Graz University of Technology

Fachbereich Ingenieurbaukunst

Comparative view on the computational and design procedures of CLT elements loaded in bending in accordance to Canadian and European standards and design guidelines

M-4-07/2012

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Vergleichende Betrachtung der Berechnungs- und Bemessungsmethoden für biege-beanspruchte Brettsperrholz-Elemente nach europäischen und kanadischen Normenwerken und Bemessungsrichtlinien

Masterarbeit

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Eingereicht am

Institut für Holzbau und Holztechnologie

Fakultät für Bauingenieurwissenschaften

Technische Universität Graz

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Einreichdatum: Oktober 2012

Comparative view on the computational and design procedures of CLT elements loaded in bending in accordance to Canadian and European standards and design guidelines

Master Thesis

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Submitted on

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Submit date: October 2012

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Guggenberger Maria 24th October 2012

DANKSAGUNG

Zuerst möchte ich Univ.-Prof. Dipl.-Ing. Dr.techn. Gerhard Schickhofer für den Kontakt zu FPInnovations und Eng. Sylvain Gagnon danken, sowie für seine Unterstützung und seinen Rat.

Bedanken möchte ich mich bei Dipl.-Ing. Dr.techn. Alexandra Thiel für die informative und konstruktive Betreuung, sowie für die freundliche und geduldige Unterstützung.

Ich danke Eng. Sylvain Gagnon für die Möglichkeit einen Teil meiner Masterarbeit bei FPInnovations zu schreiben und für seine professionelle Unterstützung. Ebenso danke ich den Angestellten von FPInnovations, die mir stets gerne Informationen zuteil kommen ließen.

Spezieller Dank geht an meine Familie, die mich ermutigte ein Studium zu absolvieren und mir immer zu Seite stand.

Ich möchte meinen FreundInnen und KollegInnen und allen weiteren Menschen, die mich bei meiner Diplomarbeit unterstützen, danken.

ACKNOWLEGEMENTS

Firstly I would like to thank Univ.-Prof. Dipl.-Ing. Dr.techn. Gerhard Schickhofer who created the contact to FPInnovations and Eng. Sylvain Gagnon, and who always offered me advice and support.

I owe thanks to Dipl.-Ing. Dr.techn. Alexandra Thiel for her informative and constructive support, as well as her friendly and patient assistance.

I thank Eng. Sylvain Gagnon for giving me the opportunity to write a part of this master thesis at FPInnovations, and for his expert support. Furthermore I thank the employees of FPInnovations who willingly provided information at any time.

Special thanks to my family who encouraged me to complete my studies and was always there for me.

I would also like to thank my friends and colleges and all people who supported me in writing this thesis.

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KURZFASSUNG

Die Berechnung und Bemessung von Brettsperrholz ist derzeit weder in Europa noch Kanada normativ geregelt. Rechtsgültigkeit haben nur Produktzulassungen. Neben diesen existieren noch einige Bemessungsvorschläge bzw. -richtlinien, wie z.B. in Österreich das BSPhandbuch vom Institut für Holzbau und Holztechnologie der TU Graz und in Kanada das CLT-Handbook von FPInnovations.

In der vorliegenden Arbeit werden die Berechnungs- und Bemessungsmethoden des CLT-Handbooks von FPInnovations und des BSPhandbuchs der TU Graz für biegebeanspruchte Brettsperrholz-Elemente untersucht und gegenübergestellt.

Zunächst werden die relevanten Normen von Kanada (CSA O86 und ANSI PRG 320) sowie von Europa (Eurocode 1 und Eurocode 5), welche als Basis für die jeweilige Bemessungsrichtlinie dienen, erläutert. Danach wird näher auf die Bemessungsvorschläge aus dem CLT-Handbook sowie dem BSPhandbuch eingegangen. Anhand von Beispielen werden die Unterschiede aufgezeigt. Dies insbesondere für die ULS Nachweise für Biegung und Schub, sowie für die SLS Nachweise für Durchbiegung und Schwingung.

Mit diesen Beispielen wird gezeigt, dass die europäischen und kanadischen Richtlinien in weiten Bereichen eine gute Übereinstimmung aufweisen, es vereinzelt aber zu Differenzen in den Ergebnissen kommen kann, da im kanadischen Bemessungsvorschlag stärkere Vereinfachungen getroffen werden und dieser daher großteils auf der konservativen Seite liegt.

Des Weiteren liefert diese Arbeit die Grundlage für die Implementierung der kanadischen Norm bzw. der Bemessungsrichtlinie nach dem CLT-Handbook in den CLTdesigner, das Bemessungstool für die Berechnung und Bemessung von Brettsperrholz der holz.bau forschungs gmbh.

ABSTRACT

Currently in Europe and in Canada there are no standards for the calculation and design of cross-laminated timber elements (CLT). The legal force is subjected to technical approvals for CLT-products. Beside technical approvals design proposals are available. For example in Austria there is the BSPhandbuch from the Institute of Timber Engineering and Wood Technology situated at Graz University of Technology and in Canada the CLT-Handbook published by FPInnovations.

In this thesis the calculation and design procedures for CLT-elements loaded out of plane according to the above mentioned proposals are examined and compared.

First the relevant standards of Canada (CSA O86 and ANSI PRG 320) and Europe (Eurocode 1 and Eurocode 5) are presented. The standards are the basis for the proposed design guidelines of the BSPhandbuch and the CLT-Handbook. To demonstrate the differences between the guidelines, examples of CLT-elements loaded out of plane were calculated. These examples deal with the ULS verification for bending and shear and the SLS verification for deflection and vibration.

It can be demonstrated that the Canadian and European guidelines show strong correlations. However, occasionally it comes to differences, because of the stronger simplifications made in the Canadian suggestions. With these simplifications the Canadian proposal is more conservative.

Further this thesis provides the basis for the implementation of the Canadian standard and the guidelines according to the CLT-Handbook in the CLTdesigner, a tool to compute and design CLT-products which is provided by the competence center holz.bau forschungs gmbh.

CHAPTER 1

1.INTRODUCTION

Building with wood gets more and more attractive besides building with concrete and steel. The increasing CO_2 pollution of the cement industry is only one indicator for changing its views on the used building materials. Wood is a renewable primary product and also the progressing and deposal is less detrimental to the environment than other materials. Besides the wood lightweight construction the wood massif construction, which cross-laminated timber (CLT) belongs to, has been developed dynamically within the last few years. A general calculation and design procedure for CLT-elements isn't available as a standard. To publish a uniform standard to design CLT-constructions for each government research was done and several guidelines (non mandatory) are proposed.

This master thesis deals with the guidelines of Graz University of Technology (Austria) and FPInnovations (Canada). The calculation and design procedures for CLT base on the existing standards for wood constructions and go specifically into the characteristic behaviour of CLT. Therefore the Canadian standard CSA O86-09 and ANSI PRG 320 will be cited. For the European approach the Eurocode 1 and Eurocode 5 will be explained. In context of this thesis cross-laminated elements loaded perpendicular to the face layer will be scrutinized. More exactly the verification for ULS (bending and shear) and SLS (deflection and vibration) are presented. Further the differences of the procedures and design methods were sift out during a parameter study.

In addition the Canadian design guideline for CLT-products shall be prepared for an implementation in the software tool CLTdesigner of the competence center holz.bau forschungs gmbh.

2. LITERATURE, LEVEL OF EDUCATION

The design of CLT-elements is not established in the national standards neither in Europe nor in Canada. Therefore architects, engineers and carpenters are related to technical approvals. These technical approvals are certifications of CLT-elements made by authorized companies to allow manufacturing procedures.

For innovative systems the designer requires science behind to support the use. For this reason several guidelines are available. These guidelines are not mandatory. They correlate strongly on the national standards and are intended to serve as basis for the future standards.

The CSA Standard O86-09 is the current standard for design wood constructions in Canada. The CSA (Canadian Standard Association) Standard Engineering design in wood was printed in 2010. The version used in this thesis contains the incorporate revisions of Update No.1 (September 2010) of the original 2009 Standard. Due to the research of CLT is still a new field in timber construction, the calculation and design guidelines for CLT are not in the CSA O86 implemented.

Source: www.amazon.ca

Figure 1: CSA Standard O86-09, CLT-Handbook and the ANSI PRG320

The CLT-Handbook is the only source of technical information in Canada to design CLT-buildings at the moment. But this is not an official standard. The Canadian

CLT-Handbook was developed to enlarge the Canadian Standard. At this time a series of tests takes place to optimize the calculation and design processes. The CLT-Handbook was published by FPInnovations, edited by Sylvain Gagnon and Ciprian Pirvu in 2011.

The American National Standard is the current official manufacturing standard in North America for CLT products. The Standard for Performance-Rated Cross-Laminated Timber of the ANSI/APA PRG 320-2011 (Approved American National Standard) is mandatory in Canada and USA.

In Europe the ON EN 1995-1-1 2010 01 Eurocode 5 Design of timber structures - Part 1-1 General Common rules and rules for buildings is valid. It is approved by the European Committee for Standardization (CEN). The Eurocode 5 maintains the design methods for wooden constructions and their fastenings. The Eurocode includes a National Annex which is defining national details chosen by the accompanying government. As mentioned before the calculation and design guidelines for CLT are not included in the Eurocode.

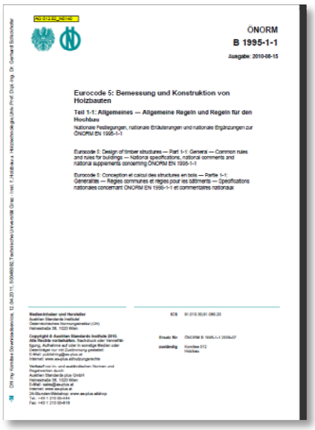


Figure 2: Eurocode 5 and BSPhandbuch

The Graz University of Technology did further experiments to lay the groundwork for a new standard to design CLT-products. The research and the results of the experiments are published in the BSPhandbuch. This thesis refers on the second edition from 2010.

Current guidelines in Europe are defined by technical approvals for products. The technical approvals are written out by qualified organisations like the DIBt („Deutsches Institut für Bautechnik“).

Examples for technical approvals (germ.: “Zulassung”) are: [6] [10]

- Zulassung ETA -09/0036: MM BSP. Wien. 2009. valid until 2014.
- Zulassung ETA -08/0242: HMS-Element. Berlin. 2009. valid until 2014.
- BauaufsichtlicheZulassung Z-9.1-638. valid until 2016.

CHAPTER 3

3.STANDARDS AND DETERMINATION GUIDELINES

3.1 The Canadian Standard

The design procedures for cross-laminated timber aren't implemented in the Canadian Timber Design Standard. To design cross-laminated timber (CLT) FPInnovations proposes suggestions in the CLT-Handbook. Before this master thesis goes into the CLT-Handbook, the general design guideline for wood constructions will be specified.

3.1.1 CSA Standards O86-09, Engineering design in wood [1]

In Canada and also in Europe the procedures designing for structural elements loaded in bending are based on elastic behaviour. The Ultimate Limit State (ULS) and the Serviceability Limit State (SLS) are used to design Cross Laminated Timber (CLT).

Load combinations for ULS

The loads which effects on a system for Ultimate Limit State verification shall be calculated according table 1 (4.2.4 CSA O86-09):

Case	Principal loads*	Companion loads
1	$1.4D$	—
2	$(1.25D$† or $0.9D) + 1.5L$‡	$0.5S$§ or $0.4W$
3	$(1.25D$† or $0.9D) + 1.5S$	$0.5L$§** or $0.4W$
4	$(1.25D$† or $0.9D) + 1.4W$	$0.5L$** or $0.5S$
5	$1.0D + 1.0E$	$0.5L$§** + $0.25S$§

*Refer to the National Building Code of Canada for loads due to lateral earth pressure (H), prestress (P), and imposed deformation (T).
†Refer to the National Building Code of Canada for a dead load (D) for soil.
‡The principal load factor of 1.5 for a live load (L) may be reduced to 1.25 for liquids in tanks.
§Refer to the National Building Code of Canada for loads on exterior areas.
**The companion load factor of 0.5 for a live load (L) shall be increased to 1.0 for storage occupancies, equipment areas, and service rooms.

Table 1: Load combinations for Ultimate Limit States [1]

The load combination for ULS shall be taken, which results amount the most unfavourable effect.

Specified loads (4.2.3)

The specified loads are defined as followed:

D... Dead load to the weight of members
E... Load due to earthquake
L... Live load to the intended use and occupancy
S... Load due to snow
W... Load due to wind
H... Permanent load due the lateral earth pressure, including groundwater
T... Load due to contraction or expansion cause by temperature changes

Importance factor for snow, wind and earthquake

The importance factors for ULS are given in table 4.2.3.2 in the CSA O86-09 Standards. The consequences of failure are taken into account related to the limit state, the use and occupancy of the building.

	Importance factors for snow loads, I_S		Importance factors for wind loads, I_W		Importance factors for earthquake loads, I_E	
Importance category	Ultimate limit state	Serviceability limit state	Ultimate limit state	Serviceability limit state	Ultimate limit state	Serviceability limit state
Low	0.8	0.9	0.8	0.75	0.8	—
Normal	1.0	0.9	1.0	0.75	1.0	—
High	1.15	0.9	1.15	0.75	1.3	
Post-disaster	1.25	0.9	1.25	0.75	1.5	—

Table 2: Important factor for determining S, W, or E loads [1]

Load combination for Serviceability Limit States

In accordance with table 4.2.4.2 (CSA O86-09) the most unfavourable effect shall be taken:

Case	Principal loads	Companion loads
1	$1.0D$*	—
2	$1.0D$* + $1.0L$	$0.5S$† or $0.4W$
3	$1.0D$* + $1.0S$	$0.5L$† or $0.4W$
4	$1.0D$* + $1.0W$	$0.5L$ or $0.5S$

**Dead loads include permanent loads due to lateral earth pressure (H) and prestress (P).*
†*Refer to the* National Building Code of Canada *for loads on exterior areas.*

Table 3: Load combination for Serviceability Limit States [1]

Bending moment resistance (5.5.4 CSA O86-09) and shear resistance (5.5.5 CSA O86-09) for Sawn Lumber

In this section the bending moment resistance and shear resistance will be explicated. The equations [3.1] to [3.4] are only valid for glued-laminated timber. An analogy for CLT is in the next section "The Canadian CLT-Handbook" explained.

The CSA O86 expresses the relation as followed:

$$M_r = \phi F_b S K_{Zb} K_L \qquad [3.1]$$

$$F_b = f_b(K_D K_H K_{Sb} K_T) \qquad [3.2]$$

where:

ϕ... resistance factor (0.9 for CLT)
f_b... specified strength in bending, MPa
S... section modulus [mm³]
K_{Zb}... size factor in bending
K_T... treatment factor
K_D... long duration factor
K_H... system factor
K_{Sb}... service condition factor for bending
K_L... lateral stability factor

The shear resistance for Sawn Lumber may be calculated as followed:

$$V_r = \phi F_v \frac{2A_n}{3} K_{Zv} \qquad [3.3]$$

$$F_v = f_v(K_D K_H K_{Sv} K_T) \quad [3.4]$$

where:

ϕ... resistance factor (0.9 for CLT)
f_v... specified strength in shear, MPa
A_n... net area of cross section, mm²
K_{Sv}... service condition factor for shear
K_{Zv}... size factor in shear

The resistance factor ϕ is used for all applicable limit states for wood members and fastenings (see 4.1.4 of CSA O86-09).

The modification factors K for Sawn Lumber are given in the following tables. Also considerations for the K factors for CLT are given in section 3.1.2 Canadian CLT-Handbook.

Load duration factor K_D (4.3.2 CSA O86-09)

Are there different loads with deviant load duration, a K_D factor with the shortest duration is recommended. It always shall be the load combination which results amount the most unfavourable effect.

Load duration	K_D	Explanatory notes
Short term	1.15	Short term loading means the condition of loading where the duration of the specified loads is not expected to last more than 7 days continuously or cumulatively throughout the life of the structure. Examples include wind loads, earthquake loads, falsework, and formwork, as well as impact loads.
Standard term	1.00	Standard term means the condition of loading where the duration of specified loads exceeds that of short-term loading, but is less than long-term loading. Examples include snow loads, live loads due to occupancy, wheel loads on bridges, and dead loads in combination with all of the above.
Long term	0.65	Long-term duration means the condition of loading under which a member is subjected to more or less continuous specified load. Examples include dead loads or dead loads plus live loads of such character that they are imposed on the member for as long a period of time as the dead loads themselves. Such loads include those usually occurring in tanks or bins containing fluids or granular material, loads on retaining walls subjected to lateral pressure such as earth, and floor loads where the specified load can be expected to be continuously applied, such as those in buildings for storage of bulk materials. Loads due to fixed machinery should be considered to be long term.

Note: *Load duration requires professional judgment by the designer. Explanatory notes in this Table provide guidance to designers about the types of loads and load combinations for which each modification factor should be applied.*

Table 4: Load duration factor K_D [1]

See also the long term load factor K_D (4.3.2.3 CSA O86-09), if the specified long term load P_L is greater than the specified standard term load P_S.

System factor K_H (5.4.4 CSA O86-09)

"*The specified strength for glued-laminated timber members in a system consisting of three or more essentially parallel members spaced not more than 610 mm apart and so arranged that they mutually support the applied load may be multiplied by a system factor, K_H, equal to 1.00 for tension parallel to grain and 1.10 for all other strength properties.*" [1]

		Case 2†		
For specified strength in	Case 1*	Visually graded	MSR	Built-up beams
Bending	1.10	1.40	1.20	1.10
Longitudinal shear	1.10	1.40	1.20	1.10
Compression parallel to grain	1.10	1.10	1.10	1.00
Tension parallel to grain	1.10	—	—	1.00
All other properties	1.00	1.00	1.00	1.00

**See Clause 5.4.4.1 for conditions applying to Case 1.*
†See Clause 5.4.4.2 for conditions applying to Case 2.

Table 5: System factor K_H [1]

Size factor K_Z (5.4.5 CSA O86-09)

Some specified strengths of visually stress-graded lumber vary with member size and shall be multiplied by a size factor.

K_Z shall be 1.0 for light framing grades, for machine stress-rated lumber and machine evaluated lumber, except K_{Zc} shall be calculated in accordance with Clause 5.5.6.2.3, K_{Zcp} may be determined in accordance with Clause 5.5.7.5, and K_{Zv} shall be as given in table 6 (Table 5.4.5 of the CSA O86-09).

Larger dimension, mm	Bending and shear K_{Zb}, K_{Zv} Smaller dimension, mm 38 to 64	89 to 102	114 or more	Tension parallel to grain, K_{Zt} All	Compression perpendicular to grain, K_{Zcp} All	Compression parallel to grain, K_{Zc} All	All other properties All
38	1.7	—	—	1.5	See Clause 5.5.7.5	Value computed using formula in Clause 5.5.6.2.3	1.0
64	1.7	—	—	1.5			1.0
89	1.7	1.7	—	1.5			1.0
114	1.5	1.6	1.3	1.4			1.0
140	1.4	1.5	1.3	1.3			1.0
184 to 191	1.2	1.3	1.3	1.2			1.0
235 to 241	1.1	1.2	1.2	1.1			1.0
286 to 292	1.0	1.1	1.1	1.0			1.0
337 to 343	0.9	1.0	1.0	0.9			1.0
387 or larger	0.8	0.9	0.9	0.8			1.0

Table 6: Size factor K_Z [1]

Service condition factor K_S (6.4.2 CSA O86-09)

The service condition factor K_S integrates a wet service condition. Thus the specified strengths are quoted for dry service condition.

K_S	For specified strength in	Glued-laminated timber Dry service conditions	Wet service conditions
K_{Sb}	Bending at extreme fibre	1.00	0.80
K_{Sv}	Longitudinal shear	1.00	0.87
K_{Sc}	Compression parallel to grain	1.00	0.75
K_{Scp}	Compression perpendicular to grain	1.00	0.67
K_{St}	Tension parallel to grain	1.00	0.75
K_{Stp}	Tension perpendicular to grain	1.00	0.85
K_{SE}	Modulus of elasticity	1.00	0.90

Table 7: Service condition factors K_S [1]

Treatment factor K_T (CSA O86-09 5.4.3)

Product	Dry service conditions	Wet service conditions
Untreated lumber	1.00	1.00
Preservative-treated unincised lumber	1.00	1.00
Preservative-treated incised lumber of thickness 89 mm or less		
Modulus of elasticity	0.90	0.95
Other properties	0.75	0.85
Fire-retardant-treated lumber	See Clause 5.4.3.2 for effects of fire-retardant treatment.	

Table 8: Treatment factor K_T

Flow chart of determining guidelines

The procedure for determining guidelines shall present in the following flow chart:

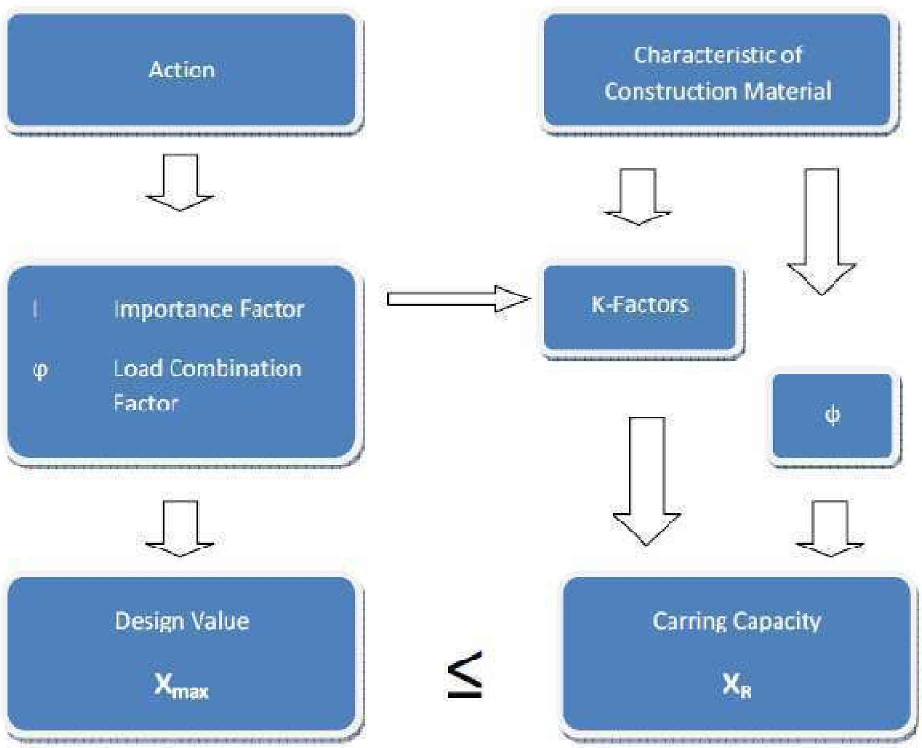


Figure 3: Flow chart for determining guidelines in ULS

3.1.2 The Canadian CLT-Handbook [2]

As mentioned the modification factors for CLT are not implemented in the CSA O86 yet. To design CLT panels there are several assumptions proposed in the Canadian CLT-Handbook.

In this list are the tables for the K-factors of the CSA O86-09 and the CLT-Handbook quoted. The appropriate K-factor to design CLT-Elements is to take out the following details:

K factor	CSA O86-09	In this thesis referred to	CLT-Handbook
K_D	Table 4.3.2.2	Table 4	Chap. 6 Table 1
K_S	Table 6.4.2	Table 7	Chap. 6 Table 2
K_H	Table 5.4.4	Table 5	Chap. 3/ 2.6.3 (K_H=1.0)
K_T	Table 4.3.4.4 and 6.4.4	Table 8	Chap. 3/ 2.6.4 (K_T=1.0)
K_L	Table 6.5.6.4 and 8.5.7		Chap. 3/ 2.6.5 (for beams)
K_{Zb}	Table 6.5.6.5.1	Table 6	Chap. 3/ 2.6.6

Table 9: Modification factors for CLT

The values of the factors K_D, K_S, K_L, K_{zb} are equal to the values in the CSA O86-09 and can be taken out from the according tables. Only for K_T and K_H a value of 1.0 is proposed.

In addition to the CLT K-factors the CLT-Handbook suggests a calculation of the bending moment resistance and shear resistance which will be explained in the following.

For the calculation of the bending moment resistance and shear resistance the CLT-Handbook refers to several methods. In this master thesis primarily the Shear Analogy Method and the proposed “Simplified Method” will be cited.

Shear Analogy Method used in Canada according the CLT-Handbook

The shear analogy method is a proven method to design multi-layer cross sections. This method takes the shear deformation in consideration. For that the initial beam is divided in two imaginary beams which are coupled with infinitely rigid web members. The two beams have the same vertical deflection. The imaginary beam on the top is usually named beam A and the other beam B. Beam A maintains the sum of the moment of inertia of the individual plies during beam B maintains the Steiner points part of the moment of inertia.

