CLT Handbook

CROSS-LAMINATED TIMBER

CANADIAN EDITION

FPInnovations

CLT Handbook

CROSS-LAMINATED TIMBER

Edited by
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and **Ciprian Pirvu**

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PREFACE

FPInnovations' Building Systems Research Program has been generating technical data to facilitate:

- Platform Frame Wood Construction
- Heavy Timber Frame Construction
- Cross-Laminated Timber Construction

Multi-disciplinary teams working in cooperation with the design and construction community and research alliances have contributed greatly to the application of Platform Frame and Heavy Timber Frame systems together with hybrid systems in Canada.

Cross-laminated timber (CLT), an emerging successful system from Europe, has been identified by the forest products industry, the research and wood design communities as a new opportunity for increasing the use of wood in non-traditional applications.

Building on the European experience, FPInnovations has prepared this peer-reviewed CLT Handbook to:

- Provide immediate support for the design and construction of CLT systems as alternative solutions in building codes;
- Provide technical information for implementation of CLT systems in building codes and standards.

This FPInnovations CLT Handbook, prepared under the Transformative Technologies Program of Natural Resources Canada, provides technical information relating to manufacturing, all aspects of design and construction, and environmental considerations.

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Richard Desjardins, M.Sc., Eng., FPInnovations

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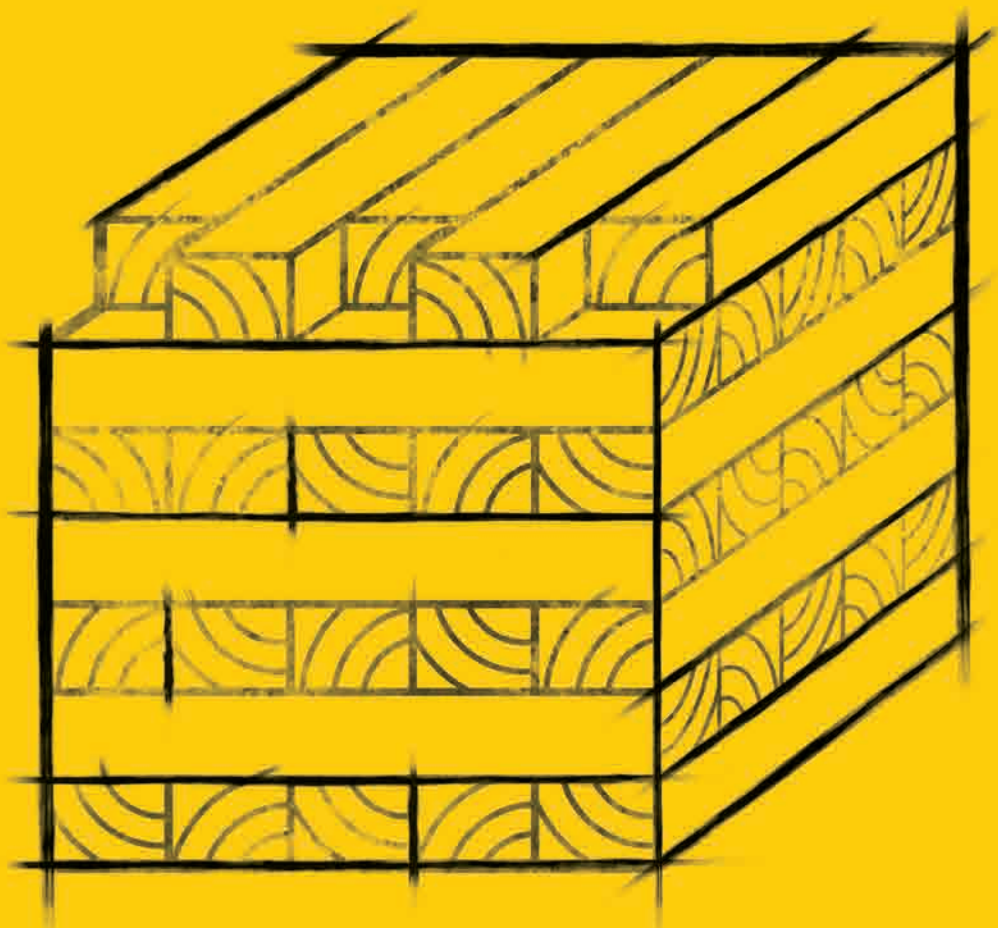

CLT
Introduction to cross-laminated timber

CHAPTER 1

Author

FPInnovations

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ABSTRACT

Cross-laminated timber (CLT), an innovative engineered wood product developed in Europe, has been gaining increasing popularity in residential and non-residential applications in several countries. Numerous impressive buildings built around the world using CLT have become a good testimony of the many advantages that this product can offer to the construction sector. In order to gain wide acceptance, cross-laminated timber, as a product and structural system, needs to be implemented in the North American codes and standards.

This chapter puts forward an introduction to CLT as a product and the CLT construction in general, along with different examples of buildings made of CLT panels. A road map for codes and standards implementation of CLT in North America is also included.

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1 BRIEF HISTORY

Cross-laminated timber (CLT) is an innovative wood product that was first developed in Austria and Germany and ever since has been gaining popularity in residential and non-residential applications in Europe. There are currently several CLT producers in Europe.

In the mid 1990s, Austria undertook an industry-academia joint research effort that resulted in the development of modern CLT. For several years, progress was slow but in the early 2000s, construction in CLT increased significantly, partially driven by the green building movement but also due to better efficiencies, product approvals, and improved marketing and distribution channels. Another important factor has been the perception that CLT is a 'non-light' construction system, like masonry and concrete, which are extensively used in residential construction in many European countries.

The use of CLT panels in buildings has increased over the last few years in Europe. Numerous impressive buildings and other types of structures built around the world using CLT have become a good testimony of the many advantages that this product can offer to the construction sector. The easy handling in construction and the high level of prefabrication involved that facilitate a quick erection time are just some of the key advantages, especially in mid-rise construction (e.g. 5 to 8 storeys). Good thermal insulation, good sound insulation and a fairly good performance under fire conditions are added benefits that come as a result of the massiveness of the wood structure.

While this product is well-established in Europe, work on the implementation of CLT products and systems has just begun in Canada and the United States. The use of CLT in North America is gaining interest in both the construction and wood industries. Several North American manufacturers are in the process of product and manufacturing assessment or have already started pilot production.

In this chapter, we put forward an introduction to CLT as a product and the CLT construction in general, along with different examples of buildings and other types of structures made with CLT panels. A road map for codes and standards implementation of CLT in North America is also included in this chapter.

2

FPINNOVATIONS RESEARCH PROGRAM AND MOTIVATION

The European experience showed that CLT construction can be competitive, particularly in mid-rise and high-rise buildings. Although CLT has barely been used in North America to date, it could be used as a viable wood-based structural solution for the shift towards sustainable densification of urban and suburban centres in Canada and the USA. In order to gain much needed wide acceptance and popularity, CLT, as a structural system, needs to be implemented in the North American codes and standards.

Under the Transformative Technologies Program of Natural Resources Canada, FPInnovations launched a multi-disciplinary research program on CLT in 2005. Based on studies and the knowledge gained from the European experience, FPInnovations has prepared this peer-reviewed CLT Handbook. Most of the work included in this Handbook has been peer-reviewed by national and international well skilled experts in wood design and construction.

Moreover, in support of FPInnovations' research activities on CLT and other new generation building systems, a new NSERC network (NEWBuildS) has been established with CLT being one of its four themes. The CLT research under the network is focused on the manufacturing and performance issues of CLT products and assemblies. Research on hybrid construction where CLT is used together with wood-based or non-wood materials is also covered under the various themes. The research is conducted by several Canadian universities in close collaboration with FPInnovations researchers.

This Handbook provides key technical information related to the manufacturing, design and performance of CLT in construction in the following areas:

- Cross-laminated timber manufacturing
- Structural design of cross-laminated timber elements
- Seismic performance of cross-laminated timber buildings
- Connections in cross-laminated timber buildings
- Duration of load and creep factors for cross-laminated timber panels
- Vibration performance of cross-laminated timber floors
- Fire performance of cross-laminated timber assemblies
- Acoustic performance of cross-laminated timber assemblies
- Building enclosure design of cross-laminated timber construction
- Environmental performance of cross-laminated timber

Finally, this comprehensive Handbook provides immediate support for the design and construction of CLT systems as alternative solutions in building codes. Additionally, it provides technical information for implementing CLT systems in building codes and standards.

Note: This document was developed using a series of reports prepared by FPInnovations to support the introduction of CLT in the North American market. The information contained in these reports represents current research results and technical information made available to FPInnovations from many sources, including researchers, wood product manufacturers, and design professionals. The information has been reviewed by staff and others including design engineers and architects, and wood product manufacturers. Although every reasonable effort has been made to make this work accurate and authoritative, FPInnovations does not warrant and assumes no liability for the accuracy or completeness of the information or its fitness for any particular purpose. It is the responsibility of users to exercise professional knowledge and judgment in the use of the information.

3

BRIEF DEFINITION OF CROSS-LAMINATED TIMBER (CLT)

CLT panels consist of several layers of boards stacked crosswise (typically at 90 degrees) and glued together on their wide faces and, sometimes, on the narrow faces as well. A cross-section of a CLT element has at least three glued layers of boards placed in orthogonally alternating orientation to the neighboring layers. In special configurations, consecutive layers may be placed in the same direction, giving a double layer (e.g. double longitudinal layers at the outer faces and additional double layers at the core of the panel) to obtain specific structural capacities. CLT products are usually fabricated with three to seven layers and even more in some cases. Figure 1 illustrates a CLT panel configuration while Figure 2 shows examples of possible CLT panel cross-sections. Figure 3 illustrates a 5-layer CLT panel with its two cross-sections.

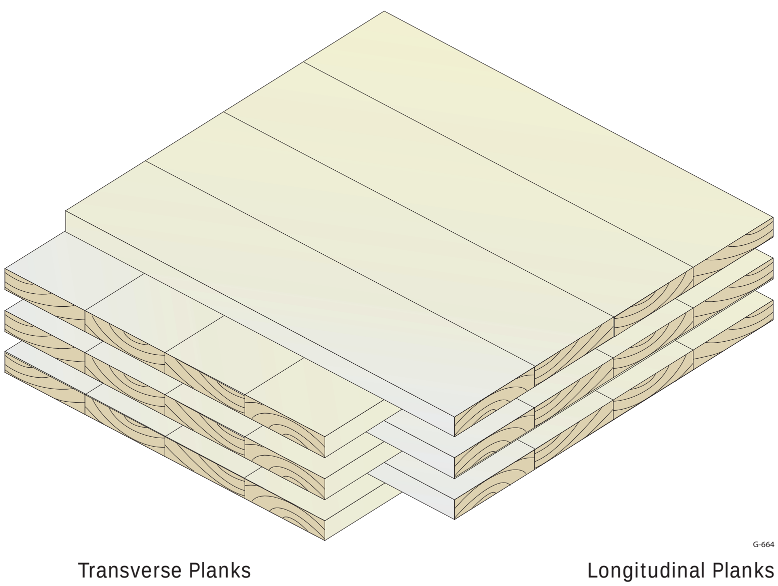


Figure 1
CLT panel configuration

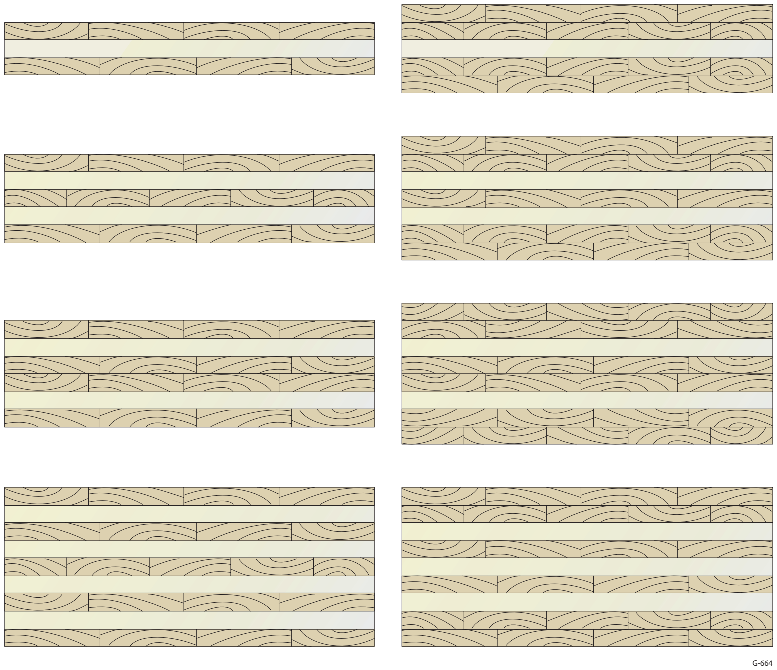


Figure 2
Examples of CLT panel cross-sections

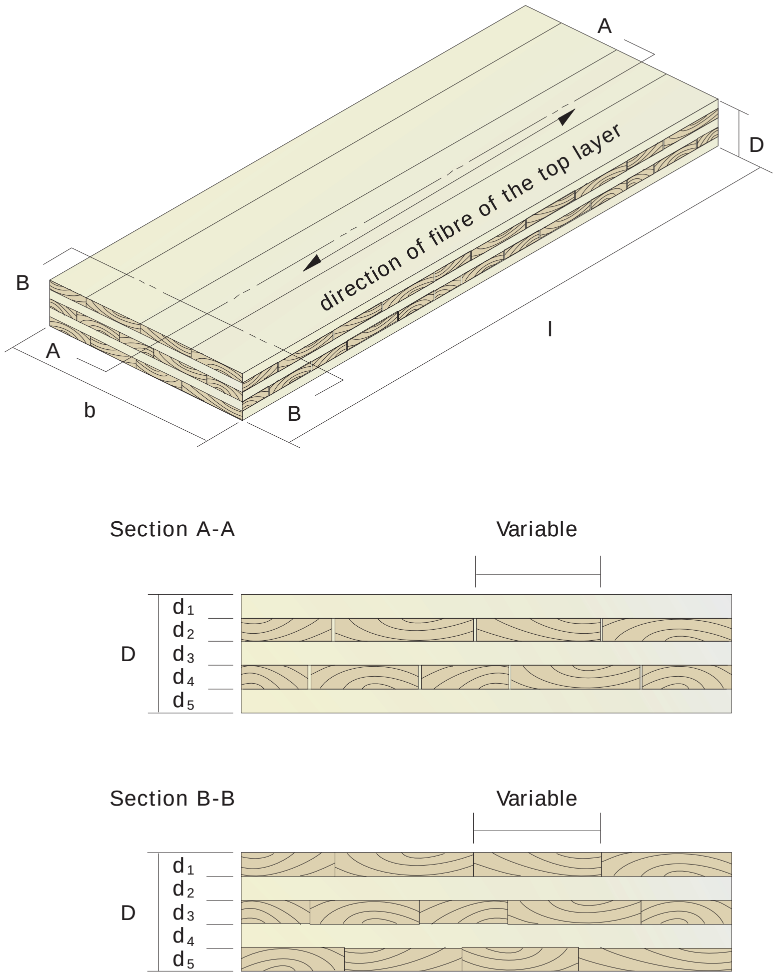


Figure 3
Example of CLT panel cross-sections and direction of fibres of the top layers

Thickness of individual boards may vary from 10 mm to 50 mm and the width may vary from about 60 mm to 240 mm. Boards are fingerjoined using structural adhesive. Boards are visually or machine stress-rated and are kiln dried. Panel sizes vary by manufacturers; typical widths are 0.6 m, 1.2 m, and 3 m (could be up to 4~5 m in particular cases) while length can be up to 18 m and the thickness can be up to 400 mm. Transportation regulations may impose limitations to CLT panel size.

The lumber or boards in the outer layers of CLT panels used as walls are normally oriented parallel to vertical loads to maximize the wall resistance. Likewise, for floor and roof systems, the outer layers run parallel to the major span direction.

CLT panels used for prefabricated wall and floor structures offer many advantages. The cross-laminating process provides improved dimensional stability to the product which allows for prefabrication of wide and long floor slabs and single storey long walls. Additionally, cross-laminating provides relatively high in-plane and out-of-plane strength and stiffness properties in both directions, giving it a two-way action capability similar to a reinforced concrete slab. The 'reinforcement' effect provided by the cross lamination in CLT also considerably increases the splitting resistance of CLT for certain types of connection systems.

4

SOME BENEFITS OF CROSS-LAMINATING

Figure 4 illustrates the primary difference between CLT and glued-laminated timber products. Figure 5a shows a floor built with four individual CLT panels acting mostly in one direction, while Figure 5b illustrates the same floor, this time built with one CLT panel only acting most likely in two directions (i.e. two-way action).

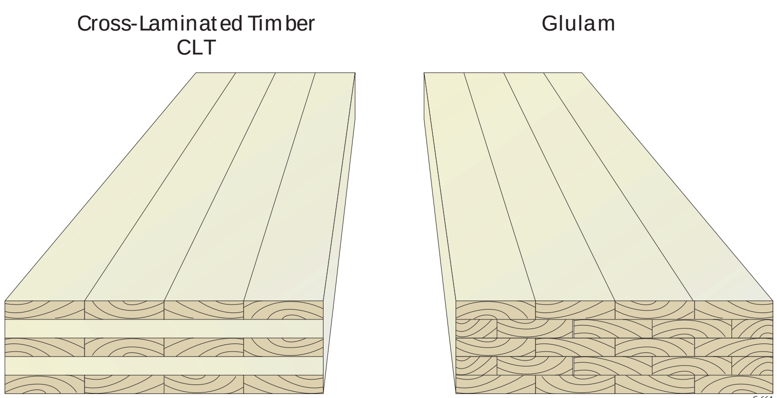
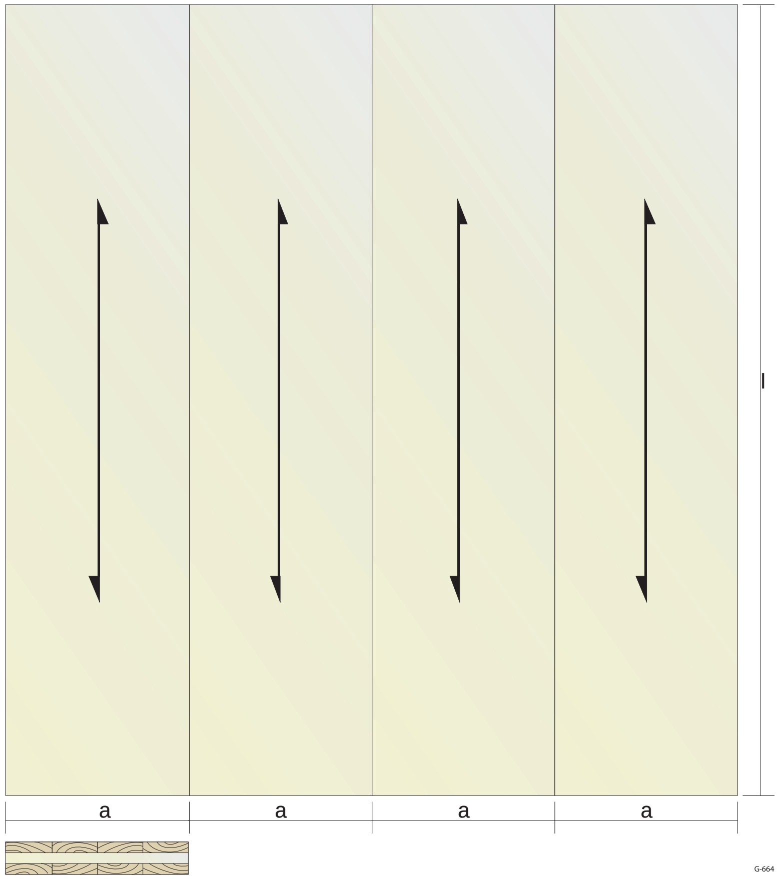
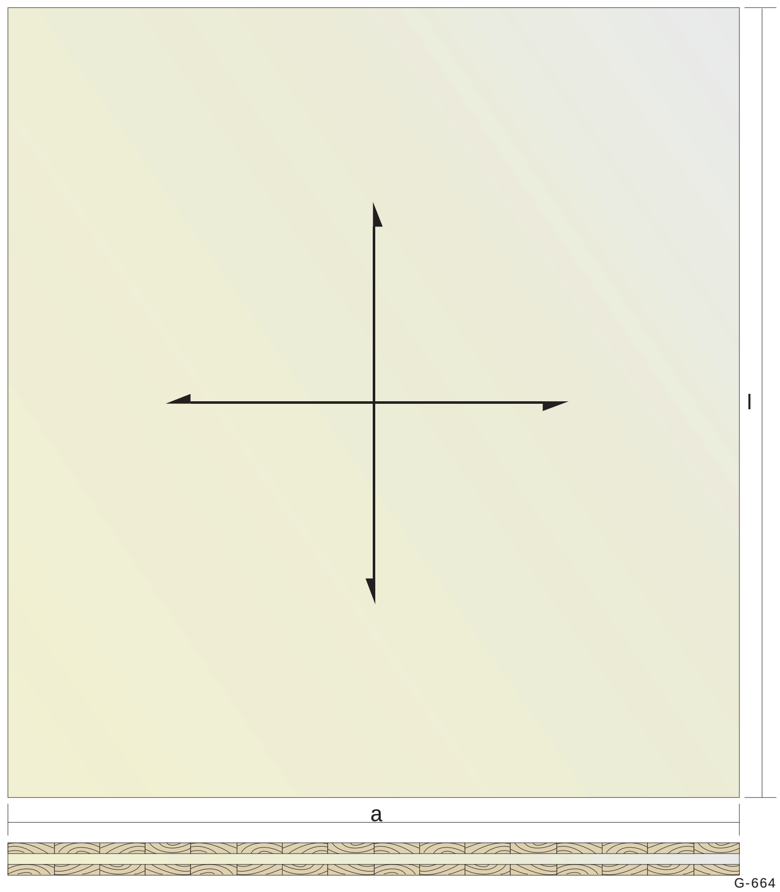


Figure 4
CLT panel vs. glued-laminated timber



(a)



(b)

Figure 5
(a) Floor assembly made of four 3-ply CLT panels acting in one direction
(b) Floor assembly made of one 3-ply CLT panel acting in both directions
Distance “a” may reach 4 meters

5 MANUFACTURING PROCESS

A typical manufacturing process of CLT includes the following steps: lumber selection, lumber grouping and planing, adhesive application, panel lay-up and pressing, and product cutting, marking and packaging. The key to a successful CLT manufacturing process is consistency in the lumber quality and control of the parameters that impact on the quality of the adhesive bond. Stringent in-plant quality control tests are required to ensure that the final CLT products will fit for the intended applications.

Chapter 2 entitled *Cross-Laminated Timber Manufacturing* provides general information about CLT manufacturing targeted mainly to engineers, designers, and specifiers. The information contained in this chapter may also be useful to potential CLT manufacturers.

Seed documents prepared by FPInnovations for a North American CLT product standard are presented and used as an example for discussing how CLT panel quality may be evaluated.

Figure 6 illustrates a typical CLT wall assembly, and Figure 7 illustrates a typical CLT floor or roof assembly.

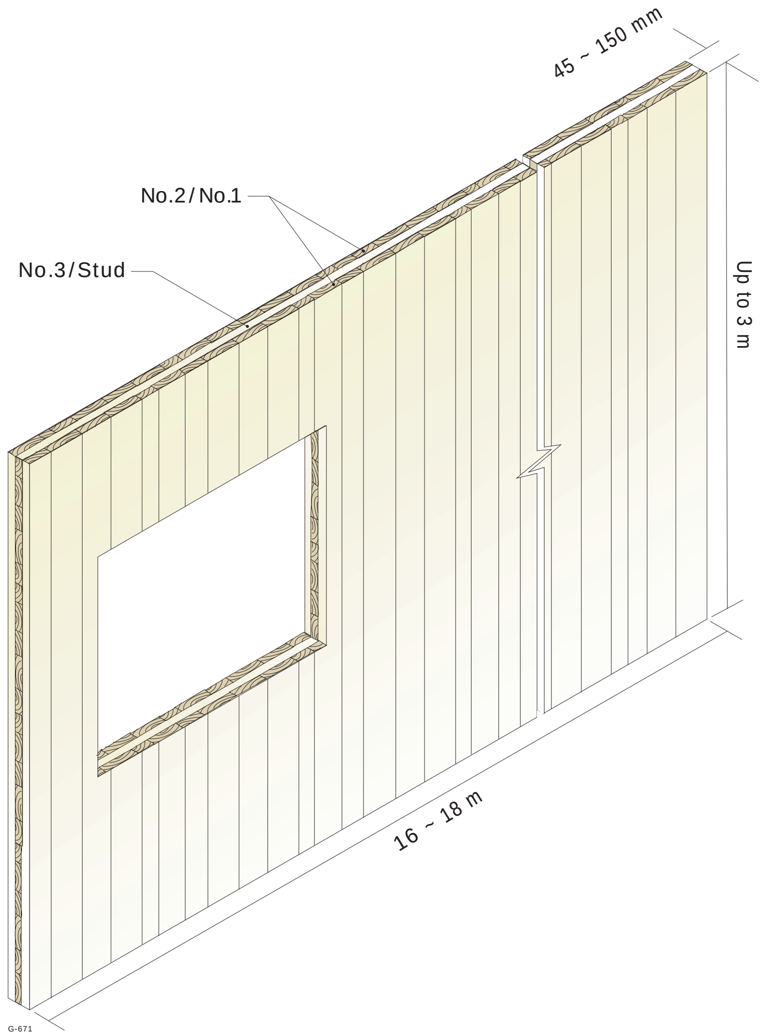


Figure 6
CLT wall assembly

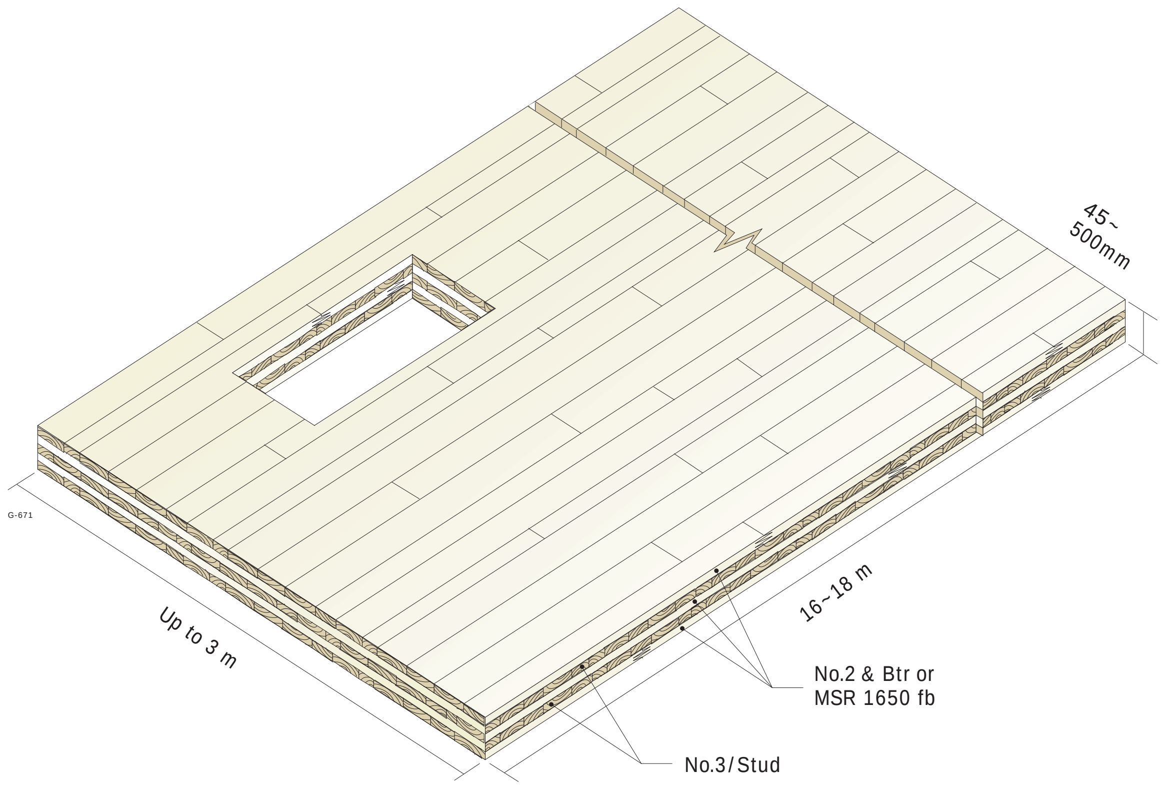


Figure 7
CLT floor or roof assembly

6 STRUCTURAL DESIGN AND SERVICEABILITY CONSIDERATIONS OF CLT

CLT panels are typically used as load-carrying plate elements in structural systems such as walls, floors and roofs. Basically, the objectives are to provide a structure which is safe and serviceable to use, economical to build and maintain, and satisfactorily performs its intended function.

For floor and roof CLT elements, key critical characteristics that must be taken into account are the following:

- In-plane and out-of-plane bending and shear strength and stiffness
- Short-term and long-term behaviour:
  - instantaneous deflection
  - long-term strength for permanent loading
  - long-term deflection (creep deformation)
- Vibration performance of floors
- Compression perpendicular to grain strength (bearing)
- Fire performance
- Acoustic performance
- Durability

For wall elements, the load-bearing capacity is critical and shall be verified together with the in-plane and out-of-plane shear and bending strength. In addition, fire and acoustic performance along with the durability of the system are key characteristics that must be taken into account at the design stage.

6.1 Proposed Analytical Design Methods

Different design methods have been adopted in Europe for the determination of basic mechanical properties of CLT. Some of these methods are experimental in nature while others are analytical. For floor elements, experimental evaluation involves determination of flexural properties by testing full-size panels or sections of panels with a specific span-to-depth ratio. The problem with the experimental approach is that every time the lay-up, type of material, or any other manufacturing parameters change, more testing is needed to evaluate the bending and shear properties of such new product configurations.

However, the analytical approach, once verified with the test data, offers a more general and less costly alternative. An analytical approach generally predicts strength and stiffness properties of CLT panels based on the input material properties of the laminate boards that make up the CLT panel.

Proposed analytical procedures for determining the basic mechanical properties of CLT panels in timber construction are given in the Chapter 3 entitled *Structural Design of Cross-Laminated Timber Elements*.

6.2 Seismic Performance of CLT Buildings

Based on the literature review of the research work conducted around the world and the results from a series of quasi-static tests on CLT wall panels conducted at FPInnovations, CLT wall panels can be used as an effective lateral load resisting system. Results to date have shown that the CLT wall panels demonstrated adequate seismic performance when nails or slender screws are used with steel brackets to connect the walls to the floors below. The use of hold-downs with nails on each end of the walls tends to further improve their seismic performance. Use of diagonally placed long screws to connect CLT walls to the floor below is not recommended in high seismic zones due to lower ductility and brittle failure mechanism. Use of step joints in longer walls can be an effective solution not only to reduce the wall stiffness and thus reduce the seismic input load, but also to improve the wall deformation capabilities. Timber rivets in smaller groups with custom made brackets were found to be effective connectors for CLT wall panels. Further research in this field is needed to further clarify the use of timber rivets in CLT and to verify performance of CLT walls with alternative types of connection systems (e.g. bearing types).

While most CLT buildings are of a platform type of structural system, they are far less susceptible to develop soft storey mechanisms than many other structural systems of the same type. Since the nonlinear behaviour (and the potential damage) is localized in the hold-down and bracket connection areas only, the panels that are also the vertical load carrying elements are virtually left intact in place, and well connected to the floor panels, even after a "near collapse" state is reached. In addition, in CLT construction, all walls in one storey contribute to the lateral and gravity resistance, thus providing a degree of redundancy and a system sharing effect.

Preliminary evaluation of the force modification factors (R-factors) for the seismic design of structures according to the National Building Code of Canada (NBCC) was also performed. Based on the experimental and analytical research work conducted in this field in Europe and at FPInnovations, the performance comparison to already existing systems in NBCC and on the equivalency performance criteria given in ICC-ES Acceptance Criteria (AC130), proposed force modification factors (R-factors) for the seismic design of CLT structures are provided in Chapter 4 entitled *Seismic Performance of Cross-Laminated Timber Buildings*.

Figure 8
Seven-storey CLT house tested at E-Defense Laboratory in Miki, Japan as a part of the SOFIE Project

6.3 Connections and Construction of CLT Structures

Connections in timber construction, including those built with CLT, play an important role in maintaining the integrity of the timber structure and in providing strength, stiffness, stability and ductility. Consequently, they require a thorough attention of the designers.

Traditional and innovative connection systems have been used in CLT assemblies in Europe. Several types of traditional and innovative connection systems for connecting CLT panels to panels, walls to walls and walls to floors are described in details in Chapter 5 entitled *Connections in Cross-Laminated Timber Buildings*.

Researchers in Europe have developed design procedures for traditional connections in CLT, including dowels, wood screws and nails which are commonly used in Europe for designing CLT assemblies. The proposed European design procedure provided in Chapter 5 deals only with ductile failure modes to determine the lateral load resistance of such connections. Empirical expressions were developed for the calculation of characteristic embedment properties of each type of fastener, depending on its location with respect to the plane of the panel (perpendicular to or on edge). Those expressions were verified by European researchers and results seem to correspond well with predictions. European Yield Model (EYM) equations as given in Eurocode 5 were adopted for the design using CLT fastener embedment strength equations.

Information on the applicability of the proposed design approach from Europe to traditional connection systems in CLT in Canada are also presented in Chapter 5. It is believed that once the embedment properties of such fasteners in CLT are established, it would be possible to apply current ductile design provisions in CSA O86-09. Due to the reinforcing effect of cross lamination in CLT, it is speculated that current minimum geometric requirements given in CSA O86-09 for dowels, screws and nails in solid timber or glulam are applicable to CLT. However, designers need to be cautious about this as further verification is required, considering the specific features of each panel. Brittle failure modes also need to be taken into account which has not been investigated yet. Further work is needed to verify possible brittle failure modes associated with each type of fasteners in CLT connections, especially for closely spaced fasteners.

Chapter 5 is mainly focused on CLT assemblies. But, since all buildings are considered to be mixed constructions to a certain extent, the scope covers hybrid constructions, where other wood-based systems (e.g. light frame, glulam, etc.) or combined with other materials such as concrete or steel are mixed with CLT to resist certain types of loads, such as the lateral loads.

6.4 Duration of Load and Creep Behaviour

Duration of load is defined as the duration of continuing application of a load or a series of periods of intermittent applications of the same load (CSA O86-09, 2009). Creep is defined as an increase in deformation of a material in time under constant loading, which translates into an increase in deformation over time.

Given the nature of CLT, with orthogonal arrangement of layers and either mechanically fastened with nails or wood dowels, or bonded with structural adhesive, it is more prone to time-dependent deformations under load (creep) than other engineered wood products such as glued-laminated timber. Therefore, special attention must be paid to the duration of load and creep behaviour of such products. Long-term behaviour of structural wood products is accounted for in the Canadian design standard by using load duration factors which are applied to adjust specified strengths. Since CLT is not covered by the CSA O86-09, load duration and service factors for CLT do not exist in CSA O86-09.

Two main options for early adopters of CLT systems in Canada are proposed in Chapter 6, entitled *Duration of Load and Creep Factors for Cross-Laminated Timber Panels*. In addition to the load duration and service factors, tentative solutions for taking into account creep in CLT structural elements are proposed. These recommendations are in line with the specifications of CSA O86-09 and NBCC.

6.5 Vibration Performance of Floors

Laboratory and field tests on CLT floor assemblies have indicated that the vibration behaviour of CLT floors is different from lightweight wood joisted floors and heavy concrete slab floors. A proposed design methodology for controlling vibrations of CLT floors under normal walking is given in the Chapter 7 entitled *Vibration Performance of Cross-Laminated Timber Floors*.

6.6 Fire Performance of Cross-Laminated Timber Assemblies

CLT panels have the potential to provide good fire resistance, often comparable to typical massive assemblies of non-combustible construction. This is due to the inherent nature of thick timber members to slowly char at a predictable rate, allowing massive wood systems to maintain significant structural capacity for extended durations when exposed to fire.

In order to facilitate the acceptance of future code provisions for the design of CLT panels with regard to fire resistance, a one-year research project has been launched at FPInnovations as of April 2010. The main objective of the project is to develop and validate a generic calculation procedure to calculate the fire resistance ratings of CLT wall and floor assemblies. A series of full-scale fire resistance experiments is currently underway to allow a comparison between the fire resistance rating measured during a standard fire resistance test and that calculated using the proposed procedure. In light of the fact that the research project has just begun, a simple but conservative design procedure is presented in Chapter 8 of the Handbook, entitled *Fire Performance of Cross-Laminated Timber Assemblies*, following the current state-of-the-art information from Europe and North America.

The Canadian Standard for Engineering Design in Wood (CSA O86) can be used to calculate the fire resistance rating of CLT panels along with the same methodology that is currently used for calculating the fire resistance ratings of glulam and "heavy" timber in the United States, New Zealand and Europe. This method is called the reduced (or effective) cross-section method and allows the use of the design values that can be found in the wood design standard, CSA O86. The calculation procedure described in Chapter 8 should be employed by a fire protection engineer familiar with wood design.

6.7 Acoustic Performance of Cross-Laminated Timber Assemblies

Adequate levels of noise/sound control in multi-family buildings are mandatory requirements of most building codes in the world. In many jurisdictions, these requirements are as strictly enforced as those for structural sufficiency and fire safety. Much effort has been spent on evaluation of sound transmission class (STC) and impact sound insulation class (IIC) of floor and wall assemblies and on studies of flanking transmission in multi-family dwellings in Canada. However, limited amount of work has been done in Canada on the acoustic performance of CLT systems.

In Chapter 9, entitled *Acoustic Performance of Cross-Laminated Timber Assemblies*, it is demonstrated that CLT floor and wall assemblies made of CLT elements can have good acoustical performance in residential, multi-residential and non-residential buildings. Estimated STC and IIC ratings of existing generic floor assemblies used in Europe are provided for benchmarking. Figures 9 and 10 show examples of floors using sound insulation systems.

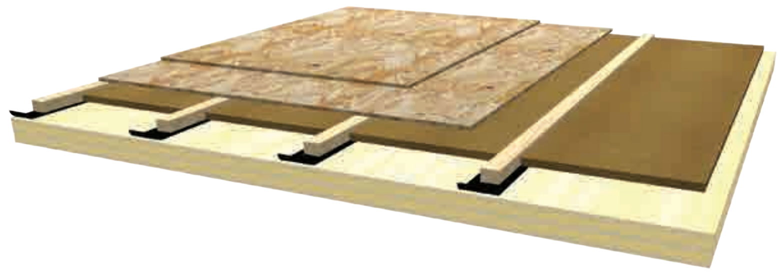


Figure 9
CLT floor sound-insulated by the top

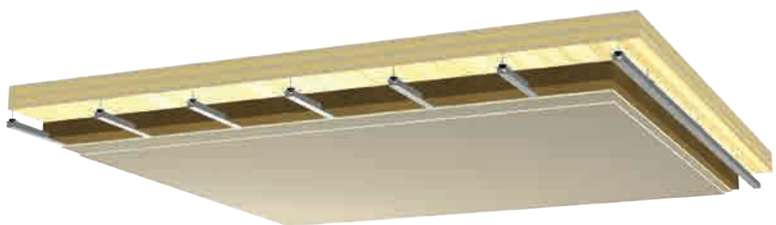


Figure 10
CLT floor sound-insulated by the bottom

6.8 Building Enclosure Design of Cross-Laminated Timber Construction

The design of CLT panels for building enclosure in North America requires considerable efforts in order to ensure their long-term durability, particularly in areas with high moisture loads such as the coastal regions. Like other wood products, the key to CLT durability is to keep it dry. There is a potential for slow drying due to the big mass of wood in the product once moisture gets into the panel. Since CLT has been used for prefabrication in Europe for over a decade, a lot of attention has been paid to protecting the CLT panels from getting wet during transportation or construction. One way of controlling the wetting during transport of CLT elements is to use closed containers. As for assembly, Europeans have adopted several methods for controlling moisture during construction. Delivering CLT panels just on time for assembly to minimize construction time is just one strategy. Other methods that have been successfully implemented in Europe involve the construction of a temporary roofing system to protect against rain and snow during construction. Other methods involve building the actual roof system of the structure on ground, then jack it up as the building goes up. Figure 11 shows a system of tent used in Sweden during the construction of an 8-storey CLT building.

CLT is usually not manufactured to be exposed to exterior environment and the panels should be protected from rain and high relative humidity levels with a properly designed building envelope. Like other wood construction types, the use of basic measures such as overhangs and the integration of drained and ventilated rainscreen walls will effectively prevent rain penetration into building assemblies. In addition, appropriate design and application of insulation materials, air control and vapour control strategies, as well as ground moisture control measures are needed. Such measures will ensure that the panels will be kept warm and dry, help prevent moisture from being trapped and accumulated within the panels during the service life, and ensure the energy efficiency of CLT building enclosure. Chapter 10 provides details on durability aspects of CLT.

Figure 11
Eight-storey building under construction protected by a tent

7 ENVIRONMENTAL PERFORMANCE OF CROSS-LAMINATED TIMBER

The environmental footprint of CLT is frequently discussed as potentially beneficial when compared to functionally equivalent concrete systems. Inherent to that discussion is an assumption that the comparative environmental profile of CLT will be lower, based on the generic life cycle analysis (LCA) profiles of wood and concrete. In particular, CLT (because it is made of wood), is assumed to have a light carbon footprint, due to relatively low embodied greenhouse gas (GHG) emissions in wood versus concrete, and due to the carbon storage capacity of wood products.

