for CLT should be regulated by using the same framework as already provided by EC5, meaning by taking into account the basic regulations for single fasteners. In future prefabricated connection systems may probably become more important.

In respect to SLS vibrations it is stated that CLT floor elements behave probably different if compared to other floor systems; there is a need for adequate regulations. It is mentioned that the calculation of vibration characteristics is simple but problems arise when fixing acceptance limits. It is commented that in Austria three requirement classes exist. Vibrations / damping in respect to tall buildings are also addressed as a current topic in Switzerland. A current project at the University of Bath in cooperation with TUM is mentioned.

It is suggested to include comments in EC5 for the design of CLT against failure modes where various approaches can be applied.

Uplift forces and how to deal with them at the connections should be also addressed for CLT structures, in particular for multi-storey buildings.

EC5 provides in some places rules how to calculate stresses accurately; for CLT the same should be provided. For connections, e.g. dowel-type fasteners, influences caused by the orthogonal layering on calculating the embedment strength should be addressed. It is responded that reports on these topics, e.g. concerning fasteners by BLASS & UIBEL (KIT) are available. It is mentioned that knowledge is available in many fields but the transfer of these findings to practitioners and standards is missing!

Two further topics important for the design are mentioned: (i) dimensional changes due to moisture changes (→ easy to calculate, but regulations required), and (ii) regulations for deformations related to compression perp. to grain, also in respect to a potential influence on the structural behaviour. It is agreed that these topics are important.

Tension perp. to grain, e.g. induced by hanging loads on floor elements, should be addressed too. It is agreed that tension perp. to grain in conjunction with CLT is still an open topic. It is mentioned that tension perp. to grain failure was also observed above the supports in bending tests on CLT elements. In general, CLT exposed to tension perp. to grain should not fail in the glue-line; the resistance against failures, which take place in timber is still open. Placement of reinforcements is seen as one solution preventing this type of failure.

Grid shells are mentioned and the possibility to realise such structures in CLT is outlined.

Once again the Canadian code for CLT is brought up. This document is said to provide also rules for CLT as bracing system (shear walls); also regulations for shear wall deformations are given. It is suggested to consider such rules also in EC5; also for architraves.

Differences in the behaviour of CLT with / without plasterboard cladding are mentioned, in particular related to different charring rates after the plasterboard has fallen down. The performance of the gypsum plasterboard in case of fire should be stated clearly.

It is stressed to keep EC5 simple, e.g. by providing tables. Reasoning is that more and more producers enter the CLT market, e.g. in Spain.

Japan, as new player and fastest growing CLT market, is mentioned. Based on a government decision there have been many research investigations. The Japanese published their CLT product standard in 2014. Also the Canadians are much faster in coming up with regulations than Europe. The same is mentioned for China. As main reason the top-down structure in japan, China and Canada is figured out. In Europe a very flat decision process takes out much speed in standardisation process. Europe aims on including production aspects. The competition between the producers, apparent when looking at the different technical approvals /

assessment documents, is a big hindrance. A harmonisation between all producers appears impossible. It is added that this type of competition is not specific for CLT but rather a common business characteristic. It is also outlined that the Canadian code for CLT with about 80 pages may appear thick, but it is complete and not complicated. Spreadsheets for the design of CLT, possible included in EC5, are mentioned. The effective cross section area relevant for designing floor and wall elements can be very different; outlining of this aspect in EC5, in particular for stability design, is suggested; at least a note should be given.

A current research project on tied elements is mentioned which may show some advantages when it comes to acoustics or vibrations.

In addition to the previously discussed moisture influence, creep is mentioned. The need for regulations in that respect is outlined, in particular when it comes to the design of multi-storey buildings. More investigations should be made on existing buildings, e.g. on the façade, to monitor how they perform. It is asked if EC5 contains k_{def} values specifically for CLT. Furthermore, the necessity for notes outlining the relevance of shear deformations in the design of CLT elements out-of-plane is stated. However, consideration of shear is not only relevant for SLS but also for ULS design.