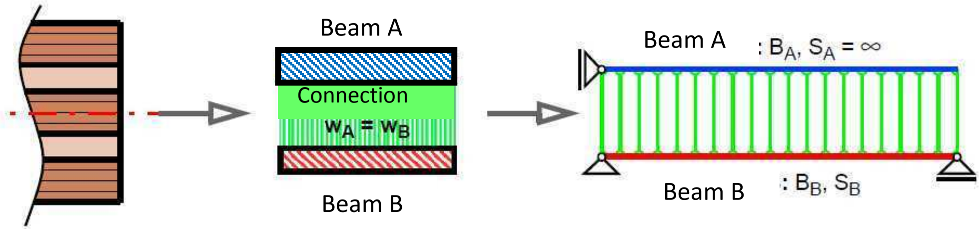


Figure 4: Concept of the Shear Analogy Method [7]

The bending and shear **stiffness** of beam A can be expressed as:

$$B_A = \sum_{i=1}^{n} E_i * I_i = \sum_{i=1}^{n} E_i * b_i * \frac{h_i^3}{12} \qquad [3.5]$$

$$S_A = \infty \qquad [3.6]$$

In addition the Beam B is as followed:

$$B_B = \sum_{i=1}^{n} E_i * A_i * z_i^2 \qquad [3.7]$$

$$\frac{1}{S_B} = \frac{1}{a^2} * \left[\sum_{i=1}^{n-1} \frac{1}{k_i} + \frac{h_1}{2 * G_1 * b_1} + \sum_{i=2}^{n-1} \frac{h_i}{G_i * b_i} + \frac{h_n}{2 * G_n * b_n}\right] \qquad [3.8]$$

where: $k_i = \frac{K_i}{s_i}$ slip of the fasteners

K_i... slip modulus of mechanical fasteners (but not present in glued CLT)

s_i... spacing between the mechanical fasteners (but not present in glued CLT)

The shear analogy method correlates the shear deflexion by using E_0 and G_0 for longitudinal layers and E_{90} as well as G_{90} for transversal layers. E_{90} is defined as $E_{90} = \frac{E_0}{30}$. In addition the shear module is given as $G_0 = \frac{E_0}{16}$ and $G_{90} = \frac{E_0}{160}$.

Furthermore the **bending and shear forces** for beam A can be calculated:

$$M_{A,i} = \frac{E_i I_i}{B_A} * M_A \qquad [3.9]$$

$$V_{A,i} = \frac{E_i I_i}{B_A} * V_A \qquad [3.10]$$

The **bending stress** and **shear stress** can be expressed as:

$$\sigma_{A,i} = \pm \frac{M_{A,i}}{I_i} * \frac{h_i}{2} \qquad [3.11]$$

$$\tau_{A,i} = \frac{E_i I_i}{B_A} * 1.5 * \frac{V_A}{b * h_i} \qquad [3.12]$$

The **axial force** and the **normal stress** for beam B may be calculated with:

$$N_{B,i} = \frac{E_i A_i * z_i}{B_B} * M_B \qquad [3.13]$$

$$\sigma_{B,i} = \frac{N_{B,i}}{b_i * h_i} = \frac{E_i * z_i}{B_B} * M_B \qquad [3.14]$$

In addition the **shear stress** may by calculated by:

$$\tau_{Bi,i+1} = \frac{V_B}{B_B} * \sum_{j=i+1}^{n} E_j * A_j * z_j \qquad [3.15]$$

where: V_B... shear force of beam B

These equations for bending, shear and normal stress are not used for the verification in Canada. For the verification only the stiffness parameters of bending and shear will be calculated according the Shear Analogy. The bending resistance M_r and the shear resistance V_r for the verification shall be calculated according to the “Simplified Method” which will be explained later in this chapter.

Deflection and Creep in the CLT-Handbook

The **deflection** expresses the sum of the contribution due to bending and shear. The maximal deflection is induced in the middle of the slab under a uniform load:

$$u_{max} = \frac{5}{384} * \frac{qL^4}{(EI)_{eff}} + \frac{1}{8} * \frac{qL^2 k}{(GA)_{eff}} \qquad [3.16]$$

where κ=1.2 the shear coefficient

The long term deflection takes the calculation of **creep** in consideration. Two options are proposed in the Canadian CLT-Handbook:

1) Option I

- The use of the load duration factor K_D (see table 9);
- The use of the service condition factor K_S (see table 9);
- The use of creep factor: The rolling shear modulus (G_{90}) is reduced with 25% to determine the elastic deflection due to total load. The permanent deformation due to long term loads is determined with a 50% reduction of the rolling shear modulus (G_{90}).

2) Option II

- Following the Eurocode 5 a k_{mod} factor for Service Classes 1 and 2 shall be taken.
- The use of the creep factor based on recommendations of Jöbstl and Schickhofer (2007) according to table 10:

$$u_{fin,P} = u_{inst,P}\left(1 + k_{def}\right)$$ for permanent loads, P [3.17]

$$u_{fin,Q,1} = u_{inst,Q,1}\left(1 + \psi_{2,1}k_{def}\right)$$ for main live loads, Q_1 [3.18]

$$u_{fin,Q,i} = u_{inst,Q,i}\left(\psi_{0,i} + \psi_{2,i}k_{def}\right)$$ for accompanying live loads, Q_i (i >1) [3.19]

Material	Service Class 1	Service Class 2	Service Class 3
CLT	0.90	1.10	N/A

Table 10: Deformation modification factor k_{def}, adjusts to CLT (CLT-Handbook chapter 6)

From table 10 is to recognize that the Canadian CLT-Handbook proposes a k_{def} factor 0.9 for service class 1 during the European approach is defined by a k_{def} factor 0.85 for service class 1.

"Simplified Method": modified procedure for Canada

The bending stiffness for the entire beam is the addition of the bending stiffness of the two imaginary beams.

$$(EI)_{eff} = B_A + B_B = \sum_{i=1}^{n} E_i * b_i * \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i * A_i * z_i^2 \qquad [3.20]$$

$$(GA)_{eff} = \frac{a^2}{\left[\frac{h_1}{2*G_1*b_1} + \sum_{i=2}^{n-1}\frac{h_i}{G_i*b_i} + \frac{h_n}{2*G_n*b_n}\right]} \qquad [3.21]$$

Bending stress for the "Simplified Method" is given as:

$$\sigma = M * y * \frac{E_1}{(EI)_{eff}} \qquad [3.22]$$

Maximum stress occurs on $y = \frac{h_{tot}}{2}$, which results to

$$\sigma_{max} = M * 0.5h_{tot} * \frac{E_1}{(EI)_{eff}}. \qquad [3.23]$$

The maximum stress is reached [1] when:

$$\sigma_{max} \leq \phi * F_b \qquad [3.24]$$

From the equations [3.2] the moment resistance can be expressed as:

$$M_r = \phi * F_b * \frac{(EI)_{eff}}{E_1} * \frac{1}{0.5h_{tot}} \qquad [3.25]$$

F_b may be calculated as proposed in the CSA O86-09. Table 9 refers to source of the K-factors for CLT.

$$F_b = f_b(K_D K_H K_{Sb} K_T) \qquad [3.26]$$

$(EI)_{eff}$ may be calculated with the γ-Method or with the Shear Analogy Method.

- γ-Method:

$$(EI)_{eff} = \sum_{i=1}^{n}\left(E_i I_i + \gamma_i E_i A_i a_i^2\right) \qquad [3.27]$$

- Shear Analogy Method:

Note: The Theory of Timoshenko gives exact the same equation as the Shear Analogy Method. Therefore the results according the theory of Timoshenko will be equal to the Shear Analogy Method.

$$(EI)_{eff} = B_A + B_B = \sum_{i=1}^{n} E_i * b_i * \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i * A_i * z_i^2$$

[3.28]=[3.10]

If the module of elasticity of the longitudinal layer is equal for the whole cross section, the resistance in bending moment can be determined as

$$M_r = \phi * F_b * \frac{I_{eff}}{0.5h_{tot}}. \qquad [3.29]$$

The shear strength of CLT is defined as

$$\tau_v = \frac{1.5 * V}{c * A_{gross}}. \qquad [3.30]$$

where $c = \frac{I_{eff}}{I_{gross}}$ ... reduction factor [3.31]

A_{gross}... gross cross-sectional area

I_{gross}... moment of inertia of the gross cross-section

With the condition [1]

$$\tau_v \leq \phi * F_v \quad [3.32]$$

the shear resistance results from the equations above to the following equation:

$$V_r = \phi F_v \frac{2A_{gross}}{3}\left(\frac{I_{eff}}{I_{gross}}\right) \quad [3.33]$$

where:

$$F_v = f_v(K_D K_H K_{Sv} K_T) \quad [3.34]$$

$$F_{v,s} = f_{v,s}(K_D K_H K_{Sv} K_T) \cong 0.5MPa \quad [3.35]$$

$F_{v,s}$... Rolling shear strength

Vibration performance (CAN)

A design procedure to grade the vibration for CLT as not irritating, the deflection shall be limited. Therefore the Uniformly Distributed Load (UDL) Deflection Method is known. Nevertheless this method provides a poor vibration performance for long span floors. For this reason FPInnovations and UNB developed a new design method. It is valid for an area mass from 15 kg/m² to 150 kg/m² and a fundamental natural frequency above 9 Hz (Hu). The new proposed design method was approved by SINTEF and included in the Canadian CLT-Handbook.

For vibration performance the design criterion is given by Gagnon S. and Hu L.:

$$\frac{f}{d^{0.7}} \geq 13.0 \quad \text{or} \quad d \leq \frac{f^{1.43}}{39} \quad [3.36]$$

f... fundamental natural frequency
d... 1 kN static deflection

The fundamental natural frequency can be calculated as followed:

$$f = \frac{3.142}{2l^2}\sqrt{\frac{(EI)_{eff}^{1m}}{\rho A}} \quad [3.37]$$

where:

l... CLT floor maximum span in meter
$(EI)_{eff}^{1m}$... effective apparent stiffness in the span direction for 1m wide panel in Nm²

ρ... density of CLT in kg/m³
A... Area of cross section of 1m wide panel [m²]

$$d = \frac{1000Pl^3}{48EI_{eff}^{1m}} \quad [3.38]$$

where: P... 1000 N

To verify vibrations as none irritating the length of the CLT element shall be limited by [3.39]. The equation [3.39] is the result of inserting the static deflection [3.38] and the fundamental natural frequency [3.37] in equation [3.36].

$$l \leq \frac{1}{9,15}\frac{\left(EI_{eff}^{1m}\right)^{0.293}}{(\rho A)^{0.123}} \quad [3.39]$$

3.1.3 American National Standard, PRG 320 [3]

The PRG 320 maintains panel dimensions and dimensional tolerances, component requirements, performance criteria, qualification and product marketing and quality assurance for CLT panels. This standard is one of the first manufacturing standards that are used together in Canada and USA.

In this section the plant pre-qualification and mechanical properties qualification are cited.

Plant Pre-Qualification

A pre-qualification panel shall be tested by six square/rectangular specimens.

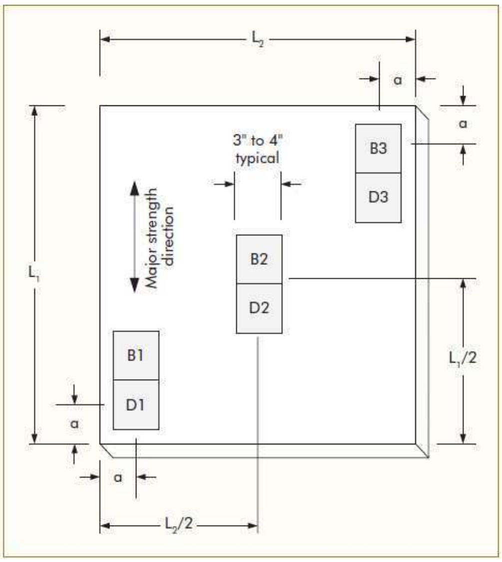


Figure 5: Block shear („B“) and delamination („D“) specimen locations: a=101.6±25.4 mm (4±1 inches), L_1=609.6 to 914.4 mm (24 to 36 inches) and L_2=457.2 to 914.4 mm (18 to 36 inches) (1 inch = 25.4mm) [3]

Three of the six are tested on block shear, named "B", the other three are tested on delamination, named "D". From each pre-qualification panel the specimens shall be extracted and labelled like it is shown in the next figure.

Mechanical properties qualification

For stiffness parameters a sample size is required that shall be sufficient for estimating the population mean within 5% precision with 75% confidence, or 10 specimens, whichever is greater. A sample size greater than 10 is required when coefficient of variation is greater than 13%. For strength capacities the sample size requires a characteristic value with 75% confidence in accordance with ASTM D2915.

The condition of the CLT samples shall have average moisture content not less than 8% according to the PRG 320. Further it is considered to store the samples in an indoor environment for a minimum of 24 hours or until the adhesive has cured sufficiently to permit evaluation, whichever is longer.

To test CLT panels on bending, the load is applied perpendicular to the face layer. In accordance with the third-point load method through ASTM D198 or ASTM D4761 the test shall be conducted flat wise. A specimen width of not less than 305 mm and the on-center span equal to approximately 30 times the specimen depth shall be used. “*The weight of the CLT panel is permitted to be included in the determination of the bending moment capacity*.” [8]

Determined from qualification tests the average bending stiffness (EI) and the characteristic bending moment (f_bS) shall meet or exceed the published allowable bending stiffness and allowable bending.

The shear test shall be conducted as the bending test according ASTM D198 or ASTM D4761. “*The bearing length shall be sufficient to avoid bearing failure, but not greater than the specimen depth. All specimens are to be cut to length with no overhangs allowed.” [8]*

The results from the shear test shall meet or exceed the published allowable interlaminar shear capacity.

In following an extract [3] of the allowable design properties for CLT grades is given by.

- E1: *1950f-1.7E Spruce-pine-fir MSR lumber in all parallel layers and No. 3 Spruce-pine-fir lumber in all perpendicular layers*
- E2: *1650f-1.5E Douglas fir-Larch MSR lumber in all parallel layers and No. 3 Douglas fir-Larch lumber in all perpendicular layers*
- E3: *1200f-1.2E Eastern Softwoods, Northern Species, or Western Woods MSR lumber in all parallel layers and No. 3 Eastern Softwoods, Northern Species, or Western Woods lumber in all perpendicular layers*
- E4: *1950f-1.7E Southern pine MSR lumber in all parallel layers and No. 3 Southern pine lumber in all perpendicular layers*
- V1: *No. 2 Douglas fir-Larch lumber in all parallel layers and No. 3 Douglas fir-Larch lumber in all perpendicular layers*
- V2: *No. 1/No. 2 Spruce-pine-fir lumber in all parallel layers and No. 3 Spruce-pine-fir lumber in all perpendicular layers*
- V3: *No. 2 Southern pine lumber in all parallel layers and No. 3 Southern pine lumber in all perpendicular layers*

Table 11: layups for design properties for CLT [3]

The next table lists the published allowable specified strength and modulus of elasticity and the published allowable bending resistance for CLT in Canada.

TABLE A3.
SPECIFIED STRENGTH AND MODULUS OF ELASTICITY(a,b,c) FOR PRG 320 CLT (FOR USE IN CANADA)

	Major Strength Direction						Minor Strength Direction			
CLT Grades	$f_{b,0}$ (MPa)	E_0 (MPa)	$f_{t,0}$ (MPa)	$f_{c,0}$ (MPa)	$f_{v,0}$ (MPa)	$f_{s,0}$ (MPa)	$f_{b,90}$ (MPa)	E_{90} (MPa)	$f_{v,90}$ (MPa)	$f_{s,90}$ (MPa)
E1	28.2	11,700	15.4	19.3	1.5	0.50	7.0	9,000	1.5	0.50
E2	23.9	10,300	11.4	18.1	1.9	0.63	4.6	10,000	1.9	0.63
E3	17.4	8,300	6.7	15.1	1.3	0.43	4.5	6,500	1.3	0.43
V1	10.0	11,000	5.8	14.0	1.9	0.63	4.6	10,000	1.9	0.63
V2	11.8	9,500	5.5	11.5	1.5	0.50	7.0	9,000	1.5	0.50

For SI: 1 MPa = 145 psi
(a) See Section 4 for symbols.
(b) Tabulated values are Limit States design values and not permitted to be increased for the lumber size adjustment factor in accordance with CSA O86. The design values shall be used in conjunction with the section properties provided by the CLT manufacturer based on the actual layup used in manufacturing the CLT panel (see Table A4).
(c) Custom CLT grades that are not listed in this table shall be permitted in accordance with Section 7.2.1.

Table 12: Specified strength and modulus of elasticity for PRG 320 CLT for use in Canada [3]

TABLE A4.
THE LSD BENDING RESISTANCES(a,b,c) FOR CLT LISTED IN TABLE A3 (FOR USE IN CANADA)

		Lamination Thickness (mm) in CLT Layup							Major Strength Direction			Minor Strength Direction		
CLT Grade	CLT t (mm)	=	⊥	=	⊥	=	⊥	=	$f_bS_{eff,0}$ (10^6 N-mm/m)	$EI_{eff,0}$ (10^9 N-mm²/m)	$GA_{eff,0}$ (10^6 N/m)	$f_bS_{eff,90}$ (10^6 N-mm/m)	$EI_{eff,90}$ (10^9 N-mm²/m)	$GA_{eff,90}$ (10^6 N/m)
E1	105	35	35	35					42	1,088	7.3	1.4	32	13
	175	35	35	35	35	35			98	4,166	15	12	837	20
	245	35	35	35	35	35	35	35	172	10,306	22	29	3,220	28
E2	105	35	35	35					36	958	8.0	0.94	36	15
	175	35	35	35	35	35			83	3,674	16	8.2	930	18
	245	35	35	35	35	35	35	35	146	9,097	24	19	3,569	26
E3	105	35	35	35					26	772	5.3	0.92	23	9.5
	175	35	35	35	35	35			60	2,956	11	8	605	14
	245	35	35	35	35	35	35	35	106	7,313	16	18	2,325	20
V1	105	35	35	35					15	1,023	8.0	0.94	36	15
	175	35	35	35	35	35			35	3,922	16	8.2	930	20
	245	35	35	35	35	35	35	35	61	9,708	24	19	3,571	27
V2	105	35	35	35					18	884	7.2	1.4	32	13
	175	35	35	35	35	35			41	3,388	14	12	837	17
	245	35	35	35	35	35	35	35	72	8,388	22	29	3,213	23

For SI: 1 mm = 0.03937 in.; 1 m = 3.28 ft
(a) See Section 4 for symbols.
(b) This table represents one of many possibilities that the CLT could be manufactured by varying lamination grades, thicknesses, orientations, and layer arrangements in the layup.
(c) Custom CLT grades that are not listed in this table shall be permitted in accordance with Section 7.2.1.

Table 13: the LSD (Limit State Design) bending resistances for CLT for Canada [3]

Symbols:

E_0 Modulus of elasticity in bending parallel to the major strength direction of CLT in MPa;

E_{90} Modulus of elasticity in bending perpendicular to the major strength direction of CLT in MPa, in the PRG 320 $E_{90} = {}^{E_0}/_{30}$ for lumber;

$F_{b,0}$ and $f_{b,0}$ allowable bending stress and characteristic bending strength parallel to the major strength direction of CLT in MPa;

$F_{b,90}$ and $f_{b,90}$ allowable bending stress and characteristic bending strength perpendicular to the major strength direction of CLT in MPa;

$F_{c,0}$ and $f_{c,0}$ allowable compressive stress and characteristic compressive strength parallel to the major strength direction of CLT in MPa;

$F_{c,90}$ and $f_{c,90}$ allowable compressive stress and characteristic compressive strength perpendicular to the major strength direction of CLT in MPa;

$F_{t,0}$ and $f_{t,0}$ allowable tensile stress and characteristic tensile strength parallel to the major strength direction of CLT in MPa;

$F_{t,90}$ and $f_{t,90}$ allowable tensile stress and characteristic tensile strength perpendicular to the major strength direction of CLT in MPa;

$F_{v,0}$ and $f_{v,0}$ allowable shear stress and characteristic shear strength parallel to the major strength direction of CLT in MPa;

$F_{s,0}$ and $f_{s,0}$ allowable interlaminar (rolling) stress and characteristic interlaminar (rolling) strength parallel to the major strength direction of CLT in MPa;

3.2 The European Standard

3.2.1 Eurocode 5 [4]

The ON EN 1995-1-1 2006 01 Eurocode 5 Design of timber structures - Part 1-1 General Common rules and rules for buildings gives the followed guideline away:

The equation to calculate the Ultimate Limit State, the basic load combination

$$E_d = \sum \gamma_{G,i} * G_{k,i} + \gamma_P * P + \gamma_{Q,1} * Q_{k,1} + \sum \gamma_{Q,i} * \psi_{0,i} * Q_{k,i} \qquad [3.40]$$

is given. For extremely load combination the equation reads:

$$E_d = \sum G_{k,i} + P + A_d + \psi_{2,1} * Q_{k,1} + \sum \psi_{2,i} * Q_{k,i} \qquad [3.41]$$

The load combination for earthquake is expressed as:

$$E_d = \sum G_{k,i} + P + A_{Ed} + \sum \psi_{2,i} * Q_{k,i} \qquad [3.42]$$

The values for the partial safety factor γ are:

- Dead load to the weight of members (G):
  $\gamma_G = 1.35$ unfavourable effect
  $\gamma_G = 1.00$ favourable effect

- Live load to the intended use and occupancy (Q):
  $\gamma_Q = 1.50$ unfavourable effect
  $\gamma_Q = 1.35$ unfavourable effect for live loads on bridges
  $\gamma_Q = 1.00$ favourable effect

- Initial tension (P):
  $\gamma_P = 1.00$

The combination factors ψ for structural engineering of the ON EN 1990 are given in the following table. Table 14 maintains the particular effects of structural engineering. The combination factors for bridges are given in the EN 1990 Annex A2.

	1	2	3	4	5
1	Action	Category[1)]	ψ_0	ψ_1	ψ_2
2	**Live loads** in building structures				
	Areas of domestic and residential activities, office areas	A, B	0.7	0.5	0.3
	Areas where people may congregate	C, D	0.7	0.7	0.6
	Shopping areas	E	1.0	0.9	0.8
	Traffic and parking area for light vehicle (≤30kN)	F	0.7	0.7	0.6
	Traffic and parking area for medium vehicle (30kN ÷ 160kN)	G	0.7	0.5	0.3
3	**Roofs**	H	0.0	0.0	0.0
4	**Load due to snow** in building structures Altitude/Region > 1000m Altitude/Region < 1000m		 0.7 0.5	 0.5 0.2	 0.2 0.0
5	**Load due to wind** in building structures		0.6	0.2	0.0
6	**Load due to contraction or expansion cause by temperature changes** (without fire) in building structures		0.6	0.5	0.0
	[1)] according ON EN 1991-1-1				

Table 14: combination factors ψ_0, ψ_1, ψ_2, according ON EN 1990 Annex A[5]

For design value of material property the equation is given by:

$$X_d = k_{mod} \frac{X_k}{\gamma_M} \qquad [3.43]$$

where:

X_K... characteristic value of a strength property

γ_M... partial factor for a material property

k_{mod}... modification factor taking into account of the duration of load and moisture content

The following table maintains the recommended partial safety factors γ_M. The next section “BSPhandbuch” contains a suggestion for CLT.

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

Table 15: Recommended partial safety factors γ_M for material properties and resistances [4]

Furthermore the values for k_{mod} in addiction of service class, load duration and material:

Material	Standard	Service class	Load-duration class				
			Permanent action	Long term action	Medium term action	Short term action	Instantaneous action
Solid timber	EN 14081-1	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90
Glued laminated timber	EN 14080	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90
LVL	EN 14374, EN 14279	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90
Plywood	EN 636						
	Part 1, Part 2, Part 3	1	0,60	0,70	0,80	0,90	1,10
	Part 2, Part 3	2	0,60	0,70	0,80	0,90	1,10
	Part 3	3	0,50	0,55	0,65	0,70	0,90
OSB	EN 300						
	OSB/2	1	0,30	0,45	0,65	0,85	1,10
	OSB/3, OSB/4	1	0,40	0,50	0,70	0,90	1,10
	OSB/3, OSB/4	2	0,30	0,40	0,55	0,70	0,90
Particle-board	EN 312						
	Part 4, Part 5	1	0,30	0,45	0,65	0,85	1,10
	Part 5	2	0,20	0,30	0,45	0,60	0,80
	Part 6, Part 7	1	0,40	0,50	0,70	0,90	1,10
	Part 7	2	0,30	0,40	0,55	0,70	0,90

Table 16: Extract of the modification factor k_{mod} [4]

The Design resistance (2.17) may be calculated with:

$$R_d = k_{mod} \frac{R_k}{\gamma_M}. \qquad [3.44]$$

R_d... Design value of load carrying capacity

R_k... Characteristic value of load carrying capacity

Further factors of influence on the strength are the factors k_{size} and k_{sys}. With this factors the member size are taken into account.

In following the specified strength and modulus of elasticity are given away. The values are used in Europe for bulk material. The values are proposed from Institute of Timber Engineering and Wood Technology (Graz University of Technology.

In addition the specified strength and modulus of elasticity for the GLh24* is cited.