Existing environmental comparisons between wood and concrete buildings generally focus on light wood framing using lumber, whereas CLT is a massive structural system involving at least three times more wood material, and added processing and auxiliary materials such as adhesives as with any engineered (composite) wood products. In other words, the footprint of a CLT building is not the same as a light-frame building, and we therefore cannot simplistically assume CLT will compare as favourably to concrete as light framed wood.

In Chapter 11, efforts are focused on quantifying the environmental footprint of CLT. Given the early stages of CLT research and development efforts in North America, this work is considered very much preliminary in nature. Some of the quantified environmental characteristics of CLT as a construction material, without conducting a full life cycle assessment (LCA), are presented in the chapter. Since no existing comparative literature on CLT has been found, efforts have been focused on trying/developing several approaches to estimate the footprint of CLT and its comparison to concrete. Using existing LCA data on Canadian glulam as a proxy, the footprint of the material itself compared to the materials in reinforced concrete, and of the material in a midrise building compared to concrete was examined. Modified glulam LCA data were used to approximate an LCA for a CLT floor section and compare it to a functionally equivalent concrete floor section. In all these cases, it is estimated that the CLT will substantially outperform concrete in every environmental metric addressed by LCA.

Finally, CLT products with different thicknesses and glue lines were tested for their volatile organic compounds (VOCs) including formaldehyde and acetaldehyde emissions in order to assist engineers and builders to better select their construction materials with less impact on indoor air quality. Emissions were evaluated according to ASTM D 5116 and were collected after 24 hours of samples exposure in the small chamber. Results are given in Chapter 11 entitled *Environmental Performance of Cross-Laminated Timber.*

8

CODES AND STANDARDS ROAD MAP FOR CLT

Under Natural Resources Canada (NRCan)'s Transformative Technologies and provincial Programs, FPInnovations has been playing a pivotal role in the identification of Next Generation Wood Building Systems to facilitate the expanded wood use outside the traditional housing market by:

- Going to new heights in Platform Frame Wood Construction (up to 6 storeys)
- Revival of Heavy Timber Frame Construction (up to 10 storeys)
- Adoption of CLT from Europe (up to 10 storeys)
- Facilitating Hybrid Construction

Notable initial successes were achieved in Canada where a number of 5- and 6-storey residential and office buildings have been built, and the discussion about changes in building codes to allow greater use of wood systems was initiated. The British Columbia Building Code already made a revision to allow platform wood-frame residential construction up to 6 storeys, and Québec allowed the construction of a heavy timber/concrete hybrid office building.

The implementation of CLT in the regulatory systems in Canada and the United States requires a multi-level strategy that includes development of a product standard, material design standard(s) and adoption of CLT on building code levels. The current state and projected short term and medium term activities of the standardization processes in those three levels are summarized in Table 1. Energy and green codes are also important aspects of regulatory systems and will be included in the future.

Table 1

Current and projected codes and standards activities for CLT

	Current Situation (November 2010)	Short Term (1 year)	Medium Term (5 years)
Product Standard Level	Proprietary route European Draft FPInnovations Drafts APA Draft ISO Work Item	Proprietary acceptance as floor/roof/wall assemblies European Draft APA/ANSI Standard ISO Draft	Proprietary CLT CLT strength classes European Standard APA/ANSI Standard ISO Standard
Material Design Standard Level	FPInnovations' CLT Handbook and other peer-reviewed information	CWC and AWC initiate the process based on FPInnovations' Handbook and other peer-reviewed information	Acceptance in the 2014 edition of CSA 086 in Canada Acceptance in the NDS in the USA
Building Code Level	FPInnovations' CLT Handbook and other peer-reviewed information	Proprietary acceptance CWC and AWC initiate the building code process based on FPInnovations' CLT Handbook and other peer-reviewed information	Acceptance in the 2015 NBCC in Canada Acceptance in the 2015 IBC in the USA

FPInnovations drafted CLT Plant Qualification and Product Standards (see Appendix in Chapter 2 for more details) and passed them to the ANSI-accredited APA Committee to be used as seed documents for the development of a single North American product standard that could be used as a basis for an ISO standard that would harmonize North American and European standards. An ISO Task Group was formed under the ISO Technical Committee on Timber Structures for the development of an ISO Standard for CLT.

It is anticipated that CLT manufacturers will use the proposed standards to gain acceptance of proprietary CLT products by code-recognized evaluation services (e.g. CCMC, ICC-ES, NTA, IAMPO).

A North American Advisory Committee on CLT was formed to advance the implementation of CLT technology. The Advisory Committee formed a Research/Standards Subcommittee so that the related activities on CLT can be streamlined. Based on the initial assessment, seismic and fire design issues have been identified as the most important ones to address. American Wood Council (AWC) and Canadian Wood Council (CWC) already initiated the process of implementing CLT in the material codes.

This CLT Handbook prepared by FPInnovations and its research collaborators includes structural (including seismic and connections) and fire design, vibration characteristics, sound transmission, building envelope and environmental performance of CLT to:

- provide immediate support for the design and construction of CLT systems as alternative solutions in building codes;
- provide technical information for implementation of CLT systems in building codes and standards.

In order to ensure that CLT is as easy to specify as non-wood systems, a strength class system (with few classes similar to steel and concrete) incorporating many CLT products is highly recommended for implementation in the next code cycle. The use of a stress class system will allow a designer to do a conceptual design using CLT panels with desired capacities readily available. This reduces the cost of design and should help ensure a positive reception of CLT by the design community.

9 CROSS-LAMINATED TIMBER IN CONSTRUCTION

The purpose of this section is to present selected interesting examples of buildings built around the world using CLT elements.

9.1 Residential Buildings

Figure 12
Single-family house in Rykkinn, Norway

Figure 13
Single-family house in Oslo, Norway

Figure 14
Single-family house in Klagenfurt, Austria (courtesy of KLH)

Figure 15
Multi-family building in Judenburg, Austria (courtesy of KLH)

Figure 16
Multi-family building in Berlin, Germany

Figure 17
Multi-family building in Växjö, Sweden

Figure 18
Multi-family building in London, United Kingdom
(courtesy of KLH and Waugh-Thistleton)

Figure 19
Multi-family building in L'Aquila, Italy (courtesy of Binderholz)

9.2 Office and Commercial Buildings

Figure 20
Impulsezentrum, Graz, Austria (courtesy of KLH)

Figure 21
Viken Skog BA, Hønefoss, Norway (courtesy of Moelven)

Figure 22
Juwi head office, Wörrstadt, Germany (courtesy of Binderholz)

Figure 23
Workshop, Fügen, Austria (courtesy of Binderholz)

Figure 24
Warehouse, Katsch, Austria (courtesy of KLH)

9.3 Hybrid Structures

Figure 25
Residential building in South Carolina, USA (courtesy of Binderholz)

Figure 26
Parking Garage in Innsbruck, Austria (courtesy of KLH)

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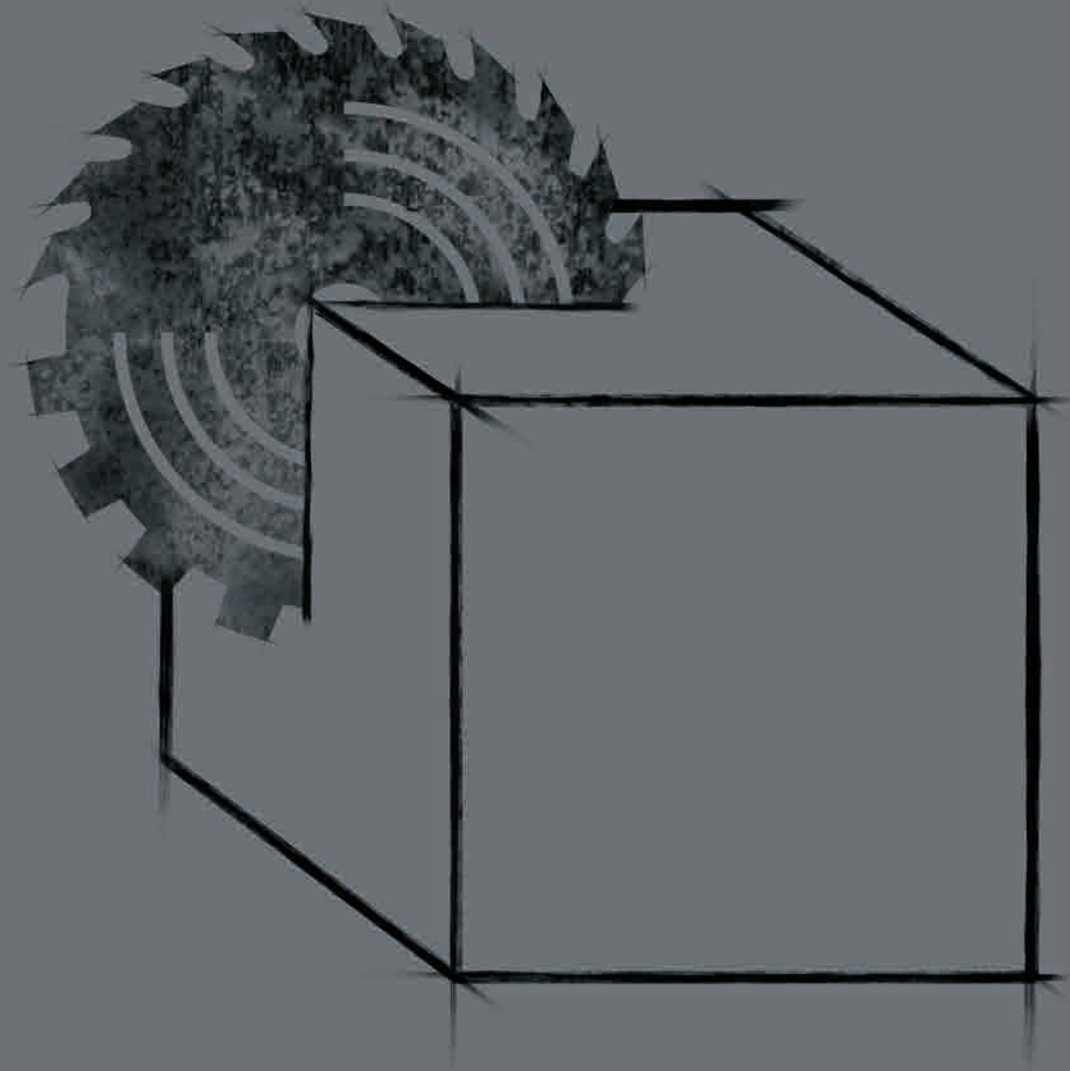

Manufacturing

Cross-laminated timber manufacturing

CHAPTER 2

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ABSTRACT

This chapter provides general information about the manufacturing of CLT that may be of interest to the design community. The information contained in this chapter may also provide guidance to CLT manufacturers in the development of their plant operating specification document.

Typical steps of the manufacturing process of CLT are described, and key process variables affecting adhesive bond quality of CLT products are discussed. Proposed methods for evaluating panel quality are presented.

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1 INTRODUCTION

Components (lumber and adhesives) selected for cross-laminated timber (CLT) and the design and operation of manufacturing processes (adhesive application, panel pressing, etc.) need to be carefully considered to ensure a reliable and consistent product. CLT products evaluated for code compliance by a recognized evaluation service, or produced and independently certified as meeting a national standard, provide product specifiers with a basis for comparing competing product performance and assurance that minimum requirements have been considered in the product design.

In North America, the desire is to support the development of CLT with a single product standard that is recognized both in Canada and the United States. While this effort does not prevent individual manufacturers from pursuing code recognition through evaluation services, it is felt that efforts specifically directed towards developing a bi-national standard will help to accelerate product awareness, and acceptance in the marketplace and amongst regulators. At the time of this report, initiatives have been launched to develop a CLT product standard. One of these has been the development of two seed documents by FPInnovations, which could after consideration by a committee, form the basis of a CLT product standard. Copies of those two seed documents were sent to the APA Standards Committee on Cross-Laminated Timber (PRG-320).

In this chapter, we will make reference to these two seed documents (hereafter referred to as the "proposed CLT standard") to facilitate the discussion on CLT manufacturing issues. The seed documents are included in Appendix 1 and Appendix 2 of this chapter.

2 RAW MATERIALS FOR CLT

2.1 Lumber

CLT is manufactured from a wide range of dimension lumber or boards in Europe, but the first generation of Canadian CLT will likely be manufactured primarily from structural dimension lumber or boards that meet the requirements of CSA O141 in Canada and PS 20 in the United States. Doing so allows manufacturers and designers to utilize design values published in the national codes (CSA O86 in Canada, and the National Design Specification in the USA) to derive capacities for CLT panels (see Chapter 3, *Structural Design of Cross-Laminated Timber Elements*, for more information). One advantage is that such lumber will typically be marked as "HT" (heat treated), meaning that the resulting CLT product will also meet national and international phytosanitary requirements.

Although any grade with published design values can be used in CLT, in most cases the visual quality requirements for lumber stock will be Structural Light Framing No. 2 & Better grade (NLGA, 2003) for the major direction, namely, the general direction of the outermost layers of the CLT panel, and No. 3 & Better grade for the minor direction perpendicular to the major one. Machine rated lumber grades such as 1650F_b-1.5E may also be specified, particularly for the major direction.

2.2 Adhesive

The proposed CLT standard requires that adhesives used in the manufacturing of CLT meet the structural adhesive standard CSA O112.10 "Evaluation of Adhesives for Structural Wood Products (Limited Moisture Exposure)" (CSA, 2008). Adhesives meeting this specification, while having a high degree of moisture resistance, are intended only for products targeted at dry service conditions. Because of the sensitivity of wood stress in rolling shear to moisture, dry service is the proposed moisture service class targeted for CLT in the proposed CLT standard (see Chapter 6, *Duration of Load and Creep Factors for Cross-Laminated Timber*, for more information).

Adhesives that traditionally have been used for laminated beam applications in Canada are also suitable for bonding CLT. Such adhesives will have met standards for adhesives suitable for exterior exposure, such as CSA O112.9 "Evaluation of Adhesives for Structural Wood Products (Exterior Exposure)" (CSA, 2010). Although adequate structural and moisture exposure performance of the adhesive are important attributes, there are other issues that need to be considered when selecting an adhesive for CLT.

For thick CLT panels, the pressing operation may become a bottleneck if commercial heat cured adhesives, such as phenol formaldehyde, are used. Structural cold-set adhesives are preferred to increase the manufacturing productivity. Appearance of the bondline and wear on cutting tools used to shape CLT panels are other considerations.

Classes of structural adhesives that could be used include:

- Phenolic types such as phenol-resorcinol formaldehyde[1] (PRF);
- Emulsion polymer isocyanate (EPI);
- One-component polyurethane (PUR).

PRF is a well-known adhesive for structural use, and commonly used for glued-laminated timber manufacturing in North America. EPI adhesive is used for wood I-joist and lamination. PUR adhesive has been commonly used in Europe to produce CLT. It should be noted that not all formulations within a class may meet the requirements of the structural adhesive standard, and there may be considerable variation in working properties within each class. Documentation from independent sources indicating that the adhesive has met the appropriate standards should be requested, and the working properties needed by the manufacturing process should be discussed with the adhesive supplier.

In addition to cost and working properties, each class may possess other attributes that may be important. Among the three adhesive types indicated above, PRF is dark brown whereas EPI and PUR are light-coloured. PUR is manufactured without the addition of solvents or formaldehyde and is moisture reactive. Due to the chemical reaction, PUR normally produces slight foaming during hardening.

Table 1 summarizes the main characteristics of the three types of structural adhesives.

Table 1

Typical characteristics of adhesives for CLT manufacturing†

Item	Units	Adhesive		
		PRF*	EPI**	PUR***
Cured adhesive colour		Dark	Light	Light
Component		Liquid, two components	Liquid, two components	Liquid, single component (isocyanate pre-polymer)
Solids content	(%)	50	43	100
Wood moisture content (MC)	(%)	6 - 15%	6 - 15%	> 8% optimal 12%
Target application rate (single spread)	(g/m^2)	375 - 400 (75 - 80 lb/msf)	275 - 325 (55 - 65 lb/msf)	100 - 180 (20 - 35 lb/msf)
Assembly time	(min)	40	20	45
Pressing time	(min)	420 - 540	60	120
Applied pressure	(psi)	120	120	120 - 200
Cost ****	($/lb)	2.0	3.5	4.8

† More information can be found in the adhesive manufacturer's technical bulletin.

Note: * Represented by Hexion's LT series;
** Represented by Hexion's EPI series;
*** Represented by Purbond's HB series;
**** Estimated price which may vary from time to time.

[1] PRF may be more appropriate for multiple panel pressing where a large number of panels are pressed consecutively in a multi opening press. Using PRF with a single panel pressing in a single opening press is not likely to be cost effective because of the long pressing times, unless there is a way of applying heat, such as preheating the lumber.

3 CLT MANUFACTURING PROCESS

Before considering the manufacturing process, it is necessary to establish the panel dimensions of interest, as this influences the choice of manufacturing technology and plant layout.

CLT is manufactured according to a wide range of specifications for various structural applications. To simplify the range of panel lay-ups available to designers, efforts are underway to establish either target performance classes or standard lay-ups for a floor, wall or roof application. Regardless of whether there will be performance classes or standard lay-ups, CLT panels will be manufactured in multiple layers consisting of three or more layers of the same or different thickness of lumber or boards in a 90° crisscross pattern.

The orthogonal arrangement of layers in CLT adds dimensional stability and two-way action capability to the product. In certain cases, two adjacent layers can be aligned in the same direction to meet certain specifications. Fundamentally, it is possible to produce any CLT thickness by combining the following layer thicknesses: 19 mm (¾ in.), 25 mm (1 in.), and 38 mm (1.5 in.) up to maximum 50 mm (2 in.). The final CLT thickness ranges from 72 mm to 400 mm. While it is possible to have panels that are not symmetrical through the thickness (e.g. top and bottom outer plies with different thickness or mechanical properties), it is likely that most panels will be symmetrical except perhaps for the layer designated as the appearance face or for fire-protection.

Panel size is generally dictated by the press size. The width of CLT panels ranges from 0.5 m to 3 m, and may reach 5 m for certain applications. Some manufacturers produce CLT panels up to 18 m long.

Figure 1 shows schematically the typical manufacturing process of CLT products, which involves the following nine basic steps:

1) Primary lumber selection,
2) Lumber grouping,
3) Lumber planing,
4) Lumber or layers cutting to length,
5) Adhesive application,
6) Panel lay-up,
7) Assembly pressing,
8) CLT on-line quality control, surface sanding[2] and cutting, and
9) Product marking, packaging and shipping.

[2] Surface sanding is optional.

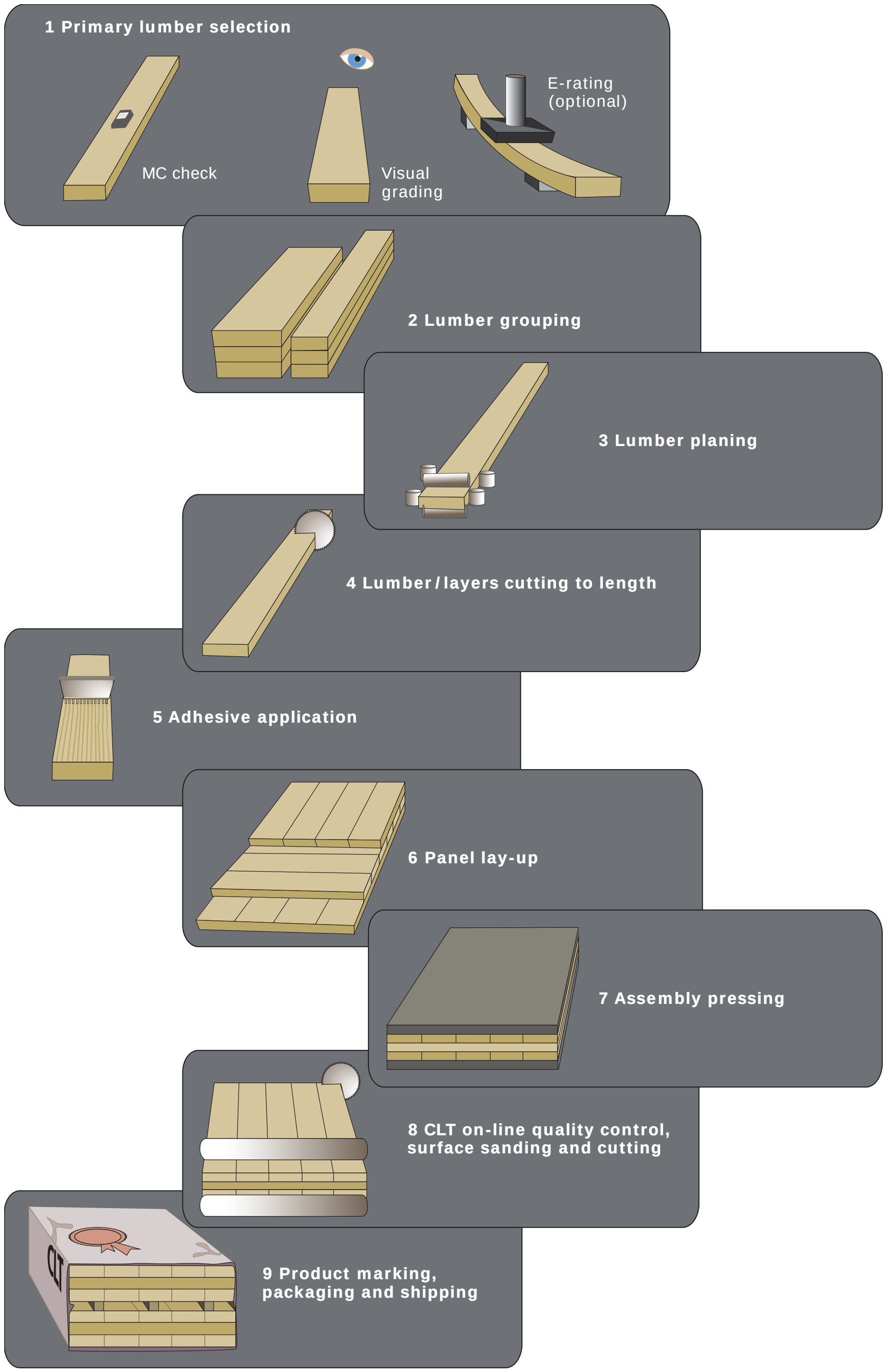

Figure 1
The manufacturing process of CLT products

Each step may include several sub-steps. Step 1 includes lumber moisture content (MC) check and quality control (QC). Lumber QC generally involves visual grading with or without E-rating. For a CLT plant with an annual capacity below 30,000 m^3, Step 3 is to plane (or surface) lumber on all four sides before cutting up to length for face-gluing. For a CLT plant with an annual capacity of 30,000 m^3 or above, Step 3 could involve secondary lumber preparation (Julien, 2010), which has the following three options: lumber end-joining only, lumber edge-gluing only, and both lumber end-joining and edge-gluing.

The key to a successful CLT manufacturing process is consistency in the lumber quality and control of the parameters that affect the quality of the adhesive bond. Much of what is described in this section should appear in the Plant Operating Specification document. This document should be in line with the requirements of the CLT product and plant qualification standards and specific to each manufacturing facility.

3.1 Primary Lumber Selection

In Europe, some manufacturers produce two grades of CLT panels: construction grade and appearance grade. Lumber stock may be selected in accordance to the grade of the CLT panel; for appearance grade CLT, the outermost layer(s) may have specific visual characteristics for aesthetic purposes. Some European manufacturers produce a so-called composite CLT by surface bonding wood composites or engineered wood products such as OSB, plywood and laminated veneer lumber to CLT. This composite CLT is outside the scope of this chapter.

Most adhesives require that surfaces be planed prior to adhesive application and pressing to ensure a strong and durable bondline. This means that graded lumber, which is usually supplied surfaced on four sides (S4S), will need to be re-planed just prior to bonding. Depending on the amount of wood removed, this may alter the grade of the lumber so a grade verification step may need to be added. While there may be savings in using rough sawn lumber (only planed once, thus resulting in higher fibre recovery), the manufacturing process will more likely have to include a lumber grading step (visual grading with or without E-rating) after planing as the amount of wood removed will be more than when using S4S lumber.

3.1.1 Lumber Moisture Content and Temperature

Packages of kiln dried lumber are usually solid-stacked and dried to a MC of 19% or less at the time of surfacing. The standard MC specification for lumber may not be suitable for all CLT manufacturing processes. Some adhesives are more sensitive to MC than others; it is best to conduct trials with production runs on lumber with representative levels of MC, remembering that MC levels may vary from season to season. Lacking information on the interaction between the manufacturing process and lumber MC, it is recommended that lumber having a MC of 12 ± 2% be targeted for CLT manufacturing to ensure proper bond quality of the product. Another reason for limiting the MC variation is to minimize the development of internal stresses between pieces due to differential shrinkage which is dependent on differential MC, growth ring orientation and species. It is recommended that the maximum difference in MC between adjacent pieces that are to be joined not exceed five percentage points.

The lumber packages should be wrapped and stored in a warehouse to prevent wetting. Storage facilities of sufficient capacity should be available to maintain the required MC and temperature of the lumber. To achieve the target MC, the package must be unpacked, stickered by row to allow air circulation and/or re-stacked for drying.

A hand-held radio-frequency MC meter (capacitance type) or an electrical resistance moisture meter can be used to check the lumber MC. Capacitance based MC meters with sets of metallic plates placed above and below the lumber to measure the electric capacitance as the lumber passes transversally at line speed can be used in production. Other on-line MC meters using emerging technologies such as a bench-type near-infrared (NIR) moisture spectroscopy or a microwave MC sensor may be installed to continuously monitor the MC of lumber pieces as they pass by. Note that the former can only measure the MC on the surface while the latter allows a deeper penetration of microwave field into the product, leading to a more accurate MC measurement. More research and development is needed to adapt the latter two emerging technologies for on-line measurement of lumber.

Wood temperature will affect the bondline quality, and the adhesive manufacturer's recommendations should be followed. The ambient temperature in the manufacturing facility may also have an effect on some process parameters such as the open assembly time and adhesive curing time; therefore it is recommended that the ambient temperature be at least 15°C. The wood temperature and MC, as well as the ambient temperature in the manufacturing facility may change throughout the year, which points to the need for a QC program that includes monitoring these parameters. As the effect of temperature and MC on the bondline and panel quality is better understood, revisions can be made to the Plant Operating Specification to better allocate monitoring resources.

3.1.2 Lumber Characteristics Affecting Adhesive Bond Quality

In addition to the lumber MC and temperature, there are other lumber characteristics that may affect the quality of the adhesive bond. These either impact on the pressure that is effectively applied to the bondline, or simply reduce the available bonding surface. Lumber warp in the form of bow, crook, cup and twist are examples of the former. Wane is a common example of the latter.

Standard grades of framing lumber permit these characteristics to varying degrees. While these limits are acceptable for wood frame construction, some of these characteristics need to be restricted when manufacturing CLT in order to ensure formation of a good bondline.

It is important that the impact of these characteristics, if permitted, be taken into account in the product manufacturing and expected bondline performance. In the proposed CLT standards, for example, this is addressed by grading to achieve an "effective bondline area[3]" of a minimum of 80%. Characteristics impacting the bondline are then permitted provided that when they are averaged over an area of 1.3 m x 1.3 m, they displace not more than 20% of the total area.

Consider wane, for example. Wane is the presence of bark or a lack of wood at the corner of a square-edged lumber piece. It will reduce the bonding area and concentrate the stresses in a CLT panel. However, wane cannot be ignored because it is a permitted characteristic in all lumber visual grades. The effect of wane can be accommodated by removing pieces with excessive amounts of wane and/or rearranging or reorienting pieces with wane.

3.2 Lumber Grouping

In production, preparation of lumber for the major direction and minor direction of the CLT may follow different steps. In grouping lumber for these two directions, the MC level and visual characteristics of lumber are primary considerations. In some cases, E-rating is also performed in conjunction with visual grading. In general, for the purpose of establishing panel capacities, all lumber for the major direction will be required to have the same engineering properties. Similarly, the lumber for the minor direction (cross plies) will have a single set of engineering properties. To ensure aesthetic quality, the exposed surfaces of the outer-most layers may be of a better visual appearance.

It may be desirable to place higher quality lumber in designated areas in a panel where fasteners will be installed to maximize the effectiveness of fastening (see Chapter 5, *Connections in Cross-Laminated Timber Buildings*, for more information).

[3]The effective bonding area is defined as the proportion of the lamination wide face averaged over its width that is able to form a close contact bond upon application of pressure.

3.3 Lumber Planing

Lumber planing (or surfacing) helps activate or "refresh" the wood surface to reduce oxidation for improved gluing effectiveness. Removal of a very thin surface layer ensures better bonding (Julien, 2010). Lumber planing must achieve the required precision to ensure optimal gluing. In most cases, planing on all four sides is required to ensure dimensional uniformity. However, in some cases, only face and back planing may suffice if the width tolerance is acceptable and lumber edges are not glued. In general, removing 0.1 in. (2.54 mm) from the thickness and 0.15 in. (3.81 mm) from the width is recommended (Julien, 2010). Due to the inevitable variations in drying efficiency and wood characteristics, it is possible for recently kiln-dried lumber pieces to exhibit higher-than-average MC after planing. If this problem is encountered, steps should be taken to remove and recondition those pieces. The suitability of those pieces for bonding after reconditioning may need to be assessed.

3.4 Lumber/Layers Cutting to Length

A cutting station rips the lumber (or layers if edge-gluing is used) lengthwise for stacking. Transverse layers may be generated from the longitudinal layers by cross-cutting into shorter sections based on the dimensions of the press, if the same grade and size of wood is used for both longitudinal and transverse layers.

3.5 Adhesive Application[4]

In a typical glue application system used in a through-feed process, which is generally seen for PUR and PRF adhesives, the extruder heads move and apply parallel lines/threads of the adhesive in an air tight system with direct supply from an adhesive container. The layers may be lightly wetted with water mist (about 15-20 g/m^2) to help the curing reaction when PUR adhesives are used. The production feed speed is generally around 20-60 m/min.

If the CLT layers are formed in advance, the glue applicator will consist of a series of side-by-side nozzles installed on a beam, and will travel longitudinally over the layers. The typical speed takes about 12 seconds for 16 m long layers (Julien, 2010).

Adhesive application should occur within 24 hours of planing to overcome such issues as surface oxidation, ageing and dimensional instability of the wood, and improve wettability and bonding effectiveness.

The actual adhesive application rate (or glue spread level) must be checked against that specified by the adhesive manufacturers. The desired rate is affected by the quality of the wood and the application system. The amount of adhesive applied must ensure uniform wetting of the wood surface. Proper application rate is evidenced by very slight but even squeeze-out along the entire bondline. The adhesive applicator and application rate are generally adhesive dependent.

The bonding surfaces of surfaced lumber must be clean and free from adhesive-repellent substances such as oils, greases or release agents, which would have a detrimental effect on bond quality. Prior to gluing, the layers should be cleaned with a compressed air jet to remove any debris.

Disruptions in the manufacturing process may be caused by issues related to adhesive application, such as exceeding the maximum allowed assembly time, which may result in adhesive pre-cure. Procedures should be in place to promptly resolve the cause of such disruptions. Such procedures should be included in the Plant Operating Specification.

[4] This chapter refers to CLT manufactured with glued laminations. However, aluminum nails or wooden dowels may also be used to assemble the laminations, although such products are not covered in the CLT Handbook.

Edge gluing of wood pieces that make up the CLT layers is not a common practice among manufacturers due to the added manufacturing cost. In order for edge-gluing to be effective, edge planing must be done in advance. As a trade-off between cost and improved product performance, edge-gluing only the surface layer lumber could be adopted.

3.6 Panel Lay-up

In general, CLT panel lay-up is similar to plywood with adjacent layers aligned perpendicular to each other, with the only difference being that each layer of the CLT panel consists of multiple lumber pieces. A minimum "effective bonding area" of 80% is recommended, although the target level may be increased or decreased depending on the structural demands placed on the panel. While there are a number of wood characteristics that may affect the available bond area, the producer is ultimately responsible to find the most effective way of meeting the requirements. In the case of wane, this may be accomplished by orienting lumber pieces such that the bark and pith faces of adjacent pieces face up. Doing this also has the advantage of reducing the tendency for the panel to warp.

The assembly time is defined as the time interval between the spreading of the adhesive on the first piece of lumber or layers and the application of target pressure to the assembly. The manufacturing process and any restart after a temporary disruption should ensure that the assembly time does not exceed the maximum target set out in the adhesive specification. In some cases, these may need to be more restrictive than the adhesive manufacturer specifications if ambient conditions are not ideal.

If the CLT layers are formed with edge-gluing in advance, the layers are generally stacked in a crisscross pattern with a vacuum gripper (Julien, 2010).

3.7 Assembly Pressing

Pressing is a critical step of the CLT manufacture accounting for proper bond development and CLT quality.

Two main types of press are used for CLT manufacturing: vacuum press (flexible membrane) and hydraulic press (rigid platen). A vacuum press generates a theoretical maximum pressure of 14.5 psi (0.1 MPa). Such a low pressure may not be sufficient to suppress the potential warping of layers and overcome their surface irregularities in order to create intimate contact for bonding. To address this deficiency, lumber shrinkage relief can be introduced to ease pressing and dissipate potential stress resulting from uneven swelling and shrinkage. Lumber shrinkage reliefs can be introduced by sawing to release the stress and in turn reduce the chances of developing cracks when CLT panels lose moisture (Figure 2). However, the relief kerfs cannot be too wide or too deep because they may reduce the bonding area and affect the panel capacity.

Figure 2
Lumber shrinkage relief

A rigid hydraulic press can generate much higher vertical pressure and side pressure than a vacuum press. To minimize the potential gaps between the lumber pieces in the main layers, application of a side pressure in the range of 40 to 80 psi is recommended concomitantly with vertical pressing.

A side pressure is sometimes needed to ensure that gaps between laminations in the major direction are not too wide. CLT product specifications may have a maximum permitted gap between adjacent laminations in the outer and inner layers. To effectively apply the side pressure to the assembly, the length of the cross plies must be less than the total width of the main laminations.

If the CLT layers are formed via edge-gluing in advance, a vertical press without side pressure would suffice. Some vertical presses allow for multiple panels to be pressed simultaneously at high pressures up to 870 psi (Julien, 2010). A lateral unloading device is generally used to un-stack multiple CLT panels loaded in a single opening press. The assembly should be pressed within the specified assembly time. Both assembly time (time between when the adhesive is applied and when the target pressure is applied) and pressing time (time under the target pressure) are dependent on the ambient temperature and air humidity. If the assembly time is shorter than the minimum recommended by the adhesive manufacturer, the pressing time may need to be increased to compensate.

During pressing, it is recommended that the ambient temperature be higher than 15°C because some adhesives may take longer to cure at low temperatures.

3.8 CLT On-line Quality Control, Surface Sanding[5] and Cutting

As a means of on-line QC, an automated visual grading system may be installed to monitor the surface quality and appearance of CLT. Advanced camera vision technologies are currently used for on-line QC of veneer and plywood; however, additional research and development is needed to adapt such technologies to on-line QC of CLT. The combination of advanced equipment and regular external and internal quality surveillance would ensure that CLT products are fit for the intended applications.

An industrial sanding machine designed for wood composite products such as plywood may be used to sand one CLT panel at a time to the target thickness with a tolerance of + 0.1 mm. Typically, the speed is about 2 m/min. Note that the material removal is generally limited to 0.5 mm in total, namely 0.25 mm per surface (Julien, 2010). The permitted tolerances of CLT products will likely be specified in design standards; however, tighter tolerances may be specified by building project.

After sanding, CLT panels are then conveyed to a machining station where a multi-axis numerically-controlled machine cuts out openings for windows and doors, splices and other required parts. Cutting is performed under strictly controlled conditions for maximum accuracy. Minor repairs are carried out manually at this stage of the manufacturing process.

[5]Surface sanding of CLT products is optional.

3.9 Product Marking, Packaging and Shipping

Product marking ensures that the correct product is specified, delivered and installed. It is also an important part product conformity assessment by providing the information to allow designers, contractors and the authority having jurisdiction to check the authenticity of the product. In the proposed CLT standard, the following information is indicated to be placed on CLT products: manufacturer's logo or mill code, reference to the CLT standard to recognize the product has met the standard's requirements, lamination grade, species and thickness to derive the capacity of CLT elements, adhesive service class (e.g. Limited Moisture Exposure, Heat Resistant Adhesive), and evidence of third-party conformity assessment (Agency's logo). Additional markings on the panels may show the main direction loading of the panels in the structure and, possibly, the zones designed to receive connectors.

Because CLT panels are intended for use under dry service conditions, the panels should be protected from weather during transportation, storage and construction on the job site.

4 PRODUCT QUALITY ASSURANCE

As there are a number of process-related issues that would affect the integrity of the bond line, there should be a process in place to qualify a plant to ensure that it has the means to assess and control the quality of the input components and the final product.

Industrial mass production of CLT panels requires an in-plant quality control (QC) program. It is proposed that the product quality assessment consists of a product pre-qualification step and an initial plant qualification of full size production. This is followed by an ongoing quality program to maintain this qualification. The plant's quality program, which includes ongoing QC testing, is required to be described in detail in the Plant Operating Specification and builds on the requirements of the applicable standard.

The surface quality of the CLT panels may need to be controlled if the panels are used for appearance applications. If the panels are to be exposed, the quality of visible surfaces should meet the appearance criteria of the specifier, which may include, for example, considerations such as knot quality, surface smoothness, and absence of surface gaps between lumber pieces. A somewhat lower appearance quality can be tolerated on construction grade panels if agreed with the client (Julien, 2010).

4.1 CLT Product and Plant Qualification

The proposed CLT standard uses delamination testing as a means to assess quality of the bond line. In the delamination test, a core specimen (see Figure 3), extracted from a pre-qualification or production panel, is saturated with water and then dried to evaluate the adhesive bond line's ability to resist the wood shrinkage and swelling stresses. The test also assesses somewhat the ability of the adhesive to withstand moisture degradation. In the delamination test, separation in the wood adjacent to the bond line, as opposed to separation in the adhesive, is not considered delamination.

Figure 3
Proposed delamination specimen

The proposed CLT standard specify limits on the amount of delamination permitted in an individual bond line, in a delamination specimen consisting of several bond lines, in a small area of the panel evaluated using several delamination specimens, and for the overall panel. When all these requirements are met, the manufacturing process is deemed to be producing CLT with bond lines of acceptable quality.

4.2 Block Shear Tests

Preliminary tests carried out at FPInnovations suggest that wood failure results from block shear specimens tested under vacuum-pressure-dry conditions could be used to assess the bond quality. A block shear test may be useful for QC assurance in lieu of, or in addition to, delamination tests for assessing the bond quality of CLT products. For additional information on this topic refer to the report on block shear testing of CLT (Casilla et al., 2010b).

4.3 Tests for Assessing Visual Quality of CLT

Wood shrinkage is not equal in all directions due to the anisotropic nature of wood. As a result, drying checks may develop in CLT panels during storage and use if the MC of the wood at the time of manufacture is significantly different from equilibrium MC of the ambient conditions (Figure 4). The shrinkage can develop tensile stresses which could exceed the local wood strength perpendicular to the grain causing checks or cracks. Although the checks may partially or fully close if exposed to higher humidity environment, they will reappear when the panel is re-dried.

Figure 4
Check development in CLT panels

Checks affect the aesthetic value of the surface, and could thus lower the product's market acceptance. Checks and gaps at the unglued edges of adjacent laminations normally will not have a significant impact on strength properties; however, some of the panel's physical properties, such as thermal conductivity and moisture diffusion may be affected. These properties may have an impact on energy performance and durability of the building assembly. The severity of checking could be used as one of the parameters in classifying the product into different "appearance" grades (Casilla et al., 2010a).

In addition to limiting the MC of the lumber at the time of manufacturing, surface checking can potentially be minimized by using quarter-sawn lumber and by laying up the outer layers in such a way that their growth rings are concave from the bond line. A disadvantage of this arrangement is that it will not help minimize panel warping. As for gaps forming between lumber pieces, this can be minimized or prevented by edge-gluing, but this will likely increase the development of checks.

Simple tests may be carried out on small 2 ft. x 2 ft. (approximately 600 mm x 600 mm) panels to assess check and gap development under the temperature and relative humidity conditions expected in-service. These tests would provide an indication of the appearance of these products after long-term exposure in service to dry conditions, or the effectiveness of steps taken to minimize checking.

5 REFERENCES

American Forest and Paper Association (AF&PA). American Wood Council (AWC). 2005. *NDS: National design specifications for wood construction*. Washington, DC: AF&PA. 174 p.

Canadian Standards Association (CSA). 2005. *Softwood lumber*. CSA O141-05. Mississauga, ON: CSA. 78 p.

______. 2008. *Evaluation of adhesives for structural wood products (limited moisture exposure)*. CSA O112.10-08. Mississauga, ON: CSA. 42 p.

______. 2009. *Engineering design in wood (limit states design)*. CSA O86-09. Mississauga, ON: CSA. 222 p.

______. 2010. *Evaluation of adhesives for structural wood products (exterior exposure)*. CSA O112.9-10. Mississauga, ON: CSA. 45 p.

Casilla, R., C. Lum, C. Pirvu, B. J. Wang, I. Chiu, and P. Symons. 2010a. Checking in CLT panel products. Draft Report. Vancouver, BC: FPInnovations.