It is asked whether or not the test configurations in EN 408 are suitable also for CLT. It is responded that a concerted standard portfolio including testing, design and properties is required addressing specialities of CLT. Apart from EN standards also ISO and other standards should be taken into account. In respect to EN 408: all needed test configurations together with regulations of system, size effects, spans and distances should be implemented.

Question 5:
Are modular systems / boxes a potential future application of CLT?

Discussion:

It is mentioned that there are already companies producing boxes as kind of a modular system by using CLT. One company prefabricates about 1,000 units per year. Modularisation takes more and more place. However, the thinking is different: rather in volumes than in square metres, also taking into account the impact on transportation. Modularisation is seen as a chance for CLT; for light-frame structures in Sweden modularisation is not considered. It is reported that an Austrian CLT producer delivers CLT modules with completely finished interior (installation & final cladding). However, how to connect these modules with each other is still a relevant task.

3. Closing

The leaders of Think Tank 1 (white) thank all participants for all their contributions, for the fruitful discussions. The good atmosphere is outlined and the Think Tank closed.

Minutes of Think Tank Nr. 2 (red)

1. Welcome

The leaders of Think Tank 2 – Jochen Köhler and Gerhard Fink – welcomed all participants and introduced the general idea of the Think Tank, asking all members to actively participate in the discussion.

2. Questions to be Discussed

Question 1:

What is essential to be included in a new Eurocode 5:2020?

Calculation methods for CLT

It was the received opinion of the participants that all necessary calculation methods for CLT have to be included in EC 5. That includes design equations for the individual failure modes.

Strength classes

The strength classes of CLT including the associated characteristic values are needed for a proper design of CLT elements. Therefore a standardization of the product is necessary. However, the participants had different perspectives regarding standardization (e.g. regulation of the layer setup) and the number of strength classes (one or more strength classes). In summary, two groups were identified: Those who want a European harmonization of the product und those who want a producer specific product.

Partial safety factor

The partial safety factors have to be included in EC 5. Some participants prefer a regulation of the partial safety factors by using the National appendix.

Serviceability limit state design

Serviceability limit state design of CLT, in particular vibration, has to be included in EC 5. Vibrations should be probably standardized on a higher level, not in the material part of the EC. Only material dependent parameters should be included in the material parts (e.g. damping factors).

Question 2:

Research vs. practice: what are the gaps between both and how can we close them?

Finite element modelling (FEM)

The use of FEM for CLT seems to be challenging for the designers. That includes several aspects such as, the choice of the material properties, system representations, design of connections, elastic/plastic design, and seismic design.

Combined stresses

It was complained from the practice that nowadays research is mainly focusing on single stresses and its influences and not on combined stresses. Research should focus more on "realistic" situations including combined stresses.

k_{mod} & moisture induced stresses

Strength reducing effects, represented by k_{mod} and moisture induced stresses, were discussed. Summarized it can be stated that most of the participants are not satisfied jet. Some prefer more simplified approaches (e.g. harmonization of k_{mod}), others prefer more scientific approaches.

Lack of education

Even it is not directly related to this question it was discussed intensively. Overall it can be stated that the practice is not satisfied with the current education of timber (and CLT) structures of the engineers.

Question 3:

Which "gaps" can realistically be solved until the end of 2017?

Necessary design equations

The necessary design equations should be implemented until 2017. That includes the implementations of failure modes like shear net section, gross section, or bond line failures as well as the solutions for the design of specific configurations (e.g. holes, cutting).

Crudeness for modelling

The level of crudeness for modelling has to be specified. This is the basis for the discussion about parameter such as k_{mod} or γ_m.

k_{mod}

The modification factor k_{mod} has to be chosen for EC 5. However, for some participants this should be finalized until 2017 for some others more time is needed.

Question 4:

Which questions have to be solved - if not until 2017 - in the near future?