$f_{m,k}$	24,0
$f_{t,0,k}$	16,5
$f_{t,90,k}$	0,5
$f_{c,0,k}$	24,0
$f_{c,90,k}$	2,7
$f_{v,k}$	3,0
$f_{r,k}$	1,0
$E_{0,mean}$	11600
$E_{90,mean}$	390
$E_{0,05}$	9667
$G_{0,mean}$	720
$G_{90,mean}$	72
$G_{0,05}$	600
ρ_k	380

GL24h

$f_{m,k}$	28,0
$f_{t,0,k}$	19,5
$f_{t,90,k}$	0,5
$f_{c,0,k}$	26,5
$f_{c,90,k}$	3,0
$f_{v,k}$	3,0
$f_{r,k}$	1,0
$E_{0,mean}$	12600
$E_{90,mean}$	420
$E_{0,05}$	10500
$G_{0,mean}$	780
$G_{90,mean}$	78
$G_{0,05}$	650
ρ_k	410

GL28h

$f_{m,k}$	32,0
$f_{t,0,k}$	22,5
$f_{t,90,k}$	0,5
$f_{c,0,k}$	29,0
$f_{c,90,k}$	3,3
$f_{v,k}$	3,0
$f_{r,k}$	1,0
$E_{0,mean}$	13700
$E_{90,mean}$	460
$E_{0,05}$	11417
$G_{0,mean}$	850
$G_{90,mean}$	85
$G_{0,05}$	708
ρ_k	430

GL32h

$f_{m,k}$	16,0
$f_{t,0,k}$	10,0
$f_{t,90,k}$	0,4
$f_{c,0,k}$	17,0
$f_{c,90,k}$	2,2
$f_{v,k}$	2,7
$f_{r,k}$	1,0
$E_{0,mean}$	8000
$E_{90,mean}$	270
$E_{0,05}$	5333
$G_{0,mean}$	500
$G_{90,mean}$	50
$G_{0,05}$	333
ρ_k	310

C16

$f_{m,k}$	24,0
$f_{t,0,k}$	14,0
$f_{t,90,k}$	0,4
$f_{c,0,k}$	21,0
$f_{c,90,k}$	2,5
$f_{v,k}$	2,7
$f_{r,k}$	1,0
$E_{0,mean}$	11000
$E_{90,mean}$	370
$E_{0,05}$	7333
$G_{0,mean}$	690
$G_{90,mean}$	69
$G_{0,05}$	460
ρ_k	350

C24

$f_{m,k}$	30,0
$f_{t,0,k}$	18,0
$f_{t,90,k}$	0,4
$f_{c,0,k}$	23,0
$f_{c,90,k}$	2,7
$f_{v,k}$	2,7
$f_{r,k}$	1,0
$E_{0,mean}$	12000
$E_{90,mean}$	400
$E_{0,05}$	8000
$G_{0,mean}$	750
$G_{90,mean}$	75
$G_{0,05}$	500
ρ_k	380

C30

Table 17: Specified strength and modulus of elasticity (bulk material) used in Europe proposed from Institute of Timber Engineering and Wood Technology

GL24h*			
$f_{m,k}$	24.0 N/mm²	$E_{0,05}$	9667.0 N/mm²
$f_{c,0,k}$	24.0 N/mm²	$E_{0,mean}$	11600.0 N/mm²
$f_{c,90,k}$	2.7 N/mm²	$E_{90,mean}$	0.0 N/mm²
$f_{t,0,k}$	16.5 N/mm²	$G_{0,mean}$	720.0 N/mm²
$f_{t,90,k}$	0.5 N/mm²	$G_{90,mean}$	72.0 N/mm²
$f_{v,k}$	3.0 N/mm²	Mean density	500.0 kg/m³
$f_{r,k}$	1.25 N/mm²		

Table 18: Specified strength and modulus of elasticity for GL24h* [6]

Symbols:

Index k (X_k)	Characteristic value
Index d (X_d)	Design value according equation [4]
$E_{0,05}$	Fifth percentile value of modulus of elasticity;
$E_{0,mean}$	Mean value of modulus of elasticity parallel to grain;
$E_{90,mean}$	Mean value of modulus of elasticity perpendicular to grain;
$G_{0,05}$	Fifth percentile value of shear modulus
$G_{0,mean}$	Mean value of shear modulus parallel to grain;
$G_{90,mean}$	Mean value of shear modulus perpendicular to grain;
F_c	Compressive force
F_t	Tensile force
N	Axial force
V	Shear force
M	Moment
$f_{c,0,k}$	Characteristic compressive strength parallel to grain
$f_{c,90,k}$	Characteristic compressive strength perpendicular to grain
$f_{m,k}$	Characteristic bending strength
$f_{t,0,k}$	Characteristic tensile strength parallel to grain
$f_{t,90,k}$	Characteristic tensile strength perpendicular to the grain
$f_{v,k}$	Characteristic shear strength (parallel to grain)
$f_{r,k}$	Characteristic shear strength perpendicular to the grain

3.2.2 BSPhandbuch [6]

The competence center competence center holz.bau forschungs gmbh and the Institute of Timber Engineering and Wood Technology Graz University of Technology indicate the following determining procedures to work with CLT. The results are based on tests.

A condition to choose a computational procedure is given by the length to high ratio L/H.

$\frac{L}{H} > 30$	Euler-Bernoulli Beam
$15 < \frac{L}{H} \leq 30$	Timoshenko Beam
$15 \geq \frac{L}{H}$	Shear-flexible Multi Layer Beam

Is the $L/_H$ ratio greater than 30, the theory of Euler Bernoulli is proposed. The shear deflection is insignificant. The theory of Timoshenko will be taken for the calculation of the internal forces and the deflection when the $L/_H$ ratio between 15 and 30. Further a ratio smaller than 15 induces a shear-flexible multilayer beam.

For CLT only the Service Class 1 and 2 conditions are permissible. This circumstance was defined to limit the swell and shrinkage properties.

In addition the long term factor k_{def} shall be considered in the calculation of the deflection. For Service Class 1 condition a k_{def} factor of 0.85 and for Service Class 2 condition a k_{def} factor of 1.1 is proposed.

The Bending stiffness for cross laminated timber is determined as

$$K_{clt} = \sum(I_i * E_i) + \sum(A_i * e_i^2 * E_i). \quad [3.45]$$

The bending elasticity module in the transversal layers may be taken as E_{90}=0.

The Shear stiffness for cross-laminated timber shall be calculated as

$$S_{clt} = S_{ges} * \kappa \quad [3.46]$$

$$S_{ges} = \sum(G_i * b_i * t_i) = \sum(G_i * A_i) \quad [3.47]$$

$$\kappa = 1 \Big/ \left(S_{ges} * \frac{1}{K_{clt}^2} * \int_h \frac{S^2\,(z, E(z))}{G(z) * b(z)} dz\right). \quad [3.48]$$

where: *S(z)...* static moment in dependence on z
G(z)... shear modulus in dependence on z
b... width of the cross section

The shear correction factor κ is a parameter which respects the material qualities and the geometry. The next figure demonstrates κ-factors for different cross section.

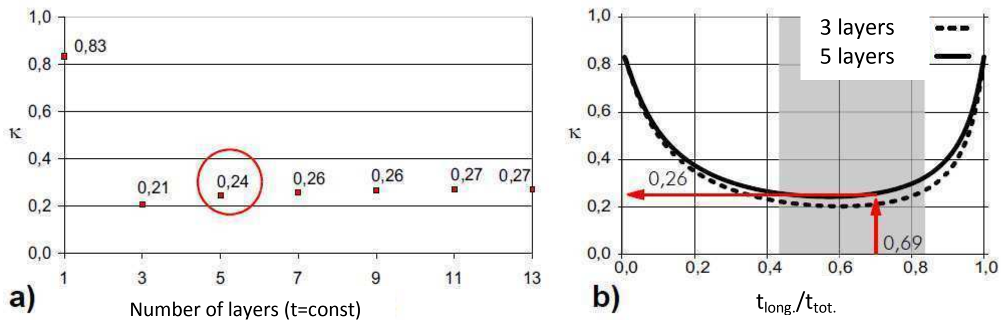


Figure 6: a) Shear correction factor for multilayer cross section with constant single layer thickness; b) Shear correction factor for 3 and 5 layer cross section with variable single layer thickness [6]

According to the BSPhandbuch the bending stress can be expressed as

$$\sigma(z) = \frac{M}{K_{clt}} * z * E(z). \qquad [3.49]$$

where: *σ(z)...* Bending stress in dependence on z
M... Bending moment
K_{clt}... Bending stiffness for CLT
z... Distance according figure 7
E(z) E-modulus in dependence on z

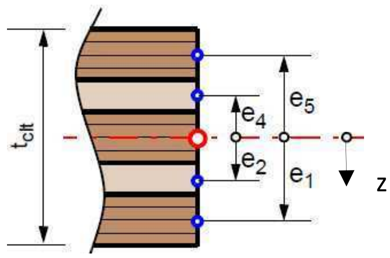


Figure 7: CLT cross section [6]

In addition the shear stress shall be

$$\tau(z_0) = \frac{V * \int_{A_0} E(z) * z * dA}{K_{clt} * b(z_0)}. \qquad [3.50]$$

where: *τ(z₀)*... shear stress depending on the coordinate of the thickness

V... shear force

A_0... Area between z_0 and the looked at edge

E(z)... E-modulus dependence on z

z... Distance according figure 7

K_{clt}... Bending stiffness for CLT

b(z)... Width of the CLT cross section

Design comprehensive stress perpendicular to the grain

$$\sigma_{c,clt,90,d} = \frac{F_d}{t_{ef} * b}. \qquad [3.51]$$

where: $\sigma_{c,clt,90,d}$... design compressive stress for CLT perpendicular to the grain

F_d... design force

t_{ef}... calculative effective thickness to calculate the total contact surface

b... width of the CLT cross section

Specified strengths for CLT:

$$f_{m,clt,k} = k_l * f_{m,glt,k} \qquad [3.52]$$

where:

$f_{m,glt,k}$... bending strength of the appropriate GLT class

$$k_l = min\begin{cases} 1.1 \\ 1 + 0.025 * n \end{cases} \quad for\, n > 1 \qquad [3.53]$$

The factor k_l (according DIBt) shall take the variance of the bulk material in consideration. It describes a system coefficient.

The Institute for timber construction and timber technology (Graz University of Technology) proposes a partial safety factor γ_M=1.25.

The verification of stress is defined according the following equations. The equations were derived from the determining procedures of the Eurocode. The approach for CLT is based on the use of the CLT parameters for strength and stiffness of the CLT cross section.

$$\frac{\sigma_{m,i,edge,d}}{f_{m,clt,d}} \leq 1.0 \qquad \sigma_{m,i,edge,d} = \frac{M_d}{K_{clt}}\left(e_i + \frac{t_i}{2}\right) * E_i \qquad [3.54]$$

$$\frac{\tau_{r,i,d}}{f_{r,clt,d}} \leq 1.0 \qquad \tau_{r,i,d} = \frac{V_{max,d} * \Sigma(S_m * E_m)}{K_{clt} * \mathrm{b_i}} \qquad [3.55]$$

$$\frac{\tau_{v,i,d}}{f_{v,clt,d}} \leq 1.0 \qquad \tau_{v,i,d} = \frac{V_{max,d} * \Sigma(S_m * E_m)}{K_{clt} * \mathrm{b_i}} \qquad [3.56]$$

where:
- b_i... width of the cross section
- i... Index of examined layer
- S_m... static moment of a specific layer
- E_m... modulus of elasticity of the appropriate layer

Deformation and creep

To calculate the deflection with the theory of Timoshenko the following equation shall be taken:

$$\mathrm{w} = \frac{1}{K_{clt}} * [\int \bar{M} * M * dx] + \frac{1}{S_{clt}} [\int \bar{V} * V * dx] \qquad [3.57]$$

Internal forces due to action load: Internal forces due to virtual force:

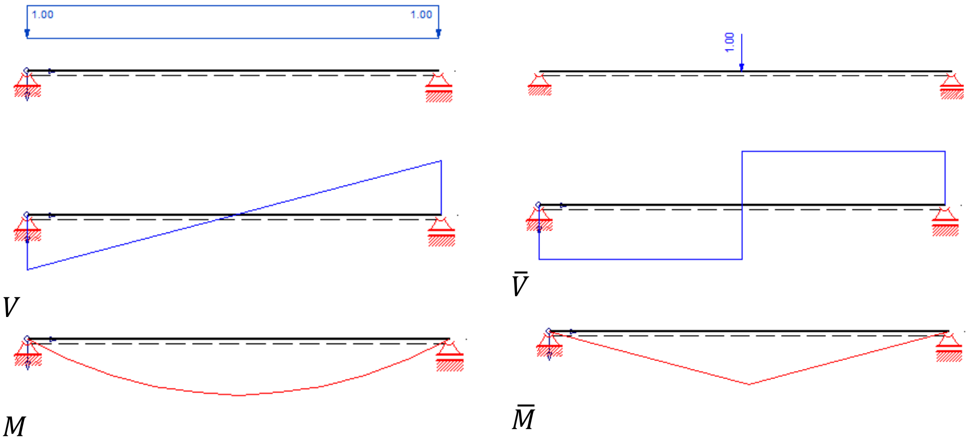


Figure 8: Internal forces for a calculation of the deflection according Timoshenko

For the verification of the deflection the deformation due to permanent load, live load and long term action permanent load shall be taken into account. The different combinations for the verification are limited according table 19.

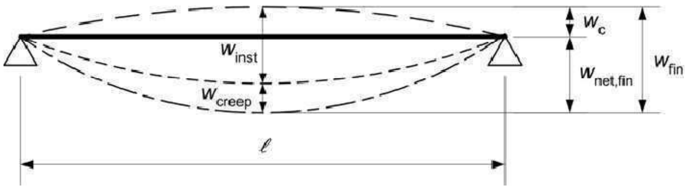


Figure 9: Components of deflection [4]

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

Table 19: Examples of limiting values for deflections of beams [4]

Vibration performance (EU)

The advantages of CLT are the high natural weight and bending stiffness in transversal direction due to the solid and flat construction. As well the damping is higher due to the longitudinal and transversal layers.

To not cause vibrations that can impair the function of the structure or causes unacceptable discomfort to the users the frequency, the deformation and the acceleration of vibration are limited.

Therefore the natural frequency of a beam can be expressed as:

$$f_{e,1} = \frac{\pi}{2L^2}\sqrt{\frac{(EI)_L}{m}} = f_{beam} \qquad [3.58]$$

Where $(EI)_L$ has to be replaced with the CLT parameter K_{clt}.

The natural frequency of a slab can be expressed as:

$$f_{slab} = f_{beam} * \sqrt{1 + \frac{1}{\alpha^4}} \qquad \alpha = \frac{b}{L} * \sqrt[4]{\frac{(EI)_L}{(EI)_Q}} \qquad [3.59]$$

Where $(EI)_L$ has to be replaced also with the CLT parameter K_{clt}. In longitudinal direction and $(EI)_Q$ with the parameter K_{clt} in transversal direction.

The BSPhandbuch lists three verification procedures on the design criterion for vibration performance. At first the DIN 1052 is cited, then the EN 1995-1-1 and also the method by Hamm/Richter.

- **DIN 1052:**

$$w_{perm} \leq 6mm$$
$$f_{e,perm} \geq 7.2Hz$$
$$a \leq 0.1 \, {}^{m}/_{s^2}$$

For lower requests the DIN 1052 allows also:

$$w_{perm} \leq 9mm$$
$$f_{e,perm} \geq 6Hz$$

The permanent deflection is expressed as:

$$w_{perm} = w_{G,inst} + \psi_2 * w_{Q,inst} = w(g) + 0.3 * w(p) \qquad [3.60]$$

- **EN 1995-1-1:**

$$f_e \geq 8Hz$$
$$w \leq 4mm$$

Where the deflection is caused by 1 kN load on the most unfavourable position on the beam.

If the natural frequency is greater than 8 Hz it should be satisfied that:

$$\frac{w}{F} \leq a \qquad [3.61]$$

$$v \leq b^{(f_1 \zeta - 1)} \qquad [3.62]$$

where:

a... according to figure 10 [mm/kN]
b... according to figure 10
w... *maximal instantaneous vertical deflection caused by a vertical concentrated static force F applied at any point on the floor, taking account of load distribution;*
v... *unit impulse velocity response, i. e. The maximum initial value of the vertical floor vibration velocity (m/s) caused by an ideal unit impulse (1 Ns) applied at the point of the floor giving maximum response. Components above 40 Hz may be disregarded;*

ζ... modal damping ratio

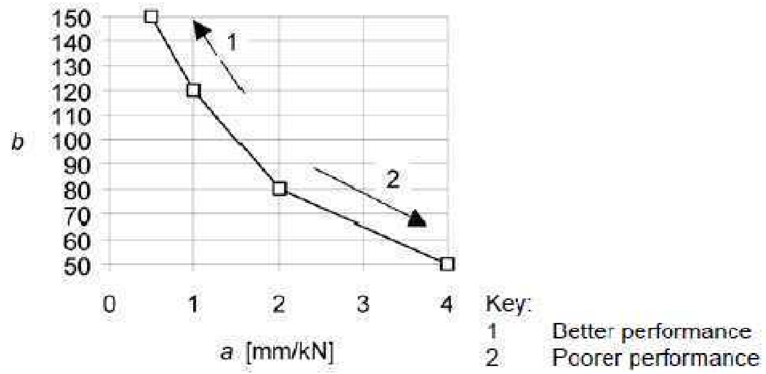


Figure 10: Recommended range of and relationship between a and b [4]

$$v = \frac{4(0.4 + 0.6n_{40})}{mbl + 200}$$

where: *v...* Unit impulse velocity response [m/(Ns²)]

n_{40}... number of first-order modes with natural frequencies up to 40 Hz

b... floor width [m]

m... mass [kg/m²]

l... floor span [m]

$$n_{40} = \left\{ \left(\left(\frac{40}{f_1} \right)^2 - 1 \right) \left(\frac{b}{l} \right)^4 \frac{(EI)_L}{(EI)_Q} \right\}^{0.25}$$

where: *$(EI)_Q$...* equivalent plate bending stiffness [Nm²/m], of the floor about an axis parallel to the beams, where $(EI)_Q < (EI)_L$

- **Hamm/Richter:**

$f_e \geq f_{grenz}$	
$f_{grenz} = 8Hz$	for evaluation from 1.0 till 1.5
$f_{grenz} = 6Hz$	for evaluation from 1.5 till 2.5
$w_{2kN} \leq w_{grenz}$	deflection under 2 kN single force at the most unfavourable position
$w_{grenz} = 0.5mm$	for evaluation from 1.0 till 1.5
$w_{grenz} = 1.0mm$	for evaluation from 1.5 till 2.5

The restriction of the acceleration of vibration or speed of vibration is given by Hamm/Richter. The acceleration of vibration is to verify when the frequency is under f_{grenz}.

$$f_{min} \leq f_e < f_{grenz}$$
$$f_{min} = 4.5Hz$$
$$a \leq a_{grenz}$$
$$a_{grenz} = 0.05\, {}^{m}/_{s^2} \qquad \text{for evaluation 1.0 to 1.5}$$
$$a_{grenz} = 0.10\, {}^{m}/_{s^2} \qquad \text{for evaluation 1.5 to 2.5}$$

Where the acceleration of vibration a [m/s²] is calculated with:

$$a\left[{}^{m}/_{s^2}\right] = \frac{0.4 * F(t)[N]}{m\left[{}^{kg}/_{m^2}\right] * 0.5 * b[m] * 0.5 * L[m] * 2 * \zeta}$$

where:	*F(t)...*	dynamic force (ex.: heeldrop)
	m...	Mass
	b...	Width
	L...	Length
	ζ...	Modal damping ratio

4 CHAPTER

4. STANDARD TEST METHODS OF STATIC TESTS OF LUMBER

4.1 Canadian Standard

4.1.1 ASTM Standard [8]

ASTM D198: Flexure and Shear

The test method for **flexure** determines the bending properties of structural beams made of solid or laminated wood or of composite constructions. Primarily the procedure is intended for beams of rectangular cross section. The test method is also valid for beams with round and irregular shapes such as round posts, I-beams or other special sections.

The beam is subjected to a bending moment. The specimen is supported near the end of the beam. The loads apply symmetrically between the supports.

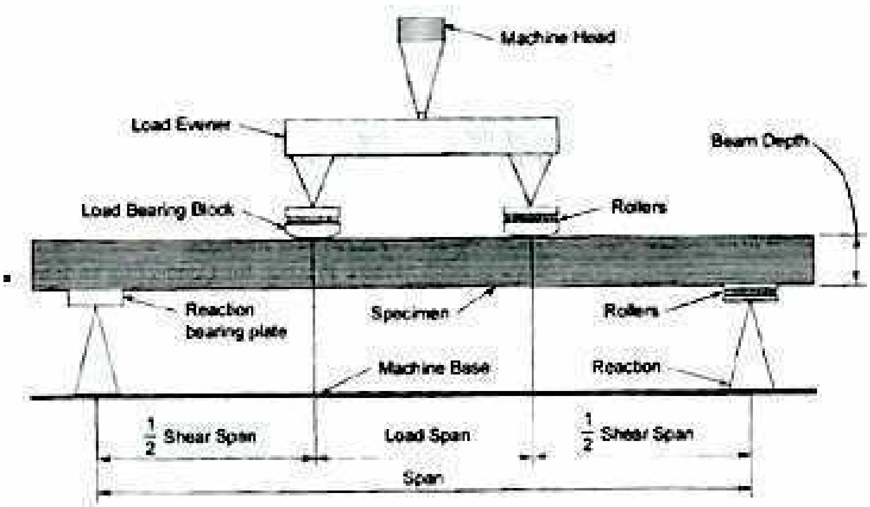


Figure 11: Flexure test method with two-point loading [8]

Figure 11 shows an example for a test set up. Obvious from the figure is the position of loading and supporting. Center-point loading or two-point loading shall be used for evaluation of shear properties.

Using the full span deflection the apparent modulus of elasticity shall be calculated. The shear free deflection is used to calculate the true or shear-free modulus of elasticity. The shear free deflection occurs there, where the bending moment is constant.

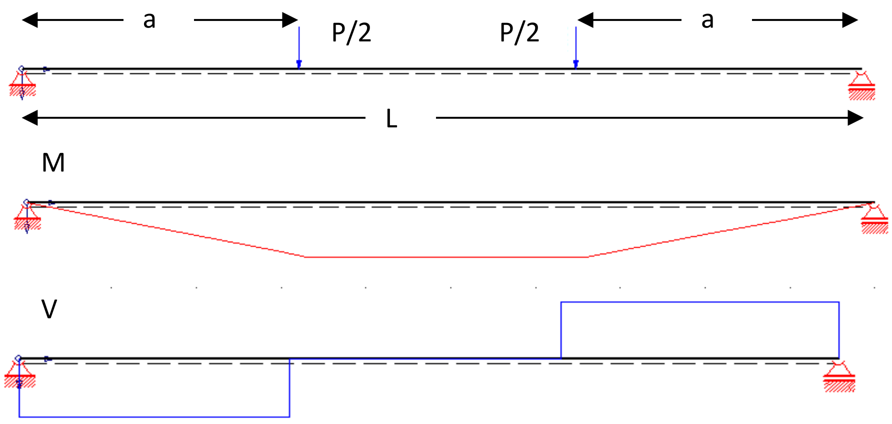


Figure 12: Test configuration, Bending moment and shear force under two point loading; shear free deflection between the loads

The identification of the specimen shall be as expansive as possible by including the origin or source of supply, species and history of drying and conditioning, chemical treatment, fabrication and other pertinent physical or mechanical details that may affect the strength. The character of the dissimilar materials and their size and location in the beam shall be reflected.

The conditioning of the specimen shall be in moisture equilibrium under the desired environment conditions, according test methods D4442 from ASTM.

The speed of testing is defined by bringing on the maximal load in about 10 min which should be reached in not less than 6 min or more than 20 min. "*A constant rate of outer strain, z, of 0.001 mm/mm*min will usually permit the tests of wood members to be complete in the prescribed time*." [8]

"The load and the deflection at first failure shall be noted, at the maximum load and at points of sudden chances. Loading shall continue until complete failure or an arbitrary terminal load has been reached." [8]

Failure shall be documented in detail as to type, manner and order of occurrence and position in beam. The documentation shall be related to drawings or photographs of the beam. The beam shall be hold until an examination will be done and the analysis of the data has been completed.

Physical and mechanical properties and their appropriate adjustment for the beam shall be calculated in accordance with the following table.

Mechanical properties	**Two Point Loading**	**Third Point loading**	**Center Point Loading**
Fiber stress at proportional limit, S_f	$\frac{3P'a}{bh^2}$	$\frac{P'L}{bh^2}$	$\frac{3P'L}{2bh^2}$
Modulus of rupture, S_R	$\frac{3P_{max}a}{bh^2}$	$\frac{P_{max}L}{bh^2}$	$\frac{3P_{max}L}{2bh^2}$
Apparent modulus of elasticity, E_f	$\frac{Pa}{4bh^3\Delta}(3L^2 - 4a^2)$	$\frac{23PL^3}{108bh^3\Delta}$	$\frac{PL^3}{4bh^3\Delta}$
Modulus of elasticity, E (shear corrected using Δ)	$\frac{Pa(3L^2-4a^2)}{4bh^3\Delta\left(1-\frac{3Pa}{5bhG\Delta}\right)}$	$\frac{23PL^3}{108bh^3\Delta\left(1-\frac{PL}{4bhG\Delta}\right)}$	$\frac{PL^3}{4bh^3\Delta\left(1-\frac{3PL}{10bhG\Delta}\right)}$
Modulus of elasticity, E (shear corrected using Δ_{Lb})	$\frac{3PaL_b^2}{4bh^3\Delta_{Lb}}$	$\frac{PLL_b^2}{4bh^3\Delta_{Lb}}$	-
Ratio between deflection at the load point and deflection at the midspan, c_2	$\frac{4a(3L-4a)+\frac{12h^2E}{5G}}{3L^2-4a^2+\frac{12h^2E}{5G}}$	$\frac{\frac{20}{9}L^2+\frac{12h^2E}{5G}}{\frac{23}{9}L^2+\frac{12h^2E}{5G}}$	-
Work to proportional limit per unit volume, W_{PL}	$\frac{P\Delta c_2}{2Lbh}$	$\frac{P\Delta c_2}{2Lbh}$	$\frac{P\Delta}{2Lbh}$
Approximate work to maximum load per unit volume, W_{ML}	$\frac{A_{ML}c_1c_2}{Lbh}$	$\frac{A_{ML}c_1c_2}{Lbh}$	$\frac{A_{ML}c_1}{Lbh}$
Approximate total work per unit volume, W_{TL}	$\frac{A_{TL}c_1c_2}{Lbh}$	$\frac{A_{TL}c_1c_2}{Lbh}$	$\frac{A_{TL}c_1}{Lbh}$
Maximum shear stress, τ_{max}	$\frac{3P_{max}}{4bh}$	$\frac{3P_{max}}{4bh}$	$\frac{3P_{max}}{4bh}$

Table 20: Flexure formulas (ASTM D198 Annex X2 Flexure)

The parameters of the length (L), force (P) and distance (a) are given in figure 12. Further the width (b) of the panel and the high (h) of the cross section is to take from the according tested specimen.