Casilla, R., C. Pirvu, B. J. Wang, C. Lum, I. Chiu, P. Symons, G. Chow, and A. Andersen. 2010b. Block shear testing of CLT panels. Draft Report. Vancouver, BC: FPInnovations.

Julien, F. 2010. Manufacturing cross-laminated timber (CLT): Technological and economic analysis, report to Quebec Wood Export Bureau. 201001259-3257AAM. Quebec, QC: FPInnovations. 176 p.

National Lumber Grades Authority (NLGA). 2003. *Standard grading rules for Canadian lumber*. New Westminster, BC: NLGA. 275 p.

U.S. Department of Commerce. National Institute of Standards and Technology (NIST). 2010. *American softwood lumber standard*. Voluntary Product Standard PS 20-10. Washington, DC: U.S. Government Printing Office. 40 p.

SEED DOCUMENT FOR PROPOSED CROSS-LAMINATED TIMBER PLANT QUALIFICATION STANDARD

DRAFT

PREPARED BY FPINNOVATIONS

Contents

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0 Introduction

Cross-laminated timber (CLT) is a wood panel product made by glue or nail laminating solid wood, composite wood, or structural wood sheathing products to a thickness corresponding to the minimum dimension for heavy timber. Components in a layer are typically arranged so that the principal axis of the components in one layer is orthogonal to the principal axis of adjacent layers. The principal axes of the outer layers are usually parallel to the long axis of the panel.

1 Scope

This Standard covers the evaluation of manufacturers producing CLT that meet the requirements of the Cross-Laminated Timber Product Standard (hereafter referred to as the Product Standard).

The Standard does not establish capacities for CLT panels. Guidance is provided in Appendix B.

2 Reference Publications

CSA (Canadian Standards Association)

CSA O112 Series Standards for Wood Adhesives

O112.7-M1977 (R2006)
Resorcinol and phenol-resorcinol resin adhesives for wood (room- and intermediate-temperature curing)

O112.9-10
Evaluation of adhesives for structural wood products (exterior exposure)

O112.10-08
Evaluation of adhesives for structural wood products (limited moisture exposure)

National Lumber Grades Authority
Standard Grading Rules for Canadian Lumber, 2007

SPS 1
Special Products Standard for Fingerjoined Structural Lumber, 2010

SPS 2
Special Products Standard for Machine Graded Lumber, 2010

SPS 6
Special Products Standard for Structural Face-Glued Lumber, 2005

National Institute of Standards and Technology
PS 20-10
American Softwood Lumber Standard

ASTM International (American Society for Testing and Materials)

D 4444-08
Standard Test Methods for Laboratory Standardization and Calibration of Hand-Held Moisture Meters

D 7438-08
Standard Practice for Field Calibration and Application of Hand-Held Moisture Meters

D 6782-05
Standard Test Methods for Standardization and Calibration of In-Line Dry Lumber Moisture Meters

3 Definitions

Adhesive – a substance capable of holding materials together

Adherend – a body held to another body by an adhesive

Agency – an independent body that is competent to evaluate a manufacturer's ability to meet the requirements of this Standard and to operate a certification system in which the interests of all parties concerned with the functioning of the system can be represented

Assembly time – the time interval between the spreading of the adhesive on the adherend and the application of the target pressure to the assembly

Note: For assemblies involving multiple layers or parts, the assembly time begins with the spreading of the adhesive on the first adherend.

> **Closed assembly time** – the time between completion of assembly of the laminations for bonding and the application of the target pressure or heat, or both, to the assembly
>
> **Open assembly time** – the time between the spreading of the adhesive on the adherend(s) and the completion of the assembly of the parts for bonding

Bondline – the layer of adhesive which attaches two adherends

> **Face bondline** – the bondline joining the wide faces of laminations in adjacent layers
>
> **Edge bondline** – the bondline joining the narrow faces of adjacent laminations within one layer

Cross-laminated timber (CLT) – the wood product made by bonding, under pressure, graded laminating stock the grain of which is essentially parallel in each layer and is produced in accordance with the requirements of this Standard

Delamination – the separation of layers in a laminate due to failure of the adhesive either in the adhesive itself or at the interface between the adhesive and the adherend

> **Bondline delamination** – the length of delamination observed in a single bondline of a delamination specimen expressed as a percentage of the total length of that bondline
>
> **Specimen delamination** – the average delamination across all face bondlines in a delamination specimen
>
> **Zone delamination** – the average of the specimen delamination of specimens sampled within a pre-qualification panel, or the area defined by the grid overlaid on a full-size panel
>
> **Panel delamination** – delamination within a full-size panel based on the average of the zone delamination results

Edge (panel edge) – the narrow face of a panel that exposes the ends or narrow faces of the laminations

> **Finished edge** – the panel edges that have been trimmed or machined to a specified quality and tolerance after pressing
>
> **Unfinished edge** – the panel edges that are neither specified as finished or as meeting the tolerances provided in this Standard

Effective bonding area – proportion of the lamination wide face averaged over its length that is able to form a close contact bond upon application of pressure

Face – one of the four longitudinal surfaces of a piece or panel

> **Lamination (or lumber) narrow face** – the face with the least dimension perpendicular to the lamination (or lumber) length
>
> **Lamination (or lumber) wide face** – the face with the largest dimension perpendicular to the lamination (or lumber) length
>
> **Panel face** – the wide face of a panel

Finger joint – a joint between two pieces, the ends of which have been formed into a series of mating fingers through either the wide or narrow faces of the pieces

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Grade – the designation of the quality of a piece of wood

Glue skip – absence of adhesive on one or both adherends

Lamination – lumber, including stress rated boards, that has been prepared for laminating

Layer – all laminations on one side of a bondline

> **Outermost** – the laminations on the panel's wide face and all adjacent layers with laminations having the same grain orientation (e.g. parallel to the major direction)
>
> **Outer** – the layer adjacent to the outermost layer and all adjacent layers with laminations having the same grain orientation (e.g. parallel to the minor direction)
>
> **Inner** – the laminations between the outermost layers

Lay-up – the number and thickness of laminations, combination of grades and species, and orientation of laminations

Length –

> **CLT length** – dimension of the CLT panel or specimen parallel to the major direction
>
> **Lamination length** – dimension of the lamination parallel-to-the-grain after planing
>
> **Lumber length** – dimension of the lumber parallel-to-the-grain at the time of stress grading
>
> **Note**: The lamination and lumber may include finger joints provided the finger joints are manufactured to a recognized specification that retains the lumber grade.
>
> **Bondline length** – dimension of a single adhesive layer along the edges of the panel
>
> **Total bondline length** – dimension of all adhesive layers along the edges of the panel

Lumber –

> **Machine evaluated lumber** – structural lumber that has been graded by means of a non-destructive test and visual grading, conforming to the requirements for machine stress-rated lumber, with the exception that the process lower fifth percentile modulus of elasticity (MOE) equals or exceeds 0.75 times the characteristic mean MOE for the grade
>
> **Machine stress-rated lumber** – structural lumber graded by means of a non-destructive test and visual grading, in accordance with the requirements of CSA O141
>
> **Sawn lumber** – the product of a sawmill not further manufactured other than by sawing, re-sawing, passing lengthwise through a standard planing mill, and cross-cutting to length
>
> **Structural lumber** – lumber in which strength is related to the anticipated end use as a controlling factor in grading or selecting
>
> **Visually stress-graded lumber** – structural lumber that has been graded in accordance with the provisions of the National Lumber Grades Authority Standard Grading Rules for Canadian Lumber

Major direction – general direction of the grain of the outermost layers of the CLT panel

Minor direction – perpendicular to the major direction

Species – the commercial name of the wood species or species combinations that are used to formulate lumber grading rules for the purposes of assigning common design properties

Note: Unless otherwise specified, the species of a panel is the species of the exposed face or the appearance face.

Specimen – a part, item, or the whole of a sample, taken as representative of a whole or a collection of items

Stress grade – a classification of CLT where combinations of laminating grades are arranged so they are suitable for resisting the type and magnitude of stress assigned to the grade

Thickness –

> **CLT thickness** – dimension of the CLT panel or specimen perpendicular to the plane of the panel
>
> **Lamination thickness** – dimension of the lamination perpendicular to the wide face after planing
>
> **Lumber thickness** – smaller cross-section dimension of the lumber at the time of stress grading
>
> **Specified thickness** – dimension of the CLT panel perpendicular to the panel for purposes of design calculation

Visual grade –

CLT (panel) visual grade – the visual grade of one or both of the outermost layers

Lumber visual grade – the visual grade at the time the lumber is grade marked

Lamination visual grade – the grade of the lamination just prior to bonding

Width –

CLT width – dimension of the CLT panel or specimen perpendicular to the major direction

Lamination width – dimension of the lamination between unglued edges of the wide face

Lumber width – larger cross-section dimension of the lumber at the time of stress grading

Note: The lamination and lumber may include edge bonds provided they are manufactured to a recognized specification that retains the lumber grade

4 Quality System

4.1 General

4.1.1 CLT shall be manufactured under a quality system that is audited by an independent third-party agency (hereafter referred to as the "Agency").

4.1.2 Manufacturers shall be qualified by the Agency to manufacture and to mark CLT conforming to the Product Standard.

4.1.3 Manufacturers shall establish a quality control program with appropriate control limits for maintaining the manufacturing process within the Specification Limits.

4.1.4 Manufacturing parameters with Specification Limits shall include, at a minimum, those parameters listed in Clause 5.2.

4.2 Plant Operating Specification

4.2.1 Operating Procedures and Records

Recordkeeping and the operating procedures necessary for ensuring the proper manufacturing of CLT shall be described in a Plant Operating Specification.

4.2.2 Specification Limits

4.2.2.1 Specification limits for manufacturing parameters including those evaluated under this Standard (see Clause 5.2) shall be listed in the Plant Operating Specification.

4.2.2.2 The quality control program and basis for the control limits shall be described in the Plant Operating Specification.

4.2.3 Product Marking

The product shall bear durable and legible markings and/or provided with a certificate that indicates, at a minimum, the following:

(a) A reference to this Standard
(b) Manufacturer's logo, or a manufacturing facility code known to the Agency
(c) Laminate grade, species and thickness in sufficient detail to derive the capacity
(d) Adhesive service class (e.g. HRA, Limited Moisture Exposure)
(e) Evidence of third-party conformity assessment (e.g. Agency logo)

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5 Tests for Facilities Qualification

5.1 General

5.1.1 Qualification tests shall be performed under the supervision of the Agency to demonstrate the capability of the manufacturing process to produce the desired product. Such tests shall be repeated should any method or material be changed.

5.1.2 Except as permitted in Clause 5.2.1, qualification samples shall be manufactured using the press, adhesive formulation, and adhesive application system from the facility to be qualified.

5.1.3 Except as required in Clause 5.2.2, the input component (lumber and adhesive) and the manufacturing parameters (clamping pressure and time, adhesive spread, etc.) shall be representative of that to be used by the facility.

5.1.4 The method used to measure the moisture content of laminations used in the qualification samples shall be similar to that to be used in production.

5.1.5 Moisture content shall be measured using a handheld moisture meter calibrated in accordance with Method D 4444 and used in accordance with Practice D 7438, or an in-line meter calibrated in accordance with Method D 6782.

5.1.6 Ambient conditions under which qualification samples are manufactured shall be representative of operating conditions. Otherwise, additional qualification testing for parameters to be used for the range of ambient conditions shall be undertaken.

5.2 Pre-Qualification

5.2.1 General

5.2.1.1 Specification Limits to be specified in the Plant Operating Specification shall be pre-qualified using full thickness qualification test panels of not less than 60 cm in the major direction, 45 cm in the minor direction, or more than 90 cm in either direction (hereafter referred to as “Pre-qualification Test Panels”).

Note: A Pre-qualification Test Panel of more than 60 cm is recommended, particularly for thicker CLT products.

5.2.1.2 Pre-qualification Test Panels shall be prepared at the facility or at an alternative facility acceptable to the Agency.

5.2.1.3 All Pre-qualification Test Panels shall be:

(a) Of the same approximate length and width at the time of pressing;
(b) Pressed individually; and
(c) Taken from approximately the geometric centre of the larger panel, if applicable.

5.2.2 Sample Preparation

5.2.2.1 Application of pressure to manufacture pre-qualification test samples shall be by a platen that has similar rigidity as that to be used in the facility to be qualified. The applicability of the results shall be documented by the Agency.

Note: For example, Pre-qualification Test Panels for facilities using a vacuum press should be clamped using a vacuum press or an air bag inserted between the sample and the rigid platen.

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5.2.2.2 Adhesive shall be applied to the laminations in a manner similar to that to be used in the facility to be qualified.

Note: In addition to considering the amount of adhesive applied, sample preparation facilities should distinguish between, for example, roller versus curtain coating and single spread versus double spread, which vary in the uniformity of the adhesive spread.

5.2.2.3 Except as permitted in Clause 5.2.2.4, the factors specified in Table 1 shall be prepared in the combinations specified in Table 2. Each panel shall be labelled to indicate the factor and measurement.

Table 1
Factors to be evaluated with the pre-qualification sample

Factor	Parameter	Deviation Δ from Nominal†	
		- (Below Nominal)	+ (Above Nominal)
A	Total assembly time§	0 minute	0 minute
B	Moisture content	2 MC PP‡	2 MC PP‡
C	Adhesive spread	90% of nominal	110% of nominal
D	Clamping pressure	90% of nominal	100% of nominal
E	Wood surface temperature	-0°C	+5°C

† The minimum and maximum levels to be evaluated shall be the greater of the deviation Δ in the Table or the permitted deviation stated in the Plant Operating Specification and monitored on an ongoing basis.

‡ Moisture content percentage points.

§ Total assembly time is the sum of the open and closed assembly time. Ambient conditions (air temperature and relative humidity) to be within the range anticipated during production. Otherwise, the adhesive formulation or assembly times evaluated should be adjusted to accommodate the anticipated ambient conditions.

Table 2
Combination of factors for pre-qualification testing

Measurement	A	B	C	D	E	Replicates
1	+	+	+	-	+	2
2	-	+	+	+	-	2
3	-	-	+	+	+	2
4	+	-	-	+	+	2
5	-	+	-	-	+	2
6	+	-	+	-	-	2
7	+	+	-	+	-	2
8	-	-	-	-	-	2

Note: Table contents adapted from ASTM E 1169, Standard Guide for Conducting Ruggedness Tests.

5.2.2.4 (a) Where two or more lay-ups with the same number of layers differ by only the thickness of the laminations, only the lay-ups with the minimum and maximum overall thickness need to be evaluated.

(b) Where lay-ups have identical outer and outermost lamination species, grade and thickness, only the lay-up with the greatest overall thickness need to be evaluated.

(c) Where lay-ups differ only by the nominal width of the laminations in one or more layers, only the lay-ups with the minimum width laminations and with the maximum width laminations need to be evaluated.

5.2.3 Sample Conditioning

Samples shall be stored in an environment maintained at 20±2°C/65±5% RH until the adhesive has cured sufficiently to permit evaluation.

Note: For panels larger than the specified pre-qualification panel size, it is permissible to trim the panels to the specified size to facilitate conditioning.

5.2.4 Specimens

5.2.4.1 Three delamination specimens shall be extracted from each Pre-qualification Panel as shown in Figure 1 and labelled to indicate the panel number and the specimen position within the panel.

5.2.4.2 Where the panel is larger than the specified Pre-qualification Test Panel size, the pre-qualification sampling area shall be 60 cm to 90 cm square located at the geometric center of the panel.

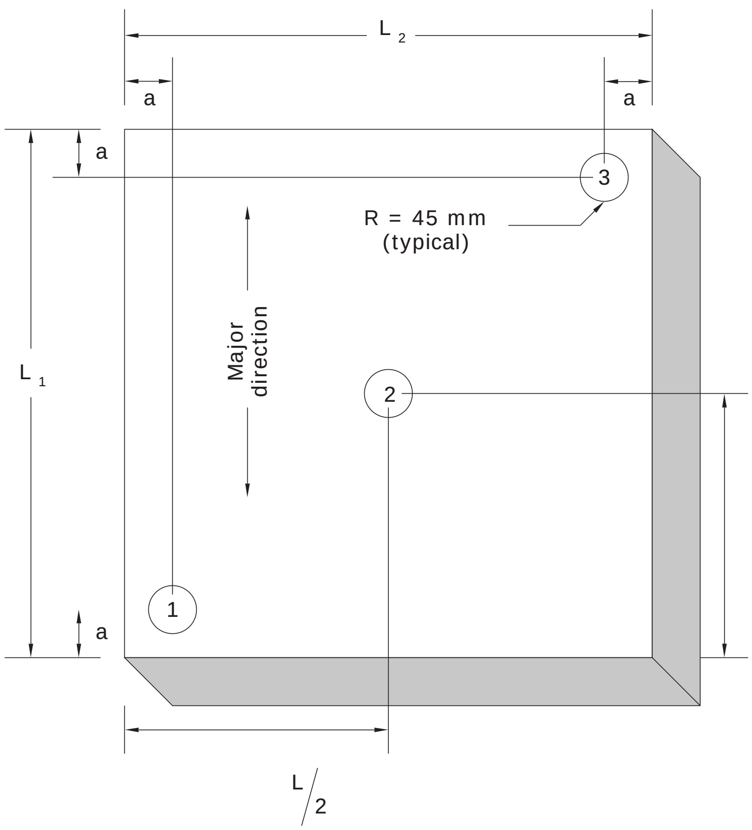

Figure 1
Delamination core specimen locations (a = 10±2 cm, L1 = 60 to 90 cm, L2 = 45 to 90 cm)

5.3 Qualification of Effective Bond Area

5.3.1 General

5.3.1.1 The manufacturer shall establish visual grade rules for the bonded faces and limit the average glue skip to maintain an average effective bond area of 80% or more.

Note: Alternatively, glue skips may be treated as delamination. See the Product Standard for additional information.

5.3.1.2 The manufacturer's visual grade rules established to achieve the target effective bond area shall include, at a minimum, limits on the characteristics listed in Table 3.

Table 3
Visual characteristics reducing the effective bond area

Characteristic	Subcategories to be Included and Limited
Knots	Knots generally classified as to quality: loose knots, knot holes, unsound knots, burls, or equivalent
Holes	Very large hole (greater than 1 inch in diameter)
Pitch streaks	All
Pockets	All
Pitch or bark seams	All
Shake	Limit as wane
Wane	All
Decay (unsound wood)	All
Compression wood	Limit as skip
Size	See size tolerances in the Product Standard
Eased edges	Exceeding the standard radius
Skip	All
Manufactured holes	Limit as wane
Grain	Chipped, torn, loosened or raised grain
Planing	Knife marks, wavy dressing, machine bite, machine gouge shall be limited

5.3.1.3 The bond area displacement shall be based on characteristic measurements consistent with the NLGA Standard Grading Rules.

5.3.2 Sample Selection and Inspection

5.3.2.1 Samples shall be drawn from representative production of laminations meeting the manufacturer's visual grade rules and positioned in accordance with the Plant Operating Specification.

5.3.2.2 The layer formed by the laminations shall be verified by the Agency to provide an effective bond area of 80% or more over any randomly selected area not more than 1.3 m x 1.3 m.

Note: A mask with a square opening of 1.3 m may be used to facilitate inspection.

5.3.2.3 All pieces within the layer shall be rotated and the opposite faces inspected.

5.4 Initial Plant Qualification

5.4.1 General

Following pre-qualification, a representative sample of the largest panels to be manufactured for each lay-up shall be subjected to qualification testing.

5.4.2 Sample Preparation

5.4.2.1 Two qualification panel samples shall be prepared from a representative sample of laminations following the procedures outlined in the Plant Operating Specification.

5.4.2.2 From each grade, species, and width used to fabricate the panel, a total of 30 pieces of laminations shall be randomly selected and the following recorded for each piece:

(a) Moisture content (see Clauses 5.1.4 and 5.1.5)
(b) Lamination thickness to the nearest 0.05 mm

5.4.3 Sample Conditioning

Panel samples shall be stored under the conditions and for the duration as specified in the Plant Operating Specification to allow the adhesive to cure.

5.4.4 Specimens

A 1.3 m x 1.3 m or smaller square grid shall be overlaid onto each panel. Three delamination cores shall be randomly taken from within the boundary of each square.

Note: A scheme that generates a random number pair for positioning of the delamination cores within each square is recommended.

5.5 Subsequent Plant Qualification

5.5.1 General

Material changes to the manufacturing process or facilities shall be subjected to subsequent qualification testing.

The requirements of Clauses 5.2, 5.3 and 5.4 shall be reapplied for material changes listed or equivalent to that listed in Table 4.

Table 4
Subsequent qualification in response to material changes

Category	Applicable Clause(s)	Material Change (Examples)	Notes
A	5.2, 5.3, 5.4	• Press equipment • Adhesive formulation class • Addition or substitution of species from a different species group • Changes to the visual grade rules that reduce the effective bond area or the effectiveness of the applied pressure (e.g. warp permitted)	Excludes replacement with identical press
B	5.2, 5.3	• Other changes to the manufacturing process or component quality not listed above. • Adhesive composition (e.g. fillers and extenders)	Additional evaluation in accordance with Clause 5.4 is at the discretion of the Agency†
C	5.4	• Increase in panel width or length of more than 20% • Table 5,Category D items when the production process is not under Level II sampling as defined in the Product Standard	

† It is recommended that changes involving two or more manufacturing parameters be subjected to evaluation in accordance with Clause 5.4.

5.5.2 Reduced Level of Qualification Testing

Reduced qualification testing is permitted for the material changes listed or equivalent to that listed in Table 5.

Table 5
Reduced qualification testing in response to material changes

Category	Applicable Clause(s)	Material Change (Examples)	Notes
D	Product Standard - Level I sampling	• Increases in width or length of not more than 20% • Increase in panel thickness of a currently qualified lay-up by two lamination thickness	Production shall be at or eligible for Level II sampling under the Product Standard

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5.6 Delamination Resistance Test

5.6.1 General

The delamination test shall be used to assess the quality of the bondlines.

5.6.2 Sample

The test specimens shall be taken from each panel prepared as described in Clause 5.2 or sampled as described in Clause 5.4.

5.6.3 Specimens

Specimen height shall be equal to the thickness of the panel from which it is sampled. Specimen diameter shall be between 80 and 90 mm. Sawn end-grain and side-grain surfaces are permitted to be sanded prior to conditioning to remove blemishes such as burn marks and facilitate inspection of the bondlines provided the overall diameter is not reduced to less than 80 mm.

5.6.4 Test

5.6.4.1 The weight of each test specimen prior to conditioning shall be recorded to the nearest gram.

5.6.4.2 The test specimen(s) shall be placed in an autoclave or pressure vessel, weighted down, and covered in water at a temperature of 18 to 27°C. All test specimen(s) shall be separated such that all bondlines are exposed to the water.

5.6.4.3 A vacuum of between 70 and 85 kPa shall be drawn and held for 30 min. The vacuum shall then be released and a pressure of 480 to 550 kPa shall be applied for 2 h.

5.6.4.4 The test specimen(s) shall then be removed from the pressure vessel and placed in a drying oven. The test specimen(s) shall be dried in air at 65 to 75°C. During the drying period, the test specimen(s) shall be placed approximately 50 mm apart and oriented with their bondlines parallel to the flow of air. The airflow rate and relative humidity shall be such that the specimen(s) are dried to within 10 to 15% above their original test weight within a period of 10 to 15 h.

5.6.4.5 When the test specimen(s) have returned to within 10 to 15% above their original test weight, delamination shall be measured and recorded.

Note: Delamination should be measured immediately after removal of the specimens from the oven. If measurement is delayed, areas of poor bond can close up because the block core dries out to a state of equilibrium with the outer block surface, or the surface can pick up moisture.

5.6.4.6 Clauses 5.6.4.2 to 5.6.4.5 shall be repeated as required for an additional delamination exposure cycle if the observed delamination exceeds the limit ("maximum permitted delamination") after one (1) cycle but less than the limit after two (2) cycles (see Table 6).

5.6.5 Measurements

5.6.5.1 Delamination is measured along the glue lines and shall exclude knots, grade defects, and wood failure in the bondline area.

Note: After all delamination exposure cycles are completed, the specimen may be chiselled apart at the bondline to further evaluate the quality of the glue bond.

5.6.5.2 Glue skips on bondable surfaces if not counted as delamination shall be averaged from all specimens from a panel and shall not exceed the maximum permitted glue skip established in Clause 5.3.1.1.

5.6.6 Requirements

5.6.6.1 Delamination specimens meeting the requirements of Table 6 for one (1) delamination exposure cycle as described in Clauses 5.6.4.2 to 5.6.4.5:

(a) Need not be subjected to a second cycle; and

(b) The results from the single delamination exposure cycle shall be used for the bondline delamination and specimen delamination assessment.

Note: See Appendix C for sample delamination calculations for a bondline, and for the average delamination in a specimen, zone and panel.

Table 6
Maximum permitted delamination

Bondline		Specimen†		Average of Specimens			
				Zone‡		Panel§	
1 cycle	2 cycles	1 cycle	2 cycles	1 cycle	2 cycles	1 cycle	2 cycles
25%	30%	20%	25%	15%	20%	10%	15%

† Result from delamination averaged across all bondlines in an individual specimen

‡ Result from delamination averaged across all specimens within the zone

§ Result from delamination averaged across all the specified zones within the panel

5.6.6.2 If the requirements of Table 6 are not met, specimen re-sampling and re-evaluation of delamination is permitted as shown in Table 7.

Table 7
Delamination re-sampling and re-evaluation

Sampling Stage	Delamination Deficiency	Re-sampling Permitted
Pre-qualification panel	Specimen or zone	Replacement panel shall be prepared and evaluated.
Full-size panel • Initial qualification • Subsequent qualification	Specimen	A replacement specimen shall be randomly selected from the same zone to replace the specimen.
	Zone	Up to two additional specimens shall be randomly selected in sequence from the same zone and combined with the original samples for computing the average delamination.
	Panel	The zone with the highest average delamination shall be re-sampled as noted above for a zone deficiency.

5.6.6.3 If glue skip is noted, the average glue skip observed shall not exceed the maximum permitted glue skip established in Clause 5.3.1.1.

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6 Records

Sufficient records shall be maintained to enable the Agency to verify that testing in accordance with this Standard has been carried out.

Appendix A	**Structural Properties CLASS for CLT Panels** [to be completed] Classification of floor, wall and roof panels
Appendix B	**Calculating Design Capacities for CLT Panels** [to be completed] Flexural capacity and stiffness
Appendix C	**Sample Delamination Calculations** [to be completed] Bondline delamination Specimen delamination Zone delamination Panel delamination

SEED DOCUMENT FOR PROPOSED CROSS-LAMINATED TIMBER PRODUCT STANDARD

DRAFT

PREPARED BY FPINNOVATIONS

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Contents

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Tables

0 Introduction

Cross-laminated timber (CLT) is a wood panel product made by glue or nail laminating solid wood, composite wood, or structural wood sheathing products to a thickness corresponding to the minimum dimension for heavy timber. Components in a layer are typically arranged so that the principal axis of the components in one layer is orthogonal to the principal axis of adjacent layers. The principal axes of the outer layers are usually parallel to the long axis of the panel.

Manufacturer must be qualified in accordance with the companion Cross-Laminated Timber Plant Qualification Standard (hereafter referred to as the Plant Qualification Standard).

1 Scope

This Standard covers the evaluation of CLT made by bonding solid wood lumber components with a structural adhesive. Although beyond the scope of this standard, the principles of this Standard may be applied to CLT made by bonding composite wood components.

This Standard does not establish capacities for solid wood lumber component or for CLT panels. See Appendix F for guidance on computing CLT panel capacities.

Fingerjoining, edge-gluing, or face-gluing of CLT panels is not permitted under this Standard.

2 Reference Publications

CSA (Canadian Standards Association)

CSA O112 Series Standards for Wood Adhesives

> O112.7-M1977 (R2006)
> Resorcinol and phenol-resorcinol resin adhesives for wood (room- and intermediate-temperature curing)
>
> O112.9-10
> Evaluation of adhesives for structural wood products (exterior exposure)
>
> O112.10-08
> Evaluation of adhesives for structural wood products (limited moisture exposure)

CSA O141-05
Softwood lumber

CAN/CSA-O86-09
Engineering design in wood

National Lumber Grades Authority

Standard Grading Rules for Canadian Lumber, 2007

SPS 1
Special Products Standard for Fingerjoined Structural Lumber, 2010

SPS 2
Special Products Standard for Machine Graded Lumber, 2010

SPS 6
Special Products Standard for Structural Face-Glued Lumber, 2005

National Institute of Standards and Technology

PS 20-10
American Softwood Lumber Standard

American Forest & Paper Association, Inc.

National Design Specification (NDS) for Wood Construction Supplement: Design Values for Wood Construction 2005 Edition

ASTM International (American Society for Testing and Materials)

D 4444-08
Standard Test Methods for Laboratory Standardization and Calibration of Hand-Held Moisture Meters

D 7438-08
Standard Practice for Field Calibration and Application of Hand-Held Moisture Meters

D 7247 – 07
Standard Test Method for Evaluating the Shear Strength of Adhesive Bonds in Laminated Wood Products at Elevated Temperatures

D 7374 – 08
Standard Practice for Evaluating Elevated Temperature Performance of Adhesives Used in End-Jointed Lumber

3 Definitions

Adhesive – a substance capable of holding materials together

Adherend – a body held to another body by an adhesive

Agency – an independent body that is competent to evaluate a manufacturer's ability to meet the requirements of this Standard and to operate a certification system in which the interests of all parties concerned with the functioning of the system can be represented

Bond – the attachment at an interface between adhesive and adherends or the act of attaching adherends together by adhesive

Bondline – the layer of adhesive which attaches two adherends

> **Face bondline** – the bondline joining the wide faces of laminations in adjacent layers
>
> **Edge bondline** – the bondline joining the narrow faces of adjacent laminations within one layer

Cross-laminated timber (CLT) – the wood product made by bonding, under pressure, graded laminating stock the grain of which is essentially parallel in each layer and is produced in accordance with the requirements of this Standard

Curing – converting an adhesive into a fixed or hardened state by chemical or physical action

Delamination – the separation of layers in a laminate due to failure of the adhesive either in the adhesive itself or at the interface between the adhesive and the adherend

> **Bondline delamination** – the length of delamination observed in a single bondline of a delamination specimen expressed as a percentage of the total length of that bondline (the circumference of the delamination specimen)
>
> **Specimen delamination** – the average delamination across all face bondlines in a delamination specimen
>
> **Zone delamination** – the average of the specimen delamination of specimens sampled within a pre-qualification panel, or the area defined by the grid overlaid on a full-size panel
>
> **Panel delamination** – delamination within a full-size panel based on the average of the zone delamination results

Edge (panel edge) – the narrow face of a panel that exposes the ends or narrow faces of the laminations

> **Finished edge** – the panel edges that have been trimmed or machined to a specified quality and tolerance after pressing
>
> **Unfinished edge** – the panel edges that are neither specified as finished or as meeting the tolerances provided in this Standard

Effective bonding area – proportion of the lamination wide face averaged over its length that is able to form a close contact bond upon application of pressure

Face – one of the four longitudinal surfaces of a piece or panel

> **Lamination (or lumber) narrow face** – the face with the least dimension perpendicular to the lamination (or lumber) length
>
> **Lamination (or lumber) wide face** – the face with the largest dimension perpendicular to the lamination (or lumber) length
>
> **Panel face** – the wide face of a panel

Finger joint – a joint between two pieces, the ends of which have been formed into a series of mating fingers through either the wide or narrow faces of the pieces

Grade – the designation of the quality of a piece of wood

In-control – is when the production continues to meet the process requirements of this Standard

Item –panels produced in sequence with the same lay-up, regardless of the panel length or width

Lamination – lumber, including stress rated boards, that has been prepared for laminating

Layer – all laminations on one side of a bondline

> **Outermost** – the laminations on the panel's wide face and all adjacent layers with laminations having the same grain orientation (e.g. parallel to the major direction)
>
> **Outer** – the layer adjacent to the outermost layer and all adjacent layers with laminations having the same grain orientation (e.g. parallel to the minor direction)
>
> **Inner** – the laminations between the outermost layers

Lay-up – the number and thickness of laminations, combination of grades and species, and orientation of laminations

Length –

> **CLT length** – dimension of the CLT panel or specimen parallel to the major direction
>
> **Lamination length** – dimension of the lamination parallel-to-the-grain after planing
>
> **Lumber length** – dimension of the lumber parallel-to-the-grain at the time of stress grading
>
> **Note:** The lamination and lumber may include finger joints provided the finger joints are manufactured to a recognized specification that retains the lumber grade.
>
> **Bondline length** – dimension of a single adhesive layer along the edges of the panel
>
> **Total bondline length** – dimension of all adhesive layers along the edges of the panel

Lumber –

> **Machine evaluated lumber** – structural lumber that has been graded by means of a non-destructive test and visual grading, conforming to the requirements for machine stress-rated lumber, with the exception that the process lower fifth percentile modulus of elasticity (MOE) equals or exceeds 0.75 times the characteristic mean MOE for the grade
>
> **Machine stress-rated lumber** – structural lumber graded by means of a non-destructive test and visual grading, in accordance with the requirements of CSA O141
>
> **Sawn lumber** – the product of a sawmill not further manufactured other than by sawing, re-sawing, passing lengthwise through a standard planing mill, and cross-cutting to length.
>
> **Structural lumber** – lumber in which strength is related to the anticipated end-use as a controlling factor in grading or selecting
>
> **Visually stress-graded lumber** – structural lumber that has been graded in accordance with the provisions of the National Lumber Grades Authority Standard Grading Rules for Canadian Lumber

Major direction – general direction of the grain of the outermost layers of the CLT panel

Minor direction – perpendicular to the major direction

Moisture content – the weight of moisture in wood expressed as a percentage of its oven-dry weight

Out-of-control – is when the process no longer meets one or more of the process requirements of this Standard

Package – one or more panels pressed together for curing

Panel – a single CLT structural element formed by bonding laminations with a structural adhesive

Piece – a single board or plank, one or more which may be used in a lamination

Sample – one or more units of product taken from a lot or batch or a portion of material taken from a panel, in order to represent that lot, batch, or panel for inspection purposes

Setting – the initial stages of curing of adhesives

Shift – that portion of production represented by the Level I or Level II quality control sample

Species – the commercial name of the wood species or species combinations that are used to formulate lumber grading rules for the purposes of assigning common design properties

Note: Unless otherwise specified, the species of a panel is the species of the exposed face or the appearance face.

Specimen – a part, item, or the whole of a sample, taken as representative of a whole or a collection of items

Stress grade – a classification of CLT where combinations of laminating grades are arranged so they are suitable for resisting the type and magnitude of stress assigned to the grade

Stress rated board (SRB) – lumber less than 2 inches in nominal thickness and 2 inches or more in nominal width that has been graded using the same grade-limiting specifications as those applied to structural dimension lumber

Structural composite lumber – the wood product that is either laminated veneer lumber (LVL), parallel strand lumber (PSL), laminated strand lumber (LSL), or oriented strand lumber (OSL), as defined in ASTM D 5456 and manufactured for use in structural applications

Thickness –

> **CLT thickness** – dimension of the CLT panel or specimen measured perpendicular to the plane of the panel
>
> **Lamination thickness** – dimension after planing of the lamination measured perpendicular to the plane of the panel
>
> **Lumber thickness** – smaller cross-section dimension of the lumber at the time of stress grading
>
> **Specified thickness** – dimension of the CLT panel perpendicular to the plane to be used for purposes of establishing the panel capacity by calculation

Visual grade –

> **CLT (panel) visual grade** – the visual grade of one or both of the outermost layers
>
> **Lumber visual grade** – the visual grade at the time the lumber is grade marked
>
> **Lamination visual grade** – the grade of the lamination just prior to bonding

Wane – the presence of bark or a lack of wood, for whatever cause, at the corner of a square-edged piece

Warp – any deviation from a true or plane surface, including crook, bow, cup, twist, or any combination of these (see NLGA Standard Grading Rules for additional details)

Width –

> **CLT width** – dimension of the CLT panel or specimen perpendicular to the major direction
>
> **Lamination width** – dimension of the lamination between unglued edges of the wide face
>
> **Lumber width** – larger cross-section dimension of the lumber at the time of stress grading
>
> **Note:** The lamination and lumber may include edge bonds provided they are manufactured to a recognized specification that retains the lumber grade.

Wood failure (per cent) – the area of wood fibre remaining at the bondline following completion of the specified shear test

Note: Wood failure is determined by means of visual examination and is expressed to the nearest 5% of the test area.

4 Panel Classification

4.1 General

Panels shall be classified and marked to indicate their lumber grade composition, appearance and panel thickness. Panels shall be further specified by their length and width, or any suitable description of their size.

4.2 Stress Grade

The stress grade shall be determined by acceptable engineering analysis or by the applicable design standard based on the panel composition.

Note: See Appendix F of this Standard for the recommended engineering analysis.

4.3 Appearance

Panel appearance shall be as agreed to between the buyer and seller.

Note: See Appendix D for guidance on specifying panel appearance.

5 Panel Tolerances

5.1 General

5.1.1 Panel and component dimensions shall be specified at a reference moisture content of 15% (See PS20 or O141 for shrinkage coefficients).

5.1.2 Textured or other face or edge finishes are permitted to alter the tolerances specified in this section. The designer shall compensate for any loss of cross-section and/or specified strength of such alternations.

5.2 Bondline Position

The actual bondline position within the panel thickness shall not deviate by more than 5% of the overall specified thickness from the bondline position based on the specified lamination thickness.

5.3 Panel Length and Width

Where panel length or width are specified and no tolerances are provided, the tolerance for both length and width shall be +0 mm, -4 mm and shall be applied to the specified width and length.

5.4 Squareness

Unless specified otherwise or designated as “unfinished”, panel face diagonals shall not differ by more than 3 mm.

5.5 Straightness

Unless specified otherwise or designated as “unfinished”, deviation of edges from a straight line between adjacent panel corners shall not exceed 2 mm.

6 Materials

6.1 Lumber

6.1.1 General Requirements

Lumber shall be obtained from structural lumber complying with the requirements of CSA O141 in Canada, or PS 20 in the United States.

6.1.2 Grading

Lumber shall be graded in accordance with the National Grade Rule and have design properties specified in either CSA O86 or the National Design Specification

6.1.3 Edge-glued and Fingerjoined Laminations

6.1.3.1 End and edge-glued joints in grade marked laminations shall meet the applicable standard for the stress grade of the lumber.

Note: Structural glued lumber meeting Standards conforming to the latest edition of the American Lumber Standard Committee Glued Lumber Policy are acceptable (e.g. NLGA SPS 1 and SPS 6).

6.1.3.2 Adhesive used for end and edge-glued joints shall meet the requirements of CSA O112.10.

6.1.4 Visual Quality

Lumber visual grade quality in each layer shall meet the visual requirements in Table 1.

Table 1
Lumber grade requirements and proportions

Grain Direction of Layer	Primary Grade	Secondary Grade	Maximum Proportion of Secondary Grade
Major	No. 2 or higher	No. 3	10%
Minor	No. 3 or higher	N/A	N/A

Note: Other structural grades such as machine graded lumber or light framing grades are permitted provided their visual grade requirements are no less restrictive than specified in this clause.

6.1.5 Minimum Sizes (lumber after surfacing)

6.1.5.1 Cross laminations shall have a width-to-thickness ratio of 3.5 or more, and a thickness not less than 17 mm.

6.1.5.2 Cross laminations (lumber after surfacing) with a width-to-thickness ratio of less than 3.5 shall be assigned rolling shear strength and modulus values developed in accordance with Appendix B.

6.1.6 Maximum Sizes (lumber after surfacing)

Laminations shall not exceed 50 mm in thickness.

Note: See the definition of lamination thickness.

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6.2 Adhesives

6.2.1 The base adhesive shall meet or exceed the requirements of CSA O112.10.

6.2.2 The specific formulation(s) to be used in production shall meet the requirements of the Plant Qualification Standard.

6.2.3 Where panels are intended for the conditions specified in Table 2, the adhesive used shall meet the additional requirements shown in Table 2 and the panels shall be marked to indicate the requirements met.

Table 2
Optional adhesive requirements for special exposure conditions

Condition	Adhesive Standard and / or Designation §
Wet service conditions - wood moisture content to exceed 19% while in service †	CSA O112.9 - Exterior ASTM D 2559 - Class B
Unprotected fire exposure - panels to be designed as heavy timber elements ‡	ASTM D 7374 - HRA (Heat Resistant Adhesive) ASTM D 7247 - Elevated temperature test of adhesive bond

† Preservative treatment of the panel or laminations may be required.
‡ Other requirements such as minimum panel and outermost lamination thickness may apply.
§ Information on Adhesive Service class to appear on the product mark. See the Plant Qualification standard.

7 Manufacturing

7.1 General

Only panel lay-ups qualified in accordance with the Plant Qualification Standard are permitted to be qualified for production under this Standard.

7.2 Lumber Preparation

7.2.1 General

The lumber surface quality and variation in thickness within and between pieces of lumber shall be limited to ensure a consistent bond.

7.2.2 Surfacing

Surfaces of laminations to be bonded shall be machine-finished to a uniformly smooth surface, but shall not be sanded.

7.2.3 Lamination Thickness

The tolerance on lamination thickness shall meet the requirements of Table 3.

Table 3
Laminate thickness variation

Layer	Under	Over
Outermost plies	0.0 mm	1.0 mm
Other plies	0.0 mm	0.5 mm

7.2.4 Reworked Lumber

7.2.4.1 General

Laminations that have been re-manufactured or re-sawn shall be re-graded, except as permitted below.