Design equations

Some more specific design solutions might not be defined until 2017 (e.g. a general solution for punching shear).

k_{mod}

See Question 3

Minutes of Think Tank Nr. 3 (green)

1. Welcome

The leaders of Think Tank 3 (Green) – Esko Mikkola and Daniel Brandon – welcomed all participants and introduced the general idea of the Think Tank, asking all members to actively participate as this team can help bring forward CLT construction in future. Approximately one hour was available in which 5 topics had to be discussed. Therefore it was the aim to discuss every topic for approximately 12 minutes.

2. Questions to be Discussed

Question 1:
What is essential to be included in a new Eurocode 5:2020?

Discussion:

Andrea Canducci suggested that it is time to talk about pre-stressing of timber. In Italy it is discouraged, because of the creep. Tests could be needed. Finite element analyses are run for analysing this.

Keerthi Ranasinghe agreed that it would be interesting, but noted that a standard should not include everything. The standard committee should look at what we have currently as the state of the art and standardize that.

Keerthi Ranasinghe suggested connections should be looked at, as there are a lot of knowledge gaps.

Nicolas Jacquier stated that there is lack of recommendations of vibration design of CLT floors. We need something to guide us when we design.

Keerthi Ranasinghe agreed with Nicolas Jacquier on this. The EC5 is not addressing the right problems regarding vibrations.

Patrick Racher stated that connection design rules focus mainly on the load bearing capacity, not on stiffness. With the evolution of the methods and connections we should be able to predict the stiffness accurately.

Esko Mikkola mentioned that, for fire conditions, connections can only be calculated for 60 minutes of a standard fire by the Eurocode 5.

Daniel Brandon stated that we don't have test results of connection fire tests that led to a fire resistance of 90 minutes or higher. As it is not possible to prove fire resistances exceeding 60 minutes it is essential that there will be tests to prove higher fire resistances.

Patrick Racher stated that it is possible to calculate for more than 90 minutes using Dhionis's model, which is currently being proposed for the Eurocode 5 among other models.

Keerthi Ranasinghe wondered if there was a technical reason for the absence of longer tests. In other words, can we test for so long?

Dhionis Dhima answered that there are no problems and that it is not more complicated than shorter tests.

Keerthi Ranasinghe mentioned that there is no clear information about the geometrical tolerances for timber connections.

Question 2:
Research vs. practice: what are the gaps between both and how can we close them?

Discussion:

Michael Klippel stated that engineers in practice should tell the researchers what they really need. Communication is of importance.

According to Esko Mikkola it is important that the research community should provide the models necessary in practice.

Keerthi Ranasinghe stated that communication between research and practice is the most important gap. Research is sufficient and the practice is sufficient.

Magnus Wålinder mentioned that there is no or very little interest in education for timber (at KTH). We need resources and to know how to improve communication.

Keerthi Ranasinghe confirmed that this is also the case in the UK as the education in the UK regarding timber engineering shrunk completely.

Magnus Wålinder stated that there seems to be gap of knowledge regarding adhesives.

Esko Mikkola thinks there is good knowledge. It is the communication to exchange knowledge that seems to be problematic.

Michael Klippel agreed with Esko and encourages the researcher to take action and talk to chemists to improve adhesives.

Alistair Bartlett stated that understanding the behaviour of delamination is important. Now we know what happens in some cases, but not why it happens.

Daniel Brandon stated that delamination can extend the fire duration. Fire tests of compartments with a limited amount (1 or 2 walls) of exposed CLT surfaces have shown a decay phase. If delamination occurs there will very likely be a second flashover. There have been adhesives tested, such as phenolic and MUF adhesives, which did not lead to delamination. Maybe we should consider them.

Keerthi Ranasinghe stated that we should ask the glue manufacturers to improve adhesives. At this moment CLT will suffer the consequences of using bad adhesives, while the responsibility should also lie with the adhesive industry.

Danny Hopkin said that we cannot predict temperatures for timber structures for anything other than the ISO fire curve. In the UK it is common practice to use performance based fire safety engineering methods. However, it is impossible to use that for timber building designs.

Pedro Palma commented on Danny Hopkins' statement and replied that we may increase the accuracy but the uncertainty is so high that a standard fire may be a sufficiently challenging at the moment.