In addition the test method for **shear stiffness** determines the coverage of the modulus of rigidity or shear modulus of structural beams made of solid or laminated wood. The measure out of the application to composite constructions gives the apparent or effective shear modulus away. Primarily the procedure is intended for beams of rectangular cross section. With an appropriate modification of the equation coefficients other sections are also applicable.

The test specimen is a straight or a slightly cambered beam of rectangular cross section. It is subjected to a bending moment and supported near the end of the beam. A single transversal load applies in the midway between the supports. A single observation of coordinate load and deflection is taken during a prescribed rate of the beam. At least four different spans shall be tested under this procedure.

Other than the mentioned facts the test set up is identical to the test method of flexure. The span of the specimen shall be choose approximately equal increments of $(h/L)^2$, within the range from 0.035 to 0.0025.

The shear modulus is proportional to the slope of the best-fit line between the plotted $1/E_f$ and $(h/L)^2$,determined in the test mehtod.

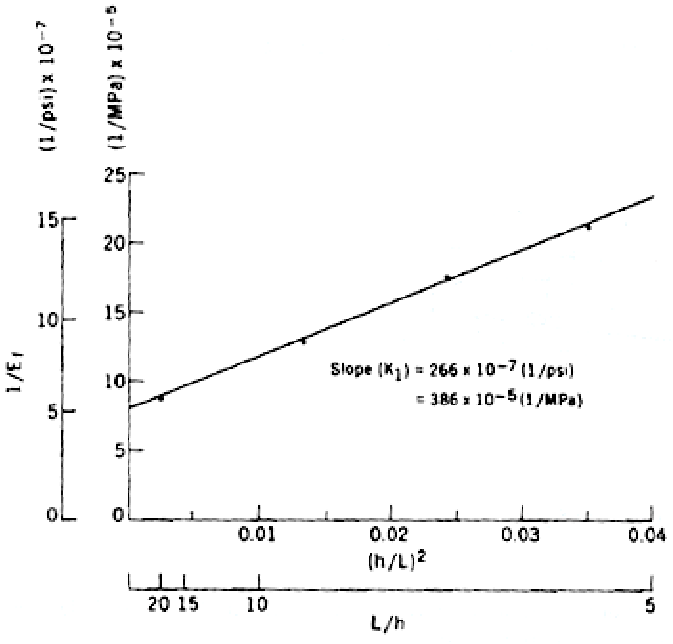


Figure 13: Determination of shear modulus according ASTM D198

Shear modulus formula

Apparent modulus of elasticity, E_f (center point loading):

$$E_f = \frac{PL^3}{48I\Delta} \qquad [4.1]$$

Shear modulus, G (rectangular section):

$$G = \frac{6}{5K_1} \qquad [4.2]$$

Shear modulus, G (circular section):

$$G = \frac{5}{6K_1} \qquad [4.3]$$

$$\frac{1}{E_f} = \frac{1}{E} + \frac{3}{4KG}\left({}^{h}/_{L}\right)^2 \qquad [4.4]$$

The factor K_1 is the slope of the line through multiple test data plotted on $(h/L)^2$ versus $(1/E_f)$ axes (see figure 13). The factor K is defined as shear coefficient. Further the shear modulus is given as G.

$$K_1 = \frac{3}{4KG} \qquad [4.5]$$

$K = \frac{5}{6}$ Rectangular section

$K = \frac{9}{10}$ Circular section

$G = \frac{6}{5K_1}$ Rectangular section

$G = \frac{5}{6K_1}$ Circular section

ASTM D1990

"Assume that it was desired to form a new species grouping from four separate species with allowable properties developed for several sizes and grades of nominal 2 in. (1.5 in. actual) thick dimension lumber. To adequately sample this matrix required sampling from at least two grades and three sizes of each grade. For this example, the grading system used was developed from the stress ratio concepts of Practice D 245. Specific grade descriptions are given in Refs (1, 2, 3, and 4). The sample matrix used consisted of Select Structural (65% bending strength ratio) and No. 2 (45% bending strength ratio) grades, of nominal 2 by 8 (1.5 by 7.25 in.) widths." [8]

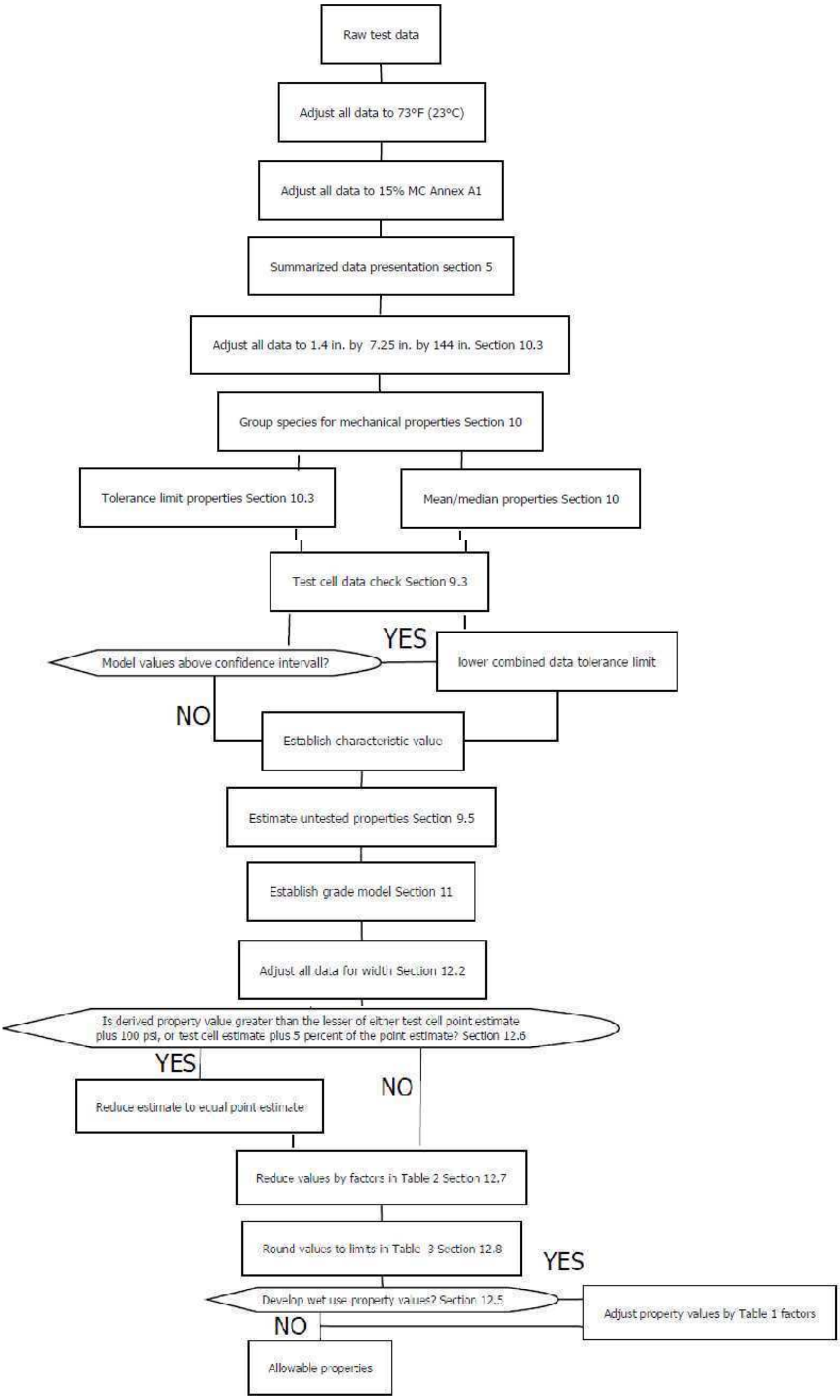


Figure 14: Flow diagram ASTM Standards (ASTM D1990)

Figure 14 demonstrates an example of allowable property development.

"It was intended to sample a minimum of approximately 200 pieces representative of the entire parent population en each size-grade test cell for each of the four species. The sampling plan chosen required taking a minimum of 10 pieces in a size/grade/species cell at a sampling site to provide additional data on small production lots. The sampling plan and availability of material in specific sizes resulted in actual sample sizes both above and below the target size. The samples were tested at the sites of production under ambient conditions in accordance with Test Methods D 4761. Tests were conducted for modulus of elasticity and modulus of rupture only." [8]

ASTM D2915: Statistical Methodology

In this section of the ASTM the statistical methodology for sampling and analysis procedures are described. Two general analyses are cited, the parametric and the nonparametric method. The nonparametric approach is more conservative than the parametric analysis because the nonparametric approach requires fewer assumptions. "*The parametric approach assumes a known distribution of the underlying population*." If this assumption is incorrect it may lead to an inaccurate result. "*Appropriate statistical test shall be employed to substantiate this choice along with measures with test adequacy*." [8]

ASTM D4761: Bending flat-wise third-point loading

Besides bending edge-wise the ASTM D4761 maintains bending flat-wise under center point loading and third-point loading. With the third-point loading strength and modulus of elasticity of stress-graded lumber and other wood based structural material in flat wise bending are determined on short span. The test specimen is simply supported. The loading effects on the wide face by two equal, concentrated forces spaced equidistant between the supports. The load shall affect at third-point. The test ends when failure occurs or a preselected load or deflection is reached.

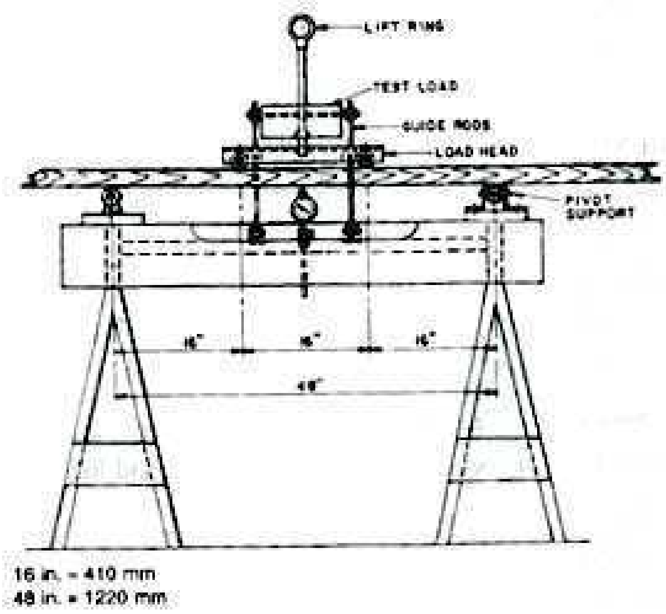


Figure 15: Schematic representation of static bending testing device

The testing machine combines three main parts*: "a reaction frame to support the specimen, a load mechanism for applying load at a specific rate or prescribed load interval and a force measuring apparatus that can be calibrated to the accuracy requirements*. *The standard span for this test is 32 times the depth for nominal 2 in. (38mm) lumber."[8]*

4.1.2 CLT test methods for "Producer A"

For a Canadian producer several CLT elements were tested and evaluated. In the following the company's name for which the certification was made will be called "Producer A". The source of the following information is from FPInnovations report for the concern company.

Bending moment capacity and stiffness (M_R and EI)

To evaluate mechanical properties of proprietary CLT panels a series of experiments was made. For "Producer A" ten specimens for each combination were tested. The apparent and true bending stiffness as well as the bending moment capacity were determined from bending tests. The ASTM Standard D198-09 (ASTM, 2011) was used.

The bending evaluation was performed with a span to depth ratio of about 30. The shear deformation was determined with different test spans. The load was applied at third-point on the top of the CLT panel using load bearing blocks.

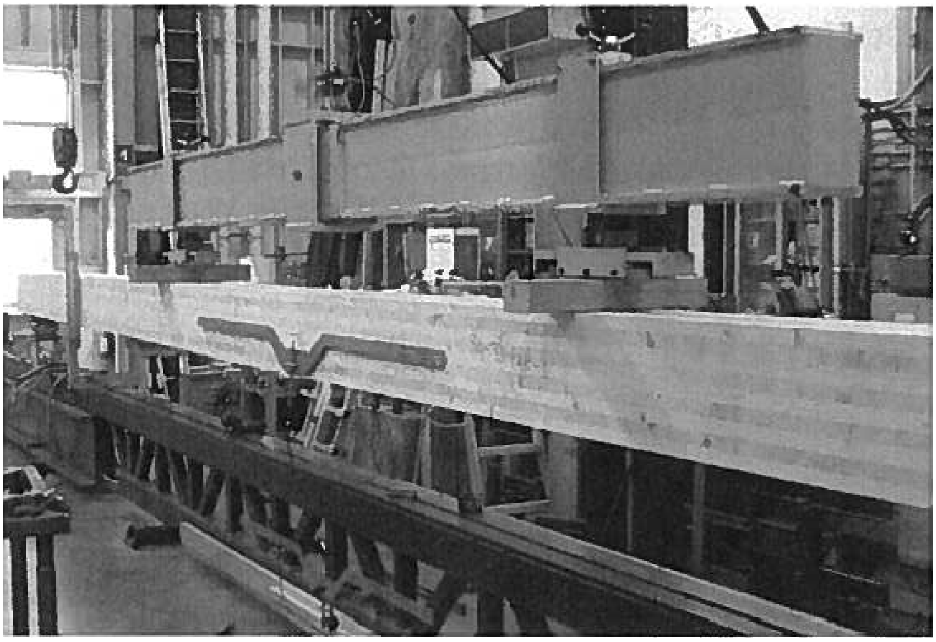


Figure 16: test set up and specimen with yokes at the neutral axis (source FPInnovations)

A way controlled procedure is used to determine the bending stiffness. First the true effective bending stiffness $(EI)_{eff(true)}$ will be determined by measuring the deflection under a constant displacement (8mm/min). The deflection will be measured by two gauges. The gauges were installed on yokes positioned at the neutral axis of the beam between the two loading points. For evaluating the apparent bending stiffness $(EI)_{app}$ load was applied until the failure of the specimen. First the electronic gauges (LVDTs) were removed and positioned at the bottom edge of the specimen at mid-span. Than the loading rate was increased to about 20 mm/min until failure occurred (5-10 min after the test has started). With this data the load-deflection graph can be plotted. The section between 0.1 F_{max} and 0.4 F_{max} is used for a linear regression analyses.

The supports and loading point allowed rotation of the beam. In addition the beam was mounted on ball bearings to allow translation. For acquisition the testing data the MTS Systems Corporation`s TestWorks™ program was used. In the failure zone of the tested specimens the moisture content and the specific gravity measurements were taken.

Apparent MOE (MOEapp) and Bending Moment Capacity (M_r)

The equations used for the calculation of MOEapp and M_r go as followed:

Moment of capacity (M_r):

$$M_r = \frac{Pa}{2} = \frac{PL}{2*3} = \frac{PL}{6} \quad [4.6]$$

Module of Elasticity:

$$MOE_{app} = \frac{PL^3}{4.7bh^3\Delta} \quad [4.7]$$

where:

P: Maximum load
L: Span
b: Width
h: Depth
Δ: Mid-span deflection (acquired from LVDTs)

True MOE evaluation

$$MOE_{true} = \frac{PLL_b^2}{4bh^3\Delta_{Lb}} \quad [4.8]$$

where:

P: Maximum load
L: Span
L_b: Yoke span
b: Width
h: Depth
Δ_{Lb}: Deflection of yoke's neutral axis measured at mid-span (acquired from LVDTs)

Horizontal Shear Strength

The horizontal shear strength (longitudinal shear strength or shear in plane) was also tested in principle with ASTM. For "Producer A" seventy CLT specimens were evaluated, which are 10 for each combination. A single point load at center span was applied at the top of the CLT panel. The panel width was determined with 305mm, which is equal to 1 foot. The span to depth ratio varied of about 6 to 8 in (15.24 to 20.32 mm). A constant displacement rate of 25 mm/min until failure occurred, loading was applied. After 2-5 minutes the test has started failure was detected.

Results

The geometric data of the examined cross sections are:

Layer	Thickness	Orientation	Material
1	25.5 mm	0	1950Fb-1.7E
2	27 mm	90	Spruce-Pine-Fir No.3
3	25.5 mm	0	1950Fb-1.7E

Width 305 mm Thickness 78 mm

Figure 17: 78-3s, 3 layer cross section

Layer	Thickness	Orientation	Material
1	35 mm	0	1950Fb-1.7E E90=0
2	35 mm	90	Spruce-Pine-Fir No.3
3	35 mm	0	1950Fb-1.7E E90=0
4	35 mm	90	Spruce-Pine-Fir No.3
5	35 mm	0	1950Fb-1.7E E90=0
6	35 mm	90	Spruce-Pine-Fir No.3
7	35 mm	0	1950Fb-1.7E E90=0

Width 305 mm Thickness 245 mm

Figure 18: 245-7s, 7 layer cross section

Layer	Thickness	Orientation	Material
1	35 mm	0	2250Fb-1.9E E90=0
2	35 mm	90	Spruce-Pine-Fir No.1/No.2
3	35 mm	0	2250Fb-1.9E E90=0
4	35 mm	90	Spruce-Pine-Fir No.1/No.2
5	35 mm	0	2250Fb-1.9E E90=0
6	35 mm	90	Spruce-Pine-Fir No.1/No.2
7	35 mm	0	2250Fb-1.9E E90=0
8	35 mm	90	Spruce-Pine-Fir No.1/No.2
9	35 mm	0	2250Fb-1.9E E90=0

Width 305 mm Thickness 315 mm

Figure 19: 314-9s, 9 layer cross section

Layer	Thickness	Orientation	Material
1	35 mm	0	2250Fb-1.9E E90=0
2	35 mm	0	2250Fb-1.9E E90=0
3	35 mm	90	Spruce-Pine-Fir No.1/No.2 E90=0
4	35 mm	0	2250Fb-1.9E E90=0
5	35 mm	90	Spruce-Pine-Fir No.1/No.2
6	35 mm	0	2250Fb-1.9E E90=0
7	35 mm	90	Spruce-Pine-Fir No.1/No.2 E90=0
8	35 mm	0	2250Fb-1.9E E90=0
9	35 mm	0	2250Fb-1.9E E90=0

Width 305 mm Thickness 315 mm

Figure 20: 314-9s, 9 layer cross section, first 2 layers longitudinal

The following graphic contains the results of the first testing phase for “Producer A”. In addition a comparison with calculated values was made. The calculation of the values, which will be compared to the results for “Producer

A", was made with the software tool Cross Section Analyser of the competence center holz.bau forschungs gmbh.

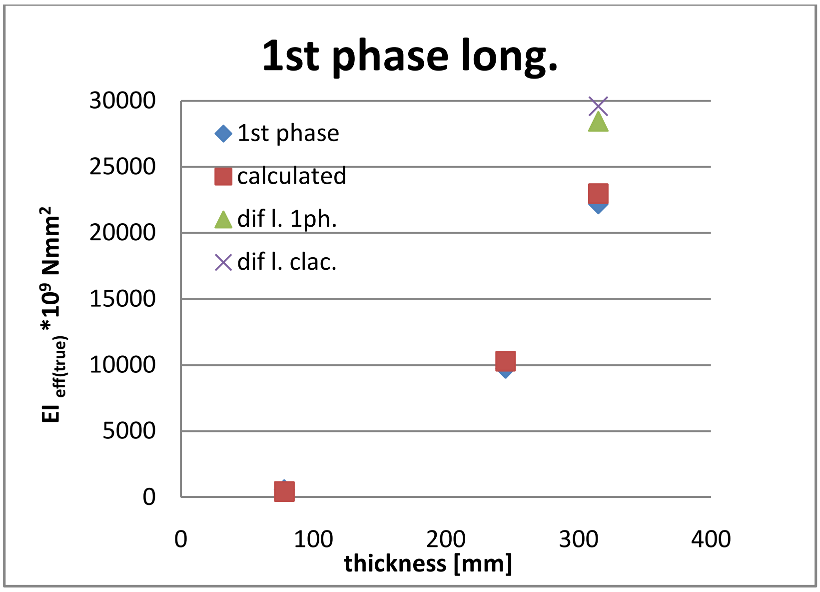


Diagram 1: True effective bending stiffness – longitudinal axis

Diagram 1 demonstrates the true bending stiffness in longitudinal axis in dependence to the thickness. The values for the measured and calculated cross sections are nearly the same (see table 21). The values for the cross section with two parallel layers on the outside (dif l. 1ph./calc) shows a higher (EI). The calculated values for $(EI)_{eff(true)}$ are higher as the measured values of the tested elements with one exception. The calculated $(EI)_{eff(true)}$ of the 3 layer panel is lower.

thickness	**$(EI)_{eff(true)}$ long.**	**(EI) long.**
mm	$x10^9$ N mm^2	$x10^9$ N mm^2
	1st phase	calculated
78	562.00	443.99
245	9748.00	10306.08
315	22187.00	22976.00
315	28431.00	29599.00

Table 21: True effective bending stiffness in longitudinal direction

The values of $(EI)_{eff(true)}$ shows good agreements. However the differences are created by calculation parameters which are not in accordance with the reality. This would mean that the used material has lower capacities as given in the certification.

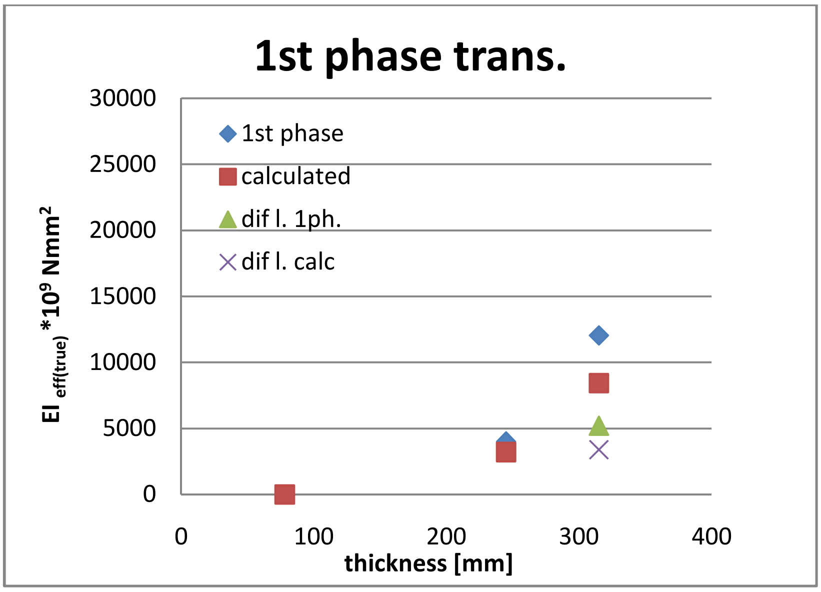


Diagram 2: True effective bending stiffness – transversal axis

Diagram 2 demonstrates the true bending stiffness in transversal axis. All calculated values are lower than the values of the experiment. Cross sections with a higher number on layers show a greater deviation as the other cross sections. The cross section with the different assembling (two parallel layers on the outside) has a lower (EI) in transversal direction.

thickness	$(EI)_{eff(true)}$ trans.	(EI) trans.
mm	$x10^9$ N mm^2	$x10^9$ N mm^2
	1st phase	calculated
78		14.76
245	3998.00	3219.70
315	12057.00	8436.00
315	5224.00	3401.00

Table 22: True effective bending stiffness in transversal direction

Also in table 22 occurs a difference which arises discrepancies.

4.2 European Standard

ON EN 789 [9]: bending properties

To test the **bending properties** the ON EN 789 2005 04 underlies the expirements.

The test piece is defined by a rectangular cross section. The width shall be 300±5 mm. The length of the specimen is (see figure 21) depending on the nominal thickness of the panel.

The load equipment shall be documented on 1% of the measurement. Appropriate loading equipment is required.

The following figure expresses the application of how the load shall be induced on the panel. The supports, positioned near the end of the panel, and the load device shall be applied by rollers with a diameter of 30±1 mm. The distance between the supports and the load point depend on the nominal thickness (16 x t). The distance l_2, according figure 21, shall not exceed 400±1 mm nor be less than 240±1 mm.

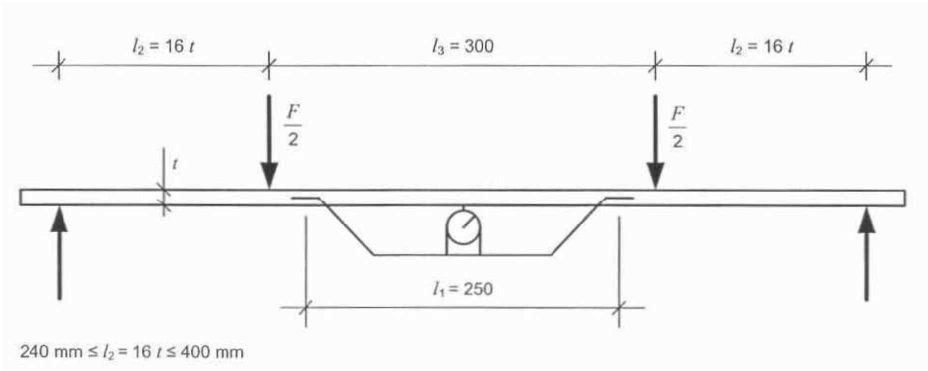


Figure 21: Arrangement for bending test (sizes in millimetre) [9]

The test procedure occurs under a continuous rate of loading until the maximum load is reached within 300±120 sec. The mean value of 300 sec is aimed.