7.2.4.2 Shrinkage Relief

In laminations, longitudinal kerfs provided to relieve shrinkage stresses shall not be more than one-half the lamination thickness, and shall not displace in total more than 10% of the lamination cross-section or more than 5% of the lamination width.

7.2.4.3 Planing Prior to Bonding

The lumber shall be visually re-graded when planing prior to bonding results in the removal of more than 3% of the original thickness from either face.

Note: The final thickness or width should be used in determining the panel capacity.

7.2.5 Moisture Content

Lumber moisture content shall be within the range qualified and specified in the Plant Operating Specification.

7.3 Panel Orientation Marking

If applicable, panels shall be clearly marked to ensure correct orientation of the panel in the structure or of any special zones in the panel specifically designed to receive connectors or treatment.

7.4 Panel Protection

Panels shall be protected from weather and mechanical damage while in the care of the manufacturer.

Note: Instructions on the care and protection of the product during transport and construction should be provided.

7.5 Repairs

7.5.1 From Delamination Sampling

7.5.1.1 General

Plugs shall be laminated and bonded using an adhesive with equal or better bond performance as that used for the panel.

Note: Laminated plugs with grain parallel to the axis of the plug are permitted to be used to repair panel holes from delamination sampling.

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7.5.1.2 Plug Locations

The use of the panel shall consider the location of repair plugs. Panels with repair plugs located near points of high concentration of loading (for example, hold down connectors or reduced area such as lintel areas) should be avoided.

7.5.1.3 Plug Material

The moisture content of the plug material shall be less than the average moisture content of the panel being repaired. Consideration shall be given to the potential differential shrinkage between the plug and the panel being repaired.

7.5.2 Other Repairs

Repair of flaws in panels shall be carried out under the supervision of a structural engineer familiar with panels and the end-use conditions for the panel.

8 Process Requirements

8.1 General

The manufacturer shall ensure that the correct size and grade of material are used in the lamination process, and that a durable and effective adhesive bond is formed between layers.

8.2 Bonding Surface Quality

8.2.1 Target Effective Bonding Area

Lumber meeting the visual grade requirements of Table 1 shall be further graded and pieces laid up to maintain an effective bonding area of 80% or better on surfaces to be bonded.

8.2.2 Lumber Growth Ring Orientation

If required to maintain the minimum effective bond area, laminations in cross-plies shall be oriented such that the bark and pith faces of adjacent pieces generally alternate.

8.2.3 Lamination Grade Limits

Grade limits intended to limit the amount of lamination warp that will not be corrected upon application of pressure shall be qualified using the Pre-qualification provisions of the Plant Qualification Standard.

8.2.4 Glue Skip in the Face Bondline

The average glue skip in a face bondline shall not exceed the level established to maintain the minimum effective bond area.

8.3 Resistance to Delamination

Bondline resistance to delamination shall meet or exceed the levels established under the Pre-qualification provisions of the Plant Qualification Standard.

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9 Quality Control

9.1 General

9.1.1 Ongoing evaluation of the process properties listed in Clause 8 shall be performed to confirm that the quality of the manufactured product remains consistent.

9.1.2 Separate quality control records shall be maintained for each lay-up regardless of panel width and length.

9.1.3 The sampling method and control forms shall be approved by the Agency.

9.1.4 Production shall be held pending results of the Quality Control testing specified in Clauses 9.2 and 9.3 on representative samples.

9.2 Quality Control Sampling

9.2.1 Increased Sampling Following Initial Plant Qualification (Level I)

9.2.1.1 General

Level I sampling shall apply after initial or any subsequent qualification as defined in the Plant Qualification Standard.

9.2.1.2 Panel Sampling

The first and last panel from each item produced in a shift shall be selected for testing.

9.2.1.3 Specimen Sampling

A uniform square grid, 1.3 m x 1.3 m or smaller, shall be overlaid onto each panel from which:

(a) Two square grids, one from first half and the second from the second half length of the panel length, shall be randomly selected for delamination specimen sampling; and

(b) Three delamination cores shall be randomly taken from within the boundary of each square.

Note: A computer generated random number pair for positioning the delamination cores within each square is recommended.

9.2.2 Reduced Sampling Following Initial Plant Qualification (Level II)

9.2.2.1 General

Level II sampling is permitted with the approval of the Agency and following at least 7 consecutive shifts of in-control production under Level I sampling.

9.2.2.2 Panel Sampling

A panel representative of the held production of each item shall be selected.

Note: It is recommended that the panel be selected at the end of the last shift in which each item is produced, provided the number of shifts does not exceed three. If the number of shifts exceeds three, the sampling should be made every three shifts. Consideration should be given to scheduling or increasing the frequency of sampling around start-up and shut-down of the production line for maintenance or shift changes.

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9.2.2.3 **Specimen Sampling**

A uniform square grid, 1.3 m x 1.3 m or smaller, shall be overlaid onto each panel from which:

(c) Two square grids shall be randomly selected for delamination specimen sampling, and
(d) One delamination core shall be randomly taken from within the boundary of each square.

Note: A computer generated random number pair for positioning the delamination cores within each square is recommended.

9.3 Delamination Resistance Testing

9.3.1 Specimen Preparation

Specimen length shall be equal to the thickness of the panel from which it is sampled. Specimen diameter shall be between 80 and 90 mm. Sawn end-grain and side-grain surfaces are permitted to be sanded prior to conditioning to remove, for example, burn marks to facilitate inspection of the bondlines, provided the overall diameter is not reduced to less than 80 mm.

9.3.1.1 The weight of each test specimen prior to conditioning shall be recorded to the nearest gram.

9.3.1.2 The test specimen(s) shall be placed in an autoclave or pressure vessel, weighted down, and covered in water at a temperature of 18 to 27°C. All test specimen(s) shall be separated such that bondlines are exposed to the water.

9.3.1.3 A vacuum of between 70 and 85 kPa shall be drawn and held for 30 min. The vacuum shall then be released and a pressure of 480 to 550 kPa shall be applied for 2 h.

9.3.1.4 The test specimen(s) shall then be removed from the pressure vessel and placed in a drying oven. The test specimen(s) shall be dried in air at 65 to 75°C. During the drying period, the test specimen(s) shall be placed approximately 50 mm apart and oriented with their bondline parallel to the flow of air. The airflow rate and relative humidity shall be such that the specimen(s) are dried to within 12 to 15% above their original test weight within a period of 10 to 15 h.

9.3.1.5 When the test specimen(s) have returned to within 12 to 15% above their original test weight, delamination shall be measured and recorded.

Note: Delamination should be measured immediately after removal of the specimens from the oven. If measurement is delayed, areas of poor bond can close up because the block core dries out to a state of equilibrium with the outer block surface, or the surface can pick up moisture.

9.3.1.6 Clauses 9.3.1.2 to 9.3.1.5 shall be repeated as required for each additional delamination exposure cycle if the observed delamination exceeds the limit after 1 cycle (see Table 4).

9.3.2 Measurements

9.3.2.1 Delamination is measured along the glue lines and shall exclude knots, grade defects, and wood failure in the bondline area.

9.3.2.2 Glue skip is permitted to be assessed as delamination.

Note: Once the test cycle is completed, the specimen may be chiselled apart at the bondline to further evaluate the quality of the glue bond. The bondline separation assessed here may be separated into glue skips and delamination.

9.3.2.3 Glue skips on bondable surfaces if not assessed as delamination shall be averaged from all specimens from a panel.

9.3.3 Requirements

9.3.3.1 Delamination of a specimen meeting the requirements of Table 4 for one (1) delamination exposure cycle as described in Clauses 9.3.1.2 to 9.3.1.5:

(a) Need not be subjected to a second cycle; and

(b) The results from the single delamination exposure cycle shall be used for the bondline and specimen delamination assessments.

Note: See Appendix G for sample delamination calculations for a bondline, and for the average delamination in a specimen, zone and panel.

Table 4
Maximum permitted delamination (and glue skips)

Bondline		Specimen†		Average of specimens			
				Zone‡		Panel§	
1 cycle	2 cycles	1 cycle	2 cycles	1 cycle	2 cycles	1 cycle	2 cycles
25%	30%	20%	25%	15%	20%	10%	15%

† Result from delamination averaged across all bondlines in an individual specimen

‡ Result from delamination averaged across all specimens within the zone

§ Result from delamination averaged across all the specified zones within the panel

9.3.3.2 If the requirements of Table 4 are not met, re-sampling and re-evaluation of delamination are permitted as shown in Table 5.

Table 5
Re-sampling and re-evaluation for delamination (or glue skip)

Sampling Stage	Deficiency	Re-sampling Permitted
Full-size Panel • Level I sampling • Level II sampling	Specimen	A replacement specimen shall be randomly selected from the same zone to replace the specimen.
	Zone	Up to two additional specimens shall be randomly selected in sequence from the same zone and combined with the original samples for computing the average delamination (or glue skips).
	Panel	The zone with the highest average delamination (or glue skips) shall be re-sampled as noted above for a zone deficiency.

9.3.3.3 If assessed separately from delamination, the average glue skip expressed as a percentage of the bond area observed shall not exceed the limit established for the plant.

10 Records

Sufficient records shall be maintained to enable the Agency to verify that testing in accordance with this Standard has been carried out, and that the requirements of this Standard have been met prior to the release of production.

11 Re-inspection

11.1 General

Panels shall be reassessed on the basis of their visual quality and, if required, resistance to delamination.

11.2 Visual Inspection

11.2.1 Visual Grade of Outer Faces

The visual grade of the lumber on the outermost layer shall be in accordance with the specified grade of the lamination and the provisions of Clauses 11.2.1.1 to 11.2.1.2

11.2.1.1 Seasoning checks need not be considered.

11.2.1.2 The specified grade limits are permitted to be adjusted to account for planing as permitted in Clause 7.2.4.

11.2.2 Growth Ring Orientation of Cross-Plies

Where cross-ply laminations are not all edge-glued, cross-ply laminations that do not have alternating growth ring patterns with adjacent laminations shall not exceed on average 10% of the total area for each bondline.

Note: The growth ring orientation of the exposed ends of laminations at the edges of panels can be surveyed. Because changes in growth ring orientation may occur along the length of a lamination due to end-jointing, opposite edges may need to be examined.

11.2.3 Bondline Separation

11.2.3.1 Except that excluded in Clause 11.2.3.2, the total length of bondline separation along the finished edges of the panel shall not exceed 10% of the total length of the bondline along the same edges.

11.2.3.2 The following are permitted to be excluded from the total length of bondline separation:

(a) Separation at the bondline not exceeding 20 mm in depth as determined by a 0.1 mm (0.004 in.) feeler gauge;
(b) Separation around characteristics other than warp that are limited by the manufacturers to achieve the minimum bond area; and
(c) Separation in wood away from the bondline.

11.3 Delamination

Random sample or cores subject to delamination testing

Requirements for the average (no check for the maximum)

Appendix A **Test Methods** [to be completed]

Modifications to ASTM test methods for properties of interest to users of this Standard

- ASTM D 4761, Standard Test Methods for Mechanical Properties for Lumber and Wood-Base Structural Material
- ASTM D 2718, Standard Test Methods for Structural Panels in Planar Shear (Rolling Shear)

Appendix B **Evaluation of Rolling Shear Properties** [to be completed]

Practice for establishing the rolling shear modulus and strength of wood using ASTM D 2718

Appendix C **Glued Lumber Requirements** [to be completed]

Glued lumber conforming to the American Lumber Standard Glued Lumber policy are permitted for use with this Standard

Appendix D **Visual Quality Class for CLT Panels** [to be completed]

Provisions for assessing checks in CLT panels used under low moisture content conditions

Appendix E **Structural Properties Class for CLT Panels** [to be completed]

Classification of floor, wall and roof panels

Appendix F **Calculating Design Capacities for CLT Panels** [to be completed]

Flexural capacity and stiffness

Appendix G **Sample Delamination Calculations** [to be completed]

Bondline delamination

Specimen delamination

Zone delamination

Panel delamination

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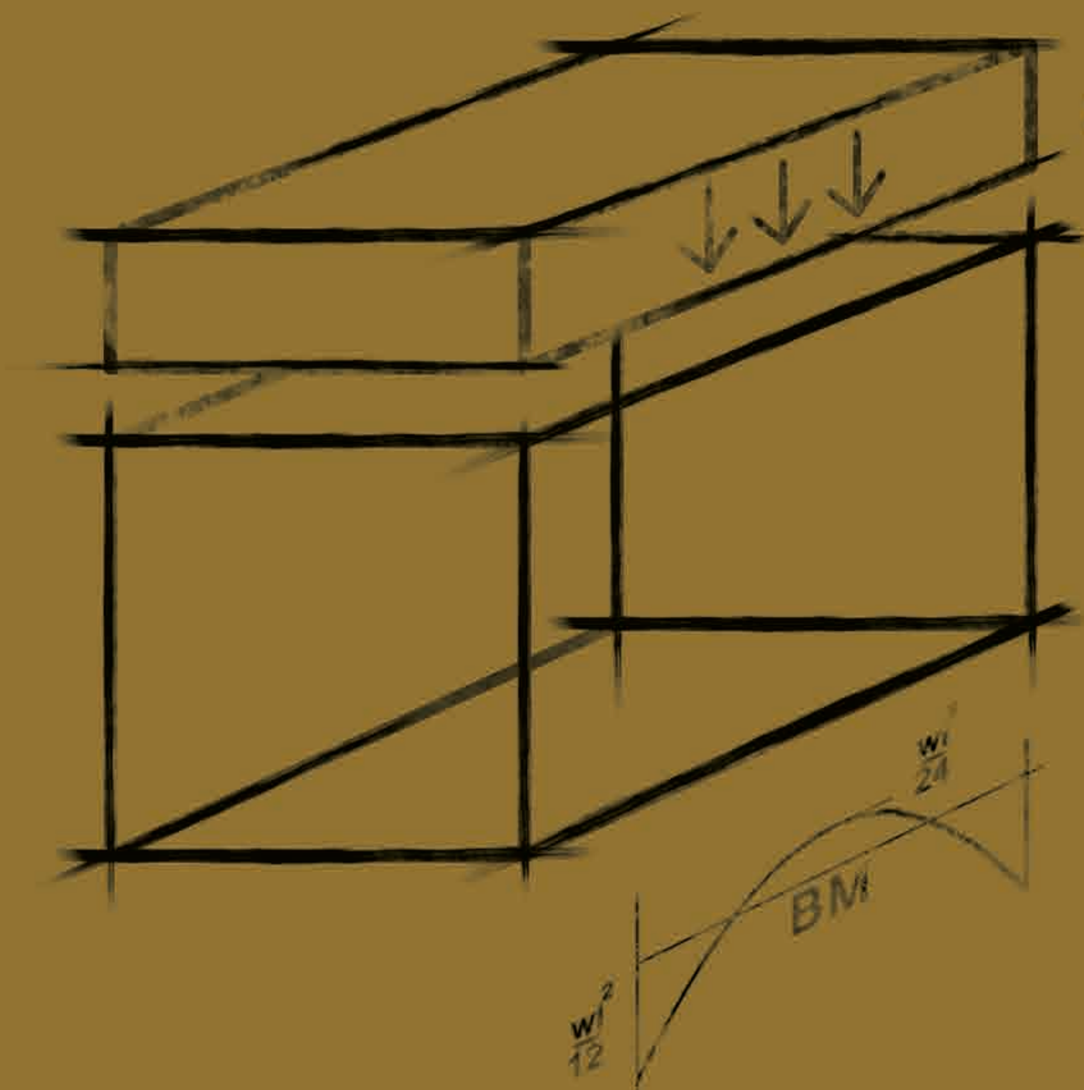
Structural

Structural design of cross-laminated timber elements



CHAPTER 3

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ABSTRACT

Various methods have been adopted in Europe for the determination of design properties of CLT. Some of these methods are experimental in nature while others are analytical. Other methods involve a combination of both empirical and analytical approaches based on model testing. Experimental evaluation involves determination of flexural properties by testing full-size panels or sections of panels with a specific span-to-depth ratio. The problem with the experimental approach is that every time the lay-up, type of material, or any of the manufacturing parameters change, more testing is needed to evaluate the bending properties of such products.

An analytical approach, once verified with the test data, offers a more general and less costly alternative. Such an analytical approach can generally predict the strength and stiffness properties of CLT panels based on the material properties of the laminate planks that make up the CLT panel.

No analytical approach has been universally accepted by European CLT manufacturers and designers. The most common analytical approach that has been adopted for CLT in Europe is based on the mechanically jointed beams theory that is available in Annex B of Eurocode 5 (EN 2004). According to this theory, the "Effective Stiffness" concept is introduced and a "Connection Efficiency Factor" (γ_i) is used to account for the shear deformation of the perpendicular layer, with $\gamma=1$ representing a completely glued member, and $\gamma=0$ no connection at all. This approach provides a closed (exact) solution for the differential equation only for simply supported beams/panels with a sinusoidal load distribution. However, the differences between the exact solution and those for a uniformly distributed load or point loads are minimal and are acceptable for engineering practice (Ceccotti, 2003).

Blass and Fellmoser (2004) have applied the "Composite Theory" (also named k-method) to predict flexural properties of CLT. However, their work did not account for shear deformation in individual layers.

More recently, a new method called "Shear Analogy" (Kreuzinger, 1999) has been developed in Europe that seems to be applicable for solid panels with cross layers. The methodology takes into account the shear deformation of the longitudinal and the cross layers and is not limited by the number of layers within a panel. This method seems to be the most accurate and adequate for prediction of stiffness properties of CLT panels.

Almost all the studies conducted in Europe so far have focused primarily on predicting the stiffness and not the strength properties of CLT panels in flexure. Also, little information is available on creep and vibration behaviours of CLT panels. While flexural stiffness of CLT panels is usually of greater interest for designers than the strength, since the structural design is mostly governed by serviceability criteria, from a product standard development point of view there is a need to characterize the strength properties as well, to ensure certain minimum panel strength in service. There is a need to adopt a design methodology for determination of the stiffness and the strength properties of CLT in flexure by further exploring the shear analogy approach. It is expected that the proposed analytical approach will be accepted in the upcoming CLT product standard. The procedure to calculate the design properties should be based on material properties for lumber published in the design standards, and should be consistent with the design philosophy in the CSA O86, the Canadian Standard for Engineering Design in Wood. Because of these potentially important features, the developed analytical method will need to be comprehensively verified against test data.

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CROSS-LAMINATED TIMBER PANELS – DEFINITION

Cross-laminated timber (CLT) panels consist of several layers of boards stacked crosswise and glued together on their faces. Therefore, a cross-section of a CLT element must have at least three glued layers of boards placed in orthogonally alternating orientation to the neighboring layers (Mestek et al., 2008). The narrow faces (edges) of the boards are usually not glued together, although sometimes boards positioned in the longitudinal direction of the panel are edge-glued. Some manufacturers will also produce panels having the transverse planks edge-glued. Also, in some cases (special configurations), consecutive board layers may be placed in the same direction, giving a double layer, e.g. double longitudinal layers at the outer faces and additional double layers at the centre of the panel. CLT products are usually fabricated with 3 to 11 board layers. Figure 1 illustrates a CLT panel configuration while Figure 2 shows examples of CLT panel cross-sections. Figure 3 illustrates a 5-layer CLT panel with its cross-sections. Finally, Figure 4a shows a floor built with four CLT panels acting mostly in one direction while Figure 4b illustrates the same floor, this time built with only one CLT panel acting most likely in two directions.

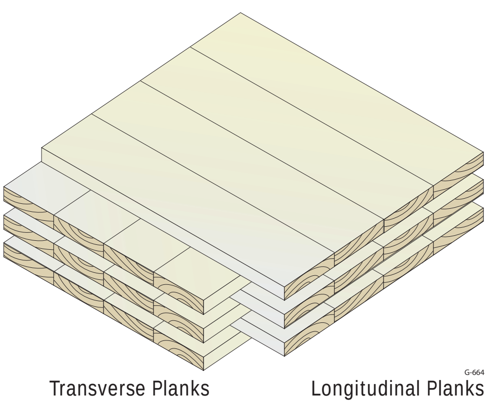


Figure 1
CLT panel configuration

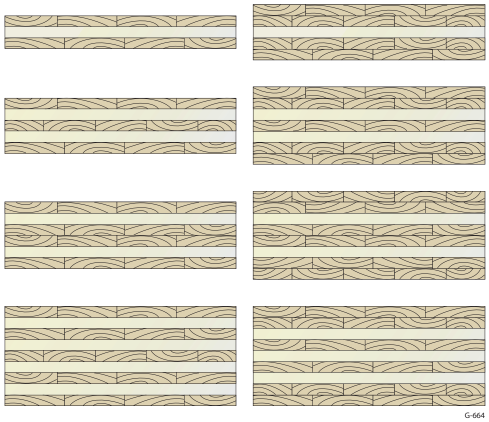


Figure 2
Examples of CLT panel cross-sections

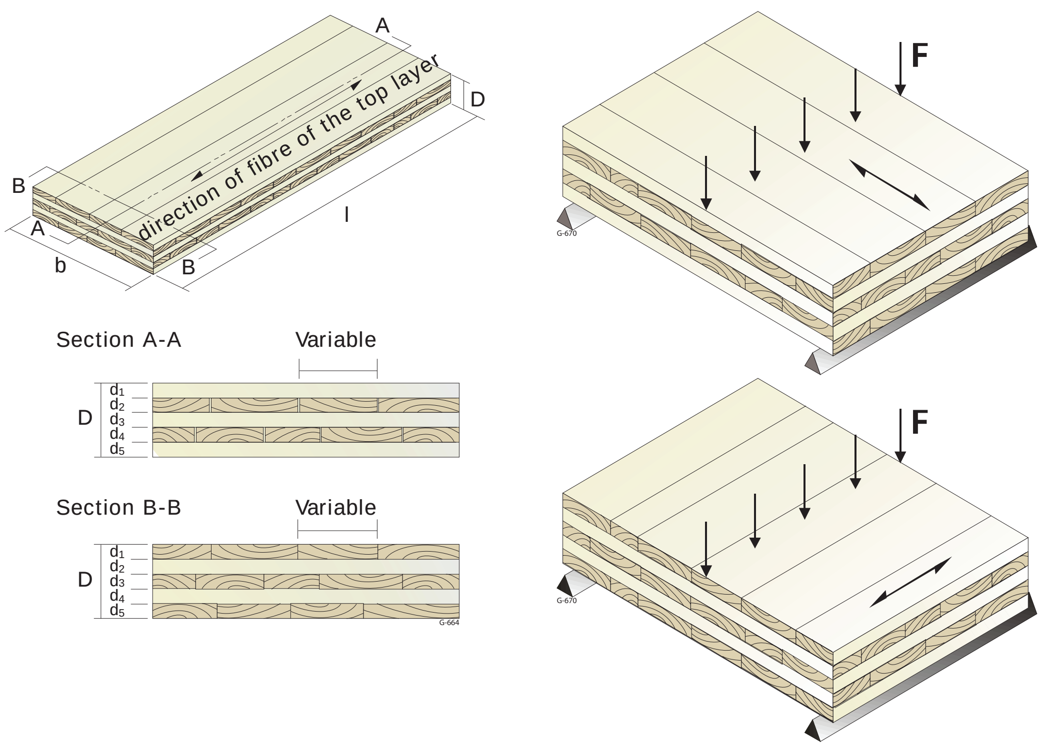
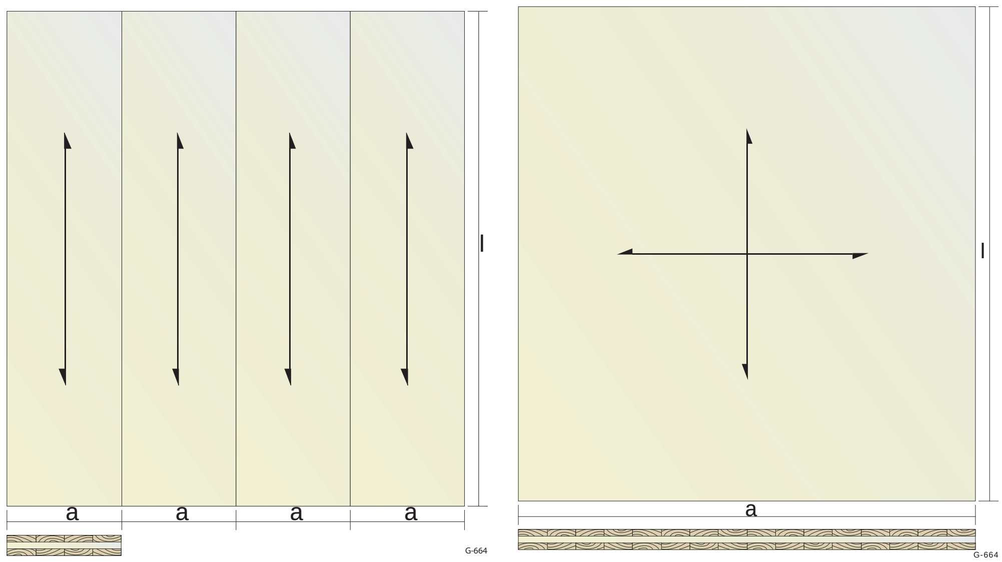


Figure 3
Example of CLT panel cross-sections and direction of fibre of the top layers



(a)

(b)

Figure 4
(a) Floor assembly made of four CLT panels acting in one direction
(b) Floor assembly made of one CLT panel acting in both directions
Distance “a” may reach 4 meters

2

ANALYTICAL PROCEDURES FOR CLT ELEMENTS

2.1 Introduction to Design Procedures used in CLT Floor, Roof and Wall Systems and their Limitations

Different methods have been adopted for the determination of basic mechanical properties of CLT in Europe. Some of these methods are experimental in nature while others are analytical. For floor elements, experimental evaluation involves determination of flexural properties by testing full-size panels or sections of panels with a specific span-to-depth ratio. The problem with the experimental approach is that every time the lay-up, type of material, or any of the manufacturing parameters change, more testing is needed to evaluate the bending properties of such new products.

Obviously, the analytical approach, once verified with the test data, offers a more general and less costly alternative. An analytical approach generally predicts strength and stiffness properties of CLT based on the material properties of the laminate planks that make up the CLT panel.

The most common analytical approach that has been adopted for CLT in Europe is based on the "Mechanically Jointed Beams Theory" (also named Gamma Method) that is available in Annex B of Eurocode 5 (EN 2004). According to this theory, the "Effective Stiffness" concept is introduced and a "Connection Efficiency Factor" (γ_i) is used to account for the shear deformation of the perpendicular layer, with $\gamma=1$ representing a completely glued member, and $\gamma=0$ no connection at all. This approach provides a closed (exact) solution for the differential equation only for simply supported beams/panels with a sinusoidal load distribution; however, the differences between the exact solution and those for uniformly distributed load or point loads are minimal and are acceptable for engineering practice (Ceccotti, 2003).

Blass and Fellmoser (2004) have applied the "Composite Theory" (also named k-method) to predict some design properties of CLT. However, this method does not account for shear deformation in individual layers and is reasonably accurate for high span-to-depth ratio.

More recently, a new method called "Shear Analogy" (Kreuzinger, 1999) has been developed in Europe that seems to be applicable for solid panels with cross layers. The methodology takes into account the shear deformation of the cross layer and is not limited to a restricted number of layers within a panel. This method seems to be the most accurate and adequate for CLT panels.

Other methods involve a combination of both empirical and analytical approaches based on model testing. No analytical approach has been universally accepted by CLT manufacturers and designers for now, and almost all of the studies have focused primarily on predicting stiffness and not strength properties of CLT panels in flexure. While flexural stiffness of CLT panels is usually of greater interest for designers than the strength, since the structural design is almost always governed by serviceability criteria, from a product standard development point of view there is a need to characterize the strength properties as well, to ensure certain minimum panel strength in service. There is a need to adopt a design methodology for determination of the stiffness and the strength properties of CLT in flexure by further exploring the shear analogy approach. It is expected that the proposed analytical approach will be accepted in the upcoming CLT product standard. The procedure to calculate the design properties should be based on material properties for lumber published in design standards, and should be consistent with the design philosophy in the CSA O86, the Canadian Standard for Engineering Design in Wood. Because of these potentially important features, the developed analytical methods will need to be comprehensively verified against test data.

Important Note: The proposed design procedures given in this chapter only apply to cross-laminated timber products manufactured with a gluing process (i.e. face-glued). Therefore, this chapter does not cover nailed or doweled CLT products.

2.2 Mechanical Properties of CLT Elements used in Floor and Wall Systems

2.2.1 Board Properties

Usually, thickness of individual boards currently produced varies from 10 mm to 50 mm and the width varies from 80 mm to 240 mm. Boards are fingerjoined using structural adhesive for longer spans. Boards are visually or machine stress-rated and are usually kiln dried to achieve average moisture content of 12% ± 2%.

Basic mechanical properties of the boards used in CLT elements vary from one producer to another. The most important European producers use boards stress graded C24 according to EN Standards (EN 338 and EN 1912) or S10 according to DIN Standard. The equivalent in Canada would be MSR 1650Fb-1.5E lumber, that gives a modulus of elasticity of about 10300 MPa (NLGA, 2010 and CSA O86). Some producers use lower grades for boards located in the inner layers and for transverse layers (e.g. C16 similar to No. 3 NLGA grade and C18 similar to 2&Btr NLGA grade). Wall elements may also be manufactured using lower grades of boards.

2.2.2 Lumber Grade and Moisture Content

Analytical design procedures given in this chapter apply to CLT elements manufactured using Canadian lumber graded in accordance with the NLGA's Standard Grading Rules for Canadian Lumber and identified by the stamp of an association or independent grading agency in accordance with CSA O141. Additionally, boards graded using in-house quality control standards may be used but shall be validated by a certification agency. It is finally recommended to use boards having a maximum moisture content of 12% ± 2% for pilot projects until further research in this area is conducted.

2.2.3 Rolling Shear Modulus and Shear Deformation – Loads Perpendicular to the Plane

2.2.3.1 Rolling Shear Modulus – Loads Perpendicular to the Plane

Rolling shear strength and stiffness in CLT has been identified as a key issue that may control the design and performance of CLT floor or wall systems. As a result of the manufacturing process of CLT panels, i.e. layers that are stacked crosswise, the load bearing behaviour of this planar element is affected by the material itself and by its constructive anisotropy (Mestek et al., 2008). Work performed at the University of British Columbia (Bejtka and Lam, 2008) on CLT panels built with Canadian lodgepole pine laminates has confirmed this finding. The magnitude of the effective bending stiffness of the panel and consequently the stress distribution in the layers depend largely on the rolling shear modulus of the cross-wise layers (Fellmoser and Blass, 2004). Little information, however, is available on the rolling shear properties of CLT panels or on the determination of such properties.

The rolling shear modulus depends on many factors such as species, cross-layer density, laminate thickness, moisture content, sawing pattern configurations (annual rings orientation), size and geometry of the board's cross-section, etc. Dynamic and numerical methods have recently been developed in Europe to measure the rolling shear modulus (Steiger et al., 2008). However, there is no general agreement among researchers, manufacturers and code officials on which method should be adopted to determine rolling shear modulus and strength. There is a lack of generalised calculations or test methods that can be adopted for the determination of rolling shear properties of CLT applicable to a wide spectrum of product lay-up details. Test methods adapted from standardized shear tests for panel type products have not been found to be satisfactory since they were developed for panels with thin layers. There is a need to develop a test method and a calculation procedure to determine the rolling shear strength and modulus of CLT.

In the literature (Mestek et al., 2008), the rolling shear modulus G_R is assumed to be 1/10 of the shear modulus parallel to the grain of the boards, G_0 (i.e. $G_R \approx G_0/10$). In Europe, the rolling shear modulus G_R of CLT panels is usually established using the Common Understanding of Assessment Procedure (CUAP) for a solid wood slab element to be used as a structural element in building (ETA request No. 03.04/06). The specified shear modulus (rolling shear modulus) of boards perpendicular to the grain (G_R) in that document is 50 MPa. The same value is proposed in Blass and Görlacher (2000). That gives a shear modulus of boards parallel to the grain, G_0, of 500 MPa.

Based on experience and the literature, the shear modulus G of wood products is generally assumed to be established between 1/12 and 1/20 of the true modulus of elasticity, i.e. $E_{true}/G_0 \approx 12$ to 20. For example, for softwood lumber, this ratio may be assumed to be 16. Using this ratio for boards made of visually graded No. 1/No. 2 SPF sawn lumber with an MOE of 9500 MPa results in G_0 being about 595 MPa and a rolling shear modulus of 59.5 MPa. In this case, the given magnitude of the rolling shear modulus in the literature seems to be on the conservative side. Thus, assuming a rolling shear modulus of 50 MPa in all cases, e.g. SPF, D Fir-L and Hem-Fir lumber, and MSR and visually graded boards, is on the conservative side. Figure 5 illustrates the rolling shear deformation behaviour of a 5-layer CLT cross-section.

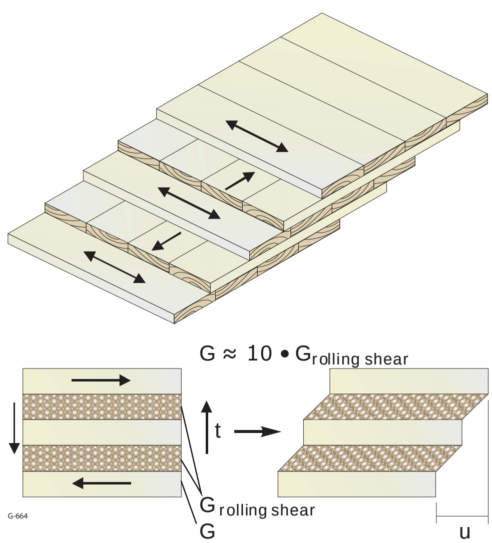


Figure 5
Rolling shear deformation of a 5-layer CLT panel

2.2.3.2 Shear Deformation – Loads Perpendicular to the Plane

It is suggested that the shear deformation of CLT panels loaded uniformly may be neglected for elements having a span-to-depth ratio (l/d) higher than 20 (Mestek et al., 2008). Other literature and CLT panel producers give as a boundary condition a minimum span-to-depth ratio of 30 before neglecting the shear deformation of the panel. This is also the ratio that is suggested for use in Canada until further research in this area is conducted. One should always be careful about setting these boundaries. Lower ratios tend to be uneconomical and have higher influence of shear deformation, while larger ones may be controlled by the vibration properties and probably creep deformation. According to preliminary calculations by the authors of this chapter using the Shear Analogy Method, for a slab with a span-to-depth ratio of 30, the contribution of shear deformation was about 11% while it was 22% for a slab with a ratio of 20.

2.3 Analytical Design Methods for CLT Elements used in Floor Systems

During the last decade, various types of analytical models for the evaluation of basic mechanical properties of CLT slab elements have been developed and proposed. This section provides more detailed information about some of the most commonly used design methods.

It is important to note that, since CLT panels are a relatively soft and light building material for slabs, the design (e.g. minimum thickness and maximum span) is often more driven by serviceability criteria (e.g. vibration, deflection and creep) than by strength ones (e.g. bending and shear strength).

2.3.1 Mechanically Jointed Beams Theory (Gamma Method)

2.3.1.1 General Assumptions and Calculations

Some CLT panel manufacturers use the design philosophy of Mechanically Jointed Beams Theory that is included in Annex B of Eurocode 5 (EN 1995: 2004). As the name suggests, this method was originally developed for beams (e.g., I or T beams) connected with mechanical fasteners with stiffness *K* uniformly spaced at distance *s* along the length of the beams. This method, also named Gamma Method (γ-method), was developed in 1955 by Professor Karl Möhler. According to this method, the stiffness properties of the mechanically jointed beams are defined using the Effective Bending Stiffness (EI_{eff}) that depends on the section properties of the beams and the connection efficiency factor γ. Factor γ depends on the slip characteristics of the fasteners (s/K ratio), being zero for no mechanical connection between the beams and equalling unity for rigidly connected (glued) beams.

Since CLT panels are glued products with no mechanical joints present, some modifications were needed to the theory to make it applicable to CLT panels. If we assume that only boards oriented in the longitudinal direction are carrying the load, then we can take into account the rolling shear stiffness (or deformability) of the cross layers as stiffness (or deformation) caused by "imaginary fasteners" connecting the longitudinal layers. In other words, the longitudinal layers of the CLT panels are taken as "beams" connected with "mechanical fasteners" that have stiffness equal to that of the rolling shear deformation of the cross layers (Figure 5). In this case, the s/K_i ratio for "fasteners" at each interface "i" in the equation for determining the factor should be replaced with the rolling shear slip (shear deformation between load carrying layers) according to equation [1].

$$\frac{s}{K_i} = \frac{\bar{h}_i}{G_R \cdot b} \qquad [1]$$

where:

G_R = shear modulus perpendicular to the grain (rolling shear modulus)
$\bar{h}_i$ = thickness of board layers in direction perpendicular to the action
b = width of the panel (normally 1 meter)
s = spacing between mechanical fasteners (but not present in glued CLT)
K_i = slip modulus of mechanical fasteners (but not present in glued CLT)

The mechanically jointed beams theory is derived using simple bending theory; therefore, all its basic assumptions are valid. Shear deformations are neglected in the "beams" (i.e. longitudinal layers of the CLT slab) and are included only for the cross layers by evaluating the rolling shear deformation. This approach provides a closed (exact) solution for the differential equation only for simply supported beams/panels with a sinusoidal (or uniform) load distribution giving a moment M = M(x) varying sinusoidally or parabolically. However, the differences between the exact solution and those for uniformly distributed load or point loads are small. Table 1 shows, for example, the ratio between mid-span deflection of timber-concrete T beam (2.5 m and 10 m spans) with deformable connections (values calculated exactly) and the deflection of the beam with perfectly rigid connections, under various loadings. As can be seen, the differences are less than 3%, and are deemed acceptable for engineering practice (Ceccotti, 2003).

Table 1

Ratio between mid-span deflection of concrete-wood T beam with deformable connections (values calculated exactly) and the deflection of the beam with perfectly rigid connections, under various loadings

Type of Load	Beam with 2.5 m Span	Beam with 10 m Span
Concentrated load at mid span	1.9313	1.3492
Concentrated load at third points	1.9060	1.3266
Uniformly distributed load	1.9039	1.3258
Sinusoidal load	1.9021	1.3190

The mechanically jointed beams theory assumes that CLT elements are simply supported and have a span of "*l*". For cantilever CLT slabs, it is suggested that the length *l* to be used in the calculations should be equal to two times the cantilever length *lc*. To determine the Effective Bending Stiffness (EI_{eff}) in continuous multi-supported beams, two approaches are suggested: a simplified procedure, and an iterative procedure. Since the γ factor (and therefore the effective stiffness) value depends on the length of the beam between the two zero-moment points (inflection points), according to the simplified procedure one can take the span in calculations to be equal to 0.8 *l*. In the iterative procedure, one can start by considering the EI_{eff} along the length of the beam calculated using a certain length l (say 0.8 *l*) and use a simple computer program to determine the points of inflection for a beam with that EI_{eff}. Then, by obtaining the new length between deflection points, one should re-calculate the EI_{eff} and do the analysis again. Usually after only a few iterations a stable solution for the EI_{eff} can be obtained.

As previously mentioned, rolling shear modulus G_R can be assumed to be 1/10 of the shear modulus parallel to the grain of the boards, G_0 (i.e. $G_R \approx G_0/10$). The rolling shear modulus G_R recommended for use in CUAP 2005 is 50 MPa. Some CLT manufacturers publish a value of 60 MPa, while others will adjust this value to the corresponding bending stiffness of lumber used in the panel (i.e. the higher the MOE, the higher the G_R). Most common values of G_R for spruce vary from 40 to 80 MPa.

The formulae and examples of calculations of the effective bending stiffness (EI_{eff}) of CLT panels (slabs) with five and seven layers are given in Section 3 of this chapter. It can be seen that only longitudinal layers, i.e. layers acting in the direction of the loading (net cross-section), are used for calculating the EI_{eff}, while the cross layers are taken into account only through their rolling shear properties. It should be noted that this calculation method applies to CLT slabs with relatively high span-to-depth ratios (i.e. 30 and higher) since it ignores the contribution of the shear deformation in the longitudinal layers.

2.3.1.2 Bending Strength and Stiffness – Loads Perpendicular to the Plane (Floor and Roof)

The evaluation process of CLT panels in most ETA product approvals in Europe employs a hybrid approach by using a mix of analytical models and mechanical testing. Tests are based on existing standards (e.g. EN, DIN) normally using the CUAP 03.04/06 (2005). This document stipulates that the bending strength of the slab needs to be defined in relation to the effective section modulus S_{eff} of the CLT element. The bending strength shall then be calculated from the test results and using the effective section modulus.

The expression for the effective section modulus is shown in equation [2]:

$$S_{eff} = \frac{2 \times I_{eff}}{h_{tot}} = \frac{I_{eff}}{0.5 \times h_{tot}} \quad [2]$$

where:

S_{eff} = effective section modulus
I_{eff} = effective moment of inertia (see Figure 6 and Section 3)
h_{tot} = total depth of the panel

$$EI_{eff} = \sum_{i=1}^{n} (E_i I_i + \gamma_i E_i A_i a_i^2) \quad [2a]$$

where $0<\gamma\leq1$ ($\gamma=1$ for rigid connection and $\gamma=0$ for no connection. But typically γ may vary from 0.85 to 0.99).