Dhionis Dhima replied to Pedro Palma by stating that we have this problem for all materials. Increasing, for example, the heat flux for steel causes inaccuracy as well.

Patrick Racher asked the group what the effect of rain during erection is. He expects that the moisture will often be trapped in the structure for a very long time. There was no immediate response from the group at this point.

Thomas Bader said that it is good that the new EC5 will allow for a simple approach and more complicated ones. It gives transparency and good guidance. Providing more material characteristics in the EC5 would help.

Alistair Bartlett added that there is a lot of data available in the literature. These can be completely different. It would give extra guidance to give more material properties in EC5.

Daniel Brandon stated that this problem also exists for the writers of the standards. How can the writers of EC5 justify the choice of material properties if the literature shows completely different results.

Michael Klippel responded and commented that data is very dependent on the way they were received. It is more important to look at how important the differences are.

Question 3:
Which "gaps" can realistically be solved until the end of 2017?

Discussion:

Esko Mikkola started the discussion by stating that 2017 is very soon.

Keerthi Ranasinghe agreed and argued that 2017 is too short to improve the standard significantly from now. Engineers should not expect too much from the standard. The people should know that there was and there always will be a limit of research at the time the standard is written.

There seemed to be general agreement that the time is too short to substantially fill major gaps.

Question 4:
Which questions have to be solved - if not until 2017 - in the near future?

Discussion:

Patrick Racher stated that we have no information about screws and nails in fires. These connections are very commonly applied. Patrick Racher suggested that we can require companies to do some testing.

Mariana Pruna proposed to introduce classes for durability and toxicity. It is important to test for toxic gases. Formaldehyde in construction is a problem.

Question 4a:
Which steps are necessary to solve these gaps?

Discussion:

Esko Mikkola started by summing up the main actions to be taken:

- Testing
- Analysis
- Communication
- Harmonization of products
- Standardization

There seemed to be a general agreement regarding these answers.

Question 5:
COMMISSION DELEGATED REGULATIONS
– Can CWFT be further utilized instead of testing?

Discussion:

Esko Mikkola asked if we can use K-classes and EI-classes for CLT in the form of tabulated data? Can we provide fire resistance classes for CLT without requiring calculations or tests? Is the reaction to fire classification included in the on-going CWFT application?

Minutes of Think Tank Nr. 4 (yellow)

1. Welcome

2. Questions to be Discussed

Question 1:
What is essential to be included in a new Eurocode 5:2020?

Discussion:

The following topics were raised in connection with essential requirements for inclusion in the next revision of Eurocode 5, however over the course of the discussion it became clear that most of these in fact needed only better guidance documents to be available as opposed to actual inclusion. Those topics, which should be included in the next revision are marked in bold below, with obvious reservations based on the response to questions 3 and 4 below.

- **Standard grading in CLT**
- Fire protection of connections
  - More information is needed, perhaps in the form of a guide
  - More information about connections is needed in the form of a guidance document for CLT in particular. More knowledge about the failure mechanisms of connections is needed.
- **Guidance about the use of CLT as deep beam elements is needed.**
- **Design of CLT elements with large openings – specifically doors and window openings since these may create stress concentrations.**
- Guidance about penetration seals
- **Stress interaction – e.g. combined axial and flexural load**
- Glued connections
  - glued in rods (adhesive anchors)(glulam vs CLT)
  - glued steel plates
  - Screwed connections
- **Timber / concrete composites**
  - Creep
  - Manufacture
- Sheer failure of CLT elements in fire should be checked. Although this is not normally a problem in fire for timber elements this may not be the case for CLT and a simple check should be carried out to ensure this. If it is an issue then it should be given due consideration in the Eurocode.

Question 2:
Research vs. practice: what are the gaps between both and how can we close them?