The measurement of the deflection shall be taken on both sides midway between two points (not less than 250 mm apart) on the axis of the test specimen.

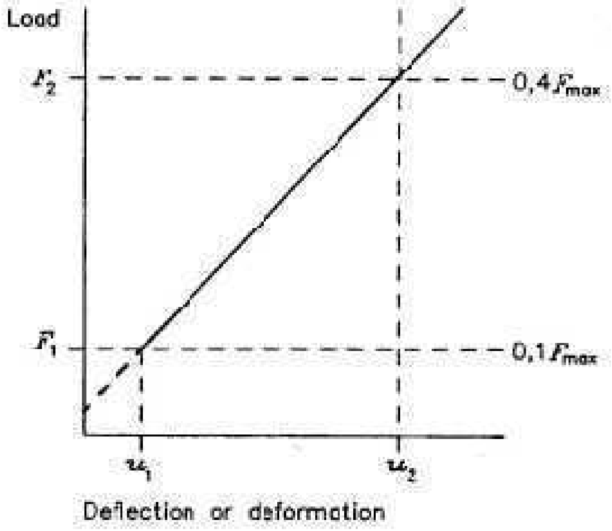


Figure 22: Load-deflection or deformation graph within the range of elastic deformation [9]

The elastic modulus of the tested specimen is the arithmetical mean of all elastic modulus of all test specimens for one category of test specimen.

The expression of the test result for bending is defined by the following formula:

- Modulus of elasticity:

$$E_m = \frac{(F_2 - F_1) l_1^2 l_2}{16 (u_2 - u_1) I} \qquad [4.9]$$

where:

F_2-F_1... increment of the load between 0.1 F_{max} and 0.4 F_{max} (figure 22) [N]
u_2-u_1... increment of the deflection corresponding to F_2-F_1 using figure 22

- Bending strength:

$$f_m = \frac{F_{max} l_2}{2W} \qquad [4.10]$$

- Moment capacity:

$$M_{max} = \frac{F_{max} l_2}{2} \qquad [4.11]$$

The following equations express the deflection of bending for a rectangular cross section according the arrangement of figure 21 when $l_2=l_3=l_{beam}/3$.

Deflection caused by bending:

$$w_B = \frac{23}{216} * \frac{M_{max}}{EI} * l^2 = \frac{23}{36} * \frac{F*a*l^2}{E*b*h^2} \quad [4.12]$$

Deflection caused by shear:

$$w_S = \frac{1}{3} * \frac{Q_{max}}{\frac{GA}{\kappa}} * l = \frac{F*l*\kappa}{6*G*b*h} \quad [4.13]$$

The entire deflection of the beam:

$$w_{ges} = w_B + w_S \quad [4.14]$$

The advantage of the four point test configuration is the shear force free section in the middle of the beam. This section gives the bending stiffness away. With the bending stiffness known, the shear stiffness can be calculated approximately.

A disadvantage of this procedure might be the high variance in the shear stiffness if the bending stiffness is subjected to dispersion.

To reduce the dispersion, a three point test configuration with variable span shall be investigated. The average of these measurements might reduce the errors. With the deflection in the middle of the beam an apparent E-modulus is available.

$$w_m = \frac{F*l^3}{48*E_{app}*\frac{b*h^3}{12}} \quad [4.15]$$

$$E_{app} = \left(4 * \frac{w_m}{F} * \frac{b*h^3}{l^3}\right)^{-1} \quad [4.16]$$

$$w_m = w_B + w_S = \frac{F*l^3}{4*E*b*h^3} + \frac{F*l}{4*G*\frac{b*h}{\kappa}} \quad [4.17]$$

$$\frac{1}{E_{app}} = \frac{1}{E} + \frac{\kappa}{G} * \left(\frac{h}{l}\right)^2 \quad [4.18]$$

Determination of shear modulus (prEN 408 page 13)

ON EN 789 [9]: shear properties

Further test method for the shear properties will be cited. It has to be distinguished shear rectangular and shear parallel to the plate. The Eurocode defines shear rectangular to the plate as panel shear and shear parallel to the plate as planar shear.

The **panel shear properties** shall be tested by loaded surfaces, which are smooth and parallel to each other and at right angles to the test piece length.

The next figure shows the specimen and the test set up according the ON EN 789:

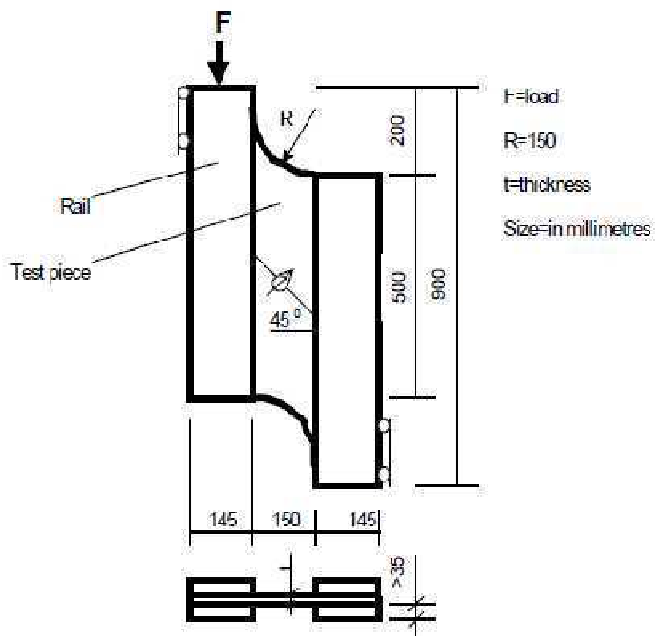


Figure 23: The specimen and test set up for panel shear test, according [9] Annex B

The shear modulus of rigidity G shall be determined by measuring the deflection with two gauges attached to both sides of the test piece, parallel to each other. The compression diagonal at 45° to the rails, which passes the center of the shear area, lays down the gauge length along the measured deformation.

The loading method occurs evenly over the top surface of the uppermost rail as single force. The load applies along the longitudinal axis of the test specimen, parallel to the rails. The next figure demonstrates an appropriate apparatus.

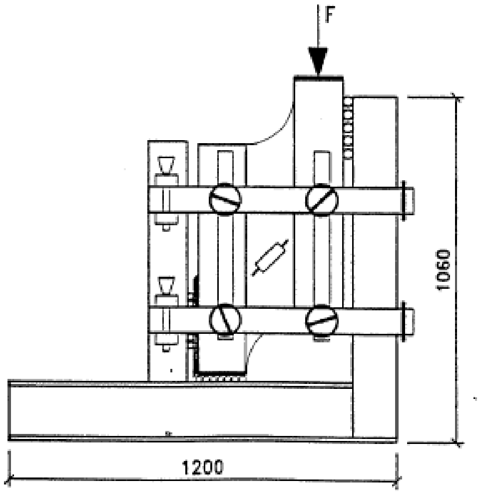


Figure 24: Loading arrangement for panel shear test (sizes in mm)

The test procedure occurs under a continuous rate of loading until the maximum load is reached within 300±120 sec. The mean value of 300 sec is aimed.

The measurement of deformation is determined as the average of the measurement taken on each side of the test piece. The measuring devices are attached to the test specimen. The deformation shall be documented to the nearest 0.005 mm.

The report shall document the failure mode of the specimen, the failure of any piece in another manner than by panel shear on the face of the specimen between the rails.

With the test data the load-deformation graph is available. The sections between 0.1 F_{max} and 0.4 F_{max} are used for a linear regression analyses.

The panel shear modulus of rigidity is expressed as:

$$G = \frac{0.5(F_2 - F_1)l_1}{(u_2 - u_1)lt} \qquad [4.19]$$

where:

F_2-F_1... increment of the load between 0.1 F_{max} and 0.4 F_{max} (figure 22)
u_2-u_1... increment of the deflection corresponding to F_2-F_1 using fig.22
u_2 *and* u_1... means of the deforamtion measured on both faces
l_1... the gauge length
l... the length of the specimen along the centre line of the shear area
t... the average thickness of the specimen

The panel shear strength can be expressed as:

$$f = \frac{F_{max}}{lt} \qquad [4.20]$$

Where F_{max} is the maximal load applied up to failure.

The **planar shear properties** shall be tested by applying a projecting end of one of the steel plates, parallel to the test piece length direction. During testing a test rig for holding the test piece is necessary. The next figure demonstrates the principle of loading.

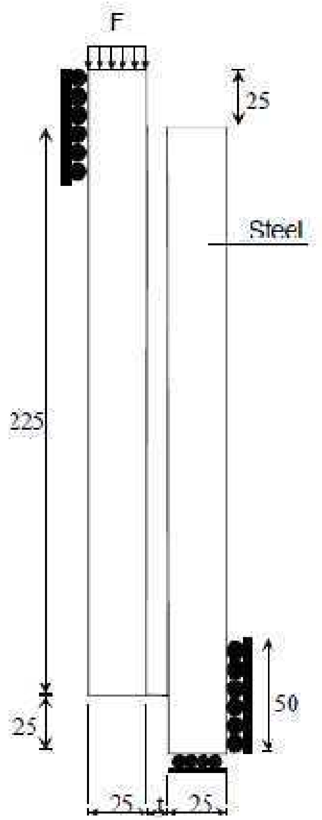


Figure 25: Loading arrangement for planar shear

The test procedure occurs under a continuous rate of loading until the maximum load is reached within 300±120 sec. The mean value of 300 sec is aimed.

The planar shear modulus of rigidity is determined by the data from load-deformation curves. The linear displacement of one of the two steel plates to the other stands for the deformation. Two dial gauges or linear transducers shall measure the deformations. One gauge is placed on each side of the test specimen in order to compensate for inaccuracy in gluing or loading. The deformation shall be documented to the nearest 0.002 mm.

Abnormally test results can occur because of different manners of failure within the thickness of the specimen. Any failure shall be reported separately.

The planar shear strength shall be expressed as:

$$f_r = \frac{F_{max}}{lb} \quad [4.21]$$

where:

F_{max}... maximum load obtained during testing
b... width of the test specimen
l... length of the test specimen

With the test data the load-deformation graph is available. The sections between 0.1 F_{max} and 0.4 F_{max} is used for a linear regression analyses.

The planar shear modulus of rigidity shall be expressed as:

$$G_r = \frac{(F_2 - F_1)t}{(u_2 - u_1)lb} \quad [4.22]$$

where:

t... panel thickness of test specimen
F_2-F_1... increment of the load between 0.1 F_{max} and 0.4 F_{max} (figure 22)
u_2-u_1... increment of the deflection corresponding to F_2-F_1 using fig.22
u_2 *and* u_1... means of the deforamtion measured on both faces
b... width of the test specimen
l... length of the test specimen

5 CHAPTER

5. COMPARISON

In this chapter several comparisons of the strength and stiffness parameters and the proposed design methods will be demonstrated. First the parameter study will be done on a single span beam. After that the parameters and design guidelines will be compared at a continuous beam.

Also a comparison of the material parameters will be done. A comparative calculation of $(EI)_{eff}$ will be cited.

Note: In the following examples the rolling shear strength will be called f_s and the rolling shear stress τ_{Vs} for the Canadian and the European parameters. This shall prevent from mixing the resistance capacities (M_r and V_r) with the shear parameters (European Index "r").

5.1 Example: 5 layer single span beam

5.1.1 Comparative view on the "Simplified Method" and the Shear Analogy Method

As first example a simple span beam with a 5-layer panel was chosen. Dead loads and live loads have an effect on the system. With this system the Shear Analogy Method will be scrutinized. More exactly declared the "Simplified Method" of the Canadian CLT-Handbook will be compared with the calculation according the Shear Analogy Method with the CLTdesigner. The European values of strength and stiffness are taken.

5.1.1.1 First part: calculation and comparison of bending stress and shear stress of each method

Before the verification of bending and shear can be compared, the bending stress, the shear stress and the rolling shear stress will be observed.

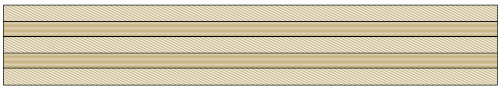


Figure 26: CLT Cross section (CLTdesigner)

$t_i \rightarrow$ 34/30/34/30/34
$t_{clt} = 162mm$
$b = 1000mm$

- Material: C24
- Service Class 1
- Office areas
- $g_k = 2.1 \, {}^{kN}/_{m^2}$
- $q_k = 3.00 \, {}^{kN}/_{m^2}$
- $L = 4800mm$

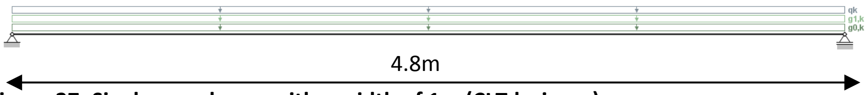


Figure 27: Single span beam with a width of 1m (CLTdesigner)

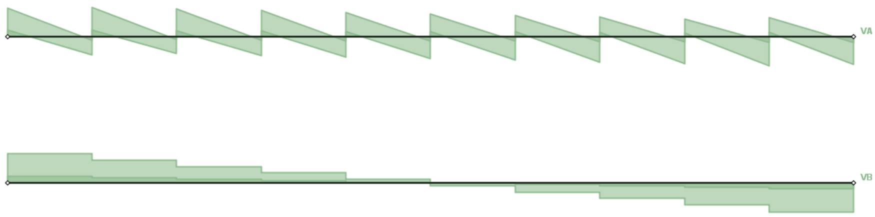
<table>
<tr><th>“Simplified Method”</th><th>CLTdesigner: Shear Analogy Method</th></tr>
<tr><td colspan="2">Predominant load combination:</td></tr>
<tr><td colspan="2">$E_d = \gamma_G * g_{k,ceiling} + \gamma_Q * q_k$</td></tr>
<tr><td colspan="2">

Figure 28: Internal forces for shear force, beam A and beam B (CLTdesigner)</td></tr>
<tr><td>According beam theory:
$V_{max} = \frac{q * l}{2}$
$= \frac{(1.35 * 2.1 + 1.50 * 3.0) * 4.8}{2}$
$= 17.1kN$</td><td>According CLTdesigner:
$V_{max,A} = 2.13kN$
$V_{max,B} = 15.45kN$</td></tr>
</table>

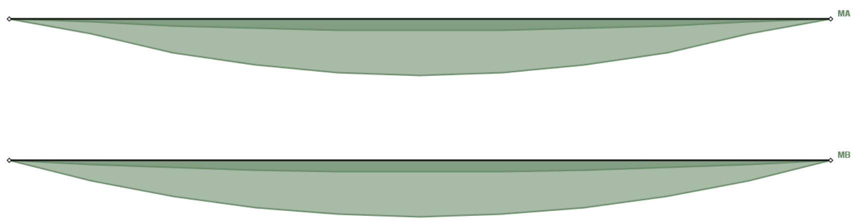


Figure 29: Internal forces for moment, beam A and beam B (CLTdesigner)

According beam theory:	According CLTdesigner:
$M_{max} = \frac{q * l^2}{8}$ $= \frac{(1.35 * 0.89 + 1.50 * 3.0) * 4.8^2}{8}$ $= 20.52kNm$	$M_{max,A} = 0.65kNm$ $M_{max,B} = 20.44kNm$

Module of elasticity

$$E_{0,mean} = 12000\, {}^{N}/_{mm^2}$$
$$G_{0,mean} = 690\, {}^{N}/_{mm^2}$$
$$G_{90,mean} = 50\, {}^{N}/_{mm^2}$$

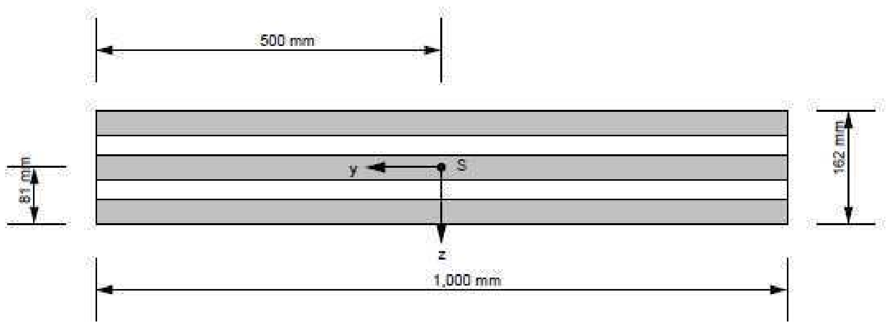


Figure 30: Cross section 5-layer panel C24

$$(EI)_{yy,A} = 1.196 * 10^{11}\text{Nmm}^2$$
$$(EI)_{yy,B} = 3.365 * 10^{12}\text{Nmm}^2$$
$$(EI)_{\text{eff}} = \text{K}_{\text{clt}} = 3.485 * 10^{12}\text{Nmm}^2$$

$$(GA)_{\text{z},A} = \infty$$
$$(GA)_{\text{z},B} = 1.26 * 10^{7}\text{N}$$

Bending stress

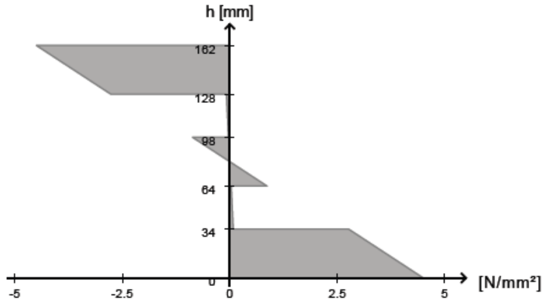


Figure 31: Bending stress (CLTdesigner)

$\boldsymbol{I_{eff} = \frac{(EI)_{eff}}{E} = \frac{3.485 * 10^{12}}{12000} = 2.90 * 10^8 mm^4}$ $\boldsymbol{\sigma_{max} = \frac{M_{max}}{I_{eff}} * 0.5 * h_{tot} = \frac{20.52 * 10^6}{2.9 * 10^8} * 0.5 * 162 = 5.73 \frac{N}{mm^2}}$	$\boldsymbol{\sigma_{max,A} = \frac{M_{max}}{B_A} * E_i * z_i}$ $\sigma_{max,B} = \frac{M_{max}}{B_A} * E_i * z_i$ $\sigma_{max} = \sigma_{max,A} + \sigma_{max,B} = \mathbf{5.78} \frac{N}{mm^2}$

Shear stress

$\boldsymbol{\tau_{max} = \frac{3 * V_{max}}{2} * \frac{I_{gross}}{I_{eff} * A_{gross}} = \frac{3 * 17.1 * 10^3 * 3.54 * 10^8}{2 * 2.9 * 10^8 * 162 * 1000} = 0.19 \frac{N}{mm^2}}$	$\boldsymbol{\tau_{max,A} = \frac{V_{maxA,d} * E_i * t_i^2}{8 * B_A}}$ $\tau_{max,B} = \frac{V_{max,B,d} * E_i * b_i * e_{s,i}}{B_B}$ $\tau_{max} = \tau_{max,A} + \tau_{max,B} = \mathbf{0.12} \frac{N}{mm^2}$

Rolling shear stress

$\tau_{s,max} = \frac{3 * V_{max}}{2} * \frac{I_{gross}}{I_{eff} * A_{gross}} = \frac{3 * 17.1 * 10^3 * 3.54 * 10^8}{2 * 2.9 * 10^8 * 162 * 1000} = \mathbf{0.19} \frac{N}{mm^2}$	$\boldsymbol{\tau_{s,max,A} = \frac{V_{maxA,d} * E_i * t_i^2}{8 * B_A}}$ $\tau_{s,max,B} = \frac{V_{max,B,d} * E_i * b_i * e_{s,i}}{B_B}$ $\boldsymbol{\tau_{s,max}} = \tau_{s,max,A} + \tau_{s,max,B} = \mathbf{0.12} \frac{N}{mm^2}$

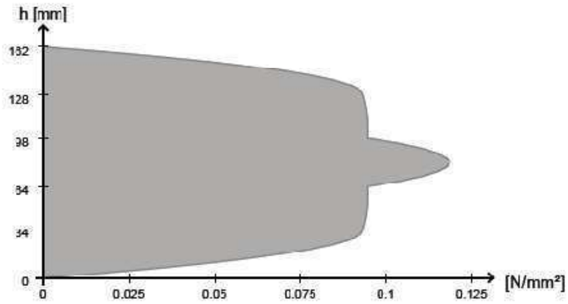


Figure 32: Shear stress (CLTdesigner)

From the calculation is to recognize that the bending and shear stress shows good agreements. In general the European bending stress is a little higher because of the higher bending moment. In comparison the shear stress is calculated a little lower due to the use of the gross cross section in the Canadian “Simplified Method”.

5.1.1.2 Second part: Calculation and comparison of “Simplified Method” and Shear Analogy Method

Further the verification of bending, shear and rolling shear for ULS and the verification of deflection and vibration for SLS will be cited. The boundary conditions and material parameters are equal to 5.1.1.1.

Verification	
Combination factors:	
$k_{mod} = 0.8$ $\gamma_G = 1.35$ $\gamma_Q = 1.50$	
Value of strength	
$f_{m,clt,k} = 24\,{}^{N}/_{mm^2}$ $f_{c,clt,90,k} = 2.7\,{}^{N}/_{mm^2}$ $f_{v,clt,k} = 2.7\,{}^{N}/_{mm^2}$ $f_{s,clt,k} = 1.5\,{}^{N}/_{mm^2}$*) $\gamma_M = 1.25\,{}^{N}/_{mm^2}$	$f_{m,clt,d} = \frac{1.1*0.8*24}{1.25} = 16.90\,{}^{N}/_{mm^2}$ $f_{c,clt,90,d} = \frac{0.8*2.7}{1.25} = 1.73\,{}^{N}/_{mm^2}$ $f_{v,clt,d} = \frac{0.8*2.7}{1.25} = 1.73\,{}^{N}/_{mm^2}$ $f_{s,clt,d} = \frac{0.8*1.5}{1.25} = 0.96\,{}^{N}/_{mm^2}$*)
f_s represents f_r (see note page 54)	

Bending	
$F_b = f_b(K_D K_H K_{Sb} K_T)$ $= 24 * (1 * 1 * 1 * 1)$ $= 24\, {}^{N}/_{mm^2}$ $M_r = \phi * F_b * \frac{I_{eff}}{0.5h_{tot}}$ $= 0.9 * 24$ $* \frac{2.90 * 10^8}{0.5 * 162}$ $= 77.33kNm$ $M_r \geq M_{max}$ $77.33kNm \geq 20.52kNm$ $\boldsymbol{\eta_M = 26.53.\%}$	$\frac{\sigma_{edge,d}}{f_{m,clt,d}} = \frac{5.78}{16.9} = 0.322 \leq 1.0$ $\boldsymbol{\eta_M = 32.2\%}$
Shear	
$c = \frac{I_{eff}}{I_{gross}} = \frac{2.90 * 10^8}{3.54 * 10^8} = 0.8$ $F_v = f_v(K_D K_H K_{Sv} K_T)$ $= 2.7 * (1 * 1 * 1 * 1)$ $= 2.7\, {}^{N}/_{mm^2}$ $V_r = \phi F_v \frac{2A_{gross}}{3}\left(\frac{I_{eff}}{I_{gross}}\right)$ $= 0.9 * 2.7 * \frac{2}{3}$ $* (1000 * 162) * 0.8$ $= 210.0kN$ $V_r \geq V_{max}$ $210.0kN \geq 17.1kN$ $\boldsymbol{\eta_V = 8.14\%}$	$\frac{\tau_{v,max,d}}{f_{v,clt,d}} = \frac{0.12}{1.72} = 0.07 \leq 1.0$ $\boldsymbol{\eta_V = 7.00\%}$
Rolling shear	

$$V_{s,r} = \phi F_{s,v} \frac{2A_{gross}}{3} \left(\frac{I_{eff}}{I_{gross}} \right) = 0.9 * 1.5 * \frac{2}{3} * (1000 * 162) * 0.8 = 116.64kN$$

$$V_{s,r} \geq V_{max}$$
$$116.64kN \geq 17.1kN$$

$$\boldsymbol{\eta_{V,s} = 14.66\%}$$

$$\frac{\tau_{s,max,d}}{f_{s,clt,d}} = \frac{0.12}{0.96} = 0.1271 \leq 1.0$$

$$\boldsymbol{\eta_{V,s} = 12.71\%}$$

Deflection

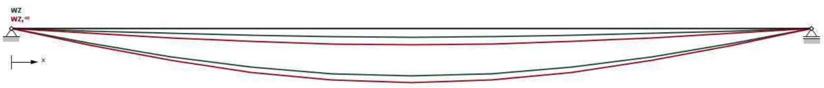


Figure 33: Deflection $w_{net,fin}$ (CLTdesigner)

$$\varphi_i = 1.0$$
$$I_D = 1.0$$
$$I_L = 1.0$$
$$k_{def} = 0.85$$

$$E_d = \varphi_D * I_D * D + \varphi_L * I_L * L$$
$$q_D = 1.0 * (1.0 * 2.1) = 2.1\, {}^{kN}/_{m}$$
$$q_L = 1.0 * (1.0 * 3.0) = 3.0\, {}^{kN}/_{m}$$

$$u_{max} = \frac{5}{384} * \frac{qL^4}{(EI)_{eff}} + \frac{1}{8} * \frac{qL^2k}{(GA)_{eff}}$$

$$\boldsymbol{u_{max,D}} = \frac{5}{384} * \frac{2.1 * 4800^4}{3.485 * 10^{12}} + \frac{1}{8} * \frac{2.1 * 4800^2 * 1.2}{1.26 * 10^7} = \boldsymbol{4.74mm}$$

$$\gamma_G = \gamma_Q = 1.0$$
$$k_{def} = 0.85$$

$\boldsymbol{u_{max,L}} = \frac{5}{384} * \frac{3.0 * 4800^4}{3.485 * 10^{12}} + \frac{1}{8} * \frac{3.0 * 4800^2 * 1.2}{1.26 * 10^7} = \mathbf{6.77}mm$ $u_{max,L} < \frac{L}{360} = \frac{4800}{360} = 13.33$ $\eta_w = 50{,}78\%$ $\boldsymbol{u_{max,tot}} = u_{max,D} + u_{max,L} = 4.74 + 6.77 = \mathbf{11.51}mm$ For verification the European limit state for Instantaneous deflection was chosen to non manipulate the results: $u_{max,tot} < \frac{L}{300} = \frac{4800}{300} = 16$ $\boldsymbol{\eta_w} = \mathbf{71{,}93}\%$ Creep: according Option II (CLT-Handbook) $k_{def} = 0.85$ Because of comparison of the method, European values were taken. Also the limits of TUGraz were verified. $u_{fin,P} = u_{inst,P}(1 + k_{def}) =$ $4.74(1 + 0.85) = 8.77mm$ $u_{fin,Q,1} = u_{inst,Q,1}(1 + \psi_{2.1}k_{def}) =$ $6.77(1 + 0.3 * 0.85) = 8.5mm$ $u_{fin} = u_{fin,P} + u_{fin,Q} \leq \frac{L}{150}$ $\boldsymbol{u_{fin}} = u_{fin,P} + u_{fin,Q} = \mathbf{17.27}mm \leq \frac{L}{150} = \frac{4800}{150} = 32$	EN 1995-1-1: deflection $\boldsymbol{w_{max}} = \mathbf{11.9mm}$ $w_{net,fin} = w_{fin} - w_c \leq \frac{L}{250}$ $\boldsymbol{\eta_w} = \mathbf{72.34.}\%$

$\eta_w = 55.01\%$	
Vibration	
$l \leq \frac{1}{9.15} \frac{(EI_{eff}^{1m})^{0.293}}{(\rho A)^{0,123}}$ $l = \frac{1}{9.15} \frac{(3.485 * 10^6)^{0.293}}{(450 * 0.162 * 1.0)^{0.123}} = 5.33\text{m}$ $l = 5.33m > l_{exist} = 4.8m$	EN 1995-1-1: vibration $f_1 = 9.0Hz \geq 8Hz$ $w_{1kN} = 0.7mm \leq 1mm$ $v = 3.7\,{}^{mm}/_{s} \leq 13.8\,{}^{mm}/_{s}$ Vibration fulfilled!