Following the CUAP 03.04/06, bending tests shall be performed using national standard EN 408, *Timber structures – Structural timber and glued laminated timber – Determination of some physical and mechanical properties* and observing the principles given in standard EN 789, *Timber structures – Test methods – Determination of mechanical properties of wood based panels.*

However, according to the mechanically jointed beams theory, and according to Appendix B of Eurocode 5, the maximum bending stress in the panel can be obtained as:

$$\sigma_{max} = \sigma_{global} + \sigma_{local} \quad [3]$$

where σ_{local} is the stress in the outside layer as a consequence of bending of that layer, while σ_{global} is the axial stress developed in the outside layer due to bending. Local and global stresses can be obtained according to the equations [4] and [5].

$$\sigma_{global} = \frac{\gamma_1 E_1 a_1 M}{(EI)_{eff}} \quad [4]$$

$$\sigma_{local} = \frac{0.5 E_1 h_1 M}{(EI)_{eff}} \quad [5]$$

The term a_1 is the distance between the centroid of the first lamina and the centroid of the panel cross-section, and h_1 is the thickness of the first (outermost) lamina (see Section 3). Having the equations [4] and [5] in mind, the maximum bending stress can be expressed as:

$$\sigma_{max} = \frac{\gamma_1 E_1 a_1 M}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M}{(EI)_{eff}} \quad [6]$$

or in other words:

$$\sigma_{max} = \frac{M E_1}{(EI)_{eff}} \cdot (\gamma_1 a_1 + 0.5 h_1) \quad [7]$$

When the modulus of elasticity of all longitudinal layers is equal, i.e. $E_1=E_2=E_3=E$, then the maximum bending stress can be obtained as:

$$\sigma_{\max} = \frac{M}{I_{eff}} \cdot (\gamma_1 a_1 + 0.5h_1) \qquad [8]$$

Note: Some producers in Europe use only local bending stresses (σ_{local}) in their calculations (see Equation B.8 from section B.3 in Eurocode 5). However, global stresses (σ_{global}) should be added to find the total bending stress in any layer (see equation B.7 from section B.3 in Eurocode 5).

If we use CSA O86 design analogy, we can let:

$$\sigma_{\max} = \sigma_{global} + \sigma_{local} \leq \phi \cdot F_b \qquad [9]$$

and determine the factored moment bending resistance M_r in terms of the specified bending strength F_b as:

$$M_r = \phi \cdot F_b \cdot \frac{I_{eff}}{(\gamma_1 a_1 + 0.5h_1)} \qquad [10]$$

Equation [10] is valid when the modulus of elasticity of all longitudinal layers is equal.

2.3.1.3 Shear Strength – Loads Perpendicular to the Plane (Floor and Roof)

Experimental methods are normally used for assessing the shear strength of a structural glued product. It is stipulated in the CUAP 03.04/06 that shear tests shall be performed using the principles of EN 408, *Timber structures – Structural timber and glued laminated timber – Determination of some physical and mechanical properties*. Tests shall be performed on simply supported slabs using loads applied to the full width of the panels and close enough to the supports to create a shear failure. The shear strength is then calculated using the following equation:

$$\tau = \frac{1.5 \times V}{A_{gross}} \qquad [11]$$

where:

τ = maximum shear strength (MPa)
V = maximum shear force (N)
A_{gross} = gross cross-sectional area of the panel = $b \times h_{tot}$ (mm^2)

According to the simple bending theory (and theory of mechanically jointed beams), maximum shear stresses occur where the normal stresses are equal to zero, and the shear stress can be obtained as:

$$\tau = \frac{V \cdot (EQ)}{(EI)_{eff} \cdot b} \qquad [12]$$

where:

τ = shear stress (MPa)
V = maximum shear force (N)
Q = static moment of area for the cross-section (mm^3)
b = width of the cross-section perpendicular to the shear flow (mm); usually 1000 mm

For a CLT panel with five layers (see Figure 6), the static moment of area, Q, for that part of the section above the centroidal axis, can be calculated as:

$$(EQ) = \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 + E'_1 \cdot A'_1 \cdot a'_1 + \gamma_2 \cdot E_2 \cdot \frac{A_2}{2} \cdot \frac{h_2}{4} \qquad [13]$$

So if we use CSA O86 design analogy, we can let:

$$\tau \le \phi \cdot F_v \qquad [14]$$

Having in mind equations [12] to [14], we can express the factored longitudinal shear resistance, V_{rL}, in terms of the specified shear strength, F_v, as:

$$V_{rL} = \frac{\phi \cdot F_v \cdot (EI_{eff}) \cdot b}{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 + E'_1 \cdot A'_1 \cdot a'_1 + \gamma_2 \cdot E_2 \cdot \frac{A_2}{2} \cdot \frac{h_2}{4}} \qquad [15]$$

In a similar way, with the appropriate modifications, the equations for CLT panels with three or seven layers can be developed. In the case of three-layered panels, it should be noted that the strength F_v should be replaced with the rolling shear strength F_{vR}.

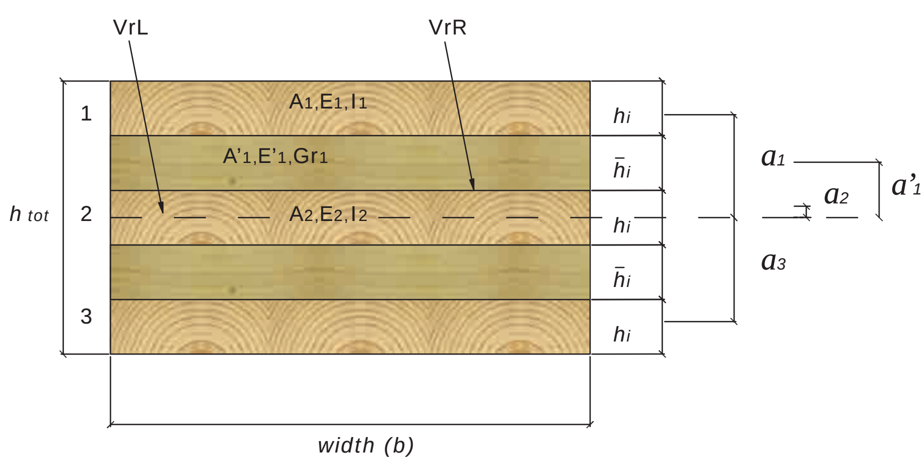


Figure 6
Cross-section of CLT panel with five layers

In CLT panels with five layers or more, the shear strength at the cross layers (rolling shear resistance) should also be checked. In this case, the static moment of area *Q* should be calculated for an axis just above the middle layer and can be expressed as:

$$(EQ) = \gamma_1 E_1 A_1 (a_1 - \frac{h_2}{2}) + E'_1 \, A'_1 \, (a'_1 - \frac{h_2}{2}) \qquad [16]$$

The factored rolling shear resistance, V_{rR}, can be expressed in terms of the specified rolling shear strength, F_{vR}, according to equation [17].

$$V_{rR} = \frac{\phi \cdot F_{vR} \cdot (EI_{eff}) \cdot b}{\gamma_1 E_1 A_1 (a_1 - \frac{h_2}{2}) + E'_1 \, A'_1 \, (a'_1 - \frac{h_2}{2})} \qquad [17]$$

The shear resistance of the CLT panel, V_r, should then be chosen as the lower value of the longitudinal shear resistance, V_{rL}, and the rolling shear resistance, V_{rR}, as shown in equation [18].

$$V_r = \min\ (V_{rL}\ or\ V_{rR}) \qquad [18]$$

2.3.2 Composite Theory – k Method

2.3.2.1 General Assumptions

This design method is well-known in the plywood industry. In the original version of this method, the plies of plywood panels stressed perpendicular to the grain are not taken into account in the calculation of the properties in bending (i.e. $E_{90} = 0$).

To overcome this deficiency, with respect to CLT panels, the general method used to calculate the effective bending stiffness (EI_{eff}) has been modified and is based on the following assumptions:

- A linear stress-strain relationship and Bernoulli's hypothesis of plane cross-sections remaining plane are assumed;
- The calculation method is based on the strength and stiffness properties of all layers; layers loaded parallel to the grain and cross layers loaded perpendicular to the grain. Stiffness of cross layers as used in the calculations is taken as: $E_{90} = E_0\ / 30$;
- Shear deformation is not taken into account. Therefore, the method may be used only for relatively high span-to-depth ratios (i.e. l/h ≥30);
- Composition factors are determined for certain loading configurations (see Table 2).

Table 2 provides the formulas to evaluate the composition ki factors for certain configurations of loading with respect to the panel orientation. For instance, the factor k1 represents the composite factor for plates loaded perpendicular to the plane and is used for calculating the properties in bending parallel to the panel. Table 3 gives the effective values of strength and stiffness for solid wood panels with cross layers (Source: Blass, 2004).

Table 2

Composition factors "k" for solid wood panels with cross layers (Source: Blass, 2004)

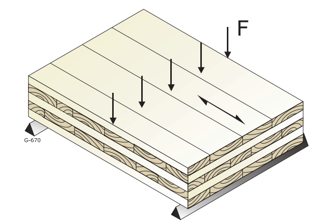
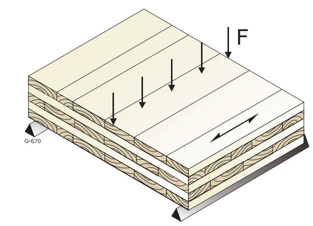
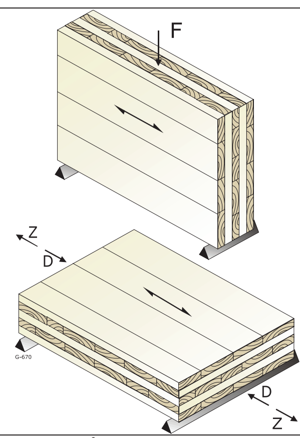
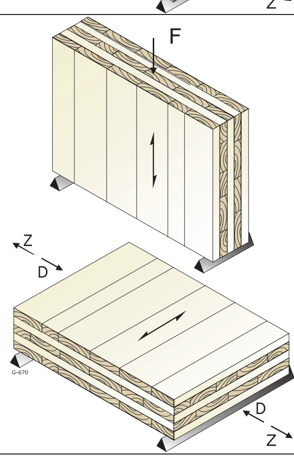
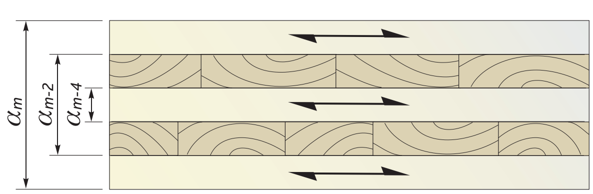
Load Configuration	k_i
	$k_1 = 1 - \left(1 - \frac{E_{90}}{E_0}\right) \cdot \frac{a_{m-2}^3 - a_{m-4}^3 + \ldots \pm a_1^3}{a_m^3}$
	$k_2 = \frac{E_{90}}{E_0} + \left(1 - \frac{E_{90}}{E_0}\right) \cdot \frac{a_{m-2}^3 - a_{m-4}^3 + \ldots \pm a_1^3}{a_m^3}$
	$k_3 = 1 - \left(1 - \frac{E_{90}}{E_0}\right) \cdot \frac{a_{m-2} - a_{m-4} + \ldots \pm a_1}{a_m}$
	$k_4 = \frac{E_{90}}{E_0} + \left(1 - \frac{E_{90}}{E_0}\right) \cdot \frac{a_{m-2} - a_{m-4} + \ldots \pm a_1}{a_m}$
	

Table 3
Effective values of strength and stiffness for solid wood panels with cross layers (Source: Blass, 2004)

Loading	To the grain of outer skins	Effective strength value	Effective stiffness value
Perpendicular to the plane loading			
Bending	Parallel	$f_{b,0,eff} = f_{b,0} \cdot k_1$	$E_{b,0,eff} = E_0 \cdot k_1$
	Perpendicular	$f_{b,90,eff} = f_{b,0} \cdot k_2 \cdot a_m/a_{m-2}$	$E_{b,90,eff} = E_0 \cdot k_2$
In-plane loading			
Bending	Parallel	$f_{b,0,eff} = f_{b,0} \cdot k_3$	$E_{b,0,eff} = E_0 \cdot k_3$
	Perpendicular	$f_{b,90,eff} = f_{b,0} \cdot k_4$	$E_{b,90,eff} = E_0 \cdot k_4$
Tension	Parallel	$f_{t,0,eff} = f_{t,0} \cdot k_3$	$E_{t,0,eff} = E_0 \cdot k_3$
	Perpendicular	$f_{t,90,eff} = f_{t,0} \cdot k_4$	$E_{t,90,eff} = E_0 \cdot k_4$
Compression	Parallel	$f_{c,0,eff} = f_{c,0} \cdot k_3$	$E_{c,0,eff} = E_0 \cdot k_3$
	Perpendicular	$f_{c,90,eff} = f_{c,0} \cdot k_4$	$E_{c,90,eff} = E_0 \cdot k_4$

2.3.2.2 Bending Strength and Stiffness – Loads Perpendicular to the Plane (Floor and Roof)

The maximum bending stress may be expressed as:

$$\sigma_{\max} = \frac{M}{S} \qquad [19]$$

If we use CSA O86 design analogy, we can let:

$$\sigma_{\max} \leq \phi \cdot F_{b,eff} \qquad [20]$$

where $F_{b,eff}$ is the effective bending strength value $f_{b,0,eff}$ obtained from Tables 2 and 3.

Thus, the factored moment bending resistance, M_r, in terms of the specified bending strength F_b, can be expressed as:

$$M_r = \phi \cdot F_{b,eff} \cdot S_{gross} \qquad [21]$$

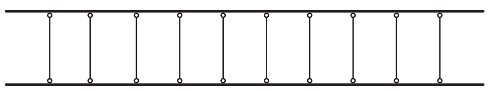
Examples are given in Section 3 for the calculation of the effective bending stiffness (EI_{eff}) and bending strength of CLT panels using the k-method.

2.3.3 Shear Analogy Method (by Kreuzinger)

2.3.3.1 General Assumptions and Procedure

This calculation method is, according to the literature (Blass and Fellmoser, 2004), the most precise design method for CLT. It is used, with the help of a plane frame analysis program, to consider the different moduli of elasticity and shear moduli of single layers for nearly any system configuration (e.g. number of layers, span-to-depth ratio). The effect of shear deformations is not neglected. In the shear analogy method, the characteristics of a multi-layer cross-section or surface (such as multi-layer CLT panels) are separated into two virtual beams

A and B. Beam A is given the sum of the inherent flexural strength of the individual plies along their own neutral axes, while beam B is given the "Steiner" points part of the flexural strength, the flexible shear strength of the panel, as well as the flexibility of all connections. These two beams are coupled with infinitely rigid web members, so that an equal deflection between beams A and B is obtained. By overlaying the bending moment and shear forces (stresses) of both beams, the end result for the entire cross-section can be obtained (Figure 7).



Beam A (bending stiffness $(EI)_A = B_A$ and shear stiffness $(GA)_A = S_A \sim \infty$)

Web members with infinite axial rigidity

Beam B (bending stiffness $(EI)_B = B_B$ and shear stiffness $(GA)_B = S_B$)

Figure 7
Beam differentiation using the shear analogy method

Beam A is assigned a bending stiffness equal to the sum of the inherent bending stiffness of all the individual layers or individual cross-sections as shown in equation [22].

$$B_A = \sum_{i=1}^{n} E_i \cdot I_i = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} \qquad [22]$$

where:

B_A = $(EI)_A$
b_i = width of each individual layer, usually taken as 1 m for CLT panels
h_i = thickness of each individual layer

The bending stiffness of beam B is calculated using Steiner's theorem (given as the sum of the Steiner points of all individual layers):

$$B_B = \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2 \qquad [23]$$

where B_B is $(EI)_B$ and z_i is the distance between the center point of each layer and the neutral axis (see Section 3).

Additionally, beam B contains the shear stiffness and the stiffness of the flexible connections, if they exist. The shear stiffness of beam B, S_B, is $(GA)_B$ and can be calculated as:

$$\frac{1}{S_B} = \frac{1}{a^2} \cdot \left[\sum_{i=1}^{n-1} \frac{1}{k_i} + \frac{h_1}{2 \cdot G_1 \cdot b_1} + \sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i} + \frac{h_n}{2 \cdot G_n \cdot b_n} \right] \qquad [24]$$

where:

$$k_i = \frac{K_i}{s_i} \qquad [25]$$

is the slip of the "fasteners" between the beams.

In the above equations, the values for E_0 shall be used for the longitudinal layers while $E_{90} = E_0/30$ is suggested to be used for cross layers. Also, in the same equations, the shear modulus for the longitudinal layer should be assumed to be G, while that for the cross layers shall be, for the rolling shear, G_R.

The auxiliary members have infinite flexural strength and shear strength and serve only to connect the two beams. The continuity of deflections between beams A and B ($\Delta_A = \Delta_B$) must be valid at every point. Using a spreadsheet, the virtual section sizes of beams A and B and the values for M_A, M_B, V_A and V_B are produced. Bending moments $M_{A,i}$ and shear forces $V_{A,i}$ of each individual layer of beam A can be obtained using the equations [26] and [27] respectively.

$$M_{A,i} = \frac{E_i \cdot I_i}{B_A} \cdot M_A \qquad [26]$$

$$V_{A,i} = \frac{E_i \cdot I_i}{B_A} \cdot V_A \qquad [27]$$

where M_A and V_A are the bending and shear forces on beam A.

Bending stresses $\sigma_{A,i}$ and shear stresses $\tau_{A,i}$ of each individual layer of beam A can be obtained using the equations [28] and [29] respectively.

$$\sigma_{A,i} = \pm \frac{M_{A,i}}{I_i} \cdot \frac{h_i}{2} \qquad [28]$$

$$\tau_{A,i} = \frac{E_i \cdot I_i}{B_A} \cdot 1.5 \cdot \frac{V_A}{b \cdot h_i} \qquad [29]$$

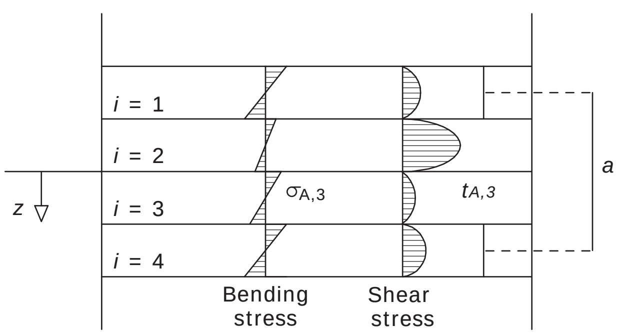


Figure 8
Bending and shear stresses in beam A using the shear analogy method (Source: Kreuzinger)

Axial forces $N_{B,i}$, normal stresses $\sigma_{B,i}$ of each individual layer of beam B, and shear stresses at the interface of the two layers of beam B, $\tau_{B,i,i+1}$ can be obtained using the equations [30], [31] and [32] respectively.

$$N_{B,i} = \frac{E_i \cdot A_i \cdot z_i}{B_B} \cdot M_B \qquad [30]$$

$$\sigma_{B,i} = \frac{N_{B,i}}{b_i \cdot h_i} = \frac{E_i \cdot z_i}{B_B} \cdot M_B \qquad [31]$$

$$t_{Bi,i+1} = \frac{V_B}{B_B} \cdot \sum_{j=i+1}^{n} E_j \cdot A_j \cdot z_j \qquad [32]$$

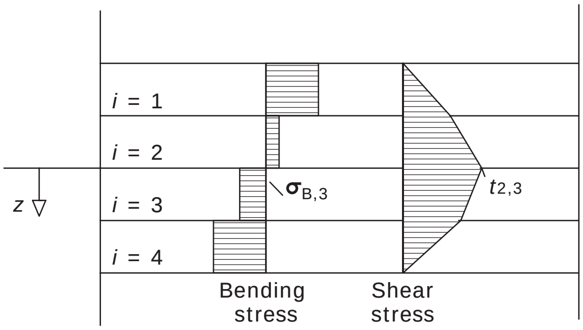


Figure 9
Normal and shear stresses in beam B using the shear analogy method (Source: Kreuzinger)

The final stress distribution obtained from the superposition of the results from beams A and B is shown in Figure 10. It should be noted that the shear distribution in Figure 10 includes the influence of the connector devices that will not be existent for a CLT panel.

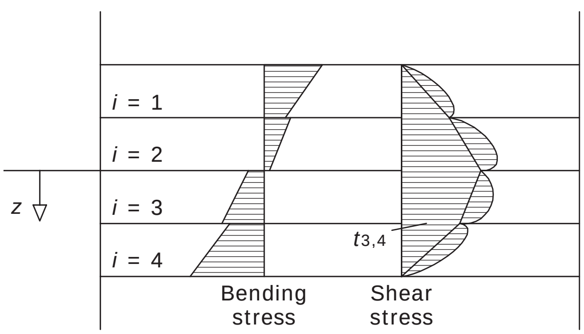


Figure 10
Final stress distribution obtained from the superposition of the results from beams A and B (Source: Kreuzinger)

Using the shear analogy method, the maximum deflection u_{max} in the middle of the CLT slab under a uniformly distributed load can be calculated as a sum of the contribution due to bending and to shear:

$$u_{\max} = \frac{5}{384} \cdot \frac{qL^4}{(EI)_{eff}} + \frac{1}{8} \cdot \frac{qL^2 k}{(GA)_{eff}} \qquad [33]$$

or in other terms:

$$u_{\max} = \frac{5}{384} \cdot \frac{qL^4}{(EI)_{eff}} \cdot \left(1 + \frac{48(EI)_{eff}\, k}{5(GA)_{eff}\, L^2}\right) \qquad [34]$$

which can be expressed as:

$$u_{\max} = \frac{5}{384} \cdot \frac{qL^4}{(EI)_{eff}} (\alpha + \beta) \qquad [35]$$

where α = 1.0 and β can be expressed according to equation [36], where *k* (kappa) is the shear coefficient form factor equal to 1.2 (i.e. 6/5 = 1.2) (see Timoshenko).

$$\beta = \frac{48(EI)_{eff}\, \kappa}{5(GA)_{eff}\, L^2} \qquad [36]$$

The effective bending stiffness can be obtained using equation [37].

$$(EI)_{eff} = B_A + B_B = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2 \qquad [37]$$

The effective shear stiffness can be obtained using equation [38].

$$(GA)_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_1 \cdot b}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i}\right) + \left(\frac{h_n}{2 \cdot G_n \cdot b}\right)\right]} \qquad [38]$$

In the case of a concentrated force *P* in the middle of the span of the CLT slab, the equation for the maximum deflection is given as:

$$u_{\max} = \frac{1}{48} \cdot \frac{PL^3}{(EI)_{eff}} + \frac{1}{4} \cdot \frac{PL}{(GA)_{eff}/\kappa} = \frac{1}{48} \cdot \frac{PL^3}{(EI)_{eff}} \left(1 + \frac{12 \cdot (EI)_{eff}\, \kappa}{(GA)_{eff}\, L^2}\right) \qquad [39]$$

which can be expressed as:

$$u_{\max} = \frac{1}{48} \cdot \frac{PL^3}{(EI)_{eff}} (\alpha + \beta) \qquad [40]$$

where α = 1.0 and β can be expressed according to the equation [41], where *k* (kappa) is the shear coefficient form factor equal to 1.2 (6/5 = 1.2).

$$\beta = \frac{12(EI)_{eff}\,\kappa}{(GA)_{eff}\,L^2} \quad [41]$$

Some examples of the calculation of the effective true bending stiffness (EI_{eff}) and effective shear stiffness (GA_{eff}) using the shear analogy method are given in Section 3.

2.3.4 Simplified Design Methods for Calculating Bending and Shear Strengths (Out-of-Plane)

The next equations are simplified design methods proposed for calculating the capacity in bending and in shear of CLT elements acting as floors and ceilings.

2.3.4.1 Bending Strength

The bending stress σ may be expressed as:

$$\sigma = M \cdot y \cdot \frac{(E_1)}{(EI)_{eff}} \quad [42]$$

The maximum stress will occur for $y = \frac{h_{tot}}{2}$, so equation [42] can be expressed as:

$$\sigma_{\max} = M \cdot 0.5h_{tot} \cdot \frac{(E_1)}{(EI)_{eff}} \quad [43]$$

If we use CSA O86 design analogy, we can let:

$$\sigma_{\max} \leq \phi \cdot F_b \quad [44]$$

and determine the factored moment bending resistance M_r in terms of the specified bending strength F_b as:

$$M_r = \phi \cdot F_b \cdot \frac{(EI)_{eff}}{E_1} \cdot \frac{1}{0.5h_{tot}} \quad [45]$$

where E_1 is the modulus of elasticity of the outer longitudinal layer in tension and $(EI)_{eff}$ is determined according to Sections 2.3.1, 2.3.2 or 2.3.3.

When the modulus of elasticity of all longitudinal layers is equal, then equation [45] can be expressed as:

$$M_r = \phi \cdot F_b \cdot \frac{I_{eff}}{0.5h_{tot}} \quad [46]$$

2.3.4.2 Shear Strength

Other methods for determining the maximum shear strength of CLT panels in the literature are shown in equations [47] and [49].

$$\tau_v = \frac{1.5 \times V}{c \cdot A_{gross}} \quad [47]$$

where coefficient "c" is a reduction factor calculated as:

$$c = \frac{I_{eff}}{I_{gross}} \quad [48]$$

According to Blass (2004), the maximum shear strength of CLT panels can be calculated as:

$$\tau_v = \frac{V \gamma_i Q_i}{I_{eff} b} \quad [49]$$

where:

Q_i = static moment of area, calculated in a way similar to equation [13].

2.3.5 Regular Two-Way Slab System Loaded Perpendicular to the Slab Plane

CLT elements used in floor assemblies will normally act in the principal direction when loaded perpendicular to the plane. In a floor/roof assembly, the slab made of CLT panels may be supported on walls, beams or columns, or by a mix of support conditions. For instance, the plate can be simply supported on two parallel sides at the extremities and free, or connected to another plate, along the other two edges. It should be noted that a CLT slab can also be supported on three or even four sides, as there are panels on the market that have a width of 3 or even 4 meters. Consequently, the two-way behaviour of the CLT slab system has to be carefully studied as well. Such an evaluation has to include the influence of the support conditions, as different support conditions may modify the relative effective stiffness of the plates at the supports:

- Plate hinged along two edges and free along two edges;
- Plate hinged along three or four edges;
- Plate supported on columns.

In the evaluation of the two-way action of CLT slabs, some of the details related to the design of concrete slabs may be adopted. According to the Concrete Design Handbook (CDH, 2005) and CSA A23.3, Design of Concrete Structures, a regular two-way slab system is one that consists of approximately rectangular panels supporting primarily uniform gravity loading. In particular, it is mentioned that this system shall meet geometric limitations such as the following:

1. Within a panel, the ratio of the longer to the shorter span, centre-to-centre of supports, is not greater than 2.0;

2. For slab systems with beams (or walls) between supports, the relative effective stiffness of beams (or walls) in the two directions is not less than 0.2 or greater than 5.0;

3. Column offsets are not greater than 20% of the span (in the direction of the offset) from either axis between centrelines of successive columns; and

4. The reinforcement is placed in an orthogonal grid (similar to CLT panel behaviour).

Figure 11 illustrates two rectangular plates in bending, where "a" is the shorter span and "b" the longer span. Case A illustrates a rectangular plate having a ratio of b/a greater than 2 (b/a > 2) while Case B shows a rectangular plate having a ratio of b/a lower or equal to 2 (b/a ≤ 2).

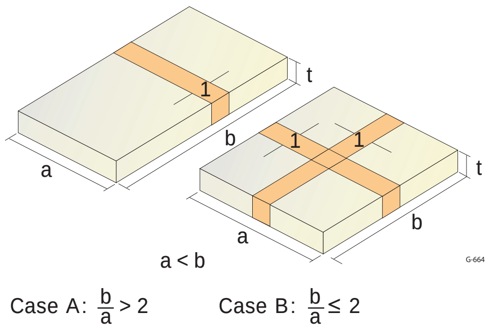


Figure 11
CLT panel loaded perpendicular to the plane

Based on the theory of plates (Timoshenko, 1959) and on the details presented above, it is suggested that plates supported on four sides should be designed in one direction (i.e. short direction) when b/a > 2. In that case, the length L used in the design should be "a". For plates supported on four sides and having a ratio of b/a ≤ 2, the design should be made in two directions with $L_1 = a$, and $L_2 = b$.

It should be noted that the calculation of bending moments and deflections of rectangular CLT plates acting in two directions is quite complex and should take into consideration many parameters, e.g. support conditions, relative effective stiffness at the supports, MOE of longitudinal and transversal layers as well as parallel and perpendicular to the action of the load, rolling shear in both directions, etc. Therefore the complexity of the design in many cases may outweigh the benefits of taking the two-way action into account. In most cases, the design of a CLT plate in a single direction will result in a more conservative solution. It is also suggested to use a minimum of 5 layers if the two-way action needs to be evaluated. The verification shall be made using the 3-layer section in the center of the panel (without the outer layers).

2.3.6 Bending Strength and Bending Stiffness – Loads Parallel to the Plane (Diaphragms)

Floor and roof diaphragms are important horizontal structural elements in wood buildings that carry vertical loads as well as lateral loads. The inertia forces caused by earthquakes or lateral forces from wind need to be transferred by the diaphragm to the supporting walls and then to the foundation. Over this load path, the in-plane stiffness and strength of the diaphragms will affect the load distribution among the wall systems, which will affect the design. It is unclear at this point if the common assumption of flexible wood diaphragms can be applied to CLT wood diaphragms. In addition, an appropriate design model for estimating the in-plane stiffness of CLT diaphragms needs to be developed. Research studies on the in-plane stiffness and strength of CLT diaphragms have not been conducted.

Consequently, it is suggested that buildings with CLT diaphragms be designed using the International Building Code analogy (IBC, 2006). One should first design the structure using the flexible diaphragm assumption, and then do the same using the rigid diaphragm approach. The more critical solution should then govern the final design.

2.3.7 Additional Stresses

If the boards used in the cross layers are not edge-bonded (glued), additional stresses perpendicular to the grain may occur in these boards due to the rotation around their longitudinal axis. These rolling shear and tensile perpendicular to the grain stresses should be verified by testing if the ratio of board width to board thickness is less than 4. If wider boards are grooved to reduce stress and to keep the boards straight prior to gluing and pressing, the grooves are regarded as free (unbonded) edges.

2.3.8 Cantilevered and Statically Indeterminate CLT Elements

The proposed analytical design procedures generally assume that CLT elements are simply supported with a span of "l". For cantilevered CLT slabs, it is suggested that the length l in the calculations be taken as two times the cantilever length l_c.

To determine the Effective Bending Stiffness (EI_{eff}) in continuous multi-supported beams, two approaches are suggested: a simplified procedure, and an iterative procedure. In the simplified procedure, the span in the calculations is taken as 0.8 l. In the iterative procedure, one can start by considering the EI_{eff} along the length of the beam calculated using a certain length l (say 0.8 l) and use a simple computer program to determine the points of inflection for a beam with that EI_{eff}. Then by obtaining the new length between deflection points, one should re-calculate the EI_{eff} and do the analysis again. Usually after only a few iterations a stable solution for the EI_{eff} can be obtained.

2.3.9 CLT Slab Supported by a Post (Compressive Resistance Perpendicular to the Grain)

As a minimum, the capacity of CLT panels in compression perpendicular to the grain should be verified using the loading surface of the post and using design provisions given in CSA O86-09, Article 6.5.9. Additionally, the panels should be verified in bending and shear in the two directions (see Section 2.3.5).

2.4 Analytical Design Methods for CLT Elements used in Wall Systems

CLT panels used as wall systems are subjected to three types of loading:

1) vertical in-plane loading from the gravity loads,
2) lateral in-plane loading coming from wind and earthquake loads, and
3) lateral out-of-plane loading that comes from wind loads.

Not much information is available in the literature for CLT walls subjected to in-plane loading.

If CLT walls were under out-of-plane wind loading only, they should be analysed in the same way as floor systems under vertical loads as in Section 2.3 of this chapter.

It should be noted that, for wall applications, especially for 3-layer panels, CLT wall panels should normally be placed with the outer layers parallel to the gravity loads.

2.4.1 CLT Wall Panels Under Axial In-Plane Loads and Out-of-Plane Loads

CLT walls under axial in-plane loads in combination with out-of-plane wind loads can be designed using different approaches. Details about these approaches are described further in this section.

2.4.1.1 Mechanically Jointed Columns Theory (Eurocode 5)

Details of the mechanically jointed columns theory are included in Annex C of Eurocode 5. Following this method and assumptions made, the effective slenderness ratio λ_{eff} can be calculated as:

$$\lambda_{eff} = l \cdot \sqrt{\frac{A_{tot}}{I_{eff}}} \qquad [50]$$

where A_{tot} is the total cross-sectional area of the panel, l is the height (buckling length) of the wall element, and the effective moment of inertia I_{eff} is given as:

$$I_{eff} = \frac{(EI)_{eff}}{E_{mean}} \qquad [51]$$

where $(EI)_{eff}$ is determined according to Sections 2.3.1, 2.3.2 or 2.3.3, and E_{mean} is the modulus of elasticity of boards acting parallel to the axial load (i.e. vertical layers).

The effective slenderness ratio λ_{eff} can then be substituted in equation 6.21 of Eurocode 5, and the compressive resistance of the CLT walls under axial loads, or under combined axial and bending loads, can be calculated using Section 6.3 of Eurocode 5.

2.4.1.2 CSA O86-09 Approach Combined with Mechanically Connected Beams Theory

Some of the cross-sectional properties for CLT panels calculated using the mechanically connected beams theory can be used in combination with Clause 5.5.6 from the Canadian Timber Design Standard CSA O86-09 to calculate the compressive resistance of CLT walls. According to this method, the resistance of the cross layers is not taken into account, or in other words, it is assumed that only the layers oriented parallel to the axial force carry the load. Using CSA O86-09 Clause 5.5.6.2.2, the slenderness ratio C_c for rectangular CLT walls can be calculated as:

$$C_c = \frac{H}{d} = \frac{H}{2\sqrt{3} \cdot r_{eff}} \qquad [52]$$

where r_{eff} can be calculated as:

$$r_{eff} = \sqrt{\frac{I_{eff}}{A_{eff}}} \qquad [53]$$

where I_{eff} can be calculated using one of the three methods proposed in Section 2.3.

A_{eff} can be calculated as:

$$A_{eff} = b \cdot h_{eff} = b \cdot \sum_i h_i \qquad [54]$$

where b is normally taken as 1000 mm and h_i is the thickness of boards parallel to the axial load.

The design procedure for determining the buckling strength can continue as specified in Clause 5.5.6 of CSA O86-09, substituting the cross-section area A with A_{eff}, and the total thickness d with the effective thickness h_{eff}. The width of the cross-section should be taken as 1000 mm. It should be noted that many producers in Europe limit the panel slenderness ratio H/r_{eff} to 150.

Using the same substitutions, including the substitution of I_{eff} for I, the compressive resistance of CLT walls with combined axial and out-of-plane (bending) loadings should be calculated using Section 5.5.10 of CSA O86-09.

In cases where the P-Δ effects need to be accounted for, then the CSA O86-09 procedure should include the factored moment that accounts for the P-Δ effects, and in such case equation [55] for beam-column capacity should be satisfied:

$$\left(\frac{P_f}{P_r}\right)^2 + \frac{M_{f,P-\Delta}}{M_r} \leq 1 \qquad [55]$$

where P_f is the factored compressive axial load, M_r is the factored bending moment resistance and $M_{f,P\text{-}\Delta}$ is the factored bending moment that includes P-Δ effects calculated as:

$$M_{f,P-\Delta} = M_f + \frac{P_f \cdot (\Delta_f + e_0 + \Delta_0)}{1 - \frac{P_f}{P_E}} \qquad [56]$$

where:

e_0 = panel deflection due to axial load eccentricity. Eccentricity should be taken as d/6, where d is the panel thickness;
Δ_0 = initial wall imperfections in the mid of the panel usually taken as H/500, where H is the wall height;
Δ_f = deflection due to out-of-plane loading (bending);
P_E = Euler buckling load in the plane of the bending moment using I_{eff} and E_{05} of boards parallel to the axial load. K_e is the effective length factor and L is the wall height. The Euler buckling load, P_E, is given as:

$$P_E = \frac{\pi^2 E_{05} I_{eff}}{(K_e \cdot L)^2} \qquad [56a]$$

Since shear deformations play a significant role in determining the properties of CLT panels, it is important to include them in the calculation of the axial load capacity of the walls. Using the basic buckling formula that accounts for shear deformations (Bazant and Cedolin, 2003, page 32) and substituting $(GA)_{eff}$ for GA, the axial load capacity is given as:

$$P_{E,v} = \frac{P_E}{1 + \frac{\kappa \cdot P_E}{(GA)_{eff}}} \qquad [57]$$

where k (kappa) is the shear coefficient form factor equal to 1.2 (see Timoshenko).

2.5 Analytical Design Procedures for CLT Elements used as Beams and Lintels

CLT elements under axial in-plane loads acting as deep beams or lintels can be designed using different approaches. Details of these approaches are described in this section. Figure 12 illustrates two 5-layer CLT systems under in-plane bending loads. The same configurations are possible for 3- and 7-layer CLT panels.

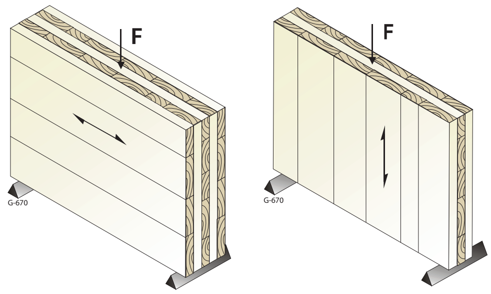


Figure 12
CLT panels (beams or lintels) under axial in-plane loads

2.5.1 Simplified Design Methods for Calculating Bending Strength (In-Plane)

The bending stress may be expressed as:

$$\sigma = M \cdot y \cdot \frac{(E_{mean})}{(EI)_{eff}} \quad [58]$$

The maximum stress will occur for $y = \frac{H}{2}$ where H is the beam depth; therefore equation [58] can be expressed as:

$$\sigma_{\max} = M \cdot 0.5H \cdot \frac{(E_{mean})}{(EI)_{eff}} \quad [59]$$

If we use the CSA O86 design analogy, we can let:

$$\sigma_{\max} \leq \phi \cdot F_b \quad [60]$$

and determine the factored moment bending resistance, M_r, in terms of the specified bending strength, F_b, as:

$$M_r = \phi \cdot F_b \cdot \frac{(EI)_{eff}}{E_{mean}} \cdot \frac{1}{0.5H} \quad [61]$$

where E_{mean} is the mean modulus of elasticity of the longitudinal layer in tension and $(EI)_{eff}$ is determined using the net cross-section.

When the moduli of elasticity of all longitudinal layers are equal, then equation [61] can be expressed as:

$$M_r = \phi \cdot F_b \cdot \frac{I_{eff}}{0.5H} \quad [62]$$

and I_{eff} can be calculated as:

$$I_{eff} = \frac{h_{eff} \cdot H^3}{12} = \frac{H^3}{12} \cdot \sum_i h_i \quad [63]$$

where H is the beam depth and h_i is the thickness of boards perpendicular to the axial load (effective boards).

It should be noted that this method assumes a composite action between effective longitudinal boards. A (much) more conservative way to evaluate the I_{eff} would be to sum the individual moments of inertia of all effective boards.

2.5.2 Composite Theory – k Method

The maximum bending stress may be expressed as:

$$\sigma_{max} = \frac{M}{S} \qquad [64]$$

If we use CSA O86 design analogy, we can let:

$$\sigma_{max} \leq \phi \cdot F_{b,eff} \qquad [65]$$

where $F_{b,eff}$ is one of the effective bending strength values $f_{m,0,eff}$ or $f_{m,90,eff}$ obtained from Tables 2 and 3.

Then, the factored moment bending resistance M_r in terms of the specified bending strength F_b can be expressed as:

$$M_r = \phi \cdot F_{b,eff} \cdot S_{gross} \qquad [66]$$

where S_{gross} can be calculated as:

$$S_{gross} = \frac{h_{tot} \cdot H^2}{6} \qquad [67]$$

and where h_{tot} is the total thickness of the CLT panel and H is the beam depth. It should be noted that this method also assumes a composite action between effective longitudinal boards. Two examples are given in Section 3 for the calculation of the effective bending strength of CLT panels under axial in-plane loads using the two previous design methods.

2.6 Modification Factors (K-factors)

As stipulated in the CSA Standard O86, the Canadian Standard for Engineering Design in Wood, the specified strengths and capacities of structural wood components shall be multiplied by the appropriate modification factors. Since CLT products are relatively new in Canada and are not yet included in CSA O86-09, some assumptions must be made by the designers for the use of these factors. Some recommendations are given in this section.

2.6.1 Load Duration Factor K_D

Special attention should be paid to the Load Duration Factor, K_D. Since a CLT floor/roof system may be heavier than a lightweight joist floor (i.e. the specified long-term load vs. the specified standard-term load is usually higher in CLT floors), it is recommended to follow design provisions given in Section 4.3.2.3 of CSA O86. See also Chapter 6 on “Duration of Load and Creep Factors for Cross-Laminated Timber Panels”.

2.6.2 Service Condition Factor K_S

For now, it is recommended to use CLT products indoor or in covered outdoor spaces (i.e. dry conditions) until further research in this area is conducted. Thus, it is recommended to use a service condition factor equal to unity ($K_S = 1.0$). For humid service (i.e. protected exterior conditions), please refer to the Chapter 6 on "Duration of Load and Creep Factors for Cross-Laminated Timber Panels".