Most of the above listed items were identified as being gaps in research. Means to close the gaps between research and practice are:

- We should influence national regulations by speeding up the process of revision of the Eurocodes to ensure that latest research is included in them.
- There should be fewer nationally determined parameters.
- Complexity of the Eurocodes should be reduced
    - This will enable greater freedom and therefore better building
    - Very simple rules should be added for lazy designers.
- We should have to pay for simplicity in the design process through a more conservative and less efficient design.
- We had a short discussion about whether or not there was an advantage to carrying out advanced analyses.
    - For example, BS 9999 has simple tabulated rules, which allow a simple trade-off between fire safety features. Something like this in the timber industry would be very beneficial.
- There should be a technical annex on penetrations / linings, i.e. practical guidance on encapsulation.
- CE marking should be possible for CLT and for the adhesives.
- Funding for R&D projects should be prioritised between needed basic and applied research.
- Better education of the work force would help to close gaps between research and industry.
- More ‘crazy architects’ are needed – this will drive innovation.

Question 3:
Which “gaps” can realistically be solved until the end of 2017?

Gaps, which can realistically be solved before the end of 2017 are:

- **Standard grading in CLT – although with due consideration for different layups of CLT.**
- Fire protection of connections
    - More information is needed, perhaps in the form of a guide
    - More information about connections is needed in the form of a guidance document for CLT in particular. More knowledge about the failure mechanisms of connections is needed.
- **Guidance about the use of CLT as deep beam elements is needed.**

- Guidance about penetration seals
- Sheer failure of CLT elements in fire should be checked. Although this is not normally a problem in fire for timber elements this may not be the case for CLT and a simple check should be carried out to ensure this. If it is an issue then it should be given due consideration in the Eurocode. If this check is done and the result shows that shear failure of CLT in fire is not an issue then this would not need to be addressed in the Eurocode.

Question 4:
Which questions have to be solved - if not until 2017 - in the near future?

Long-term knowledge gaps, which should be solved in the future are:

- **Design of CLT elements with large openings – specifically doors and window openings since these may create stress concentrations.**
- **Stress interaction – e.g. combined axial and flexural load**
- Glued connections
  - glued in rods (adhesive anchors)(glulam vs CLT)
  - Glued steel plates
  - Screwed connections
- **Timber / concrete composites**
  - **Creep**
  - **Manufacture**

Question 5:
How can education in CLT be improved?

Courses on CLT and timber engineering are not normally offered at universities. The same is true for many specialist topics. One way to increase education in timber engineering would be to increase research funding, which would then attract more people to do research in the area and then to transfer this knowledge into their teaching.

Minutes of Think Tank Nr. 5 (blue)

1. Welcome

The leaders of Think Tank 5 – Stefan Winter (SW) and Philipp Dietsch (PD) – welcomed all participants and introduced the general idea of the Think Tank, asking all members to actively participate as this team can help bring forward CLT construction in future.

2. Questions to be Discussed

Question 1:
What is essential to be included in a new Eurocode 5:2020?

Discussion:

Delegate asked about tabulated data in the fire part.

Francois Colling (FC) replied that tabulated data is of no use if we look the heterogeneous products from different producers, e.g. with multiple parallel layers in one direction.

Stefan Winter (SW) advocated to stick to the state of the art.

Member proposed to compare different codes (think of small Swiss code vs. Comprehensive German code). Think of users that don't use timber on a daily basis.

Delegate stated that we need a methodology for in-plane shear that everybody can agree on.

Member proposed to have the tables as part of the standard.

Jorgen Munch-Andersen (JMA) stated that Eurocodes should give us a common basis, e.g. the equations needed to derive tables, tables should be part of other documentation as they will blow up. Eurocode is for Engineers.

Thomas Orskaug (TO) stated that there is a lack on provisions and principles for vibrations.

FC: Make the code short but clear. Think of commented versions of codes. Then we could bring the principles in the code and the application tables in the comment.

SW: For fire design tables could be helpful as they are then authorized by building authorities.

Delegate: Eurocode 5 is used by people that are not necessarily trained in timber.

TO: Look at the Canadian regulations, they are very useful.

Question 2:
Research vs. practice: what are the gaps between both and how can we close them?