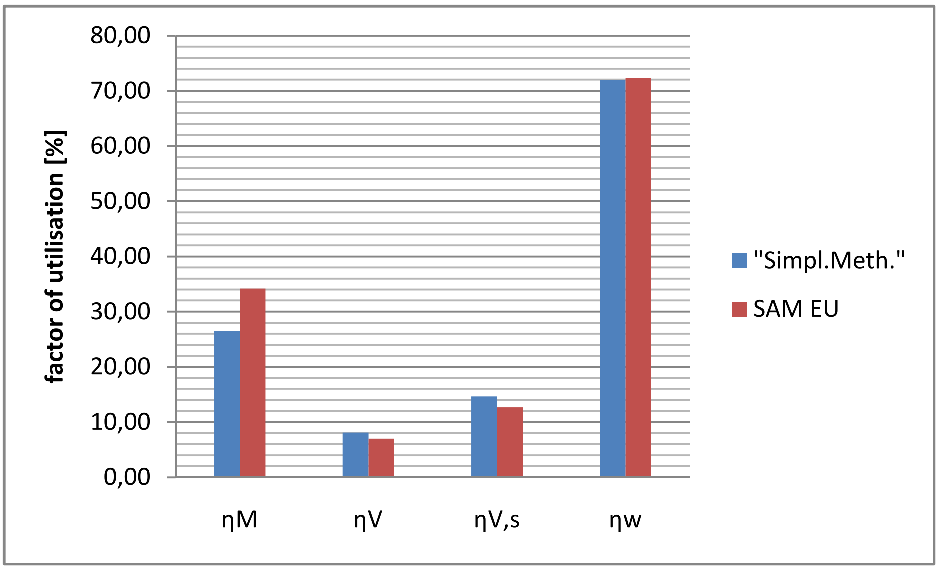


Diagram 3: Comparison of the results from the calculation according the „Simplified Method" and the Shear Analogy Method

The significant verification of the simple beam is the verification of vibration. From diagram 3 is to recognize that the factor of utilisation for bending is higher according the calculation with the CLTdesigner. This may be due the higher calculated bending moment resistance, because of the resistance factor ϕ. The European procedure using the design values demonstrates a more critical verification.

The shear stress has been calculated higher (use of A_{gross}) as well as the shear resistance. It results a quite good agreement for shear.

The result of the Canadian calculation of the deflection is lower than the European approach. This may occur due to the higher shear stiffness. Also a different load combination is significant for the verification.

The verification of vibration in the CLTdesigner is fulfilled for the guidelines by DIN 1052, EN 1995-1-1 and ON B 1995-1-1/NA 2009-07. According the proposed method by Hamm/Richter the verification is not fulfilled because of lower limits for the stiffness criterion. The verification of vibration for the "Simplified Method" is below the limit value.

5.1.2 Comparative calculation with the "Simplified Method" and the model "BSP Graz" (Timoshenko)

In addition a comparison of the "Simplified Method" with the model "BSP Graz" is made. The calculation of the model "BSP Graz" underlies the theory of Timoshenko because of the length to high relation L/H=29.6. The same boundary conditions are available: a simple span beam with a 5-layer panel. The material C24 is chosen.

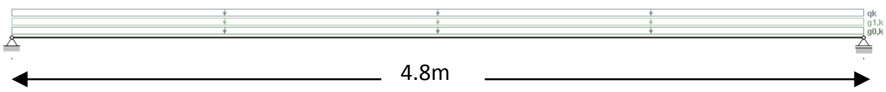


Figure 34: Single span beam with a width of 1m (CLTdesigner)

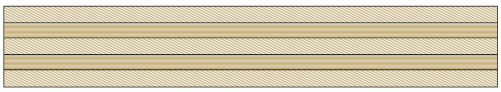
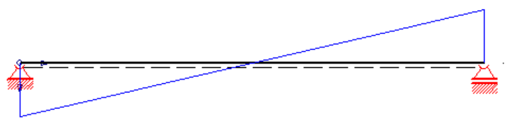
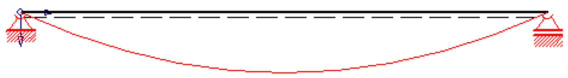


Figure 35: CLT Cross section (CLTdesigner)

t_i → 34/30/34/30/34
$t_{clt} = 162mm$
$b = 1000mm$

- Material: C24
- Service Class 1
- Office areas
- $g_{i,k} = 2.1\, {}^{kN}/_{m^2}$
- $q_k = 3.00\, {}^{kN}/_{m^2}$
- $L = 4800mm$

"Simplified Method"	**Model "BSP-Graz" (Timoshenko)**
Combination factors:	
$\varphi_i = 1.0$ $I_D = 1.25$ $I_L = 1.50$	$k_{mod} = 0.8$ $\gamma_G = 1.35$ $\gamma_Q = 1.50$
Predominant load combination:	
$E_d = \gamma_G * g_{k,ceiling} + \gamma_Q * q_k$	$E_d = \gamma_G * g_{k,ceiling} + \gamma_Q * q_k$
 **Figure 36: Internal forces for shear force (RuckZuck)**	

$V_{max} = \frac{q*l}{2} = \frac{(1.25*2.1+1.50*3.0)*4.8}{2} = 17.1N$	$V_{max} = \frac{q*l}{2} = \frac{(1.35*2.1+1.50*3.0)*4.8}{2} = 17.60kN$
 **Figure 37: Internal forces for moment (RuckZuck)**	
$M_{max} = \frac{q*l^2}{8} = \frac{(1.25*2.1+1.50*3.0)*4.8^2}{8} = 20.52kNm$	$M_{max} = \frac{q*l^2}{8} = \frac{(1.35*2.1+1.50*3.0)*4.8^2}{8} = 21.12kNm$
Value of strength	
$f_b = 24\,{}^{N}/_{mm^2}$ $f_{c,90} = 2.7\,{}^{N}/_{mm^2}$ $f_v = 2.7\,{}^{N}/_{mm^2}$ $f_s = 1.5\,{}^{N}/_{mm^2}$	$f_{m,clt,d} = k_l * k_{mod}\frac{f_b}{\gamma_M} = 1.1 * 0.8 * \frac{24}{1.25} = 16.9\,{}^{N}/_{mm^2}$ $f_{c,clt,90,d} = k_{mod}\frac{f_{c,clt,90}}{\gamma_M} = 0.8 * \frac{2.7}{1.25} = 1.73\,{}^{N}/_{mm^2}$ $f_{v,clt,d} = k_{mod}\frac{f_v}{\gamma_M} = 0.8 * \frac{2.7}{1.25} = 1.73\,{}^{N}/_{mm^2}$ $f_{s,clt,d} = k_{mod}\frac{f_s}{\gamma_M} = 0.8 * \frac{1.5}{1.25} = 0.96\,{}^{N}/_{mm^2}$*) *) $f_{s,clt,d}$ stands for $f_{r,clt,d}$
Module of elasticity	
$E_{0,mean} = 12000\,{}^{N}/_{mm^2}$ $G_{0,mean} = 690\,{}^{N}/_{mm^2}$ $G_{90,mean} = 50\,{}^{N}/_{mm^2}$	

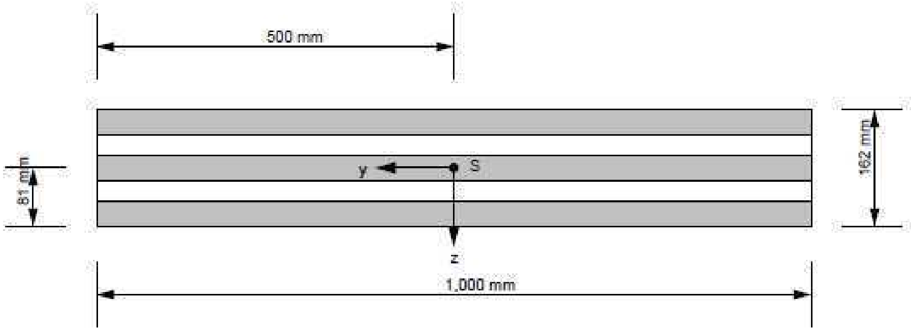


Figure 38: Cross section 5-layer panel C24

$$(EI)_{eff} = B_A + B_B$$

$$(EI)_{eff} = 1.196E11 + 3.365E12 = 3.485 * 10^{12} Nmm^2$$

$$(GA)_{eff} = \frac{a^2}{\left[\frac{h_1}{2*G_1*b_1} + \sum_{i=2}^{n-1}\frac{h_i}{G_i*b_i} + \frac{h_n}{2*G_n*b_n}\right]} = \frac{128^2}{\left[\frac{34}{2*690*10^3} + \left(\frac{2*30}{50*10^3} + \frac{34}{2*690*10^3}\right) + \frac{34}{2*690*10^3}\right]} = 1.286 * 10^7 N$$

$$K_{clt} = \sum(I_i * E_i) + \sum(A_i * e_i^2 * E_i)$$

$$K_{clt} = 3 * 1000 * \frac{34^3}{12} * 12000 + 2 * 1000 * 34 * 64^2 * 12000 = 3.485 * 10^{12} Nmm^2$$

$$S_{ges} = \sum(G_i * b_i * t_i)$$

$$S_{ges} = 3 * 69 * 10000 * 34 + 2 * 50 * 1000 * 30 = 7.34 * 10^7 N$$

$$\kappa = 0.257$$

$$S_{clt} = S_{ges} * \kappa$$

$$S_{clt} = 7.34 * 10^7 * 0.257 = 1.90 * 10^7 N$$

Bending stress

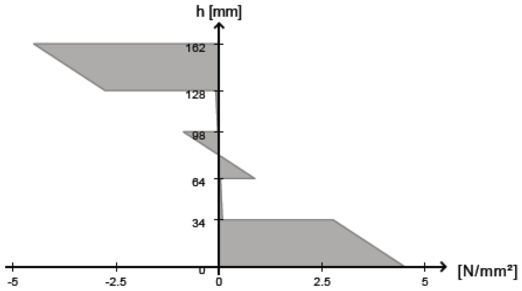


Figure 39: Bending stress (CLTdesigner)

$$I_{eff} = \frac{(EI)_{eff}}{E} = \frac{3.485 * 10^{12}}{12000} = 2.90 * 10^{8} mm^{4}$$

$$\sigma_{max} = \frac{M_{max}}{I_{eff}} * 0.5 * h_{tot} = \frac{20.52 * 10^{6}}{2.9 * 10^{8}} * 0.5 * 162 = 5.73 \frac{N}{mm^{2}}$$

$$\sigma_{max} = \frac{M_{max}}{K_{clt}} * 1200 * 0.5 * h_{tot} = \frac{21.12 * 10^{6}}{3.485 * 10^{12}} * 12000 * 0.5 * 162 = 5,91 \frac{N}{mm^{2}}$$

Shear stress

$$\tau_{max} = \frac{3 * V_{max}}{2} * \frac{I_{gross}}{I_{eff} * A_{gross}} = \frac{3 * 17.1 * 10^{3} * 3.54 * 10^{8}}{2 * 2.9 * 10^{8} * 162 * 1000} = 0.19 \frac{N}{mm^{2}}$$

$$\tau_{max} = \frac{V_{max,d} * \sum(S_m * E_m)}{K_{clt} * b_i} = \frac{17.6 * 10^{3}}{3.485 * 10^{12} * 1000} * 12000 * 1000 * (34 * 64 + 17 * 17/2) = 0.14 \frac{N}{mm^{2}}$$

Rolling shear stress

$$\tau_{s,max} = \frac{3 * V_{max}}{2} * \frac{I_{gross}}{I_{eff} * A_{gross}} = \frac{3 * 17.1 * 10^{3} * 3.54 * 10^{8}}{2 * 2.9 * 10^{8} * 162 * 1000} = 0.19 \frac{N}{mm^{2}}$$

$$\tau_{s,max} = \frac{V_{max,d} * \sum(S_m * E_m)}{K_{clt} * b_i} = \frac{17.6 * 10^{3}}{3.485 * 10^{12} * 1000} * 12000 * 1000 * (34 * 64) = 0.13 \frac{N}{mm^{2}}$$

Verification	
Bending	
$F_b = f_b(K_D K_H K_{Sb} K_T) = 24 * (1 * 1 * 1 * 1) = 24\, {}^{N}/_{mm^2}$ $I_{eff} = \frac{(EI)_{eff}}{E} = \frac{3.485*10^{12}}{12000} = 2.90 * 10 \quad mm^4$ $M_r = \phi * F_b * \frac{I_{eff}}{0.5h_{tot}} = 0.9 * 24 * \frac{2.90 * 10^8}{0.5 * 162} = 77.33kNm$ $M_r \geq M_{max}$ $77.33kNm \geq 20.52kNm$ $\boldsymbol{\eta_M = 26.54\%}$	$\frac{L}{H} = \frac{4800}{162} = 29{,}63\ldots$ Timoshenko $\sigma_{edge,d} = \frac{M_d}{K_{clt}} * \left(e_i + \frac{t_i}{2}\right) * E_i = \frac{21.2*10^6}{3.485*10^{12}} * \frac{162}{2} * 12000 = 5.91 \frac{N}{mm^2}$ $\frac{\sigma_{edge,d}}{f_{m,clt,d}} = \frac{5.91}{16.9} = 0.35 \leq 1.0$ $\boldsymbol{\eta_M = 34.98\%}$
Shear	

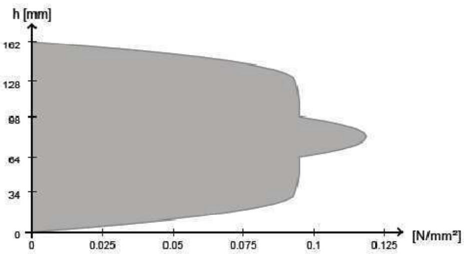


Figure 40: Shear stress (CLTdesigner)

$c = \frac{I_{eff}}{I_{gross}} = \frac{2.90 * 10^8}{3.54 * 10^8} = 0.8$ $F_v = f_v(K_D K_H K_{Sv} K_T) = 2.7 * (1 * 1 * 1 * 1) = 2.7\, {}^{N}/_{mm^2}$ $V_r = \phi F_v \frac{2A_{gross}}{3}\left(\frac{I_{eff}}{I_{gross}}\right) = 0.9 * 2.7 * \frac{2}{3} * (1000 * 162) * 0.8 = 210.0kN$ $V_r \geq V_{max}$ $210.0kN \geq 17.1kN$ $\boldsymbol{\eta_V = 8.14\%}$	$\tau_{v,i,d} = \frac{V_{max,d} * \sum(S_m * E_m)}{K_{clt} * b_i} = \frac{17.6 * 10^3}{3.485 * 10^{12} * 1000} * \left(12000 * 34 * 1000 * 64 + 12000 * 1000 * \frac{17^2}{2}\right) = 0.14 \frac{N}{mm^2}$ $\frac{\tau_{v,max,d}}{f_{v,clt,d}} = \frac{0.14}{1.73} = 0.08 \leq 1.0$ $\boldsymbol{\eta_V = 8.13\%}$
Rolling shear	
$V_{s,r} = \phi F_{s,v} \frac{2A_{gross}}{3}\left(\frac{I_{eff}}{I_{gross}}\right) = 0.9 * 1.5 * \frac{2}{3} * (1000 * 162) * 0.8 = 116.64kN$ $V_{s,r} \geq V_{max}$ $116.64kN \geq 17.1kN$ $\boldsymbol{\eta_{V,s} = 14.66\%}$	$\tau_{s,i,d} = \frac{V_{max,d} * \sum(S_m * E_m)}{K_{clt} * b_i} = \frac{17.6 * 10^3}{3.485 * 10^{12} * 1000} * (12000 * 34 * 1000 * 64) = 0.13 \frac{N}{mm^2}$ $\frac{\tau_{s,max,d}}{f_{s,clt,d}} = \frac{0.13}{0.96} = 0.14 \leq 1.0$ $\boldsymbol{\eta_{V,s} = 13.74\%}$
Deflection	

Figure 41: Deflection $w_{net,fin}$ (CLTdesigner)

$$\varphi_i = 1.0$$

$$I_D = 1.0$$

$$I_L = 1.0$$

$$k_{def} = 0.90$$

$$E_d = \varphi_D * I_D * D + \varphi_L * I_L * L$$

$$q_D = 1.0 * (1.0 * 2.1) = 2.1\, {kN}/{m}$$

$$q_L = 1.0 * (1.0 * 3.0) = 3.0\, {kN}/{m}$$

$$u_{max} = \frac{5}{384} * \frac{qL^4}{(EI)_{eff}} + \frac{1}{8} * \frac{qL^2 k}{(GA)_{eff}}$$

$$\boldsymbol{u_{max,D}} = \frac{5}{384} * \frac{2.1 * 4800^4}{3.485 * 10^{12}} + \frac{1}{8} * \frac{2.1 * 4800^2 * 1.2}{1.26 * 10^7} = \mathbf{4.74}mm$$

$$\boldsymbol{u_{max,L}} = \frac{5}{384} * \frac{3.0 * 4800^4}{3.485 * 10^{12}} + \frac{1}{8} * \frac{3.0 * 4800^2 * 1.2}{1.26 * 10^7} = \mathbf{6.77}mm$$

$$u_{max,L} < \frac{L}{360} = \frac{4800}{360} = 13.33$$

$$\eta_w = 50{,}78\%$$

$$\gamma_G = \gamma_Q = 1.0$$

$$k_{def} = 0.85$$

$$\boldsymbol{w_{inst,G} = 5.2mm}$$

$$\boldsymbol{w_{inst,Q} = 5.99mm}$$

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$$\boldsymbol{u_{max,tot}} = u_{max,D} + u_{max,L} = 4.74 + 6.77 = \mathbf{11.51}mm$$

$$u_{max,tot} < \frac{L}{240} = \frac{4800}{240} = 20$$

$$\boldsymbol{\eta_w} = \mathbf{57.55}\%$$

Creep

$$u_{fin,P} = u_{inst,P}(1 + k_{def}) = 4.74(1 + 0.9) = 9.0mm$$

$$u_{fin,Q,1} = u_{inst,Q,1}(1 + \psi_{2.1}k_{def}) = 6.77(1 + 0.3 * 0.9) = 8.60mm$$

$$u_{fin} = u_{fin,P} + u_{fin,Q} \leq \frac{L}{150}$$

$$\boldsymbol{u_{fin}} = u_{fin,P} + u_{fin,Q} = \mathbf{17.61}mm \leq \frac{L}{150} = \frac{4800}{150} = 32$$

$$\eta_w = 55.01\%$$

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$$\boldsymbol{w_{inst}} = w_{inst,G} + w_{inst,Q} = 5.2 + 5.99 = \mathbf{11.19}mm \leq \frac{L}{300} = \frac{4800}{300} = 16mm$$

$$\boldsymbol{\eta_w} = \mathbf{69.93}\%$$

Creep

$$w_{inst,Q,perm} = \psi_{2,1} * w_{inst,Q} = 0.3 * 5.99 = 1.79mm$$

$$\boldsymbol{w_{creep}} = (w_{inst,G} + w_{inst,Q,perm}) * k_{def} = (5.2 + 1.79) * 0.85 = \mathbf{5.94}mm$$

$$w_{fin} = w_{inst} + w_{creep} = 5.2 + 5.94 = 11.14mm \leq \frac{L}{150} = \frac{4800}{150} = 32mm$$

$$\eta_w = 34.48\%$$

$$w_{net,fin} = w_{fin} - w_c = 11.14mm \leq \frac{L}{250} = \frac{4600}{250} = 18.4mm$$

$$\eta_w = 60.54\%$$

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<tr>
<td colspan="2">Vibration</td>
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$l \leq \frac{1}{9.15} \frac{\left(EI_{eff}^{1m}\right)^{0.293}}{(\rho A)^{0.123}}$ $l = \frac{1}{9.15} \frac{(3.485 * 10^6)^{0.293}}{(450 * 0.162 * 1.0)^{0.123}}$ $= 5.33\text{m}$ $l = 5.33m > l_{exist} = 4.8m$	EN 1995-1-1: vibration $f_1 = \frac{\pi}{2L^2}\sqrt{\frac{(EI)_L}{m}}$ $= \frac{\pi}{2 * 4.8^2}\sqrt{\frac{(3.46 * 10^6)}{350 * 0.162 * 1}}$ $= 16.84$ $f_1 = 16.84Hz \geq f_e = 8Hz$ $w_{1kN} = 0.67mm \leq 4mm$ Vibration fulfilled!

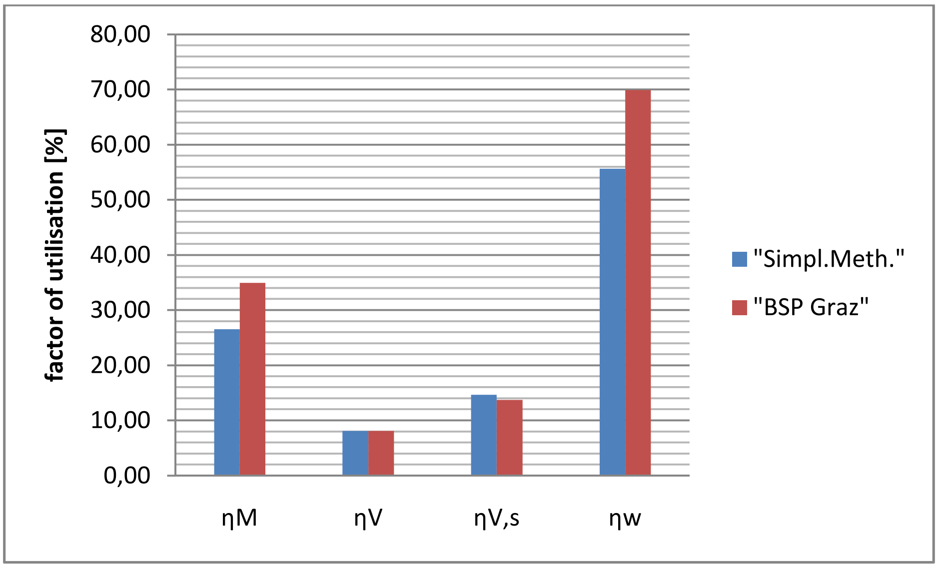


Diagram 4: Comparison of the results from the calculation according the „Simplified Method“ and the model “BSP Graz”

The verification of vibration is predominant again. Diagram 4 shows a good agreement for shear. The utilisation factor for bending according the model “BSP Graz” is higher than the calculation after the “Simplified Method”. The deflection in the “Simplified Method” has a lower factor of utilisation. This may be due the different limit states is significant for the deflection. The arguments from example 5.1.1 are also valid for this example.

5.2 Example: 5 layer continuous beam

5.2.1 Comparable calculation of a CLT-element with the "Simplified Method"

For this comparison the example from the BSPhandbuch of the Institute for timber construction and timber technology (Graz University of Technology) was taken. The example shows a continuous beam with a five layers panel. Dead loads and live loads have an effect on the system. With this comparable calculation the strength and stiffness parameters will be examined. The parameters for strength and stiffness are E1 for the Canadian example and GLh24* for the European example. This assumption was made because of the similar values in bending (see 5.5 "Comparison of the material").

5.2.1.1 Calculation and comparison of the bending stress, shear stress and rolling shear stress

Like in example 5.1 the stresses will be calculated and compared.