2.6.3 System Factor K_H

For the moment, it is recommended to use a system factor, K_H, equal to unity ($K_H = 1.0$). Further work is needed to determine if CLT construction can benefit from the use of the system factor.

2.6.4 Treatment Factor K_T

For now, it is recommended to use CLT products indoor (i.e. dry exposure) or in covered outdoor spaces until further research in this area is conducted. Then, no treatment would be required and K_T should be equal to unity. However, if a CLT product is impregnated with any strength-reducing chemicals, it is recommended to test the product as stipulated in CSA O86, Sections 4.3.4.4 and 6.4.4, and to use an appropriate value for the K_T factor that corresponds to the influence of the strength-reducing chemicals.

2.6.5 Lateral Stability Factor K_L for Beams and Lintels

The bending moment capacity of beams and lintels shall take into account the lateral stability of the element by evaluating the factor K_L. Some design provisions in CSA O86 could be used by designers as guidance. In particular, Sections 6.5.6.4 and 8.5.7 of CSA O86 could be helpful.

2.6.6 Size Factor for Bending K_{Zb}

As demonstrated for Glulam beams (Foschi, 1993), the bending resistance of a CLT product may also be controlled by the tensile strength of the end-joints used in the outer tension laminations. Therefore, it is suggested to follow design provisions given in Section 6.5.6.5.1 of CSA O86 for calculating K_{Zbg}.

2.6.7 Curvature Factor K_X and Radial Resistance K_R

This chapter does not cover curved CLT products.

2.7 Creep Behaviour of CLT in Bending

CLT panels are used as load-carrying slab/plate elements in structural systems, and as such, duration of load and creep behaviour are critical characteristics that should be taken into account in design. FPInnovations proposed two different options to take care of creep and duration of load for cross-laminated timber panels. Those provisions are provided in Chapter 6 of this Handbook.

2.8 Vibration of CLT Floors

Laboratory tests performed by FPInnovations on CLT floors (Gagnon and Hu, 2007) showed that the vibration behaviour of CLT floors is different from lightweight wood joisted floors and heavy concrete slab floors. CLT floors are heavier than conventional wood joisted floors and lighter than concrete slab floors. FPInnovations proposed a design method for controlling vibrations in CLT floors. Additional design provisions are provided in Chapter 7 of this Handbook.

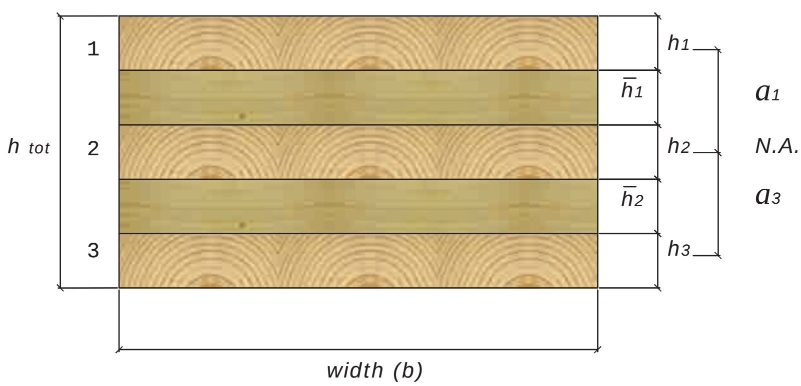
3

DESIGN EXAMPLES

The main purpose of the following examples is to illustrate the proposed design methods for calculating the basic design properties of cross-laminated timber panels used in North American buildings. Engineers should be aware that not all the necessary checks are included in each example.

3.1 Calculation of Effective Bending Stiffness (EI_{eff}) and Bending Strength using the Mechanically Jointed Beams Theory (Gamma Method)

3.1.1 Five-Layer CLT Panel



Cross-section of a 5-layer CLT panel

Where:

h_i = Thickness of board layers in direction of actions
$\bar{h}_i$ = Thickness of board layers in direction perpendicular to actions

$$EI_{eff} = \sum_{i=1}^{3}(E_i I_i + \gamma_i E_i A_i a_i^2)$$

$E_1 = E_2 = E_3$ (Note: could be different)

$$A_i = b_i \cdot h_i \quad \text{and} \quad I_i = \frac{b_i h_i^3}{12}$$

$$\gamma_2 = 1$$

$$\gamma_1 = \frac{1}{1+\left(\pi^2 \cdot \frac{E_1 \cdot A_1}{l^2} \cdot \frac{\overline{h_1}}{G_R \cdot b}\right)} \quad \text{and} \quad \gamma_3 = \frac{1}{1+\left(\pi^2 \cdot \frac{E_3 \cdot A_3}{l^2} \cdot \frac{\overline{h_2}}{G_R \cdot b}\right)}$$

where $\frac{\overline{h_i}}{G_R \cdot b}$ = slip modulus due to shear deformation between layers and G_R = shear modulus perpendicular to the grain or rolling shear modulus.

$$a_1 = \frac{h_1}{2} + \overline{h_1} + \frac{h_2}{2} - a_2 \quad \text{and} \quad a_3 = \frac{h_2}{2} + \overline{h_2} + \frac{h_3}{2} + a_2$$

$$a_2 = \frac{\gamma_1 \cdot E_1 \cdot A_1 \cdot \left(\frac{h_1}{2} + \overline{h_1} + \frac{h_2}{2}\right) - \gamma_3 \cdot E_3 \cdot A_3 \cdot \left(\frac{h_2}{2} + \overline{h_2} + \frac{h_3}{2}\right)}{\sum_{i=1}^{3} \left(\gamma_i \cdot E_i \cdot A_i\right)}$$

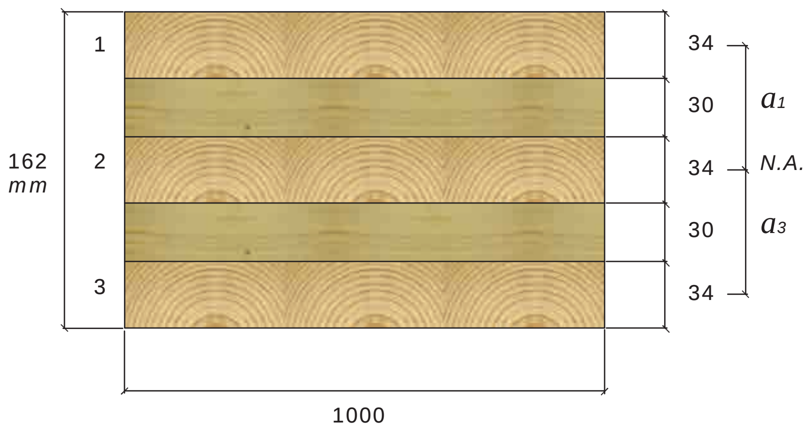
In the case where:

$$E_1 = E_3$$
$$A_1 = A_3$$
$$\frac{h_1}{2} + \overline{h_1} + \frac{h_2}{2} = \frac{h_2}{2} + \overline{h_2} + \frac{h_3}{2}$$

then: $a_2 = 0$

Panel properties for this example:

$$b = 1000\ mm$$
$$h_1 = h_2 = h_3 = 34\,mm \qquad \overline{h_1} = \overline{h_2} = 30\,mm \qquad h_{tot} = 162\ mm$$
$$l = 6000\,mm \quad (simple\ \ span;\ in\ direction\ of\ \ action\ //)$$
$$E = 12\,000\,MPa \qquad fb = 30\,MPa$$
$$G_R = 50\,MPa$$
$$a_2 = 0$$



1) Calculation of Effective Bending Stiffness using the Mechanically Jointed Beams Theory (Gamma Method)

$$EI_{eff\,//} = \sum_{i=1}^{3} (E_i I_i + \gamma_i E_i A_i a_i^{\,2})$$

$$EI_{eff\,//} = (E_1 I_1 + \gamma_1 E_1 A_1 a_1^{\,2}) + (E_2 I_2) + (E_3 I_3 + \gamma_3 E_3 A_3 a_3^{\,2})$$

For longitudinal application (i.e. l = 6000 mm)

$$A_1 = A_3$$
$$E_1 = E_2 = E_3$$
$$I_1 = I_2 = I_3 = \frac{b_1 h_1^{\,3}}{12}$$
$$a_2 = 0$$
$$a_1 = a_3 = \frac{h_1}{2} + \overline{h_1} + \frac{h_2}{2} = \frac{h_2}{2} + \overline{h_2} + \frac{h_3}{2}$$

$$\gamma_1 = \gamma_3 \quad and \quad \gamma_2 = 1$$

then,

$$EI_{eff\,//} = E\left[\left(I_1 + \gamma_1 A_1 a_1^{\,2}\right) + I_2 + \left(I_3 + \gamma_3 A_3 a_3^{\,2}\right)\right]$$

$$EI_{eff\,//} = EI\left[\left(1 + \frac{\gamma A a^2}{I}\right) + (1) + \left(1 + \frac{\gamma A a^2}{I}\right)\right]$$

$$EI_{eff\,//} = EI\left[3 + \frac{2 \cdot \gamma A a^2}{I}\right]$$

where:

$$\gamma_1 = \gamma_3 = \frac{1}{1 + \frac{\pi^2 EA}{l^2} \cdot \frac{\bar{h}}{G_R \cdot b}}$$

$$\gamma_1 = \gamma_3 = \frac{1}{1 + \frac{\pi^2 \cdot 12\,000 \cdot (1000 \cdot 34)}{6000^2} \cdot \frac{30}{50 \cdot 1000}}$$

$$\gamma_1 = \gamma_3 = 0.9371$$

$$A = b \cdot h = 1000 \cdot 34 = 34\,000\ mm^2$$

$$a = \frac{h_1}{2} + \overline{h_1} + \frac{h_2}{2} = \frac{h_2}{2} + \overline{h_2} + \frac{h_3}{2} = \frac{34}{2} + 30 + \frac{34}{2} = 64\,mm$$

$$I = \frac{bh^3}{12} = \frac{1000 \cdot 34^3}{12} = 3.275 \times 10^6\,mm^4$$

we find:

$$EI_{eff\,//} = EI\left[3 + \frac{2 \cdot \gamma A a^2}{I}\right]$$

$$EI_{eff\,//} = 12\,000 \cdot \frac{1000 \cdot 34^3}{12}\left[3 + \frac{2 \cdot 0.9371 \cdot (1000 \cdot 34) \cdot 64^2}{\frac{1000 \cdot 34^3}{12}}\right]$$

$$EI_{eff\,//} = 3250 \times 10^9\,N \cdot mm^2$$

$$I_{eff\,//} = \frac{3250 \times 10^9}{12\,000}$$

$$I_{eff\,//} = 270.84 \times 10^6\,mm^4$$

2) Calculation of Bending Strength using the Mechanically Jointed Beams Theory (Gamma Method)

$$M_r = \phi \cdot F_b \cdot \frac{I_{eff}}{(\gamma_1 a_1 + 0.5 h_1)} \qquad \textit{(E1=E2=E3)}$$

In that case,

$$\phi = 0.9 \quad and \quad Fb = 30\ MPa \quad (if\ K_D = K_S = K_T = K_H = 1)$$
$$\gamma_1 = 0.9371$$
$$a_1 = 64\ mm$$
$$h_1 = 34\ mm$$

then,

$$M_r = \phi \cdot F_b \cdot \frac{I_{eff}}{(\gamma_1 a_1 + 0.5 h_1)} = 0.9 \cdot 30 \cdot \frac{270.84 \times 10^6}{(0.9371 \cdot 64 + 0.5 \cdot 34)} \times 10^{-6}$$

$$M_r = 95.0\, kN - m$$

3) Calculation of Bending Strength using the Simplified Method

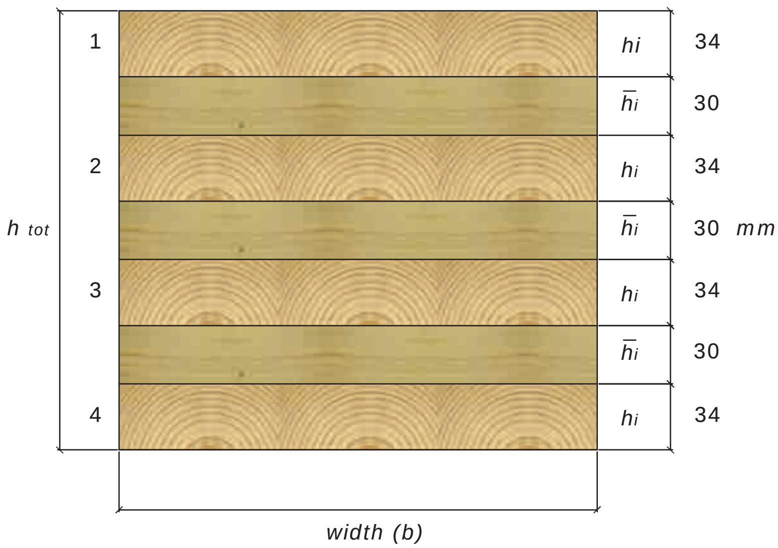
$$M_r = \phi \cdot F_b \cdot \frac{I_{eff}}{0.5 h_{tot}}$$

$$M_r = 0.9 \cdot 30 \cdot \frac{270.84x10^6}{0.5 \cdot 162} x10^{-6}$$

$$M_r = 90.3\, kN - m$$

3.1.2 Seven-Layer CLT Panel

1) Calculation of Effective Bending Stiffness using the Mechanically Jointed Beams Theory (Gamma Method)



Cross-section of a 7-layer CLT panel

Panel properties for this example:

$$b = 1000\ mm$$

$$h_i = 34\,mm \quad \overline{h_i} = 30\,mm \quad h_{tot} = 226\ mm$$

$$l = 6000\,mm \quad (simple\ \ span;\ in\ direction\ of\ \ action\ //)$$

$$E = 12\,000\ MPa \quad fb = 30\ MPa$$

$$G_R = 50\ MPa$$

$$h_i = Thickness\ of\ \ board\ \ layers\ \ in\ direction\ of\ \ actions$$

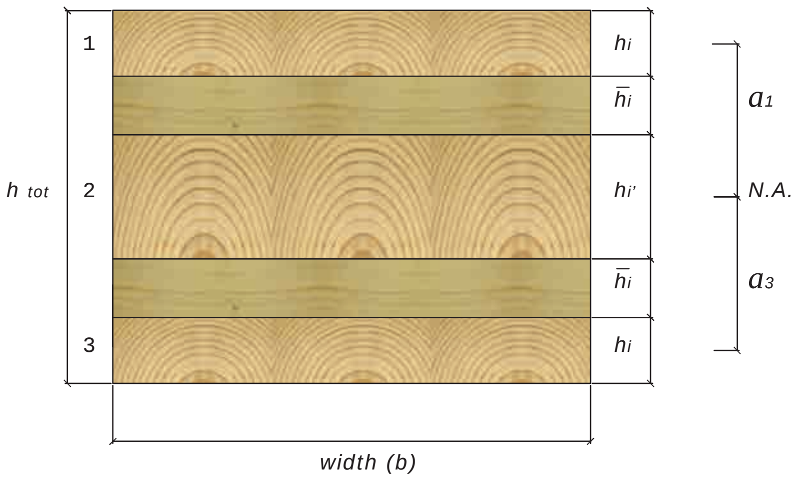
$$\overline{h}_i = Thickness\ of\ \ board\ \ layers\ \ in\ direction\ \ perpendicular\ to\ actions$$

$$EI_{eff}\ final = EI(1) - EI(2) + EI(3)$$

Where:

$EI(1)$ is the effective bending stiffness of a 5-layer cross-section (crosswise)
$EI(2)$ is the bending stiffness of the 3 middle layers (with the 3 layers acting longitudinally)
$EI(3)$ is the effective bending stiffness of the 3 middle layers (crosswise)

a) Calculation of $EI(1)$ using a 5-layer cross-section



$$EI(1) = \sum_{i=1}^{3} (E_i I_i + \gamma_i E_i A_i a_i^{\,2})$$

$$EI(1) = (E_1 I_1 + \gamma_1 E_1 A_1 a_1^{\,2}) + (E_2 I_2 + \gamma_2 E_2 A_2 a_2^{\,2}) + (E_3 I_3 + \gamma_3 E_3 A_3 a_3^{\,2})$$

where:

$$a_1 = a_3 = \frac{h_i}{2} + \overline{h_i} + \frac{h_i'}{2} = \frac{34}{2} + 30 + \frac{98}{2} = 96\ mm$$

$$a_2 = \frac{\gamma_1 E_1 A_1 (h_1 + h_2) - \gamma_3 E_3 A_3 (h_2 + h_3)}{2 \sum_{i=1}^{3} \gamma_i E_i A_i}$$

where:

$$E_1 = E_3$$
$$A_1 = A_3$$
$$h_1 + h_2 = h_2 + h_3$$
$$\gamma_1 = \gamma_3$$

then: $a_2 = 0$

$$\gamma_i = \frac{1}{1 + \pi^2 \dfrac{E_i A_i}{l^2} \cdot \dfrac{\overline{h_i}}{G_R \cdot b}} \quad \text{for} \quad i = 1\ and\ i = 3$$

$$E_1 = E_2 = E_3$$

$$I_1 = I_3 = \frac{b_1 h_1^{\ 3}}{12}$$

$$I_2 = \frac{b_2 h_2^{\ 3}}{12}$$

$$\gamma_1 = \gamma_3 \quad and \quad \gamma_2 = 1$$

$$\gamma_1 = \gamma_3 = \frac{1}{1 + \frac{\pi^2 EA}{l^2} \cdot \frac{\overline{h}}{G_R \cdot b}}$$

$$\gamma_1 = \gamma_3 = \frac{1}{1 + \frac{\pi^2 \cdot 12000 \cdot (1000 \cdot 34)}{6000^2} \cdot \frac{30}{50 \cdot 1000}} = 0.9371$$

then,

$$EI(1) = (E_1 I_1 + \gamma_1 E_1 A_1 a_1^{\ 2}) + (E_2 I_2) + (E_3 I_3 + \gamma_3 E_3 A_3 a_3^{\ 2})$$

$$EI(1) = E\left[\left(I_1 + \gamma_1 A_1 a_1^{\ 2}\right) + I_2 + \left(I_3 + \gamma_3 A_3 a_3^{\ 2}\right)\right]$$

where:

$$A_1 = A_3 = b \cdot h = 1000 \cdot 34 = 34\,000\ mm^2$$

$$a_1 = a_3 = 96\ mm$$

$$\gamma_1 = \gamma_3$$

$$I_1 = I_3 = \frac{bh^3}{12} = \frac{1000 \cdot 34^3}{12} = 3.275 \times 10^6\ mm^4$$

$$I_2 = \frac{b_2 h_2^{\ 3}}{12} = \frac{1000 \cdot 98^3}{12} = 78.433 \times 10^6\ mm^4$$

we find:

$$EI(1) = E\left[\left(I_1 + \gamma_1 A_1 a_1^{\,2}\right) + I_2 + \left(I_3 + \gamma_3 A_3 a_3^{\,2}\right)\right]$$

$$EI(1) = E\left[2\left(I_1 + \gamma_1 A_1 a_1^{\,2}\right) + I_2\right]$$

$$EI(1) = 12000\left[2\left(\frac{1000 \cdot 34^3}{12} + 0.9371 \cdot 1000 \cdot 34 \cdot 96^2\right) + \frac{1000 \cdot 98^3}{12}\right]$$

$$EI(1) = 8067.03 \times 10^9\, N \cdot mm^2$$

$$I(1) = \frac{8067.03 \times 10^9}{12\,000}$$

$$I(1) = 672.25 \times 10^6\, mm^4$$

b) Calculation of *EI(2)*

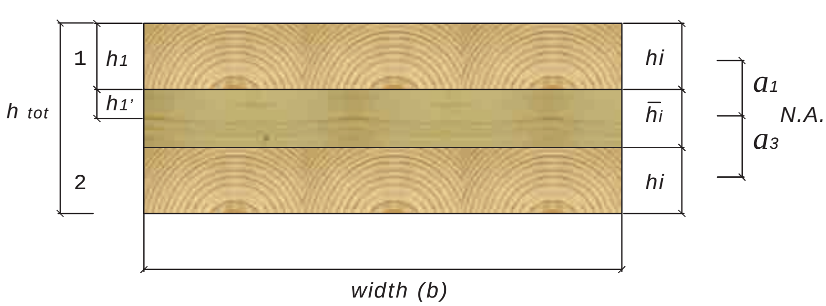
$$EI(2) = E \cdot I_2 = E \cdot \frac{b_2 \cdot h_2^{\,3}}{12} = 12000 \cdot \frac{1000 \cdot 98^3}{12}$$

$$EI(2) = 941.19 \times 10^9\, N \cdot mm^2$$

$$I(2) = \frac{941.19 \times 10^9}{12\,000}$$

$$I(2) = 78.433 \times 10^6\, mm^4$$

c) Calculation of *EI(3)* using a 3-layer cross-section



$$EI(3)=\sum_{i=1}^{2}(E_iI_i+\gamma_iE_iA_ia_i^{\,2})$$

$$EI(3)=(E_1I_1+\gamma_1E_1A_1a_1^{\,2})+(E_2I_2+\gamma_2E_2A_2a_2^{\,2})$$

where:

$$A_1=A_2=b\cdot h_i=1000\cdot 34=34\,000\ mm^2$$

$$a_1=a_2=\frac{h_i}{2}+\frac{\overline{h_i}}{2}=\frac{34}{2}+\frac{30}{2}=32\,mm$$

$$I_1=I_2=\frac{bh_i^{\,3}}{12}=\frac{1000\cdot 34^3}{12}=3.275\times 10^6\,mm^4$$

then:

$$EI(3)=2E(I+\gamma Aa^2)$$

where:

$$\gamma_i=\frac{1}{1+\pi^2\,\frac{E_iA_i}{l^2}\cdot\frac{h_1'}{G_R\cdot b}}\qquad \text{for } i=1$$

$$\gamma_1=\frac{1}{1+\pi^2\,\frac{12000\cdot 1000\cdot 34}{6000^2}\cdot\frac{30/2}{50\cdot 1000}}=0.9675$$

we find:

$$EI(3)=2\cdot 12000\cdot(\frac{1000\cdot 34^3}{12}+0.9675\cdot 1000\cdot 34\cdot 32^2)$$

$$EI(3)=887.04\times 10^9\ N\cdot mm^2$$

$$I(3)=\frac{887.04\times 10^9}{12000}$$

$$I(3)=73.919\times 10^6\,mm^4$$

Finally,

$$EI_{eff}\,final = EI(1) - EI(2) + EI(3)$$

$$EI_{eff}\,final = 8067.03\times10^{9} - 941.19\times10^{9} + 887.04\times10^{9}$$

$$EI_{eff}\,final = 8013\times10^{9}\,N\cdot mm^{2}$$

$$I_{eff}\,final = \frac{8013\times10^{9}}{12000}$$

$$I_{eff}\,final = 667.75\times10^{6}\,mm^{4}$$

2) Calculation of Bending Strength using the Mechanically Jointed Beams Theory (Gamma Method)

$$M_r = \phi\cdot F_b\cdot\frac{I_{eff}}{(\gamma_1 a_1 + 0.5h_1)} \qquad (E1=E2=E3)$$

In that case,

$$\phi = 0.9 \quad and \quad Fb = 30\ MPa \quad (if\ K_D = K_S = K_T = K_H = 1)$$

$$\gamma_1 = 0.9371 \quad (from\ the\ 5-layer\ section)$$
$$a_1 = 96\,mm$$
$$h_1 = 34\,mm$$

then,

$$M_r = \phi\cdot F_b\cdot\frac{I_{eff}}{(\gamma_1 a_1 + 0.5h_1)} = 0.9\cdot30\cdot\frac{667.75\times10^{6}}{(0.9371\cdot96+0.5\cdot34)}\times10^{-6}$$

$$M_r = 168.6\,kN-m$$

3) Calculation of Bending Strength using the Simplified Method

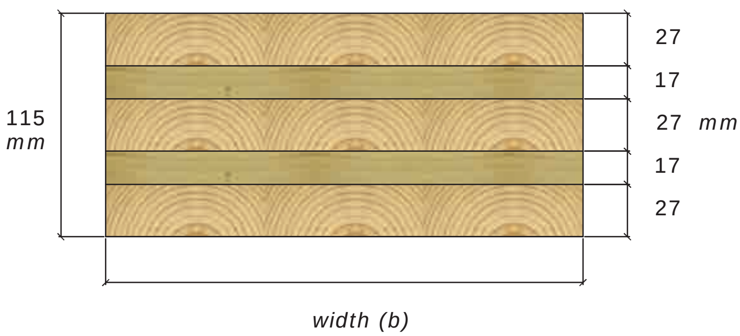
$$M_r = \phi \cdot F_b \cdot \frac{I_{eff}}{0.5 h_{tot}}$$

$$M_r = 0.9 \cdot 30 \cdot \frac{667.75x10^6}{0.5 \cdot 226} x10^{-6}$$

$$M_r = 159.6\, kN - m$$

3.2 Calculation of Effective Bending Stiffness (EI_{eff}) According to Composite Theory (k-Method)

3.2.1 Five-Layer CLT Panel – 115 mm thick



$$E_0 = 10000\, MPa$$
$$E_{90} = \frac{10000}{30} = 333\, MPa$$

From Table 2:

$$m = 5$$
$$a_m = a_5 = 115\, mm$$
$$a_{m-2} = a_3 = 17 + 27 + 17 = 61 mm$$
$$a_{m-4} = a_1 = 27\, mm$$

$$k_1 = 1 - \left[\left(1 - \frac{E_{90}}{E_0} \right) \cdot \left(\frac{a_3^3 - a_1^3}{a_5^3} \right) \right]$$

$$k_1 = 1 - \left[\left(1 - \frac{333}{10000} \right) \cdot \left(\frac{61^3 - 27^3}{115^3} \right) \right] = 1 - [(0.9667) \cdot (0.1363)]$$

$$k_1 = 0.8682$$

From Table 3:

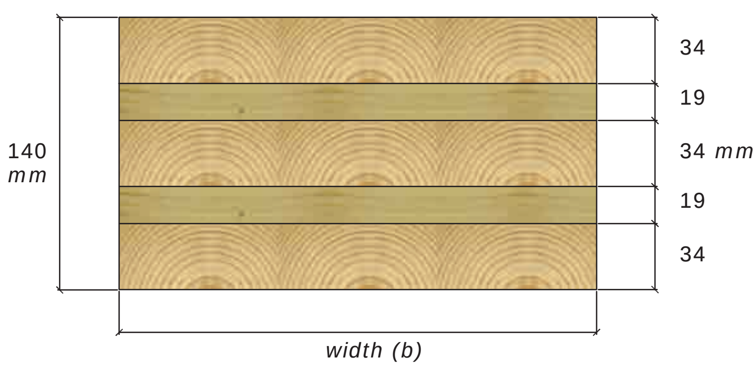
$$E_{b,0,eff} = E_0 \cdot k_1$$

$$E_{b,0,eff} = 12000 \cdot 0.8824 = 10588 \ MPa$$

$$for \quad b = 1000\, mm$$

$$EI_{eff} = 10588 \cdot \frac{1000 \cdot 140^3}{12} = 2421 x 10^9 \ N \cdot mm^2$$

3.2.2 Five-Layer CLT Panel – 140 mm thick



$$E_0 = 12000 \ MPa$$

$$E_{90} = \frac{12000}{30} = 400 \ MPa$$

From Table 2:

$$a_m = a_5 = 140\, mm$$

$$a_{m-2} = a_3 = 19 + 34 + 19 = 72\, mm$$

$$a_{m-4} = a_1 = 34 mm$$

$$k_1 = 1 - \left[\left(1 - \frac{E_{90}}{E_0}\right) \cdot \left(\frac{a_3^3 - a_1^3}{a_5^3}\right)\right]$$

$$k_1 = 1 - \left[\left(1 - \frac{400}{12000}\right) \cdot \left(\frac{72^3 - 34^3}{140^3}\right)\right] = 1 - [(0.9667) \cdot (0.1217)]$$

$$k_1 = 0.8824$$

From Table 3:

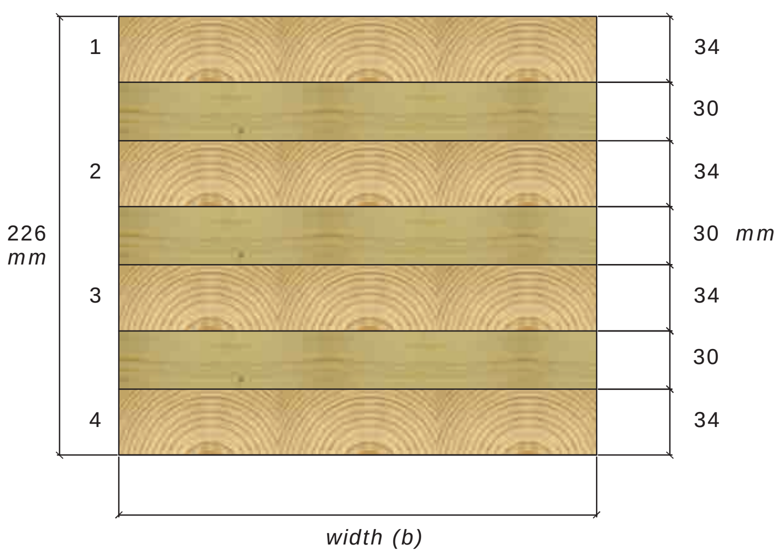
$$E_{b,0,eff} = E_0 \cdot k_1$$

$$E_{b,0,eff} = 12000 \cdot 0.8824 = 10588 \ MPa$$

$$for \quad b = 1000\, mm$$

$$EI_{eff} = 10588 \cdot \frac{1000 \cdot 140^3}{12} = 2421 x 10^9 \ N \cdot mm^2$$

3.2.3 Seven-Layer CLT Panel – 226 mm thick



$$E_0 = 12000 \ MPa$$

$$E_{90} = \frac{12000}{30} = 400 \ MPa$$

From Table 2:

$$a_m = a_7 = 226mm$$
$$a_{m-2} = a_5 = 30+34+30+34+30 = 158\,mm$$
$$a_{m-4} = a_3 = 34+30+34 = 98mm$$
$$a_{m-6} = a_1 = 30mm$$
$$k_1 = 1-\left[\left(1-\frac{E_{90}}{E_0}\right)\cdot\left(\frac{a_5^3-a_3^3+a_1^3}{a_7^3}\right)\right]$$
$$k_1 = 1-\left[\left(1-\frac{400}{12000}\right)\cdot\left(\frac{158^3-98^3+30^3}{226^3}\right)\right] = 1-[(0.9667)\cdot(0.2625)]$$

$$k_1 = 0.7462$$

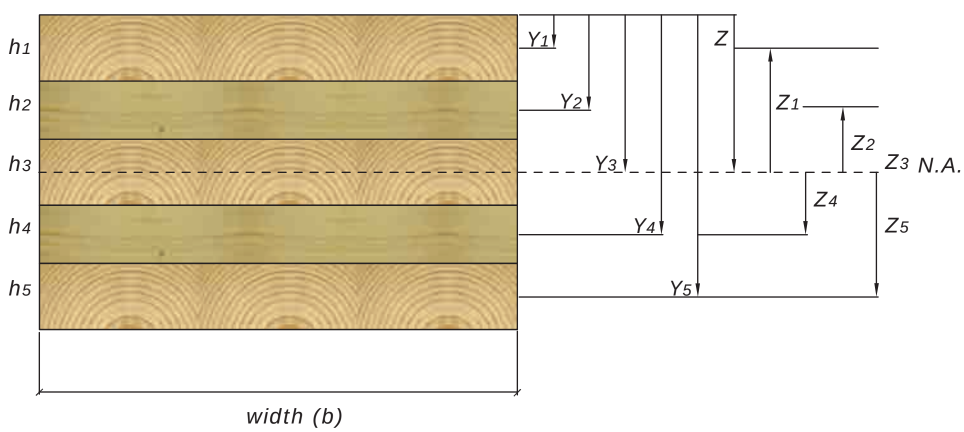
From Table 3:

$$E_{b,0,eff} = E_0 \cdot k_1$$
$$E_{b,0,eff} = 12000 \cdot 0.7462 = 8954\ MPa$$

$$for \quad b = 1000\,mm$$
$$EI_{eff} = 8954 \cdot \frac{1000 \cdot 226^3}{12} = 8614\,x10^9\,N \cdot mm^2$$

3.3 Calculation of True Effective Bending Stiffness (EI_{eff}) and Effective Shear Stiffness (GA_{eff}) According to the Shear Analogy Method (Kreuzinger)

3.3.1 True Bending Stiffness (EI_{eff}) of a Five-Layer CLT Panel – 140 mm thick



Where:

$h_1 = 32$ mm	$E_0 = 11000$ MPa	$E_{90} = 370$ MPa ($\approx 11000/30$)
$h_2 = 21$ mm	$E_0 = 7000$ MPa	$E_{90} = 230$ MPa ($\approx 7000/30$)
$h_3 = 34$ mm	$E_0 = 7000$ MPa	$E_{90} = 230$ MPa ($\approx 7000/30$)
$h_4 = 21$ mm	$E_0 = 7000$ MPa	$E_{90} = 230$ MPa ($\approx 7000/30$)
$h_5 = 32$ mm	$E_0 = 11000$ MPa	$E_{90} = 370$ MPa ($\approx 11000/30$)

h total = $h_1 + h_2 + h_3 + h_4 + h_5 = 140$ mm and b = 1000 mm

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

1) Determine location of the Neutral Axis Z

$$Z = \frac{\sum_{i=1}^{n}(E_i A_i) \cdot Y_i}{\sum_{i=1}^{n}(E_i A_i)}$$ (Note that for symmetric panels and same E, $Z = h_{total}/2$)

$$E_1 A_1 = E_1 \cdot b \cdot h_1 = 11000 \cdot 1000 \cdot 32 = 3.520x10^8\, N$$
$$E_2 A_2 = E_2 \cdot b \cdot h_2 = 230 \cdot 1000 \cdot 21 = 4.830x10^6\, N$$
$$E_3 A_3 = E_3 \cdot b \cdot h_3 = 7000 \cdot 1000 \cdot 34 = 2.380x10^8\, N$$
$$E_4 A_4 = E_4 \cdot b \cdot h_4 = 230 \cdot 1000 \cdot 21 = 4.830x10^6\, N$$
$$E_5 A_5 = E_5 \cdot b \cdot h_5 = 11000 \cdot 1000 \cdot 32 = 3.520x10^8\, N$$

$$\sum_{i=1}^{5}(E_i A_i) = 3.520x10^8 + 4.830x10^6 + 2.380x10^8 + 4.830x10^6 + 3.520x10^8$$

$$\sum_{i=1}^{5}(E_i A_i) = 9.517x10^8\, N$$

$$Y_1 = \frac{h_1}{2} = \frac{32}{2} = 16mm$$

$$Y_2 = h_1 + \frac{h_2}{2} = 32 + \frac{21}{2} = 42.5mm$$

$$Y_3 = h_1 + h_2 + \frac{h_3}{2} = 32 + 21 + \frac{34}{2} = 70mm$$

$$Y_4 = h_1 + h_2 + h_3 + \frac{h_4}{2} = 32 + 21 + 34 + \frac{21}{2} = 97.5mm$$

$$Y_5 = h_1 + h_2 + h_3 + h_4 + \frac{h_5}{2} = 32 + 21 + 34 + 21 + \frac{32}{2} = 124mm$$

$$E_1 A_1 y_1 = 3.520x10^8 \cdot 16 = 5.632x10^9\, N \cdot mm$$
$$E_2 A_2 y_2 = 4.830x10^6 \cdot 42.5 = 2.053x10^8\, N \cdot mm$$
$$E_3 A_3 y_3 = 2.380x10^8 \cdot 70 = 1.666x10^{10}\, N \cdot mm$$
$$E_4 A_4 y_4 = 4.830x10^6 \cdot 97.5 = 4.709x10^8\, N \cdot mm$$
$$E_5 A_5 y_5 = 3.520x10^8 \cdot 124 = 4.365x10^{10}\, N \cdot mm$$

$$\sum_{i=1}^{5} (E_i A_i) \cdot Y_i = 5.632x10^9 + 2.053x10^8 + 1.666x10^{10} + 4.709x10^8 + 4.365x10^{10}$$

$$\sum_{i=1}^{5} (E_i A_i) \cdot Y_i = 6.662x10^{10}\, N \cdot mm$$

Then:

$$Z = \frac{\sum_{i=1}^{5} (E_i A_i) \cdot Y_i}{\sum_{i=1}^{5} (E_i A_i)} = \frac{6.662x10^{10}}{9.517x10^8} = 70mm$$

and,

$$Z_1 = Z - \frac{h_1}{2} = 70 - \frac{32}{2} = 54mm$$

$$Z_2 = Z - h_1 - \frac{h_2}{2} = 70 - 32 - \frac{21}{2} = 27.5mm$$

$$Z_3 = Z - h_1 - h_2 - \frac{h_3}{2} = 70 - 32 - 21 - \frac{34}{2} = 0mm$$

$$Z_4 = -Z_2 = -27.5mm$$

$$Z_5 = -Z_1 = -54mm$$

2) Calculation of $B_A = \sum_{i=1}^{n} E_i \cdot I_i = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12}$

$$E_1 I_{a,1} = E_1 \frac{b \cdot h_1^{\ 3}}{12} = 11000 \cdot \frac{1000 \cdot 32^3}{12} = 3.004x10^{10}\, N \cdot mm^2$$

$$E_2 I_{a,2} = E_2 \frac{b \cdot h_2^{\ 3}}{12} = 230 \cdot \frac{1000 \cdot 21^3}{12} = 1.775x10^{8}\, N \cdot mm^2$$

$$E_3 I_{a,3} = E_3 \frac{b \cdot h_3^{\ 3}}{12} = 7000 \cdot \frac{1000 \cdot 34^3}{12} = 2.293x10^{10}\, N \cdot mm^2$$

$$E_4 I_{a,4} = E_4 \frac{b \cdot h_4^{\ 3}}{12} = 230 \cdot \frac{1000 \cdot 21^3}{12} = 1.775x10^{8}\, N \cdot mm^2$$

$$E_5 I_{a,5} = E_5 \frac{b \cdot h_5^{\ 3}}{12} = 11000 \cdot \frac{1000 \cdot 32^3}{12} = 3.004x10^{10}\, N \cdot mm^2$$

Then,

$$B_A = 3.004x10^{10} + 1.775x10^{8} + 2.293x10^{10} + 1.775x10^{8} + 3.004x10^{10}$$

$$B_A = 8.337x10^{10}\, N \cdot mm^2$$

3) Calculation of $B_B = \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$

$$E_1 I_{b,1} = E_1 A_1 \cdot Z_1^{\,2} = 11000 \cdot 1000 \cdot 32 \cdot 54^2 = 1.026x10^{12}\,mm^4$$

$$E_2 I_{b,2} = E_2 A_2 \cdot Z_2^{\,2} = 230 \cdot 1000 \cdot 21 \cdot 27.5^2 = 3.653x10^{9}\,mm^4$$

$$E_3 I_{b,3} = E_3 A_3 \cdot Z_3^{\,2} = 7000 \cdot 1000 \cdot 34 \cdot 0 = 0mm^4$$

$$E_4 I_{b,4} = E_4 A_4 \cdot Z_4^{\,2} = 230 \cdot 1000 \cdot 21 \cdot (-27.5^2) = 3.653x10^{9}\,mm^4$$

$$E_5 I_{b,5} = E_5 A_5 \cdot Z_5^{\,2} = 11000 \cdot 1000 \cdot 32 \cdot (-54^2) = 1.026x10^{12}\,mm^4$$

Then,

$$B_B = 1.026x10^{12} + 3.653x10^{9} + 0 + 3.653x10^{9} + 1.026x10^{12}$$

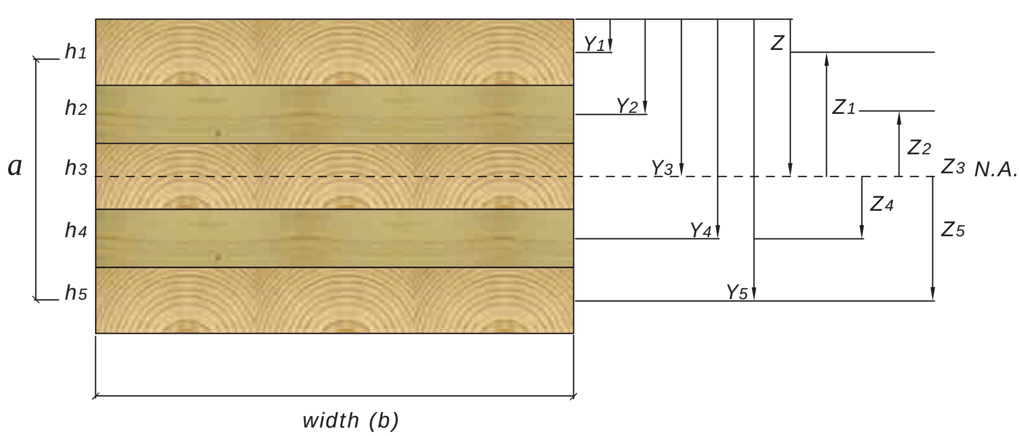
$$B_B = 2.059x10^{12}\,N \cdot mm^2$$

Finally:

$$(EI)_{eff} = B_A + B_B = 8.337x10^{10} + 2.059x10^{12}$$

$$(EI)_{eff} = 2.143x10^{12}\,N \cdot mm^2$$

3.3.2 Shear Stiffness (GA_{eff}) of a Five-Layer CLT Panel – 140 mm thick



Where:

$h_1 = 32$ mm $\quad G_0 = 690$ MPa $\quad G_{90} = 69$ MPa $(\approx G_0/10)$
$h_2 = 21$ mm $\quad G_0 = 440$ MPa $\quad G_{90} = 44$ MPa $(\approx G_0/10)$
$h_3 = 34$ mm $\quad G_0 = 440$ MPa $\quad G_{90} = 44$ MPa $(\approx G_0/10)$
$h_4 = 21$ mm $\quad G_0 = 440$ MPa $\quad G_{90} = 44$ MPa $(\approx G_0/10)$
$h_5 = 32$ mm $\quad G_0 = 690$ MPa $\quad G_{90} = 69$ MPa $(\approx G_0/10)$

h total = h1+ h2 + h3 + h4 + h5 = 140 mm and b = 1000 mm

$$a = h_{total} - \frac{h_1}{2} - \frac{h_n}{2} \qquad where\ n = 5$$

$$a = 140 - \frac{32}{2} - \frac{32}{2} = 108mm$$

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

$$(GA)_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_1 \cdot b}\right) + \left(\sum_{i=2}^{4} \frac{h_i}{G_i \cdot b_i}\right) + \left(\frac{h_5}{2 \cdot G_5 \cdot b}\right)\right]}$$

$$(GA)_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_1 \cdot b}\right) + \left(\frac{h_2}{G_2 \cdot b} + \frac{h_3}{G_3 \cdot b} + \frac{h_4}{G_4 \cdot b}\right) + \left(\frac{h_5}{2 \cdot G_5 \cdot b}\right)\right]}$$

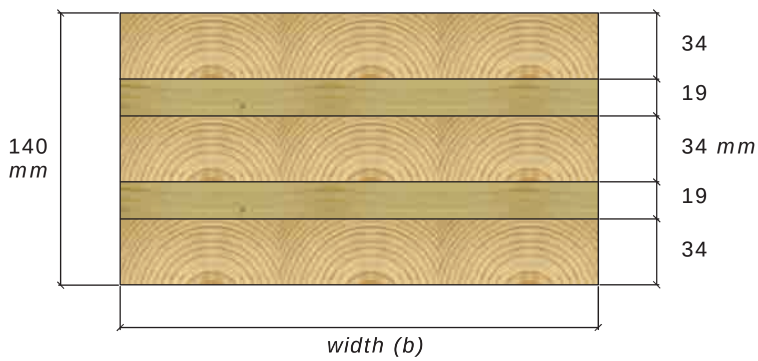
$$(GA)_{eff} = \frac{108^2}{\left[\left(\frac{32}{2 \cdot 690 \cdot 1000}\right) + \left(\frac{21}{44 \cdot 1000} + \frac{34}{440 \cdot 1000} + \frac{21}{44 \cdot 1000}\right) + \left(\frac{32}{2 \cdot 690 \cdot 1000}\right)\right]}$$

Then,

$$(GA)_{eff} = 1.082x10^7\ N$$

3.4 Calculation of Effective Bending Stiffness (EI_{eff}) and Deflection under Live Load Using the Three Proposed Design Methods

3.4.1 Bending Stiffness (EI_{eff}) of a Five-Layer CLT Panel – 140 mm thick



Panel properties and parameters for this example:

$$h_{tot} = 140mm \quad b = 1000\ mm \quad l = 4500\ mm \quad (simple\ \ span;\ in\ direction\ of\ \ action\ //)$$

$$l/h_{tot} = 32$$

$$Uniform\ Live\ Load = 1.9\ kPa \quad \Rightarrow w = 1.9\ kN/m \quad for\ 1\ m\ wide\ strip$$

$$Longitudinal\ layers:$$

$$E_0 = 11000\ MPa$$

$$E_{90} \approx \frac{E_0}{30} = \frac{11000}{30} = 367 \approx 370\ MPa$$

$$G_0 \approx \frac{E_0}{16} = \frac{11000}{16} = 688 \approx 690\ MPa$$

$$G_R \approx \frac{G_0}{10} = \frac{690}{10} = 69\ MPa$$

$$Perpendicular\ layers:$$

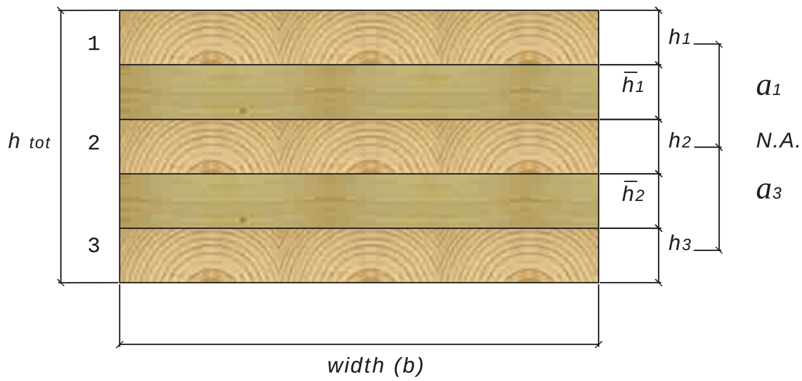
$$E_0 = 9000\ MPa$$

$$E_{90} \approx \frac{E_0}{30} = \frac{9000}{30} = 300\ MPa$$

$$G_0 \approx \frac{E_0}{16} = \frac{9000}{16} = 563 \approx 560\ MPa$$

$$G_R \approx \frac{G_0}{10} = \frac{560}{10} = 56\ MPa$$

1) Mechanically Jointed Beams Theory (Gamma Method)



$$h_1 = h_2 = h_3 = 34\,mm \qquad \overline{h_1} = \overline{h_2} = 19\,mm \qquad h_{tot} = 140\ mm$$

$$E_1 = E_2 = E_3 = 11\,000\,MPa \ \ (only\ longitudinal\ layers\ \ participate)$$

$$G_R = 56\,MPa \ \ (transverse\ layers)$$

$$EI_{eff\,//} = \sum_{i=1}^{3} (E_i I_i + \gamma_i E_i A_i a_i^{\ 2})$$

$$EI_{eff\,//} = (E_1 I_1 + \gamma_1 E_1 A_1 a_1^{\ 2}) + (E_2 I_2) + (E_3 I_3 + \gamma_3 E_3 A_3 a_3^{\ 2})$$

For longitudinal application (i.e. L = 4500 mm)

$$A_1 = A_3$$
$$E_1 = E_2 = E_3$$
$$I_1 = I_2 = I_3 = \frac{b_1 h_1^{\ 3}}{12}$$
$$a_2 = 0$$
$$a_1 = a_3 = \frac{h_1}{2} + \overline{h_1} + \frac{h_2}{2} = \frac{h_2}{2} + \overline{h_2} + \frac{h_3}{2}$$

$$\gamma_1 = \gamma_3 \quad and \quad \gamma_2 = 1$$

then,

$$EI_{eff\,//} = EI\left[3 + \frac{2 \cdot \gamma A a^2}{I}\right]$$

where:

$$\gamma_1 = \gamma_3 = \frac{1}{1 + \frac{\pi^2 EA}{l^2} \cdot \frac{\bar{h}}{G_R \cdot b}} = \frac{1}{1 + \frac{\pi^2 \cdot 11000 \cdot (1000 \cdot 34)}{4500^2} \cdot \frac{19}{56 \cdot 1000}} = 0.9418$$

$$A = b \cdot h = 1000 \cdot 34 = 34\,000\ mm^2$$

$$a_1 = a_3 = \frac{h_1}{2} + \overline{h_1} + \frac{h_2}{2} = \frac{h_2}{2} + \overline{h_2} + \frac{h_3}{2} = \frac{34}{2} + 19 + \frac{34}{2} = 53\,mm$$

$$I = \frac{bh^3}{12} = \frac{1000 \cdot 34^3}{12} = 3.275 \times 10^6\,mm^4$$

we find:

$$EI_{eff\,//} = 11000 \cdot \frac{1000 \cdot 34^3}{12}\left[3 + \frac{2 \cdot 0.9418 \cdot (1000 \cdot 34) \cdot 53^2}{\frac{1000 \cdot 34^3}{12}}\right]$$

$$EI_{eff\,//} = 2087 \times 10^9\,N \cdot mm^2$$

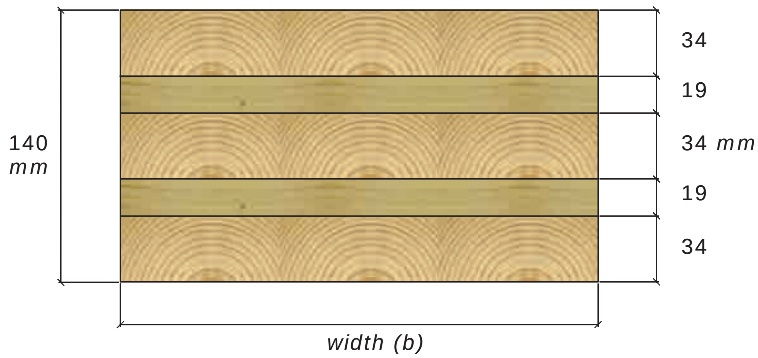
Then, the deflection under uniform Live Load can be calculated as:

$$\Delta_L = \frac{5 \cdot w \cdot L^4}{384 \cdot EI_{eff}}$$

$$where \quad w = 1.9\,kN/m = 1.9\,N/mm \quad and \quad L = 4500\,mm$$

$$\Delta_L = \frac{5 \cdot 1.9 \cdot 4500^4}{384 \cdot 2087 \times 10^9} = 4.9\,mm$$

2) Composite Theory (k-Method)



$$E_0 = 11000\ MPa$$
$$E_{90} = 300\ MPa \quad (from\ \ perpendicular\ layers)$$

From Table 2:

$$a_m = a_5 = 140\,mm$$
$$a_{m-2} = a_3 = 19 + 34 + 19 = 72\,mm$$
$$a_{m-4} = a_1 = 34mm$$

$$k_1 = 1 - \left[\left(1 - \frac{E_{90}}{E_0}\right)\cdot\left(\frac{a_3^3 - a_1^3}{a_5^3}\right)\right] \qquad k_1 = 1 - \left[\left(1 - \frac{300}{11000}\right)\cdot\left(\frac{72^3 - 34^3}{140^3}\right)\right] = 0.8816$$

From Table 3:

$$E_{b,0,eff} = E_0 \cdot k_1 = 11000 \cdot 0.8816 = 9698\,MPa$$

$$for \quad b = 1000\,mm$$

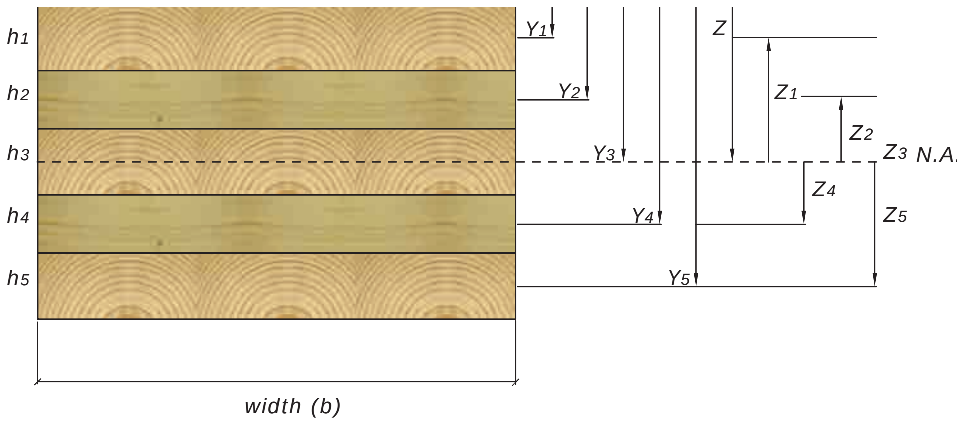
$$EI_{eff} = 9698 \cdot \frac{1000 \cdot 140^3}{12} = 2218x10^9\,N \cdot mm^2$$

Then, the deflection under uniform Live Load can be calculated as:

$$\Delta_L = \frac{5 \cdot w \cdot L^4}{384 \cdot EI_{eff}} \quad where \quad w = 1.9\ kN/m = 1.9\ N/mm \quad and \quad L = 4500\ mm$$

$$\Delta_L = \frac{5 \cdot 1.9 \cdot 4500^4}{384 \cdot 2218x10^9} = 4.6\ mm \quad (correct\ \ for\ L/h_{tot} \geq 30)$$

3) Shear Analogy Method (Kreuzinger)



Where:

h_1 = 34 mm	E_0 = 11000 MPa	E_{90} = 370 MPa (≈ 11000/30)
h_2 = 19 mm	E_0 = 9000 MPa	E_{90} = 300 MPa (≈ 9000/30)
h_3 = 34 mm	E_0 = 11000 MPa	E_{90} = 370 MPa (≈ 11000/30)
h_4 = 19 mm	E_0 = 9000 MPa	E_{90} = 300 MPa (≈ 9000/30)
h_5 = 34mm	E_0 = 11000 MPa	E_{90} = 370 MPa (≈ 11000/30)

h total = $h_1 + h_2 + h_3 + h_4 + h_5$ = 140 mm and b = 1000 mm

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

a) Determine location of the Neutral Axis Z

$$Z = \frac{\sum_{i=1}^{n} (E_i A_i) \cdot Y_i}{\sum_{i=1}^{n} (E_i A_i)}$$

$$E_1 A_1 = E_1 \cdot b \cdot h_1 = 11000 \cdot 1000 \cdot 34 = 3.74x10^8 \, N$$
$$E_2 A_2 = E_2 \cdot b \cdot h_2 = 300 \cdot 1000 \cdot 19 = 5.7x10^6 \, N$$
$$E_3 A_3 = E_3 \cdot b \cdot h_3 = 11000 \cdot 1000 \cdot 34 = 3.74x10^8 \, N$$
$$E_4 A_4 = E_4 \cdot b \cdot h_4 = 300 \cdot 1000 \cdot 19 = 5.7x10^6 \, N$$
$$E_5 A_5 = E_5 \cdot b \cdot h_5 = 11000 \cdot 1000 \cdot 34 = 3.74x10^8 \, N$$

$$\sum_{i=1}^{5} (E_i A_i) = 11.334x10^8 \, N$$

$$Y_1 = \frac{h_1}{2} = \frac{34}{2} = 17mm$$
$$Y_2 = h_1 + \frac{h_2}{2} = 34 + \frac{19}{2} = 43.5mm$$
$$Y_3 = h_1 + h_2 + \frac{h_3}{2} = 34 + 19 + \frac{34}{2} = 70mm$$
$$Y_4 = h_1 + h_2 + h_3 + \frac{h_4}{2} = 34 + 19 + 34 + \frac{19}{2} = 96.5mm$$
$$Y_5 = h_1 + h_2 + h_3 + h_4 + \frac{h_5}{2} = 34 + 19 + 34 + 19 + \frac{34}{2} = 123mm$$

$$E_1 A_1 y_1 = 3.740x10^8 \cdot 17 = 6.358x10^9\, N \cdot mm$$
$$E_2 A_2 y_2 = 5.7x10^6 \cdot 43.5 = 2.480x10^8\, N \cdot mm$$
$$E_3 A_3 y_3 = 3.740x10^8 \cdot 70 = 2.618x10^{10}\, N \cdot mm$$
$$E_4 A_4 y_4 = 5.7x10^6 \cdot 96.5 = 5.501x10^8\, N \cdot mm$$
$$E_5 A_5 y_5 = 3.740x10^8 \cdot 123 = 4.600x10^{10}\, N \cdot mm$$

$$\sum_{i=1}^{5} (E_i A_i) \cdot Y_i = 7.934x10^{10}\, N \cdot mm$$

Then:

$$Z = \frac{\sum_{i=1}^{5} (E_i A_i) \cdot Y_i}{\sum_{i=1}^{5} (E_i A_i)} = \frac{7.934x10^{10}}{11.334x10^8} = 70mm$$

and,

$$Z_1 = Z - \frac{h_1}{2} = 70 - \frac{34}{2} = 53mm$$
$$Z_2 = Z - h_1 - \frac{h_2}{2} = 70 - 34 - \frac{19}{2} = 26.5mm$$
$$Z_3 = Z - h_1 - h_2 - \frac{h_3}{2} = 70 - 34 - 19 - \frac{34}{2} = 0mm$$
$$Z_4 = -Z_2 = -26.5mm$$
$$Z_5 = -Z_1 = -53mm$$

b) Calculation of $B_A = \sum_{i=1}^{n} E_i \cdot I_i = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12}$

$$E_1 I_{a,1} = E_1 \frac{b \cdot h_1^{\ 3}}{12} = 11000 \cdot \frac{1000 \cdot 34^3}{12} = 3.603x10^{10}\, N \cdot mm^2$$

$$E_2 I_{a,2} = E_2 \frac{b \cdot h_2^{\ 3}}{12} = 300 \cdot \frac{1000 \cdot 19^3}{12} = 1.715x10^{8}\, N \cdot mm^2$$

$$E_3 I_{a,3} = E_3 \frac{b \cdot h_3^{\ 3}}{12} = 11000 \cdot \frac{1000 \cdot 34^3}{12} = 3.603x10^{10}\, N \cdot mm^2$$

$$E_4 I_{a,4} = E_4 \frac{b \cdot h_4^{\ 3}}{12} = 300 \cdot \frac{1000 \cdot 19^3}{12} = 1.715x10^{8}\, N \cdot mm^2$$

$$E_5 I_{a,5} = E_5 \frac{b \cdot h_5^{\ 3}}{12} = 11000 \cdot \frac{1000 \cdot 34^3}{12} = 3.603x10^{10}\, N \cdot mm^2$$

Then,

$$B_A = 1.084x10^{11}\, N \cdot mm^2$$

c) Calculation of $B_B = \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2$

$$E_1 I_{b,1} = E_1 A_1 \cdot Z_1^{\ 2} = 11{,}000 \cdot 1{,}000 \cdot 34 \cdot 53^2 = 1.051x10^{12}\, mm^4$$

$$E_2 I_{b,2} = E_2 A_2 \cdot Z_2^{\ 2} = 300 \cdot 1{,}000 \cdot 19 \cdot 26.5^2 = 4.003x10^{9}\, mm^4$$

$$E_3 I_{b,3} = E_3 A_3 \cdot Z_3^{\ 2} = 11{,}000 \cdot 1{,}000 \cdot 34 \cdot 0 = 0 mm^4$$

$$E_4 I_{b,4} = E_4 A_4 \cdot Z_4^{\ 2} = 300 \cdot 1{,}000 \cdot 19 \cdot (-26.5^2) = 4.003x10^{9}\, mm^4$$

$$E_5 I_{b,5} = E_5 A_5 \cdot Z_5^{\ 2} = 11{,}000 \cdot 1{,}000 \cdot 34 \cdot (-53^2) = 1.051x10^{12}\, mm^4$$

Then,

$$B_B = 2.110x10^{12}\, N \cdot mm^2$$

d) Calculation of EI_{eff}:

$$(EI)_{eff} = B_A + B_B = 1.084x10^{11} + 2.110x10^{12}$$

$$(EI)_{eff} = 2218x10^{9}\, N \cdot mm^2$$

e) Calculation of Shear Stiffness

h total = $h_1 + h_2 + h_3 + h_4 + h_5$ = 140 mm and b = 1000 mm

$$a = h_{total} - \frac{h_1}{2} - \frac{h_n}{2} \qquad where\ n = 5$$

$$a = 140 - \frac{34}{2} - \frac{34}{2} = 106\ mm$$

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

$$(GA)_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_1 \cdot b}\right) + \left(\sum_{i=2}^{4} \frac{h_i}{G_i \cdot b_i}\right) + \left(\frac{h_5}{2 \cdot G_5 \cdot b}\right)\right]}$$

$$(GA)_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_1 \cdot b}\right) + \left(\frac{h_2}{G_2 \cdot b} + \frac{h_3}{G_3 \cdot b} + \frac{h_4}{G_4 \cdot b}\right) + \left(\frac{h_5}{2 \cdot G_5 \cdot b}\right)\right]}$$

$$(GA)_{eff} = \frac{106^2}{\left[\left(\frac{34}{2 \cdot 690 \cdot 1000}\right) + \left(\frac{19}{56 \cdot 1000} + \frac{34}{690 \cdot 1000} + \frac{19}{56 \cdot 1000}\right) + \left(\frac{34}{2 \cdot 690 \cdot 1000}\right)\right]}$$

$$(GA)_{eff} = 1.446x10^7\ N$$

Then, the deflection under uniform Live Load can be calculated as:

$$\Delta_L = \frac{5}{384} \cdot \frac{wL^4}{(EI)_{eff}} \cdot \left(1 + \frac{48(EI)_{eff}\, k}{5(GA)_{eff}\, L^2}\right)$$

$$\Delta_L = \frac{5}{384} \cdot \frac{1.9 \cdot 4500^4}{2218x10^9} \cdot \left(1 + \frac{48 \cdot 2218x10^9 \cdot 1.2}{5 \cdot 1.446x10^7 \cdot 4500^2}\right) = 5.0\ mm$$

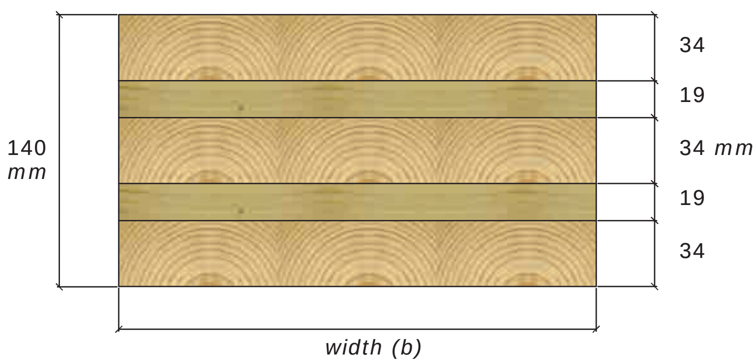
Summary for a 140 mm 5-layer panel:

Mechanically Jointed Beams Theory (Gamma Method): ΔL = 4.9 mm (correct for l/h>30)
Composite Theory (k-Method): ΔL = 4.6 mm (correct for l/h>30)
Shear Analogy Method (Kreuzinger): ΔL = 5.0 mm

It can be seen that the final results are very similar using the three proposed methods and for a span-to-depth ratio of about 30.

3.5 Calculation of Out-of-Plane Bending Strength

3.5.1 Out-of-Plane Bending Strength of a Five-Layer CLT Panel – 140 mm thick



Panel properties and parameters for this example:

$$b = 1000\ mm \qquad h_{tot} = 140 mm$$

$$l = 4500\ mm \quad (simple\ \ span; in\ direction\ \ of\ \ action\ //)$$

$$l/h_{tot} = 32$$

$$Longitudinal\ layers: \quad MSR\ \ 1800\ fb - 1.6E$$

$$E_0 = 11000\ MPa \qquad E_{90} \approx \frac{E_0}{30} = \frac{11000}{30} = 367 \approx 370\ MPa$$

$$G_0 \approx \frac{E_0}{16} = \frac{11000}{16} = 688 \approx 690\ MPa \qquad G_R \approx \frac{G_0}{10} = \frac{690}{10} = 69\ MPa$$

$$f_b = 26.1\ MPa$$

$$Perpendicular\ layers: \quad SPF\ \ No.3 / Stud$$

$$E_0 = 9000\ MPa \qquad E_{90} \approx \frac{E_0}{30} = \frac{9000}{30} = 300\ MPa$$

$$G_0 \approx \frac{E_0}{16} = \frac{9000}{16} = 563 \approx 560\ MPa \qquad G_R \approx \frac{G_0}{10} = \frac{560}{10} = 56\ MPa$$

$$f_b = 7.0\ MPa$$

1) Mechanically Jointed Beams Theory (Gamma Method)

The maximum bending stress may be expressed as:

$$\sigma_{\max} = \sigma_{global} + \sigma_{local}$$

$$\sigma_{\max} = \frac{\gamma_1 E_1 a_1 M}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M}{(EI)_{eff}} = \frac{M E_1}{(EI)_{eff}} \cdot (\gamma_1 a_1 + 0.5 h_1)$$

From Figure 6, the term a_1 is the distance between the centroid of the first lamina and the centroid of the panel cross-section, and the term h_1 is the thickness of the first (outermost) lamina.

In this example, the modulus of elasticity of all longitudinal layers is equal to 11000 MPa:

$$\sigma_{\max} = \frac{M}{I_{eff}} \cdot (\gamma_1 a_1 + 0.5 h_1)$$

If we use the CSA O86 design analogy, we can let: $\sigma_{\max} = \sigma_{global} + \sigma_{local} \leq \phi \cdot F_b$

Subsequently, the factored moment bending resistance Mr in terms of the specified bending strength F_b may be determined as:

$$M_r = \phi \cdot F_b \cdot \frac{I_{eff}}{(\gamma_1 a_1 + 0.5 h_1)}$$

From Figure 6 and from the example given for mechanically jointed beams theory in Section 3.4, we obtained:

$$a_1 = 53mm \qquad h_1 = 34mm \qquad \gamma_1 = \gamma_3 = 0.9418$$

$$EI_{eff\,//} = 2087 \times 10^9 \, N \cdot mm^2$$

$$I_{eff} = \frac{2087 \times 10^9}{11000} = 189.7 \times 10^6 \, mm^4$$

In that case,

$$\phi = 0.9 \quad and \quad Fb = 26.1 \, MPa \quad (if \; K_D = K_S = K_T = K_H = 1)$$

then,

$$M_r = \phi \cdot F_b \cdot \frac{I_{eff}}{(\gamma a_1 + 0.5 h_1)} = 0.9 \cdot 26.1 \cdot \frac{189.7 \times 10^6}{(0.9418 \cdot 53 + 0.5 \cdot 34)} \times 10^{-6}$$

$$M_r = 66.6 \, kN - m$$

2) Composite Theory (k-method)

The maximum bending stress may be expressed as:

$$\sigma_{\max} = \frac{M}{S}$$

If we use the CSA O86 design analogy, we can let:

$$\sigma_{\max} \leq \phi \cdot F_{b,eff}$$

where $F_{b,eff}$ is the effective bending strength value obtained from Tables 2 and 3. Then, the factored moment bending resistance M_r in terms of the specified bending strength F_b can be expressed as:

$$M_r = \phi \cdot F_{b,eff} \cdot S_{gross}$$

From Section 3.4 for the example given for the composite theory we obtained:

$$k_1 = 0.8816$$

From Table 3:

$$f_{b,0,eff} = f_{b,0} \cdot k_1$$

where:

$f_{b,0} = f_b = 26.1\ MPa$ (from CSA O86)

then,

$$f_{b,0,eff} = f_{b,0} \cdot k_1 = 26.1 \cdot 0.8816 = 23.0\ MPa$$

In that case,

$$\phi = 0.9$$
$$Fb = 23.0\ MPa \quad (if\ K_D = K_S = K_T = K_H = 1)$$

then,

$$M_r = \phi \cdot F_{b,eff} \cdot S_{gross}$$

$$M_r = 0.9 \cdot 23.0 \cdot \frac{1000 \cdot 140^2}{6} \times 10^{-6}$$

$$M_r = 67.6\ kN - m$$

3) Simplified Method

The maximum bending stress may be expressed as:

$$\sigma_{\max} = M \cdot 0.5h_{tot} \cdot \frac{(E_1)}{(EI)_{eff}}$$

If we use the CSA O86 design analogy, we can let:

$$\sigma_{\max} \leq \phi \cdot F_b$$

Subsequently, the factored moment bending resistance M_r in terms of the specified bending strength F_b may be determined as:

$$M_r = \phi \cdot F_b \cdot \frac{(EI)_{eff}}{E_1} \cdot \frac{1}{0.5h_{tot}}$$

In this example, the modulus of elasticity of all longitudinal layers is equal i.e. E_1=E_2=E_3=E = 11000 MPa. Then the maximum bending stress can be obtained as:

$$M_r = \phi \cdot F_b \cdot \frac{I_{eff}}{0.5h_{tot}}$$

From Section 3.4 for the example given for the Shear Analogy Method (Kreuzinger) we obtained:

$$EI_{eff\,//} = 2218x10^9\, N \cdot mm^2$$

$$I_{eff} = \frac{2218x10^9}{11000} = 201.6 \times 10^6\, mm^4$$

In that case,

$$\phi = 0.9$$

$$Fb = 26.1\, MPa \quad (if\ K_D = K_S = K_T = K_H = 1)$$

then,

$$M_r = \phi \cdot F_b \cdot \frac{I_{eff}}{0.5h_{tot}} = 0.9 \cdot 26.1 \cdot \frac{201.6 \times 10^6}{0.5 \cdot 140} \times 10^{-6}$$

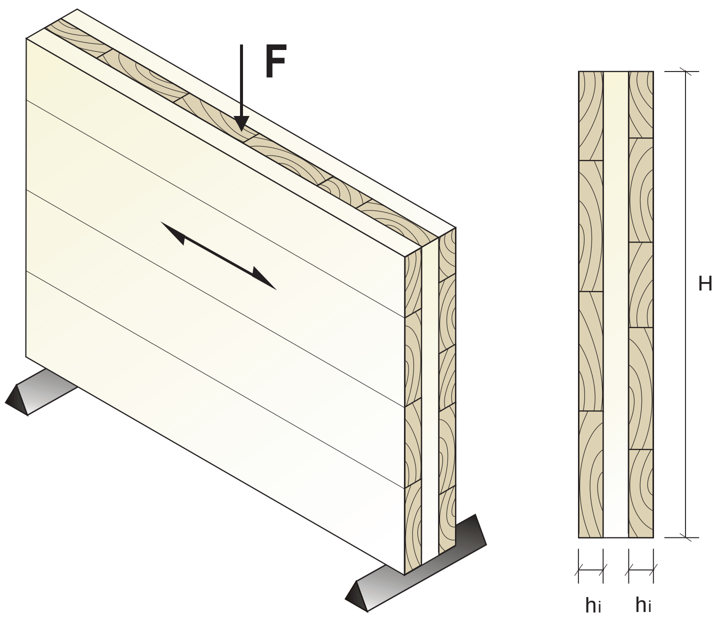
$$M_r = 67.7\, kN - m$$

Summary for a 140 mm 5-layer panel:

Mechanically Jointed Beams Theory (Gamma Method):	M_r = 66.6 kN-m
Composite Theory (k-Method):	M_r = 67.6 kN-m
Simplified Method:	M_r = 67.7 kN-m

3.6 Calculation of In-Plane Bending Strength (Lintels or Beams)

3.6.1 In-Plane Bending Strength of a Three-Layer CLT Panel – 94 mm thick



Panel properties and parameters for this example:

$$H = 1000\ mm \qquad h_{tot} = 94\ mm$$

$$Longitudinal\ layers: \quad MSR\ 1800\,fb - 1.6E$$

$$E_0 = 11000\ MPa \qquad E_{90} \approx \frac{E_0}{30} = \frac{11000}{30} = 367 \approx 370\ MPa$$

$$G_0 \approx \frac{E_0}{16} = \frac{11000}{16} = 688 \approx 690\ MPa \qquad G_R \approx \frac{G_0}{10} = \frac{690}{10} = 69\ MPa$$

$$f_b = 26.1\ MPa$$

$$Perpendicular\ layer: \quad SPF\ No.3\,/\,Stud$$

$$E_0 = 9000\ MPa \qquad E_{90} \approx \frac{E_0}{30} = \frac{9000}{30} = 300\ MPa$$

$$G_0 \approx \frac{E_0}{16} = \frac{9000}{16} = 563 \approx 560\ MPa \qquad G_R \approx \frac{G_0}{10} = \frac{560}{10} = 56\ MPa$$

$$f_b = 7.0\ MPa$$

Note: In this example, the compression edge of the beam is supported throughout its span *l* (i.e. K_L=1.0).

1) Simplified Method

In this example, the modulus of elasticity of all longitudinal layers is equal. Thus, the factored moment bending resistance, M_r, in terms of the specified bending strength, F_b, may be expressed as:

$$M_r = \phi \cdot F_b \cdot \frac{I_{eff}}{0.5H}$$

And I_{eff} can be calculated as:

$$I_{eff} = \frac{h_{eff.} \cdot H^3}{12} = \frac{H^3}{12} \cdot \sum_i h_i$$

where *H* is the beam depth equal to 1000 mm and h_i is the thickness of boards perpendicular to the axial load:

$$I_{eff} = \frac{h_{eff.} \cdot H^3}{12} = \frac{1000^3}{12} \cdot (30 + 30) = 5000x10^6\ mm^4$$

In that case,

$$\phi = 0.9$$

$$F_b = 26.1\,MPa \quad (if\ K_D = K_S = K_T = K_H = K_L = 1)$$

then,

$$M_r = \phi \cdot F_b \cdot \frac{I_{eff}}{0.5H} = 0.9 \cdot 26.1 \cdot \frac{5000x10^6}{0.5 \cdot 1000}\,x10^{-6}$$

$$M_r = 234.9\,kN - m$$

2) Composite Theory (k-Method)

In this example, the modulus of elasticity of all longitudinal layers is equal. Thus, the factored moment bending resistance, M_r, in terms of the specified bending strength, F_b, may be expressed as:

$$M_r = \phi \cdot F_{b,eff} \cdot S_{gross}$$

where $F_{b,eff}$ is the effective bending strength values $f_{b,0,eff}$ obtained from Tables 2 and 3 for the longitudinal boards.

From Table 2 with *m*=3:

$$a_m = a_3 = 94\,mm$$
$$a_{m-2} = a_1 = 34\,mm$$

$$k_3 = 1 - \left[\left(1 - \frac{E_{90}}{E_0}\right) \cdot \left(\frac{a_1}{a_3}\right)\right]$$

$$k_3 = 1 - \left[\left(1 - \frac{300}{11000}\right) \cdot \left(\frac{34}{94}\right)\right]$$

$$k_3 = 0.6482$$

From Table 3 using $f_{b,0} = f_b = 26.1$ MPa for longitudinal boards (from CSA O86):

$$f_{b,0,eff} = f_{b,0} \cdot k_3 = f_b \cdot k_3 = 26.1 \cdot 0.6482 = 16.9\,MPa$$

Thus,

$$M_r = \phi \cdot F_{b,eff} \cdot S_{gross}$$

where:

$$\phi = 0.9$$

$$F_{b,eff} = f_{b,0,eff} = 16.9\ MPa \quad (if\ K_D = K_S = K_T = K_H = K_L = 1)$$

$$S_{gross} = \frac{h_{tot} \cdot H^2}{6} = \frac{94 \cdot 1000^2}{6} = 15667x10^3\ mm^3$$

$$M_r = \phi \cdot F_{b,\,eff} \cdot S_{gross} = 0.9 \cdot 16.9 \cdot 15667x10^3\ x10^{-6}$$

$$M_r = 238.3\ kN - m$$

4

CONCLUSION AND RECOMMENDATIONS

It was demonstrated in this chapter that various methods have been adopted in Europe for the determination of design properties of CLT. However, no analytical approach has been universally accepted by CLT manufacturers and designers so far.

It seems that the most common analytical approach that has been adopted for CLT in Europe is based on the mechanically jointed beams theory that is available in Annex B of Eurocode 5 (EN 2004). This approach provides a closed solution for the differential equation only for simply supported beams/panels with a sinusoidal load distribution. However, the differences between the exact solution and those for a uniformly distributed load or point loads are minimal and are acceptable for engineering practice (Ceccotti, 2003).

Another design methodology has been proposed by Blass and Fellmoser (2004). This method applies the "Composite Theory" (also named k-method) to predict flexural properties of CLT. However, this method does not account for shear deformation in individual layers.

More recently, a new method called "Shear Analogy" (Kreuzinger, 1999) has been developed in Europe that seems to be applicable for solid panels with cross layers. The methodology takes into account the shear deformation of the longitudinal and the cross layers and is not limited by the number of layers within a panel. This method seems to be the most accurate and adequate for prediction of stiffness properties of CLT panels.

It was also found in the literature that almost all studies conducted in Europe so far have focused primarily on predicting the stiffness and not the strength properties of CLT panels in flexure. While flexural stiffness of CLT panels is usually of greater interest for designers than the strength, since the structural design is mostly governed by serviceability criteria (i.e. deflection and vibration), from a product standard development point of view, there is a need to characterize the strength properties as well, to ensure certain minimum panel strength in service. Design methods to evaluate out-of-plane and in-plane bending strength have been proposed. Design methods for walls were also proposed in this chapter.

There is a need to adopt a design methodology for determination of the stiffness and the strength properties of CLT in flexure by further exploring the shear analogy approach. It is expected that the proposed analytical approach will be accepted in the upcoming CLT product standard. The procedure to calculate the design properties should be based on material properties for lumber published in the design standards, and should be consistent with the design philosophy in CSA O86, the Canadian Standard for Engineering Design in Wood. Because of these potentially important features, the developed analytical method will need to be comprehensively verified against test data.

5
REFERENCES

Aicher, S., and G. Dill-Langer. 2000. Basic considerations to rolling shear modulus in wooden boards. *Otto-Graf-Journal* 11:158-165.

Bazant, Z. P., and L. Cedolin. 2003. *Stability of structures: Elastic, inelastic, fracture, and damage theories.* Mineola, NY: Dover Publications. 1011 p.

Bejtka, I., and F. Lam. 2008. Cross laminated timber as innovative building material. In *Proceedings of the CSCE Annual Conference, Québec, QC*. Montréal, QC: Canadian Society for Civil Engineering. CD-ROM.

Blass H. J., and P. Fellmoser. 2004. Design of solid wood panels with cross layers. In *Proceedings of the 8th World Conference on Timber Engineering, June 14-17, 2004, Lahti, Finland*, 2:543-548.

Blass, H. J., and T. Uibel. 2007. Edge joints with dowel type fasteners in cross laminated timber. In *Proceedings of CIB-W18 Timber Engineering, University of Karlsruhe, Karlsruhe, Germany*, paper 40-7-2.

Bogensperger, T., T. Moosebrugger, and G. Schickhofer. 2007. New test configuration for CLT-wall elements under shear load. In *Proceedings of CIB-W18 Timber Engineering, University of Karlsruhe, Karlsruhe, Germany*, paper 40-21-2.

Canadian Standard Association (CSA). 2004. *Design of concrete structures.* CSA A23.3-04. Rexdale, ON: CSA. 214 p.

______. 2009. *Engineering design in wood (limit states design)*. CSA O86-09. Rexdale, ON: CSA. 222 p.

Ceccotti A. 2003. Composite structures. In *Timber Engineering*, ed. S. Thelandersson and H. J. Larsen, chapter 21. West Sussex, England: John Wiley and Sons.

Chen, J. Y. 2009. Development of cross lamination technology for MPB engineered wood products thick laminated MPB wood plates: Report submitted to Forestry Innovation Investment Ltd. Contract number FII-MDP-09-0083. Vancouver, BC: University of British Columbia. Department of Wood Science.

European Committee for Standardization. 2004. *Eurocode 5: Design of timber structures. Part 1-1: General – Common rules and rules for buildings*. EN 1995-1-1. Brussels: CEN. 124 p.

Fellmoser, P., and H. J. Blass. 2004. Influence of rolling shear modulus on strength and stiffness of structural bonded timber elements. In *Proceedings of CIB-W18 Meeting, Edinburgh, United Kingdom*, paper 37-6-5.

Foschi, R. O. 1993. Design recommendations for timber bridges: Report to CSA-S6 Committee. Report No. 1.

Jöbstl, R. A., T. Bogensperger, and G. Schickhofer. 2008. In-plane shear strength of cross laminated timber. In *Proceedings of CIB-W18 Timber Engineering, University of Karlsruhe, Karlsruhe, Germany*, paper 41-12-3.

Jöbstl, R.A., T. Moosbrugger, T. Bogensperger and G. Schickhofer. 2006. A contribution to the design and system effect of cross laminated timber (CLT). In *Proceedings of CIB-W18 Timber Engineering, University of Karlsruhe, Karlsruhe, Germany*, paper 39-12-5.

Jöbstl, R. A., and G. Schickhofer. 2007. Comparative examination of creep of GTL and CLT-slabs in bending. In *Proceedings of CIB-W18 Timber Engineering, University of Karlsruhe, Karlsruhe, Germany*, paper 40-12-3.

Kreuzinger H. 1995. Mechanically jointed beams and columns. In *Timber Engineering – STEP 1*, ed. H. J. Blass et al., B11/1-8. Almere, The Netherlands: Centrum Hout.

______. 1999. Platten, Scheiben und Schalen – ein Berechnungsmodell für gängige Statikprogramme. *Bauen mit Holz* 1: 34-39.

Mestek, P., H. Kreuzinger, and S. Winter. 2008. Design of cross laminated timber (CLT). Paper presented at the *10th World Conference on Timber Engineering*, June 2-5, 2008, Miyazaki, Japan.

Österreichisches Institut für Bautechnik (OIB). 2005. CUAP (Common Understanding of Assessment Procedure): Solid wood slab element to be used as a structural element in buildings. ETA request no. 03.04/06. Wien, Austria: OIB. 28 p.

Steiger, R., and A. Gülzow. 2009. Validity of bending tests on strip-shaped specimens to derive bending strength and stiffness properties of cross laminated solid timber (CLT). In *Proceedings of CIB-W18 Timber Engineering, University of Karlsruhe, Karlsruhe, Germany*, paper 42-12-4.

Steiger, R., A. Gülzow, and D. Gsell. 2008. Non-destructive evaluation of elastic material properties of cross laminated timber (CLT). In *Proceedings of the COST E53 Conference, October 29-30, 2008, Delft, The Netherlands*, p. 171-182.