Discussion:

TO: We are missing more information on connectors.

Andrew Lawrence: there is a lack of provisions for openings, this would help to reduce engineers design cost.

Delegate: Durability. In the UK there is a big demand for pressure treatment of CLT, e.g. insurance companies ask for it.

SW: Chemical treatment does not help with the exception of termites.

AL: In the UK, CLT is sometimes completely encapsulated.

Tobias Wiegand: Should this be part of the Eurocode?

SW: One place could be a CEN Technical Specification. We need knowledge transfer and maybe some tests to prove that structures are safe towards moisture. Mentioning of tall timber facades project.

JMA: This cannot be a subject of Eurocode 5.

TO: Could this become part of a execution standard?

SW: Developments starting in the area of implementing layers in CLT as fire stops, heating, reinforcement etc. Plus CLT with hardwoods and the mix of lamellas with wood-based panels.

TW: Wood-based panel layers could be used for joints. Hardwoods could be used to increase certain properties.

Question 3:
Which "gaps" can realistically be solved until the end of 2017?

Discussion:
TW: Curvatures of elements. Design of CLT as a beam: we have research results with homogeneous layup, will it work for other strength classes and layups?

Ad Lejten: Glued-in rods in CLT.

SW: How to calculate connections in the side-faces of CLT.

Andrea Frangi (AF): The decision on the safety factor is important. It should be based on a scientific basis.

FC: Most design situations are Serviceability related.

TO: The vibration chapter can be solved.

JMA: Connections: consider density, the fastener will sit in one piece with certain density not in an element with a mean density. When looking at fasteners in gaps, also consider the amount of fasteners used.

FC: CLT of different layups / strength classes could be solved. In addition: built-up beams (T-beams). For both we have the theoretical background.

TW: Safety factors: to which property do we relate it? (Bending?, Rolling shear?, stability?, ...).

AF: We just have to declare the correct characteristic value based one common safety factor.

SW: This is not possible. We have to use correction factors.

Fernando Perez: To include connections/connectors in CLT is key. SW/AF: punching shear / simple supports and their reinforcement.

Question 4:
Which questions have to be solved - if not until 2017 - in the near future?

Discussion:

FC: Bring together all manufacturers and tell them that standardization of a product is good for timber construction.

AL: CLT from hardwood.

TW: Boxed elements.

Delegate: Standardization of refabricated elements.

Eero Tukhanen: Can we standardize boxed elements in Eurocode 5? Probably not.

Delegate: How to connect functions from one element to the whole building?

TW: Apart from seismic and fire we need acoustics.

FP: Acoustics in joints is essential.

SW: Better basis for modelling (FE-modelling). Currently the models are not checkable and full of mistakes.

Delegate: 3-D models are a challenge as changing one property (e.g. connection) will change the results in all elements and connections.

FC: Composite beams of e.g. timber (CLT) and concrete (pre-fabricated?). Consider that timber structures are dry structures.

TO: For modelling we need the basic material properties as pure as possible without modification factors.

AF: In the future we should only design ductile structures.

Delegate: Relation of FE-Models with test results.

Question 5:
Will LCA be part of future building regulations?
(Material efficiency vs. Carbon storage)

Discussion:

AL: Definitely.

JMA/AL: We just need to fight against the concrete industry.

Delegate: This is also a recycling question. A CLT building can be reused.

SW: For this we need research for design for recycling.

JMA: It should become part of the essential requirements.

TO: In Norway it is already part of the system and it is a matter of time that it becomes part of building regulations.

TW: There is an organization of footprints (land-use). In future you will be punished if you can't demonstrate the possibility for recycling.

3. Closing

The leaders of Think Tank 5 (blue) thank all participants for all their contributions, for the fruitful discussions. The good atmosphere is outlined and the Think Tank closed.

COST Action FP1402 – FP1404
"Joint Conference of COST Action FP1402 and FP1404 – Cross Laminated Timber – a competitive wood product for visionary and fire safe buildings"

Thursday 10th - Friday 11th March 2016, KTH Stockholm, Sweden

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