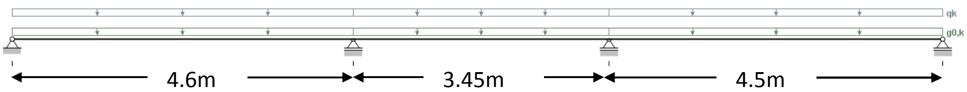


Figure 42: Continuous beam with a width of 1m (CLTdesigner)

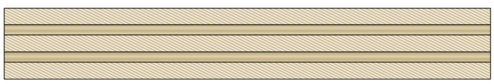


Figure 43: Cross section of the CLT slab (CLTdesigner)

t_i → 34/22/34/22/34
$t_{clt} = 146mm$
$b = 1000mm$

- Material: GL24h* or E1
- Service Class 1
- Areas for domestic and residential activities
- $g_{1,k} = 2.10\, {}^{kN}/_{m^2}$
- $q_k = 2.00\, {}^{kN}/_{m^2}$
- $L_{max} = 4600mm$
- $x_{M=0} = 3970mm$

Material E1 (ANSI PRG 320) "Simplified Method"	**Material GL24h* (BSPhandbuch)** "Simplified Method"
Predominant load combination:	
$E_d = \varphi_D * I_D * D + \varphi_L * I_L * L$ Due to the "Simplified Method" is interpreted for single span beams, the internal forces will be calculated on a substitute single span with L=3.97m.	

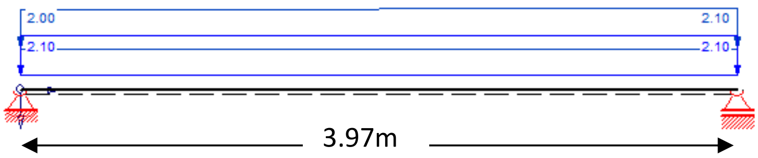


Figure 44: Load assumption for maximal moment [RuckZuck]

$$M_{max} = 11.08kNm$$

$$V_{max} = 11.16kN$$

Module of elasticity

$$E_{0,mean} = 11700\, {}^{N}/_{mm^2}$$
$$G_{0,mean} = 730\, {}^{N}/_{mm^2}$$
$$G_{90,mean} = 73\, {}^{N}/_{mm^2}$$

$$E_{0,mean} = 11600\, {}^{N}/_{mm^2}$$
$$G_{0,mean} = 720\, {}^{N}/_{mm^2}$$
$$G_{90,mean} = 72\, {}^{N}/_{mm^2}$$

Value of stiffness calculated with the Shear Analogy Method:

$$(EI)_{eff} = B_A + B_B = \sum_{i=1}^{n} E_i * b_i * \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i * A_i * z_i^2$$

$$(EI)_{eff} = 2.62 * 10^{12} Nmm^2$$
$$(GA)_{eff} = 1.428 * 10^{7} Nmm^2$$

$$(EI)_{eff} = 2.59 * 10^{12} Nmm^2$$
$$(GA)_{eff} = 1.778 * 10^{7} Nmm^2$$

Bending stress

$$\boldsymbol{\sigma_{max}} = \frac{M_{max}}{I_{eff}} * 0.5 * h_{tot} = \frac{11.08 * 10^6}{2.24 * 10^8} * 0.5 * 146 = \mathbf{3.58} \frac{N}{mm^2}$$

$$I_{eff} = \frac{(EI)_{eff}}{E} = \frac{2.62 * 10^{12}}{11700} = 2.24 * 10 \quad mm^4$$

$$\boldsymbol{\sigma_{max}} = \frac{M_{max}}{I_{eff}} * 0.5 * h_{tot} = \frac{11.08 * 10^6}{2.24 * 10^8} * 0.5 * 146 = \mathbf{3.58} \frac{N}{mm^2}$$

$$I_{eff} = \frac{(EI)_{eff}}{E} = \frac{2.59 * 10^{12}}{11600} = 2.24 * 10 \quad mm^4$$

Shear stress

$\boldsymbol{\tau_{max} = \frac{3 * V_{max}}{2} * \frac{I_{gross}}{I_{eff} * A_{gross}} = \frac{3 * 11.16 * 10^3 * 2.59 * 10^8}{2 * 2.24 * 10^8 * 146 * 1000} = 0.13 \frac{N}{mm^2}}$	$\boldsymbol{\tau_{max} = \frac{3 * V_{max}}{2} * \frac{I_{gross}}{I_{eff} * A_{gross}} = \frac{3 * 11.16 * 10^3 * 2.59 * 10^8}{2 * 2.24 * 10^8 * 146 * 1000} = 0.13 \frac{N}{mm^2}}$
Rolling shear stress	
$\tau_{s,max} = \mathbf{0.13} \frac{N}{mm^2}$	$\tau_{s,max} = \mathbf{0.13} \frac{N}{mm^2}$

The calculation of the stresses remains the same due to the elimination in the equation of the bending stiffness.

5.2.1.2 Verification according "Simplified Method" and model "BSP-Graz"

With the same parameters the ULS and SLS verification will be cited.

Verification	
Combination factors:	
$\varphi_i = 1.0;\ I_D = 1.25; I_L = 1.50$	
Value of strength	
$f_b = 28.2 \, {}^{N}/_{mm^2}$ $f_c = 19.3 \, {}^{N}/_{mm^2}$ $f_v = 1.5 \, {}^{N}/_{mm^2}$ $f_s = 0.5 \, {}^{N}/_{mm^2}$	$f_{m,clt,k} = k_l * f_{m,glt,k} = 1.1 * 24 = 26.4 \, {}^{N}/_{mm^2}$ $f_{v,clt,k} = 3.0 \, {}^{N}/_{mm^2}$ $f_{s,clt,k} = 1.25 \, {}^{N}/_{mm^2}$
Bending	

<table>
<tr>
<td>$F_b = f_b(K_D K_H K_{Sb} K_T) = 28.2 * (1 * 1 * 1 * 1) = 28.2\, {}^{N}/_{mm^2}$

$I_{eff} = \frac{(EI)_{eff}}{E} = \frac{2.62 * 10^{12}}{11700} = 2.24 * 10 \quad mm^4$

$M_r = \phi * F_b * \frac{I_{eff}}{0.5 h_{tot}} = 0.9 * 28.2 * \frac{2.24 * 10^8}{0.5 * 146} = 77.87 kNm$

$M_r \geq M_{max}$
$77.87 kNm \geq 11.08 kNm$

$\boldsymbol{\eta_M = 14.12\%}$</td>
<td>$F_b = f_b(K_D K_H K_{Sb} K_T) = 26.4 * (1 * 1 * 1 * 1) = 26.4\, {}^{N}/_{mm^2}$

$I_{eff} = \frac{(EI)_{eff}}{E} = \frac{2.59 * 10^{12}}{11600} = 2.24 * 10 \quad mm^4$

$M_r = \phi * F_b * \frac{I_{eff}}{0.5 h_{tot}} = 0.9 * 26.4 * \frac{2.24 * 10^8}{0.5 * 146} = 72.58 kNm$

$M_r \geq M_{max}$
$72.58 kNm \geq 11.08 kNm$

$\boldsymbol{\eta_M = 15.16\%}$</td>
</tr>
<tr>
<td colspan="2">Shear</td>
</tr>
<tr>
<td>$c = \frac{I_{eff}}{I_{gross}} = \frac{2.24 * 10^8}{2.6 * 10^8} = 0.86$

$F_v = f_v(K_D K_H K_{Sv} K_T) = 1.5 * (1 * 1 * 1 * 1) = 1.5\, {}^{N}/_{mm^2}$</td>
<td>$c = \frac{I_{eff}}{I_{gross}} = \frac{2.24 * 10^8}{2.6 * 10^8} = 0.86$

$F_v = f_v(K_D K_H K_{Sv} K_T) = 3.0 * (1 * 1 * 1 * 1) = 3.0\, {}^{N}/_{mm^2}$</td>
</tr>
</table>

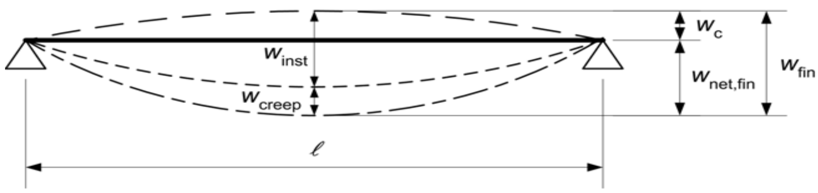
<table>
<tr>
<td>$$V_r = \phi F_v \frac{2A_{gross}}{3}\left(\frac{I_{eff}}{I_{gross}}\right)$$ $$= 0.9 * 1.5 * \frac{1}{1.5}$$ $$* (1000 * 146) *$$ $$* 0.86 = 113.00kN$$ $$V_r \geq V_{max}$$ $$113.00kN \geq 11.16kN$$ $$\boldsymbol{\eta_V = 9.7\%}$$</td>
<td>$$V_r = \phi F_v \frac{2A_{gross}}{3}\left(\frac{I_{eff}}{I_{gross}}\right)$$ $$= 0.9 * 3.0 * \frac{1}{1.5}$$ $$* (1000 * 146)$$ $$* 0.86 = 226.00kN$$ $$V_r \geq V_{max}$$ $$226.00kN \geq 11.16kN$$ $$\boldsymbol{\eta_V = 4.9\%}$$</td>
</tr>
<tr>
<td colspan="2">Rolling shear</td>
</tr>
<tr>
<td>$$V_{s,r} = \phi F_{s,v} \frac{2A_{gross}}{3}\left(\frac{I_{eff}}{I_{gross}}\right)$$ $$= 0.9 * 0.5 * \frac{1}{1.5}$$ $$* (1000 * 146) *$$ $$* 0.86 = 37.67kN$$ $$V_{s,r} \geq V_{max}$$ $$37.67 \geq 11.16kN$$ $$\boldsymbol{\eta_{V,s} = 29.20\%}$$</td>
<td>$$V_{s,r} = \phi F_{s,v} \frac{2A_{gross}}{3}\left(\frac{I_{eff}}{I_{gross}}\right)$$ $$= 0.9 * 1.25 * \frac{1}{1.5}$$ $$* (1000 * 146)$$ $$* 0.86 = 94.17kN$$ $$V_{s,r} \geq V_{max}$$ $$94{,}17kN \geq 11.16kN$$ $$\boldsymbol{\eta_{V,s} = 11.68\%}$$</td>
</tr>
<tr>
<td colspan="2">Deflection</td>
</tr>
<tr>
<td colspan="2">

Figure 45: Definition of deflection according [4]</td>
</tr>
<tr>
<td>$\varphi_i = 1.0$
$I_D = 1.0$
$I_L = 1.0$
$k_{def} = 0.90$</td>
<td>$\varphi_i = 1.0$
$I_D = 1.0$
$I_L = 1.0$
$k_{def} = 0.90$</td>
</tr>
</table>

$E_d = \varphi_D * I_D * D + \varphi_L * I_L * L$ $q_D = 1.0 * (1.0 * 2.1) = 2.1\, {}^{kN}/_{m}$ $q_L = 1.0 * (1.0 * 2.0) = 2.0\, {}^{kN}/_{m}$ $u_{max} = \frac{5}{384} * \frac{qL^4}{(EI)_{eff}} + \frac{1}{8} * \frac{qL^2k}{(GA)_{eff}}$ $\boldsymbol{u_{max,D}} = \frac{5}{384} * \frac{2.1 * 3970^4}{2.62 * 10^{12}} + \frac{1}{8} * \frac{2.1 * 3970^2 * 1.2}{1.428 * 10^7} = \mathbf{2.94} mm = u_{inst,P}$ $\boldsymbol{u_{max,L}} = \frac{5}{384} * \frac{2.0 * 3970^4}{2.62 * 10^{12}} + \frac{1}{8} * \frac{2.0 * 3970^2 * 1.2}{1.428 * 10^7} = \mathbf{2.8} mm = u_{inst,Q1}$ $u_{max,L} < \frac{L}{360} = \frac{3970}{360} = 11.03$ $\eta_w = 26.38\%$	$E_d = \varphi_D * I_D * D + \varphi_L * I_L * L$ $q_D = 1.0 * (1.0 * 2.1) = 2.1\, {}^{kN}/_{m}$ $q_L = 1.0 * (1.0 * 2.0) = 2.0\, {}^{kN}/_{m}$ $u_{max} = \frac{5}{384} * \frac{qL^4}{(EI)_{eff}} + \frac{1}{8} * \frac{qL^2k}{(GA)_{eff}}$ $\boldsymbol{u_{max,D}} = \frac{5}{384} * \frac{2.1 * 3970^4}{2.59 * 10^{12}} + \frac{1}{8} * \frac{2.1 * 3970^2 * 1.2}{1.778 * 10^7} = \mathbf{2.90} mm = u_{inst,P}$ $\boldsymbol{u_{max,L}} = \frac{5}{384} * \frac{2.0 * 3970^4}{2.59 * 10^{12}} + \frac{1}{8} * \frac{2.0 * 3970^2 * 1.2}{1.778 * 10^7} = \mathbf{2.76} mm = u_{inst,Q1}$ $u_{max,L} < \frac{L}{360} = \frac{3970}{360} = 11.03$ $\eta_w = 25.02\%$
$\boldsymbol{u_{max,tot}} = u_{max,D} + u_{max,L} = 2.94 + 2.8 = \mathbf{5.74} mm$ $u_{max,tot} < \frac{L}{240} = \frac{3970}{240} = 16.54$ $\boldsymbol{\eta_w} = \mathbf{34.70}\%$	$\boldsymbol{u_{max,tot}} = u_{max,D} + u_{max,L} = 2.9 + 2.76 = \mathbf{5.66} mm$ $u_{max,tot} < \frac{L}{240} = \frac{3970}{240} = 16.54$ $\eta_w = 34.2\%$

Creep	Creep
$u_{fin,P} = u_{inst,P}(1 + k_{def}) =$ $2.94(1 + 0.9) = 5.59mm$	$u_{fin,P} = u_{inst,P}(1 + k_{def}) =$ $2.9(1 + 0.9) = 5.51mm$
$u_{fin,Q,1} = u_{inst,Q,1}(1 + \psi_{2,1}k_{def}) =$ $2.8(1 + 0.3 * 0.9) = 3.56mm$	$u_{fin,Q,1} = u_{inst,Q,1}(1 + \psi_{2,1}k_{def}) =$ $2.76(1 + 0.3 * 0.9) = 3.51mm$
$u_{fin} = u_{fin,P} + u_{fin,Q} \leq \frac{L}{150}$	$u_{fin} = u_{fin,P} + u_{fin,Q} \leq \frac{L}{150}$
$\boldsymbol{u_{fin}} = u_{fin,P} + u_{fin,Q} = \mathbf{9.14}mm \leq \frac{L}{150} = \frac{3970}{150} = 26.47mm$	$\boldsymbol{u_{fin}} = u_{fin,P} + u_{fin,Q} = \mathbf{9.02}mm \leq \frac{L}{150} = \frac{3970}{150} = 26.47mm$
$\eta_w = 34.53\%$	$\boldsymbol{\eta_w} = \mathbf{34.07}\%$

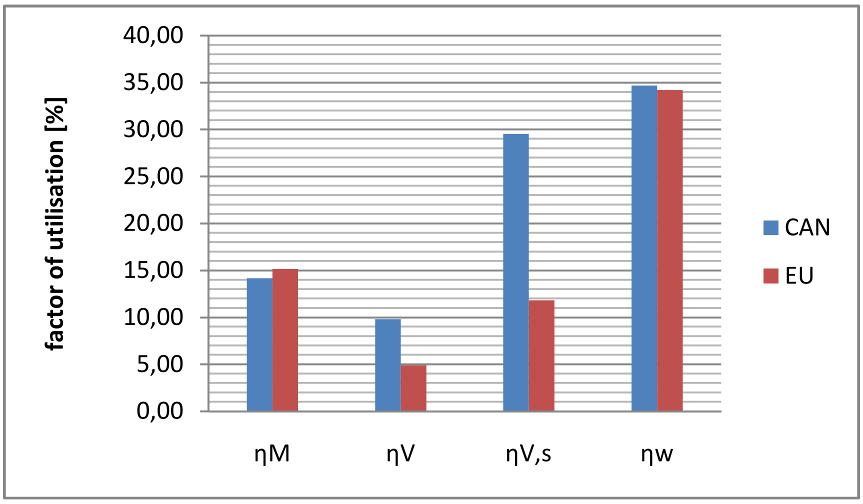


Diagram 5: Comparison of the results from the calculation according the „Simplified Method" with different parameters

The comparable view on the parameters demonstrates good agreements for bending and deflection.

The verification of shear shows strong differences. This affects because of the very different values of shear strength ($f_{v,CAN=}$ 1.5 N/mm² and $f_{v,clt,k,EU}$= 3.0 Nmm²) and rolling shear strength ($f_{s,CAN}$= 0.5 N/mm² and $f_{r,ckt,k,EU}$= 1,25 N/mm²).

5.2.2 Comparison of the calculation methods between the "Simplified Method" and model "BSP-Graz" with the same parameters

Another comparable view was made on the "Simplified Method" and the model "BSP-Graz" (Timoshenko). For the example the material E1 of the Canadian Standard PRG 320 was chosen.

The boundary conditions are defined as before from the BSPhandbuch of Institute for timber construction and timber technology (Graz University of Technology). It is a continuous beam with a five layers panel. Dead loads and live loads have an effect on the system.

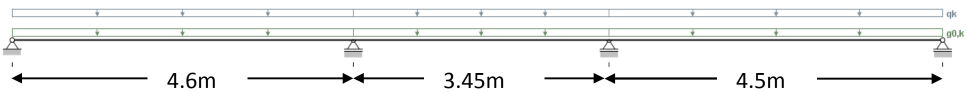


Figure 46: Continuous beam with a width of 1m (CLTdesigner)

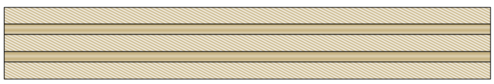


Figure 47: Cross section of the CLT slab (CLTdesigner)

$$t_{clt} = 146mm$$
$$b = 1000mm$$

- Material: E1
- Service Class 1
- Areas for domestic and residential activities
- $g_{k,ceiling} = 2.10\, {kN}/{m^2}$
- $q_k = 2.00\, {kN}/{m^2}$
- $L_{max} = 4600mm$
- $x_{M=0} = 3970mm$

"Simplified Method"	**Model "BSP-Graz" (Timoshenko)**
Combination factors:	
$\varphi_i = 1.0$ $I_D = 1.25$ $I_L = 1.50$	$k_{mod} = 0.8$ $\gamma_G = 1.35$ $\gamma_Q = 1.50$
Predominant load combination:	
$E_d = \varphi_D * I_D * D + \varphi_L * I_L * L$	$E_d = \gamma_G * g_{k,ceiling} + \gamma_Q * q_k$

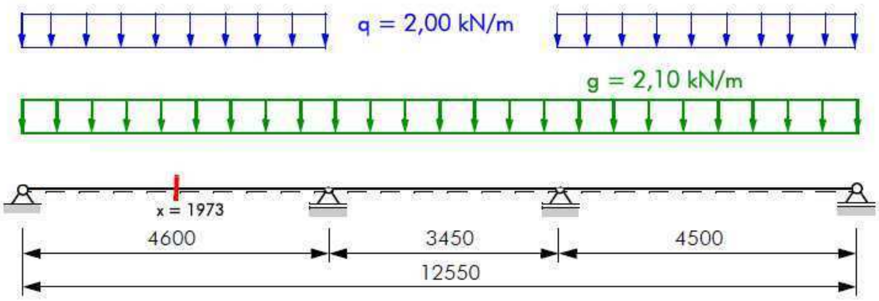


Figure 48: Load assumption for maximal moment [6]; g → dead load, q → live load

Internal forces calculated on a single span beam with a length of 3.97 m	Internal forces calculated on the continuous beam
$M_{max} = 11.08kNm$ $A_{max} = 11.16kN$	$M_{max} = 11.36kNm$ $A_{max} = 11.51kN$

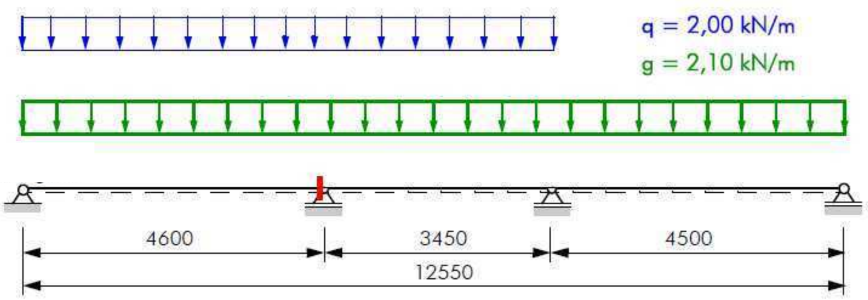


Figure 49: Load assumption for maximal shear force [6]; g → dead load, q → live load

$V_{max} = 11.16kN$	$V_{max} = 15.85kN$

Value of strength

$f_b = 28.2\, {}^{N}/_{mm^2}$ $f_c = 19.3\, {}^{N}/_{mm^2}$ $f_v = 1.5\, {}^{N}/_{mm^2}$ $f_s = 0.5\, {}^{N}/_{mm^2}$	$f_{m,clt,d} = k_l * k_{mod} \frac{f_b}{\gamma_M} = 1.1 * 0.9 * \frac{28.2}{1.25} = 22.32\, {}^{N}/_{mm^2}$ $f_{c,clt,0,d} = k_{mod} \frac{f_{c,0}}{\gamma_M} = 0.9 * \frac{19.3}{1.25} = 13.9\, {}^{N}/_{mm^2}$ $f_{v,clt,d} = k_{mod} \frac{f_v}{\gamma_M} = 0.9 * \frac{1.5}{1.25} = 1.08\, {}^{N}/_{mm^2}$

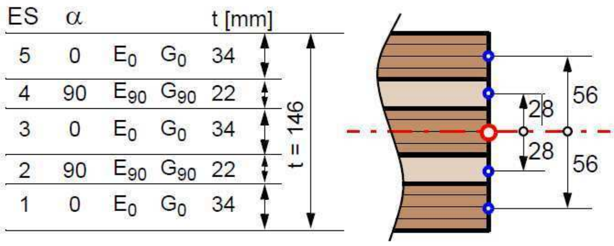
<table>
<tr><td></td><td>$$f_{s,clt,d} = k_{mod}\frac{f_s}{\gamma_M} = 0.9 * \frac{0.5}{1.25} = 0.36\, {}^{N}/_{mm^2}$$</td></tr>
<tr><td colspan="2">Module of elasticity</td></tr>
<tr><td>$E_{0,mean} = 11700\, {}^{N}/_{mm^2}$
$G_{0,mean} = 730\, {}^{N}/_{mm^2}$
$G_{90,mean} = 73\, {}^{N}/_{mm^2}$</td><td>$E_{0,mean} = 11700\, {}^{N}/_{mm^2}$
$G_{0,mean} = 730\, {}^{N}/_{mm^2}$
$G_{90,mean} = 73\, {}^{N}/_{mm^2}$</td></tr>
<tr><td colspan="2">

Figure 50: Definition of the laminated cross section [6]</td></tr>
<tr><td>Value of stiffness calculated with the Shear Analogy Method:
$$(EI)_{eff} = B_A + B_B = \sum_{i=1}^{n} E_i * b_i * \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i * A_i * z_i^2$$
$$(EI)_{eff} = 2.62 * 10^{12} Nmm^2$$
$$(GA)_{eff} = \frac{a^2}{\left[\frac{h_1}{2*G_1*b_1}+\sum_{i=2}^{n-1}\frac{h_i}{G_i*b_i}+\frac{h_n}{2*G_n*b_n}\right]} = \frac{112^2}{\left[\frac{34}{2*730*1000}+\left(\frac{2*22}{73*1000}+\frac{34}{2*730*1000}\right)+\frac{34}{2*730*1000}\right]} = 1.428 * 10^7 N$$</td><td>$$K_{clt} = \sum(I_i * E_i) + \sum(A_i * e_i^2 * E_i)$$
$$K_{clt} = 3 * 1000 * \frac{34^3}{12} * 11700 + 2 * 1000 * 34 * 56^2 * 11700 = 2.62 * 10^{12} Nmm^2$$
$$S_{ges} = \sum(G_i * b_i * t_i)$$
$$S_{ges} = 730 * 3 * 34 * 1000 + 73 * 2 * 22 * 1000 = 7.77 * 10^7 N$$
$$\kappa = 0.258$$
$$S_{clt} = S_{ges} * \kappa$$
$$S_{clt} = 7.77 * 10^7 * 0.258 = 2.00 * 10^7 N$$</td></tr>
<tr><td colspan="2">Verification</td></tr>
</table>

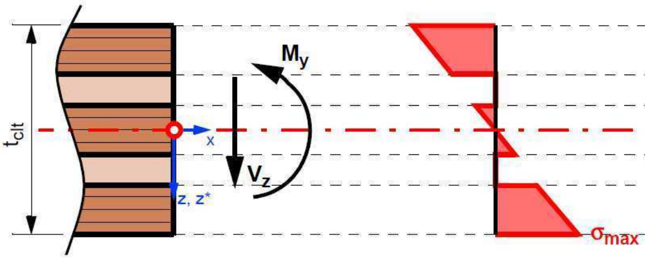
<table>
<tr><td colspan="2">Bending</td></tr>
<tr><td colspan="2">

Figure 51: Normal stress $\sigma_{max}=\sigma_{edge,d}$ [6]</td></tr>
<tr><td>

$$F_b = f_b(K_D K_H K_{Sb} K_T) = 28.2 * (1*1*1*1) = 28.2\,{}^{N}/_{mm^2}$$

$$I_{eff} = \frac{(EI)_{eff}}{E} = \frac{2.62 * 10^{12}}{11700} = 2.24 * 10 \quad mm^4$$

$$M_r = \phi * F_b * \frac{I_{eff}}{0.5h_{tot}} = 0.9 * 28.2 * \frac{2.24 * 10^8}{0.5 * 146} = 77.88kNm$$

$$M_r \geq M_{max}$$
$$77.88kNm \geq 11.08kNm$$

$$\boldsymbol{\eta_M = 14.23\%}$$

</td><td>

$$\sigma_{edge,d} = \frac{M_d}{K_{clt}} * \left(e_i + \frac{t_i}{2}\right) * E_i = \frac{11.36 * 10^6}{2.62 * 10^{12}} * \frac{146}{2} * 11700 = 3.70 \frac{N}{mm^2}$$

$$\frac{\sigma_{edge,d}}{f_{m,clt,d}} = \frac{3.70}{22.32} = 0.16 \leq 1.0$$

$$\boldsymbol{\eta_M = 16.57\%}$$

</td></tr>
<tr><td colspan="2">Shear</td></tr>
</table>

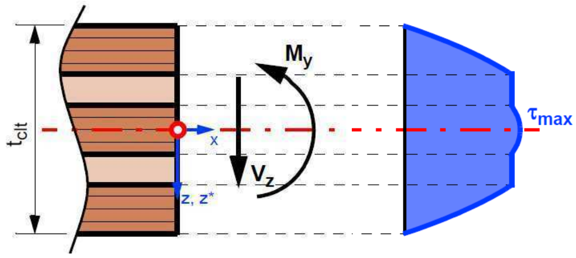


Figure 52: Shear stress $\tau_{max}=\tau_{V,d}$ [6]

$c = \frac{I_{eff}}{I_{gross}} = \frac{2.24 * 10^8}{2.62 * 10^8} = 0.85$	
$F_v = f_v(K_D K_H K_{Sv} K_T) = 1.5 * (1 * 1 * 1 * 1) = 1.5 \, N/_{mm^2}$ $V_r = \phi F_v \frac{2A_{gross}}{3}\left(\frac{I_{eff}}{I_{gross}}\right) = 0.9 * 1.5 * \frac{1}{1.5} * (1000 * 146) * 0.85 = 111.69kN$	$\tau_{v,i,d} = \frac{V_{max,d} * \sum(S_m * E_m)}{K_{clt} * b_i} = \frac{15.85 * 10^3}{2.62 * 10^{12} * 1000} * \left(11700 * 34 * 1000 * 56 + 11700 * 1000 * \frac{17^2}{2}\right) = 0.145 \frac{N}{mm^2}$
$V_r \geq V_{max}$ $111.69N \geq 11.16kN$ $\boldsymbol{\eta_V = 9.99\%}$	$\frac{\tau_{v,max,d}}{f_{v,clt,d}} = \frac{0.145}{1.08} = 0.13 \leq 1.0$ $\boldsymbol{\eta_V = 13.43\%}$
Rolling shear	
$V_{s,r} = \phi F_{s,v} \frac{2A_{gross}}{3}\left(\frac{I_{eff}}{I_{gross}}\right) = 0.9 * 0.5 * \frac{1}{1.5} * (1000 * 146) * 0.85 = 37.23kN$	$\tau_{s,i,d} = \frac{V_{max,d} * \sum(S_m * E_m)}{K_{clt} * b_i} = \frac{15.85 * 10^3}{2.62 * 10^{12} * 1000} * (11700 * 34 * 1000 * 56) = 0.135 \frac{N}{mm^2}$
$V_{s,r} \geq V_{max}$ $37.23kN \geq 11.16kN$ $\boldsymbol{\eta_{V,r} = 39.97\%}$	$\frac{\tau_{s,max,d}}{f_{s,clt,d}} = \frac{0.135}{0.36} = 0.38 \leq 1.0$ $\boldsymbol{\eta_{V,r} = 37.5\%}$

deflection

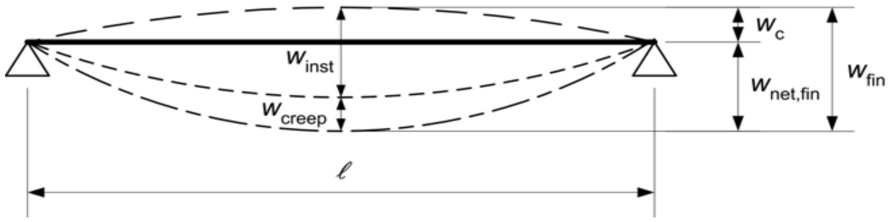


Figure 53: Definition of deflection [4]

$$\varphi_i = 1.0$$
$$I_D = 1.0$$
$$I_L = 1.0$$
$$k_{def} = 0.90$$

$$E_d = \varphi_D * I_D * D + \varphi_L * I_L * L$$

$$q_D = 1.0 * (1.0 * 2.1) = 2.1\, {}^{kN}\!/_{m}$$

$$q_L = 1.0 * (1.0 * 2.0) = 2.0\, {}^{kN}\!/_{m}$$

$$u_{max} = \frac{5}{384} * \frac{qL^4}{(EI)_{eff}} + \frac{1}{8} * \frac{qL^2k}{(GA)_{eff}}$$

$$\boldsymbol{u_{max,D}} = \frac{5}{384} * \frac{2.1 * 3970^4}{2.62 * 10^{12}} + \frac{1}{8} * \frac{2.1 * 3970^2 * 1.2}{1.428 * 10^7} = \mathbf{2.94}mm = u_{inst,P}$$

$$\boldsymbol{u_{max,L}} = \frac{5}{384} * \frac{2.0 * 3970^4}{2.62 * 10^{12}} + \frac{1}{8} * \frac{2.0 * 3970^2 * 1.2}{1.428 * 10^7} = \mathbf{2.8}mm = u_{inst,Q1}$$

$$u_{max,L} < \frac{L}{360} = \frac{3970}{360} = 11.03$$

$$\eta_w = 26.38\%$$

$$\gamma_G = \gamma_Q = 1.0$$

$$k_{def} = 0.85$$

The calculation of the deflection was made with the software RStab.

Permanent load field 1

$$\boldsymbol{w_{inst,G} = 3.12mm}$$

Variable load field 1

$$\boldsymbol{w_{inst,Q} = 3.51mm}$$

$$u_{max,tot} = u_{max,D} + u_{max,L}$$
$$= 2.94 + 2.8$$
$$= 5.74mm$$

$$u_{max,tot} < \frac{L}{240} = \frac{3970}{240} = 16.54$$

$$\boldsymbol{\eta_w = 34.7\%}$$

Creep

$$u_{fin,P} = u_{inst,P}\left(1 + k_{def}\right) =$$
$$2.94(1 + 0.9) = 5.586mm$$

$$u_{fin,Q,1} = u_{inst,Q,1}\left(1 + \psi_{2,1}k_{def}\right) =$$
$$2.8(1 + 0.3 * 0.9) = 3.556mm$$

$$u_{fin} = u_{fin,P} + u_{fin,Q} \le \frac{L}{150}$$

$$\boldsymbol{u_{fin}} = u_{fin,P} + u_{fin,Q} = 9.142mm$$
$$\le \frac{L}{150} = \frac{3970}{150}$$
$$= \boldsymbol{26.47}mm$$

$$\eta_w = 34.53\%$$

Creep

$$w_{inst,Q,perm} = \psi_{2,1} * w_{inst,Q}$$
$$= 0.3 * 3.51$$
$$= 1.05mm$$

$$w_{creep} = \left(w_{inst,G} + w_{inst,Q,perm}\right)$$
$$* k_{def}$$
$$= (3.12 + 1.05)$$
$$* 0.85 = 3.54mm$$

$$w_{inst} = w_{inst,G} + w_{inst,Q}$$
$$= 6.63mm \le \frac{L}{300}$$
$$= \frac{4600}{300} = 18.4mm$$
$$\eta_w = 36.03\%$$

$$\boldsymbol{w_{fin}} = w_{inst} + w_{creep} = 10.17mm$$
$$\le \frac{L}{150} = \frac{4600}{150}$$
$$= \boldsymbol{30.67}mm$$

$$\eta_w = 33.16\%$$

$$w_{net,fin} = w_{fin} - w_c = 10.17mm$$
$$\le \frac{L}{250} = \frac{4600}{250}$$
$$= 18.4mm$$
$$\boldsymbol{\eta_w = 55.27\%}$$

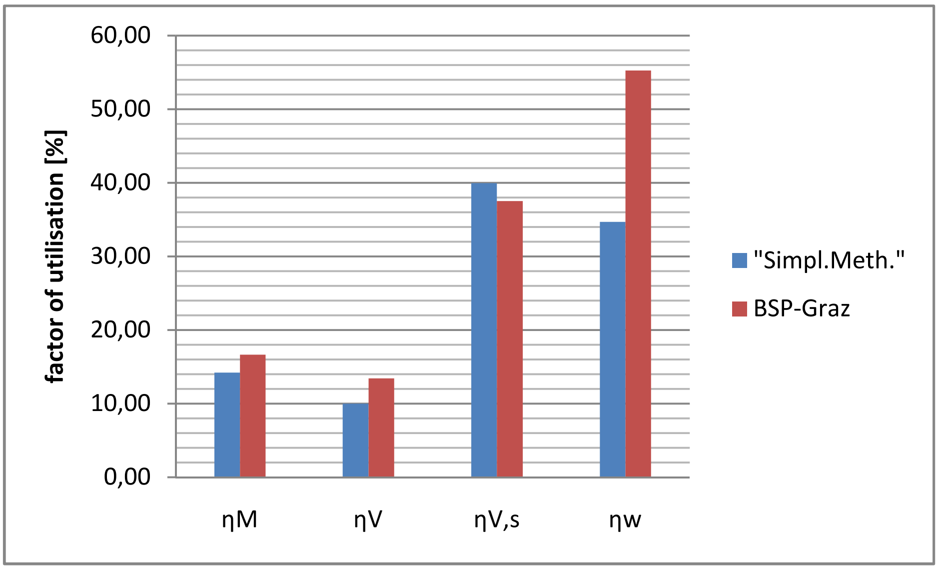


Diagram 6: Comparison of the results from the calculation according the „Simplified Method" and the model „BSP-Graz" with European parameters

At this results stand out that the deflection is more conservative according to the model BSP-Graz. This difference occurs because of the different limit states for the verification of deflection which are significant for this example. Predominant is the verification of vibration.

5.3 Summary of the results from the examples

The calculation above shall demonstrate how the parameters behave during a design procedure and how the methods of the guidelines have an influence on the verification.

First it can be observed that the results of the comparison of the methods show similar values in all examples. The European verification of bending shows higher factors of utilisation as the Canadian approach. The verification of shear demonstrates good agreements in example 5.1 (single span beam). However the Canadian results of shear verification show little higher factors of utilisation. Also in example 5.2 (continuous beam) the Canadian shear verification has a higher factor of utilisation.

This may occur due to the European calculation with design values, which is a more critical procedure than the use of the Canadian resistance factor ϕ.

The European verification of deflection shows good agreements in example 5.1.1, 5.1.2 and 5.2.1. In example 5.2.2 and 5.2.3 great differences appear for the verification of deflection. During the calculated values of deflection between the guidelines don't differ that much, the limits vary within the proposed design procedures. Due to this limit states different load combinations are significant.

Further the material parameters shall be compared. The verification of bending demonstrates higher factors of utilisation for Europe. This occurs due the higher values of the Canadian bending strength and stiffness parameters. The Canadian rolling shear in example 5.2.1 and 5.2.3 shows high factors of utilisation. The reason lies in the low rolling shear strength of the used material from ANSI PRG 320.

Note: Under the subtitle "Comparison of the material" will be explained why the material GL24h* and E1 were chosen and compared.

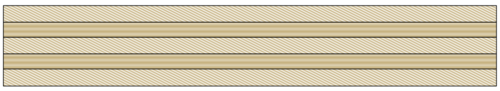
Furthermore must be mentioned that the Canadian Standard is interpreted for single span beams. Therefore the results of example 5.1 agree more with the European approach then in example 5.2.

5.4 Comparative view on the influence of the stiffness parameters between Timoshenko and Shear Analogy Method

Table 23 contains a listing of factors of utilisation according a calculation with the theory of Timoshenko and the Shear Analogy Method. The calculation was made with the CLTdesigner. The boundary conditions are given by the example of the BSPhandbuch (see figure 55).

The differences in the calculation are created by strength and stiffness parameters taken one time from the BSPhandbuch and another time from the ANSI PRG 320.

Proposed values of BSPhandbuch	Values of ANSI PRG 320
• Material: GL24h* • Service Class 1 • Areas for domestic and residential activities • $g_{k,ceiling} = 2.10 \,{}^{kN}/_{m^2}$ • $q_k = 2.00 \,{}^{kN}/_{m^2}$ • $L_{max} = 4600mm$	• Material: E1 • Service Class 1 • Areas for domestic and residential activities • $g_{k,ceiling} = 2.10 \,{}^{kN}/_{m^2}$ • $q_k = 2.00 \,{}^{kN}/_{m^2}$ • $L_{max} = 4600mm$



CLT Cross section (CLTdesigner)

t_i → 34/30/34/30/34

$t_{clt} = 162mm$

$b = 1000mm$

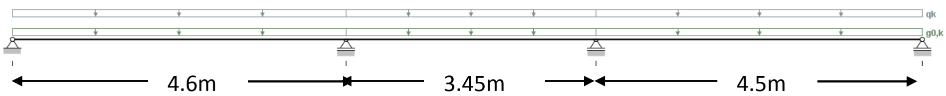


Figure 54: Geometric data for the calculation

		EU		CAN	
%		Timosh.	SAM	Timosh.	SAM
Bending	η_M	15.70	19.50	13.30	17.1
Rolling shear	$\eta_{V,s}$	11.9	9.7	29.90	24.0
Deflection	η_w	38.90	38.90	39.3	39.2
Vibration		fulfilled	fulfilled	Fulfilled	fulfilled
Fire: bending	$\eta_{M,fi}$	6.70	6.40	5.70	5.6
Fire: shear	$\eta_{V,fi}$	4.2	3.70	10.50	9.1

Table 23: Comparison of Timoshenko and Shear Analogy Method

An aspect of the comparison is the difference between the methods (Timoshenko – Shear Analogy Method) itself. Bending and shear deviate from each other, during the results of deflection behaves similarly. It stands out that the factor of utilisation for the verification of bending is according the Shear Analogy Method more conservative, during the verification of shear shows lower factors of utilisation. This occurs due to the peak stress on the supports.

The following diagrams demonstrate the numbers as a bar chart.

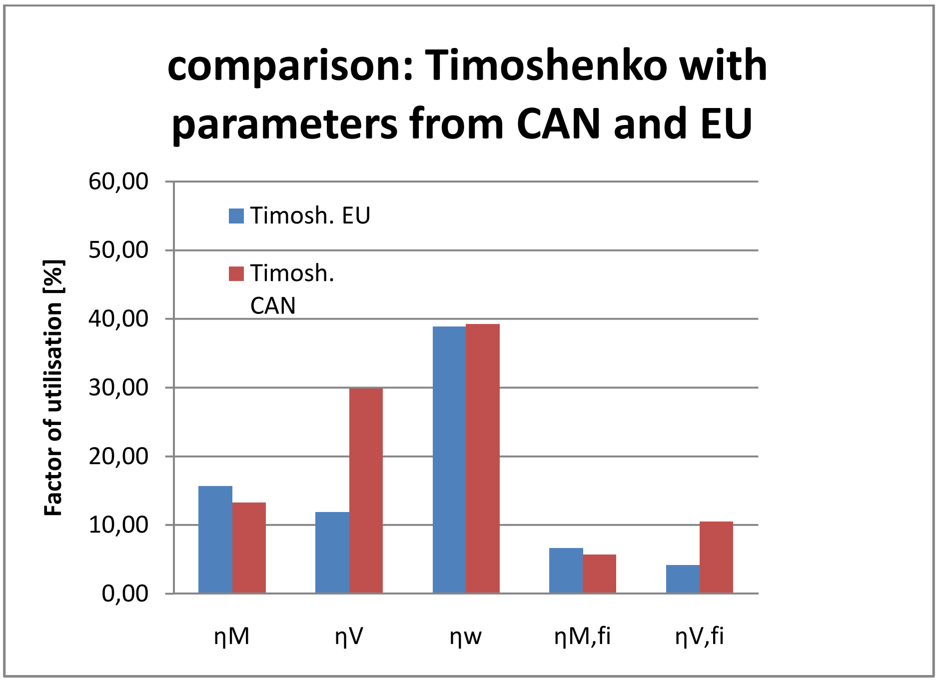


Diagram 7: Comparison of the calculation according Timoshenko with parameters form the Canadian and European Standard

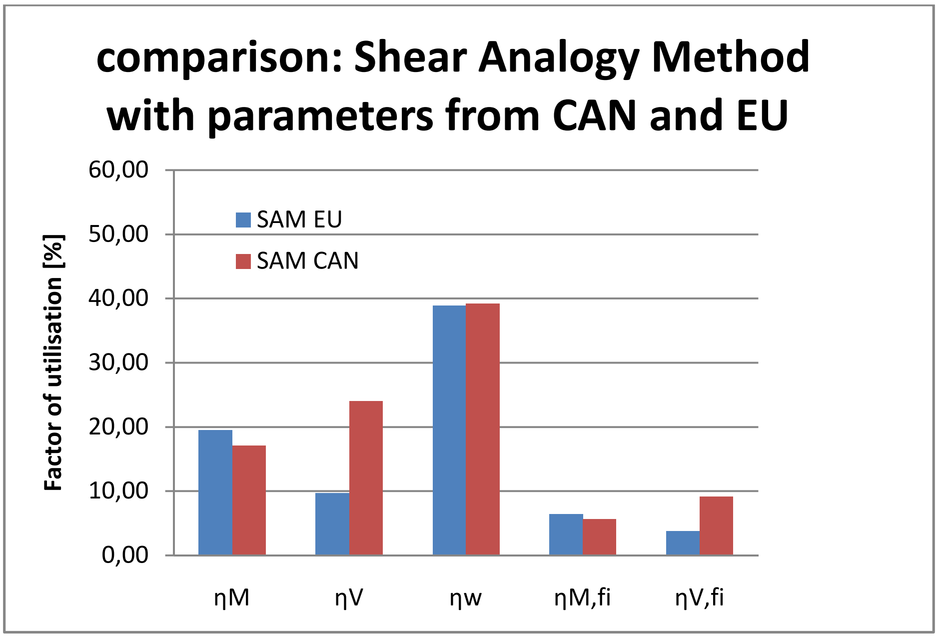


Diagram 8: Comparison of the calculation according the Shear Analogy Method with parameters from the Canadian and European Standard

where: Factors of utilisation according:

η_M... bending moment

η_V... shear force

$\eta_{c,90}$... Lateral pressure

η_w... deflection
$\eta_{M,fi}$... bending moment under fire
$\eta_{V,fi}$... shear force under fire

The degree of utilisation in bending is higher for the calculation according the European parameters. The verification of shear is more conservative for the Canadian parameters. This can be explained by the low values of the shear strength.

5.5 Comparison of the material

In this section shall be explained why the material E1 and GL24h* for the comparison was chosen.

The following table contains the material parameters for the CLT-grades E1 (ANSI PRG 320) and GL24h* (proposed in BSPhandbuch). The table also includes the bulk material for GL24h*, named C24. The CLT-grade E1 contains in longitudinal direction the material 1950f-1.7E spruce-pine-fir lumber and in transversal direction the material No. 3 spruce-pine-fir lumber.

E1 [N/mm²]			GL24h* [N/mm²]		C24 [N/mm²]
	1950f-1.7E	*No.3*			
$f_{b,0}$	28.2	*7.0*	$f_{m,k}$	24.0	24.0
$f_{c,0}$	19.3	*9.0*	$f_{c,0,k}$	24.0	21.0
$f_{c,90,k}$	5.3	*5.3*	$f_{c,90,k}$	2.7	2.5
$f_{t,0}$	15.4	*3.2*	$f_{t,0,k}$	16.5	14.0
$f_{t,90,k}$	0.5	*0.5*	$f_{t,90,k}$	0.5	0.4
$f_{v,0}$	1.5	1.5	$f_{v,k}$	3.0	2.7
$f_{s,0}$	0.5	0.5	$f_{r,k}$	1.2	1.0
			$E_{0,05}$	9667.0	7333.0
E_0	11700.0	*9000.0*	$E_{0,mean}$	11600.0	11000.0
E_{90}	390.0	*300.0*	$E_{90,mean}$	0.0	370.0
G_0	731.3	*562.0*	$G_{0,mean}$	720.0	690.0
G_{90}	73.1	*56.2*	$G_{90,mean}$	72.0	69.0

Table 24: Material grade for E1, GL24h* and C24

The parameters for bending strength and bending stiffness are similar for 1950f-1.7 and GL24h*. This was the main reason of the comparative calculation in the examples 5.2. Neither the parameters for tension and pressure aren't very different.

The values of the Canadian shear strength are much lower as the European approach. Therefore a conservative shear design was to expect.

Also from table 24 is to observe that only the values for bending strength, bending stiffness and shear stiffness (1950f-1.7 and GL24h*) are higher on the contrary to other values.

6 CHAPTER

6. CONCLUSION

6.1 Résumé

The beginning of this master thesis deals with the basis of the design of wood constructions according the Canadian and European standards. During this procedure the safety concept was examined. It stood out that the Canadian load combination factor for dead loads lies on 10% under the European factor. The partial safety factors or importance factors are also different to the European approach: during the European factors depend on the use of the building the Canadian values are defined with 0.5 for live loads and snow loads and 0.4 for wind loads.

The design methods according the Canadian CLT-Handbook of FPInnovations and the BSPhandbuch of the Institute of Timber Engineering and Wood Technology (Graz University of Technology) were examined. More exactly the "Simplified Method" according the Shear Analogy Method in the Canadian CLT-Handbook was observed. For the European approach the theory of Timoshenko and the Shear Analogy method were examined.

Further a parameter study was made. The above proposed methods and the parameters of the European (proposed in BSPhandbuch) and Canadian strength and stiffness values were compared. For this several examples were calculated.

Within the comparison of the methods it was demonstrated that due to the design values of the European safety concept a more critical design procedure is available. Further the calculation of the bending stress and the shear stress shows good agreements.

During the comparison of the material in the ULS verification stood out that the Canadian bending parameters are a little higher than the European values. Therefore a lower factor of utilisation in the calculation of verification is to expect. The Canadian rolling shear properties are much more lower in relation to the European approach. However the shear parameters (shear parallel to grain) come closer to the European values.

In the SLS verification it was to observe that the calculation of the deflection reveals similar results. Due to the limits for the verification of deflection vary within the design guidelines different combinations are significant. Therefore the factors of utilisation of the verification may not agree.

The Canadian and European verification of vibration depends on the frequency and the static deflection under a 1kN load. In addition the European values depend also on the acceleration of vibration. From the equations of the frequency and the static deflection the CLT-Handbook gives away a formula to limit the length of the panel to verify the vibration. The European verification limits the frequency, deflection and the acceleration of vibration by limit states of DIN 1052, EN 1995-1-1 and according Hamm/Richter.

Overview of the difference of the design methods

Here a summary of the calculation and design procedures shall be presented. Therefore the keywords of the Canadian and European proposed methods were confronted. With this listing a quick overview on the calculation and design guidelines shall be demonstrated.

Canadian Method	European Method
For the verification the **internal forces with the resistance forces** are compared	For the verification the **stresses** with the strength properties are compared
Use of resistance factor **ϕ** and **K-factors**	**k_{mod}**: includes qualities of service conditions and long term duration; other k-factors; **γ_M**
Combination factor for dead load **1.25**	Combination factor for dead load **1.35**
Use of **importance factors**	Partial system **ψ_i**
Values of strength: for **bending higher** and for **shear lower** in relation to the European approach	Values of strength: for bending lower and for shear higher in relation to the Canadian approach
Calculation of the stiffness for verification according Shear Analogy Method [**$(EI)_{eff}$ $(GA)_{eff}$**]	Calculation of the stiffness for verification with **K_{clt}** and **S_{clt}** or with Shear Analogy Method **$(EI)_{Aeff}$, $(EI)_{Beff}$** and **$(GA)_{B,eff}$**
Calculation of deflection only valid for simple beam under uniform load	Different methods to calculate deflection (Timoshenko ecc.)

Two options for Creep: I) suitable KD, KS factor, rolling shear G reduced; II) like Europe proposal, but different k_{def}	Creep: $\mathbf{k_{def}=0.85}$ for Service Class 1 and $\mathbf{k_{def}=1.1}$ for Service Class 2
Vibration: **limiting** of the element **length** (in dependence of frequency and static deflection)	Vibration: **limiting** of the **frequency**, limiting of the **deflection** and limiting of the **acceleration of vibration**
When theory according SAM: internal forces according beam theory	When theory according SAM: internal forces for beam A and beam B
Simplified Method: Addition of stiffness of the two beams ($(EI)_{Aeff}$ + $(EI)_{Beff}$)	Shear Analogy Method: Separate calculation: Addition of stresses

Table 25: Comparison of the design methods in Canada and Europe

6.2 Future works

The future prospects for FPInnovations are to work on the Canadian CLT-Handbook. In general the Chapter 8 for fire will be updated. For that and to improve the Canadian Standard more experiments will be done.

Also cooperation with the USA is underway. FPInnovations will produce a CLT-Handbook for USA in cooperation with partners from USA.

The research for timber construction in Europe is aimed to create a Eurocode conform standard for CLT. Therefore the Institute for timber construction and timber technology and the competence center holz.bau forschungs gmbh is working on a series of experiments. Furthermore a standardisation of CLT-products for architects, engineers and carpenters is planned.

LIST OF TABLES

LIST OF FIGURES

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