Timoshenko, S., and S. Woinowsky-Krieger. 1959. *Theory of plates and shells.* New York: McGraw-Hill. 580 p.

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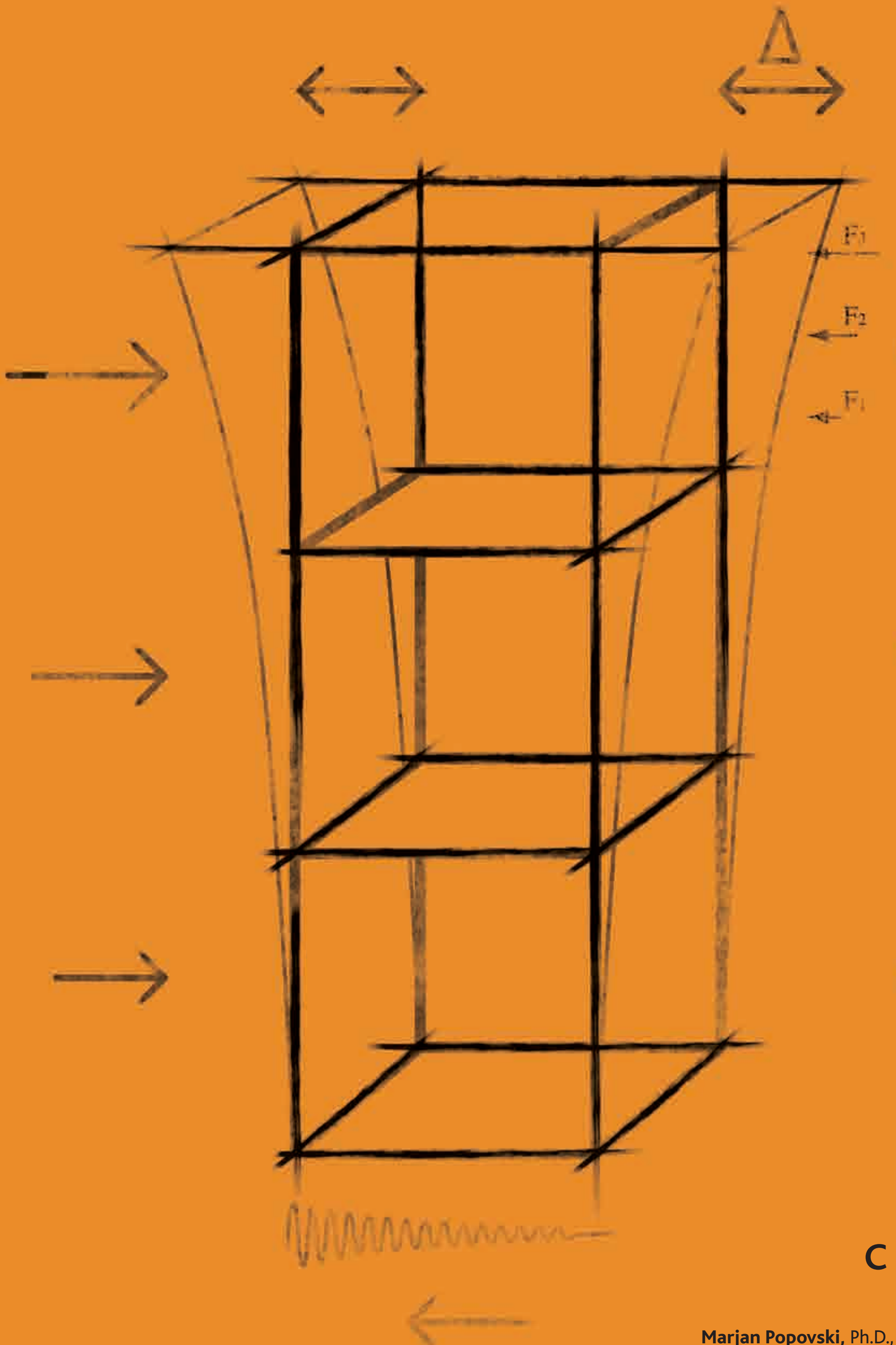

Seismic

Seismic performance of cross-laminated timber buildings

CHAPTER 4

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ABSTRACT

Cross-laminated timber (CLT) is an innovative wood product that was first developed some 20 years ago in Austria and Germany and ever since has been gaining popularity in residential and non-residential applications in Europe. European experience shows that this system can be competitive, particularly in mid-rise and high-rise buildings.

In this chapter, a literature review on the research work conducted around the world related to the seismic performance of cross-laminated timber (CLT) wall panels and structures is included. This is followed by the results from a series of quasi-static tests on CLT wall panels that were conducted at FPInnovations' Wood Products laboratory in Vancouver. CLT wall panels with various configurations and connection details were tested. These configurations included single panel walls with three different aspect ratios, multi-panel walls with step joints and different types of screws to connect them, as well as two-storey wall assemblies. Connections for securing the walls to the foundation included off-the-shelf steel brackets with annular ring nails, spiral nails, and screws; combination of steel brackets and hold-downs; diagonally placed long screws; and custom made brackets with timber rivets. Results showed that CLT walls can have adequate seismic performance when nails or screws are used with the steel brackets. Use of hold-downs with nails on each end of the wall improves their seismic performance. Use of diagonally placed long screws to connect the CLT walls to the floor below is not recommended in high seismic zones due to less ductile wall behaviour and to the sudden screw pull-out failure mechanism. Use of step joints in longer walls can be an effective solution not only to reduce the wall stiffness and thus reduce the seismic input load, but also to improve the wall deformation capabilities. Timber rivets in small groups with custom made brackets were found to be effective connectors for CLT wall panels.

In addition, this chapter includes a survey of potentially available methods for development and assessment of R-factors for different structural systems. Studies conducted in Europe on the assessment of the behaviour *q*-factor (European R-factor equivalent) for CLT structures and their findings are also discussed. Finally, based on all available information, estimates were made on the values of R-factors for CLT structures according to the National Building Code of Canada, and capacity-based design procedures for CLT structures were drafted.

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1 INTRODUCTION

Cross-laminated timber (CLT) is an innovative wood product that was first developed some 20 years ago in Austria and Germany and has since been gaining popularity in residential and non-residential applications in Europe. By using cross-laminated solid timber boards for prefabricated wall and floor panels, this system offers many advantages. The cross lamination process provides improved dimensional stability to the product that allows for prefabrication of long floor slabs and single-storey walls. Openings for windows and doors can be pre-cut using sophisticated Computer Numerical Controlled (CNC) machines. CLT panels are easy to process and to assemble with ordinary tools. Quick erection of solid and durable structures is possible even for non-highly-skilled workers. Good thermal insulation and a fairly good behaviour in case of fire are added benefits resulting from the massiveness of the wood structure. European experience shows that this system can be competitive, particularly in mid-rise and high-rise buildings. Although CLT has rarely been used in North America to date, it could be used as a viable wood-based structural solution for the shift towards sustainable densification of urban and suburban centres in Canada and the US. In order to gain much needed wide acceptance and popularity, CLT as a structural system needs to be implemented in the North American codes arena.

For these reasons, the Wood Products Division of FPInnovations has undertaken a multi-disciplinary research project on determining the structural properties of typical CLT construction. One of the important parts of the project is to quantify the seismic resistance of structures with CLT panels, including the development of the force modification factors (R-factors) for seismic design according to the National Building Code of Canada. In this chapter, some of the results from a series of quasi-static monotonic and cyclic tests on CLT wall panels are presented, which are the first of their kind conducted in North America. This is followed by a survey of the potentially available methods for development or assessment of R-factors for different structural systems, and some findings from Europe related to CLT as a structural system. Finally, based on the available information, conservative estimates are made for the R-factors for CLT structures appropriate for the National Building Code of Canada.

2

PREVIOUS RESEARCH IN THE FIELD

The most comprehensive study to quantify the seismic behaviour of low- and mid-rise CLT construction was part of the SOFIE project in Italy. This project was undertaken by the Trees and Timber Institute of the National Research Council of Italy (CNR-IVALSA) in collaboration with National Institute for Earth Science and Disaster Prevention in Japan (NIED), Shizuoka University, and the Building Research Institute (BRI) in Japan. The testing program included tests on connections; in-plane cyclic tests on CLT wall panels with different layouts of connections and openings (Ceccotti et al., 2006); pseudo-dynamic tests on a one-storey 3-D specimen in three different layouts (Lauriola and Sandhaas, 2006); shake table tests on a three-storey, 7 m x 7 m in plan and 10 m high building under different earthquakes (Ceccotti and Follesa, 2006); and finally a series of full-scale shaking table tests on a seven-storey CLT building conducted at E-Defense facility in Miki, Japan.

Results from quasi-static tests on CLT wall panels showed that the connection layout and design has a strong influence on the overall behaviour of the wall (Ceccotti et al., 2006). Hysteresis loops were found on average to have an equivalent viscous damping of 12%. Similar to the cyclic tests, the pseudo-dynamic tests showed that the construction system is very stiff but still ductile (Lauriola and Sandhaas, 2006). It was found that the initial stiffness of the 3-D specimen with asymmetric configuration (openings of 4.0 m on one side and 2.25 m on the other) was similar to that of the symmetric configuration (openings of 2.25 m on both sides), suggesting that the larger opening on one side did not affect the building stiffness very much. It thus confirmed that the behaviour of the wall is due to the connections and not to the wooden panel for lower levels of lateral force.

Figure 1
Three-storey CLT house tested at NIED Laboratory in Tsukuba, Japan

Shaking table tests on the 3-storey house conducted in the laboratories of the NIED in Tsukuba, Japan (Figure 1) showed that the CLT construction survived 15 destructive earthquakes without any severe damage (Ceccotti and Follesa, 2006). The collapse state definition for the tests was defined to be failure of one or more hold-down anchors, which was reached only during the last test that used the Nocera Umbra earthquake record with peak ground acceleration (PGA) of 1.2 g. An analytical model of the 3-storey house was developed using the DRAIN 3-DX computer program. The model was used to predict the behaviour of the 3-storey house during the shaking table tests, and showed good correlation with the test results. Using the same analytical model, a number of non-linear time-history dynamic analyses were conducted using eight different earthquake records and an evaluation of the behaviour factor q for seismic design according to Eurocode 8 was conducted (Ceccotti et al., 2006; Ceccotti, 2008). The behaviour factor q was defined as the ratio between the PGA that caused the failure (uplift of 25.5 mm at one or more hold-down positions in the walls) vs. the design PGA. For seven out of eight earthquakes, the q-factor was greater than 3.0 and in two cases even greater than 4.0, with an average of 3.4.

The next series of shaking table tests from the SOFIE project was conducted in October 2007 at the Hyogo Earthquake Engineering Research Centre in Miki, Japan. The building had a floor plan of 13.5 m x 7.5 m, and was comprised of seven storeys with a total height of 23.5 m (Figure 2). The building walls were made of CLT panels with a thickness of 142 mm on the first two floors, 125 mm on floors three and four, and 85 mm on the last three floors, where less loads were expected. The walls were connected to each other using self-drilling (tapping) screws. Each wall consisted of several 2.5 m long panels connected together with screws. The floors were also made with CLT panels with a thickness of 142 mm, and were connected to the walls with steel brackets and screws. The building was designed using a q-factor of 3, and importance factor of 1.5 according to Eurocode. The testing consisted of several consecutive applications in all three orthogonal directions of two earthquake ground motions, including the record from the Great Hanshin-Awaji Earthquake from 1995, also known as Kobe Earthquake (M=7.2) with 100% intensity (0.6 g acceleration in shorter X-direction, 0.82 g in longitudinal Y-direction, and 0.34 g in vertical Z-direction). The structure withstood all tests without any significant damage. The first storey drift was 38 mm (1.3% drift) in the Y-direction and 29 mm (1% drift) in the X-direction, with the total deflection at the top of the building being 175 mm and 287 mm, respectively.

Figure 2
Seven-storey CLT house tested at E-Defense Laboratory in Miki, Japan

The most comprehensive study to determine the seismic behaviour of 2-D CLT wall panels was conducted at the University of Ljubljana, Slovenia. During the project that was partially supported by KLH Massiveholz GmbH from Austria, numerous quasi-static monotonic and cyclic tests were carried out on walls with lengths of 2.44 m and 3.2 m and heights of 2.44 m and 2.72 m (Dujic et al., 2004). Walls were subjected to combined constant vertical load and either monotonic or cyclic horizontal loads. Wall panels were tested with various boundary conditions which enabled the development of load vs. wall deformation relations from cantilever to pure shear wall behaviour. Influence of boundary conditions, magnitude of vertical load and types of anchoring systems were investigated (Dujic et al., 2005, 2006). Differences in mechanical properties between monotonic and cyclic responses were also studied (Dujic and Zarnic, 2006), as was the influence of openings on the shear wall properties (Dujic et al., 2006, 2007). Two configurations of walls with equal dimensions, one with no opening and one with a door and a window, were tested under the same boundary conditions. Analytical models of CLT wall panels were developed in the computer program SAP 2000, and were verified against the test results. The verified analytical models were used for a parametric study that included 36 mathematical models having different patterns of openings (Dujic et al., 2008). Results of the parametric study were used to develop mathematical formulas describing the relationship between the shear strength and stiffness of CLT wall panels with and without openings. As part of the project, shaking table tests were conducted on two single-storey box CLT models at the Dynamic Testing Laboratory of the Institute of Earthquake Engineering and Engineering Seismology (IZIIS) in Skopje, Macedonia (Dujic et al., 2006; Hristovski et al., 2005). The intent was to make a correlation between the results from the quasi-static tests and the results from the shaking table tests. Based on these tests, the main characteristics of the dynamic response of the tested models were determined.

Finally, CLT wall tests were carried out by Karlsruhe Institute of Technology in order to compare the performance of such a modern system vs. the "traditional" timber-frame construction (Schädle et al., 2010).

3

CLT WALL SPECIMENS IN THE FPINNOVATIONS STUDY

In the testing program at FPInnovations in Vancouver, a total of 32 monotonic and cyclic tests were performed. All walls were 3-ply CLT panels with a thickness of 94 mm. They were made of European spruce and manufactured at KLH Massiveholz GmbH in Austria, one of the largest CLT manufacturers in Europe. Since the CLT panels had to be shipped in a container over the ocean, one of the panel dimensions was limited to 2.3 m, which was the height and width of the container. CLT walls with 12 different configurations were tested. Details about the testing matrix and the different wall configurations I to XII are given in Tables 1, 2 and 3. In Table 1, walls with aspect ratio of 1:1 are shown (2.3 m high and 2.3 m long), while in Table 2 walls with aspect ratio of 1:1.5 are shown (2.3 m high and 3.45 m long). In Table 3, two-storey assemblies of 2.3 m x 2.3 m walls are presented along with tall CLT walls that had a height of 4.9 m and a length of 2.3 m (aspect ratio of 2.1:1).

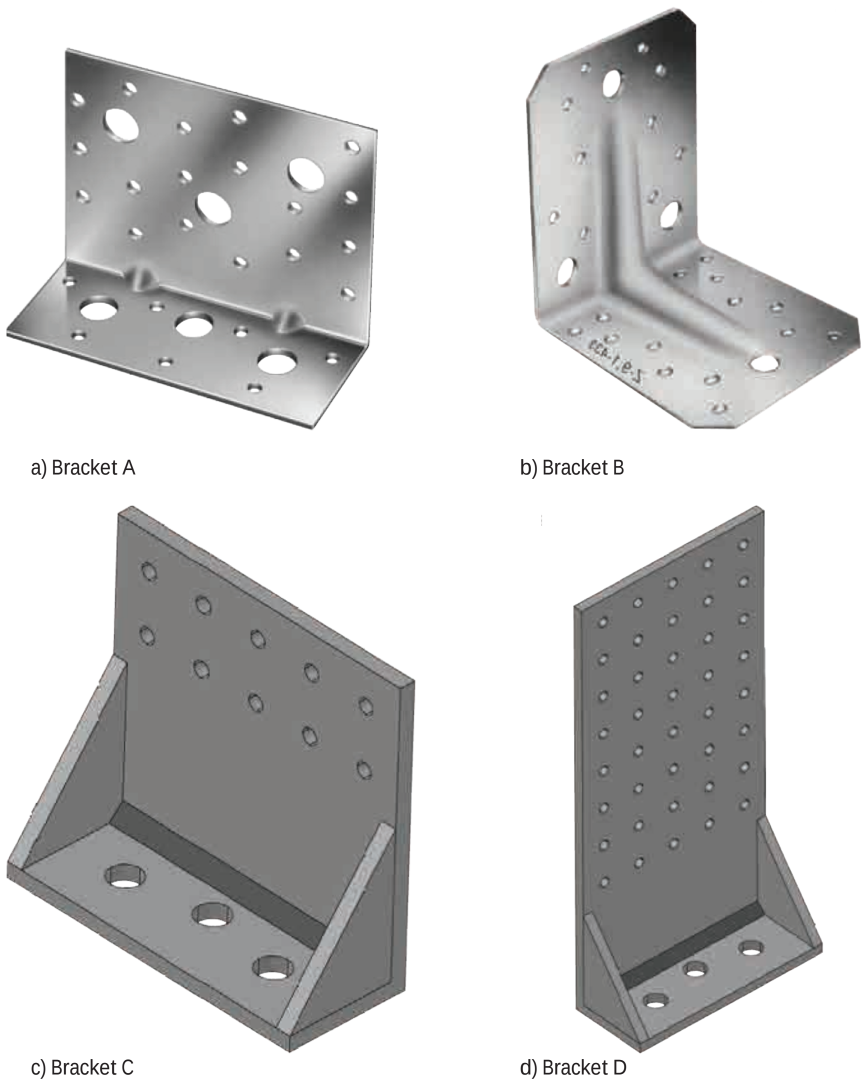
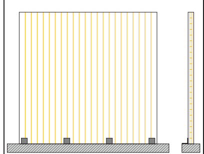
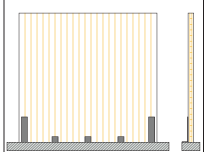
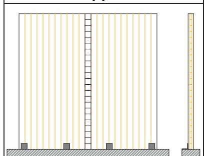
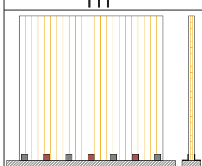
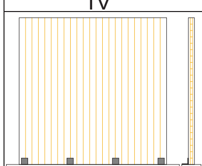
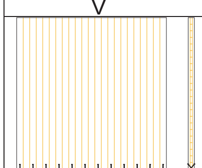
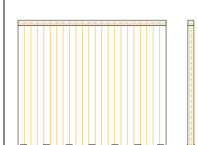


Figure 3
Brackets for CLT walls used in the tests

Four different types of brackets (A, B, C, and D) were used to connect the walls to the steel foundation beam or to the CLT floor panel below (Figure 3). Bracket A (BMF), 90 mm x 48 mm x 116 mm (W x D x H), and bracket B (Simpson Strong Tie), 90 mm x 105 mm x 105 mm, are off-the-shelf products that are commonly used in CLT applications in Europe. Brackets C and D were custom made out of 6.4 mm thick steel plates to accommodate the use of timber rivets. The designations of the tests shown in Tables 1, 2 and 3 were developed to show the bracket type and the fastener type used in the tests. For example, designation **CA-SNH-08A** means that the **C**LT wall had type **A** brackets, **S**piral **N**ails as fasteners, had **H**old-downs and was test number **08A**. The following acronyms were also used in the test designations: TR for **T**imber **R**ivets, RN for Annular **R**ing **N**ails, **S1** for SFS screws 4 x 70 mm, **S2** for SFS screws 5 x 90 mm, and **WT** for SFS WT-T type screws.

Table 1

Test matrix for 2.3 m long and 2.3 m high walls

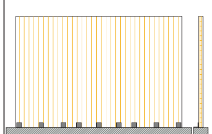
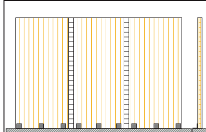
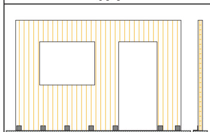
Wall Configuration	Test Designation	Brackets and Fasteners	Vertical Load [kN/m]	Lateral Load
 I	CA-SN-00	Bracket A SN 16d, n=18 D=3.9 mm L=89 mm	0	CUREE
	CA-SN-01		10	Monotonic
	CA-SN-02		10	CUREE
	CA-SN-03		20	CUREE
	CA-RN-04	RN 10d (3.4 x 76 mm), n=12	20	CUREE
	CA-S1-05	S1 (4 x 70 mm), n=18	20	CUREE
	CA-S2-06	S2 (5 x 90 mm), n=10	20	CUREE
	CC-TR-09	Bracket C, Rivets L=65 mm, n=10	20	Monotonic
	CC-TR-10A	Bracket C, Rivets L=65 mm, n=10	20	CUREE
 II	CA-SNH-07	SN 16d (3.9 x 89 mm), n=18 Same on Hold-Down	20	Monotonic
	CA-SNH-08	SN 16d (3.9 x 89 mm), n=18 Same on Hold-Down	20	CUREE
	CA-SNH-08A	SN 16d (3.9 x 89 mm), n=18 12d (3.3 x 63 mm), n=18 on HD	20	CUREE
 III	CA-SN-11	SN 16d (3.9 x 89 mm), n=18 WT-T (3.8 x 89 mm), n=12	20	CUREE
	CA-SN-12	SN 16d (3.9 x 89 mm), n=18 SFS2 (5 x 90 mm), n=12	20	CUREE
	CA-SN-12A	SN 16d (3.9 x 89 mm), n=18 Between panels SFS2 (5 x 90 mm), n=12	20	ISO
 IV	CA-SN-20	Bracket A SN 16d, n=18 D=3.9 mm, L=89 mm 3 brackets on the back side	20	CUREE
 V	CA-SN-21	Bracket A SN 16d, n=6 D=3.9 mm L=89 mm	20	CUREE
 VI	CS-WT-22	WTT-T, n=18 D=6.5 mm L=130 mm	20	CUREE
	CS-WT-22B	WTT-T, n=34 D=6.5 mm L=130 mm	20	CUREE
 VII	CA-SN-23	Bracket A SN 16d, n=6 D=3.9 L=89 mm 3 brackets on the back side	20	CUREE

Walls in configuration I had four brackets spaced at 710 mm o.c. Walls 00 through 03 used type A brackets, which were connected to the wall using eighteen 16d spiral nails with D=3.9 mm and L=89 mm (Figure 4a). Wall 04 used type A brackets and twelve annular ring nails with D=3.4 mm and L=76 mm (Figure 4c). Wall 05 used type A bracket and eighteen SFS screws with D=4.0 mm and L=70 mm (Figure 4e), while wall 06 used ten SFS screws with D=5.0 mm and L=90 mm (Figure 4d). Walls 09 and 10A used type C brackets with two rows of five L=65 mm timber rivets (Figure 4g). In addition to three type A brackets spaced at 550 mm o.c. nailed with eighteen spiral nails (D=3.9 mm; L=89 mm), walls 07, 08 and 08A of configuration II had Simpson Strong Tie HTT-16 hold-downs at both ends. The hold-downs were nailed using eighteen 16d spiral nails for walls 07 and 08, while wall 08A used eighteen spiral nails with D=3.3 mm and L=63 mm (Figure 4b).

Walls from configuration III (11, 12 and 12A) consisted of two panels that were connected to the foundation in the same way as walls from configuration I. The two panels that formed the wall were connected together using a continuous 65 mm long step joint (lap joint) with no gap, and one vertical row of screws. Twelve SFS WTT-T type screws with D=3.8 mm and L=89 mm, spaced at 200 mm were used in wall 11 (Figure 4i) to connect panels, while panels in walls 12 and 12A were connected to each other using SFS screws with D=5.0 mm and L=90 mm (Figure 4d). These walls were designed to investigate the effect of gaps in the walls on the overall wall performance under lateral loads.

Table 2

Test matrix for 3.45 m long and 2.3 m high walls

Wall Configuration	Test Designation	Brackets and Fasteners	Vertical Load [kN/m]	Lateral Load
 VIII	CB-SN-13	Bracket B (9 brackets) SN 16d (3.9 x 89 mm), n=10	20	Monotonic
	CB-SN-14	Bracket B (9 brackets) SN 16d (3.9 x 89 mm), n=10	20	ISO
 IX	CB-SN-15	Bracket B (9 brackets) SN 16d (3.9 x 89 mm), n=10 SFS2 (5 x 80 mm), n=8	20	Monotonic
	CB-SN-16	Bracket B (9 brackets) SN 16d (3.9 x 89 mm), n=10 SFS2 (5 x 80 mm), n=8	20	ISO
 X	CB-SN-19	Bracket B SN 16d, n=10 D=3.9 L=89 mm	20	ISO

Only one CLT panel (wall 20) was tested from configuration IV. In addition to four type A brackets on the front side, this wall had three additional brackets on the back side, spaced right in the middle between the front brackets, for a total of seven brackets. This configuration was representative of an inside wall where both sides of the wall are available for connecting.

Table 3

Test matrix for two-storey assemblies and tall walls

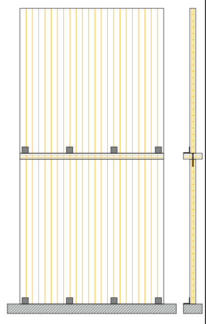
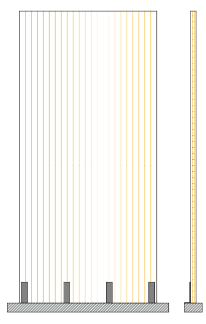
Wall Configuration	Test Designation	Brackets and Fasteners	Vertical Load [kN/m]	Lateral Load
 XI	CA-SN-28	Bracket A, SN 16d (3.9 x 89 mm), n=6 Slab-to-wall screws (8 x 200 mm) at 200	20	Monotonic
	CA-SN-29B	Bracket A, SN 16d, n=6 on both floors	20	ISO
	CA-SN-29C	Bracket A, SN 16d, n=8 at the bottom n=6 on the top storey	20	ISO
 XII	CD-TR-24	Bracket D, Rivets L=65mm, n=40	20	Monotonic
	CD-TR-25	Bracket D, Rivets L=65 mm, n=40	20	ISO
	CD-TR-26	Bracket D, Rivets L=90mm, n=40	20	ISO
	CD-TR-27	Bracket D, Rivets L=90 mm, n=20	20	ISO

Figure 4

Fasteners used in the testing program

To investigate the effect of the foundation stiffness in a real case scenario, walls in configurations V, VI and VII were placed over a 94 mm thick CLT slab with a width of 400 mm. Wall 21 used four type A brackets spaced at 710 mm o.c., while wall 23 used a total of seven brackets (four in front and three on the back) in the same arrangement as in wall 20. Each of the brackets had six 16d spiral nails. The brackets were connected to the CLT floor slab using three SFS WFC screws with D=10 mm and L=80 mm. Wall 22 used nine pairs of SFS WT-T 6.5 x 130 mm screws (Figure 4h) placed at an angle of 45 degrees to the slab and spaced at 280 mm. Wall 22B used seventeen pairs of the same screws with five pairs being closely grouped near each end of the wall (spaced at 40 mm) to simulate a hold-down effect. The rest of the screws were spaced at 320 mm.

Walls from configurations VIII, IX, and X were 3.45 m long and 2.3 m high. Walls 13 and 14 (configuration VIII) were single-panel walls that had a total of nine type B brackets, each with ten 16d spiral nails. Brackets had different spacing, varying from 320 mm to 460 mm. Walls 15 and 16 (configuration IX) were three-panel walls, with the same number and position of the brackets as the walls of configuration VIII. The panels were connected to each other using step joints and fasteners. Walls 15 and 16 used eight SFS screws of 5 x 90 mm spaced at 300 mm. Wall 19 of configuration X was the only wall in the entire research program with openings. The door was 1.9 m high and 0.8 m wide, with the door post being 500 mm wide, while the window was 1.15 m wide and 0.8 m high. The wall was connected using seven type B brackets, each using ten 16d spiral nails.

Configuration XI included three two-storey wall assemblies consisting of a lower and upper storey wall (2.3 m x 2.3 m) with a 94 mm CLT slab in between. Both walls were connected at the bottom using type A brackets, spaced at 710 mm o.c. Walls 28 and 29B used six 16d spiral nails, while wall 29C used eight such nails. The floor panel was connected to the bottom wall using SFS screws with D=8 mm and L=200 mm, spaced at 200 mm. Finally, configuration XII consisted of four single-panel tall walls (2.3 m x 4.9 m) that were connected to the steel foundation beam using four type D brackets spaced at 710 mm. Walls 24 and 25 had forty rivets in each bracket (L=65 mm), wall 26 had the same number of 90 mm long rivets (Figure 4f), while wall 27 had twenty L=90 mm rivets.

4

TEST SET-UP AND LOADING

A sketch of the test set-up with a specimen ready for testing is shown in Figure 5. Steel "I" beams with stiffeners provided a foundation to which the specimens were bolted. Another stiff steel beam that was bolted to the top of CLT walls was used as a spreader bar for the lateral load. Lateral guides with rollers were also used to ensure a steady and consistent unidirectional movement of the walls. Vertical load was applied using a single 13.3 kN hydraulic actuator located in the middle of the wall when testing 2.3 m long walls (Figure 5), or using two such actuators located at third points for 3.45 m long walls. Only wall 00 was tested without any vertical load. Walls 01 and 02 were tested with a 10 kN/m vertical load, which approximately corresponds to a wall being at the bottom of a two-storey structure. All other walls were tested using a 20 kN/m vertical load, which corresponds to a wall being at the bottom of a four-storey structure.

The walls were subjected to either monotonic or cyclic lateral loading using a 110 kN hydraulic actuator (Figure 5). Walls 01, 07, 09, 13 and 15 were tested under monotonic (ramp) loads with a displacement rate of 0.2 mm/s, while walls 24 and 28 were tested with a rate of 0.4 mm/s. All other walls, as shown in Tables 1, 2 and 3, were tested either using CUREE (Method C) or ISO 16670 cyclic testing protocols (Method B), as specified in ASTM E 2126 (ASTM, 2009), with a rate of 5 mm/s. Instrumentation included displacement at the top and bottom of the wall, uplift at both ends, as well as deformation of the wall along the wall diagonals.

Figure 5
Sketch of the test set-up used for CLT walls

5 RESULTS AND DISCUSSION

As expected, the CLT wall panels behaved almost as rigid bodies during the testing. Although slight shear deformations in the panels were measured, most of the panel deflections occurred as a result of the deformation in the joints connecting the walls to the foundation. In case of multi-panel walls, deformations in the step joints also had significant contribution to the overall wall deflection. Selected average properties of the CLT walls, based on the envelope curves of both sides of the hysteretic loops obtained from the experimental program, are given in Table 4. Analysis of the test data was conducted using the procedure specified in ASTM Standard E 2126 (ASTM, 2009). After determining the envelope curves for the cyclic tests, the Equivalent Energy Elastic Plastic (EEEP) curves were defined and main properties based on these curves were determined. In Table 4, K_y is the initial stiffness, Δ_y the yield displacement, F_{max} the maximum load, Δ_{Fmax} the displacement at maximum load, and Δ_u the ultimate displacement. It should be noted that most findings presented here are based on a single wall test for any different wall arrangement.

The axial load value had some impact on the lateral resistance of the walls, although not as significant as expected. Wall 00 with no vertical load had a maximum lateral resistance of 88.9 kN, while wall 02 with a 10 kN/m vertical load had a lateral resistance of 90.3 kN. When the vertical load was increased to 20 kN/m (wall 03), the lateral resistance increased to 98.1 kN, an increase of 10% (Figures 6 and 7). It seems that the axial load had to be at least 20 kN/m or higher to have any significant influence on the lateral load resistance. The amount of vertical load, however, had a higher influence on the wall stiffness. The stiffness of wall 03 was 28% higher than that of wall 00. In addition, higher values of vertical load influenced the shape of the hysteresis loop near the origin (Figures 6 and 7). It should be noted, however, that on a system (building) level, the vertical load had relatively significant influence on the seismic performance of CLT buildings, especially at higher deformation levels, when CLT panels basically turn into rocking structural elements.

Table 4

Selected average wall properties obtained from the experimental program

Wall	K_y [kN/mm/m]	Δ_y [mm]	F_{max} [kN]	Δ_{Fmax} [mm]	Δ_u [mm]	Δ_u [% drift]	Ductility [Δ_u / Δ_y]	Energy Dissipation [kJ]^{II}
00	4.18	20.6	88.9	44.9	66.6	2.9	3.5	29.30
01*	5.13	18.2	108.6	47.7	54.9	2.4	3.0	-
02	4.42	18.3	90.3	41.7	71.5	3.1	3.9	33.75
03	5.36	17.0	98.1	44.1	63.6	2.8	3.9	35.98
04	5.52	16.9	102.3	39.2	59.6	2.6	3.6	27.97
05	5.26	17.6	102.7	35.3	53.7	2.3	3.1	31.97
06	5.01	17.4	100.1	42.2	50.1	2.2	2.9	30.26
09*	12.67	6.6	93.5	29.2	47.8	2.1	7.3	-
10A	11.78	7.8	102.4	33.1	49.0	2.1	6.3	34.62
07*	5.10	21.3	121.2	52.7	88.0	3.8	4.1	-
08**	8.89	11.7	118.2	53.1	63.3	2.8	5.4	39.75
08A	9.68	11.4	107.1	41.7	57.6	2.5	5.8	36.05
11	3.62	19.2	79.5	54.1	60.3	2.6	3.2	25.35
12	4.19	19.7	92.5	52.8	72.0	3.1	3.7	34.36
12A	4.24	19.4	90.6	41.0	56.6	2.5	3.0	44.04
20	7.20	20.1	152.1	40.9	70.5	3.1	3.6	57.74
21	5.41	8.7	54.1	43.5	84.9	3.7	9.8	19.87
22	8.97	4.9	41.2	12.1	31.6	1.4	7.5	8.19
22B	8.80	5.1	50.6	11.7	24.9	1.1	5.0	11.83
23	4.97	13.5	72.2	46.6	79.8	3.5	6.1	25.77
13*	17.85	10.1	201.2	39.9	70.4	3.1	6.9	-
14	19.23	9.5	190.9	35.8	67.7	2.9	7.4	91.89
15*	8.82	16.2	156.3	86.9	89.4	3.9	5.5	-
16	6.39	18.3	130.2	77.8	107.1	4.7	5.9	66.68
19‡	8.99	14.1	141.0	33.5	64.6	2.8	4.5	61.93
28*	0.94	27.8	29.1	89.3	176.1	3.8	6.3	-
29B‡	1.11	24.2	29.4	73.9	117.3	2.5	4.9	6.53
29C	1.22	23.3	31.7	74.5	142.2	3.0	6.1	8.16
24*	1.94	47.3	101.2	68.8	70.5	1.5	1.5	-
25	2.19	30.6	75.0	44.5	78.8	1.7	2.8	41.97
26	2.29	36.2	92.9	59.6	75.0	1.6	2.0	35.52
27	2.01	43.1	92.4	100.2	113.7	2.4	2.7	49.00

* Value from a single monotonic test; ** Hold-down fatigue failure observed; ‡ One of the values in the loop for F_u was at 90% of F_{max}; ^{II}Energy dissipated until the end of the test.

The maximum loads obtained from the monotonic tests were greater than the corresponding values obtained from the cyclic tests for each of the two cyclic protocols, while the ultimate deformations and loads at these deformation levels were underestimated. An example of this is given in Figure 8. It was also observed that, during the static tests, more deformation demand was induced on the brackets themselves than on the fasteners used to connect them. It is therefore suggested that cyclic tests be used for determining the properties of CLT wall panels under seismic loads.

Wall 04, with twelve 10d annular ring nails per bracket, exhibited slightly higher resistance than wall 03 with eighteen 16d spiral nails per bracket. This was mainly due to the higher withdrawal resistance of the ring nails. The ductility of wall 04, however, was slightly lower than that of wall 03 (Figure 9). The failure mode observed at the brackets of wall 04 was also slightly different than that of wall 03. While spiral nails in the brackets exhibited mostly bearing failure combined with nail deformation and withdrawal, ring nails in withdrawal had a tendency to pull out small chunks of wood along the way, as shown in Figure 10.

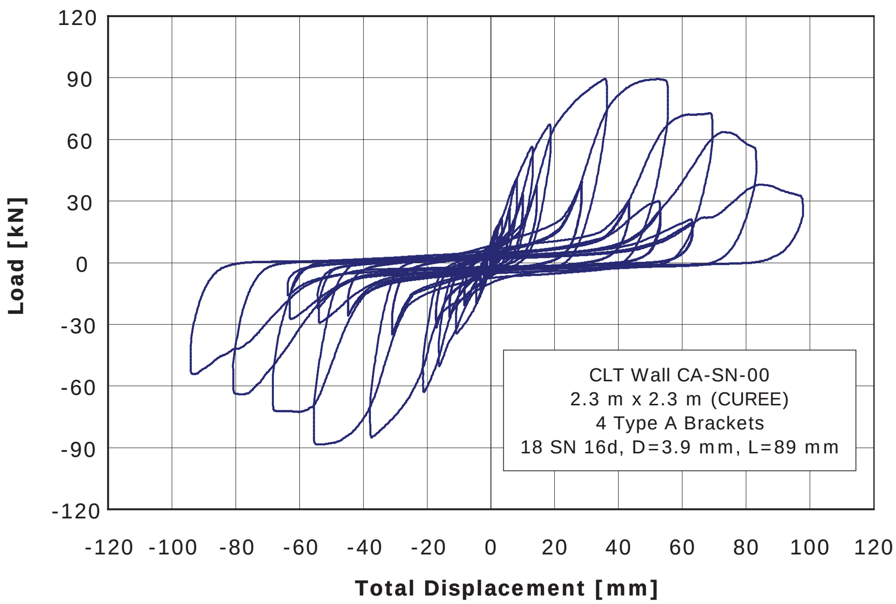


Figure 6
Hysteretic behaviour for wall 00 with no vertical load

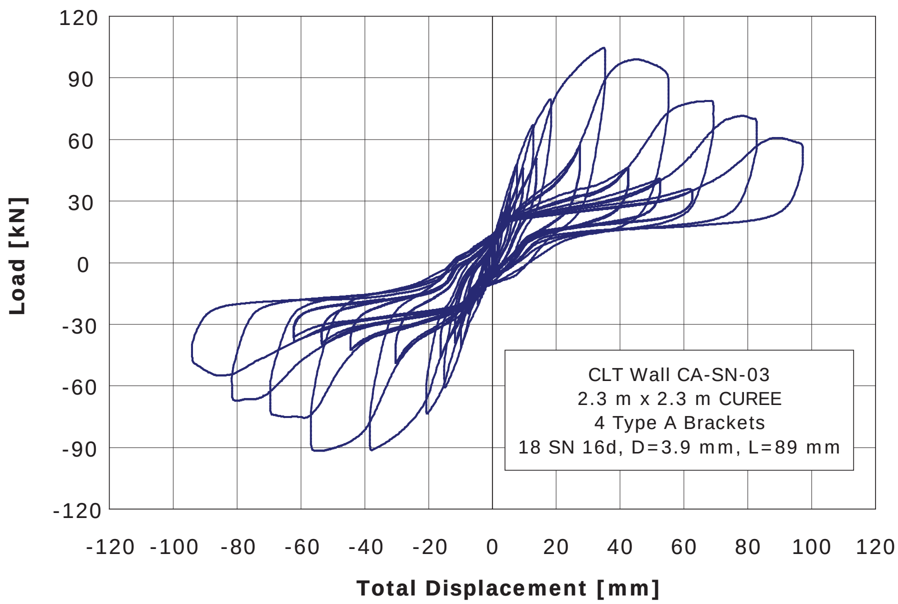


Figure 7
Hysteretic behaviour for wall 03 with 20 kN/m vertical load

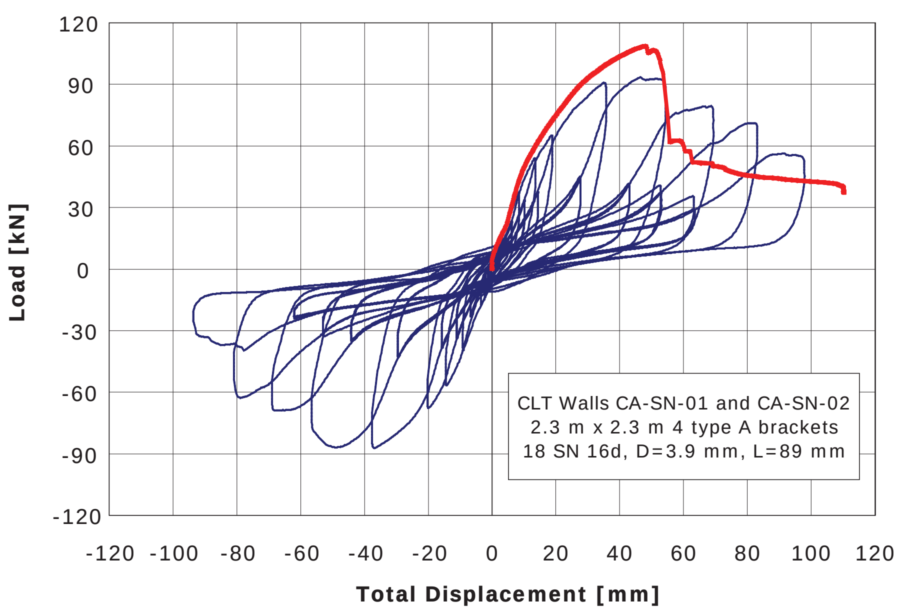


Figure 8
Results from monotonic and cyclic tests on CLT walls with the same configuration

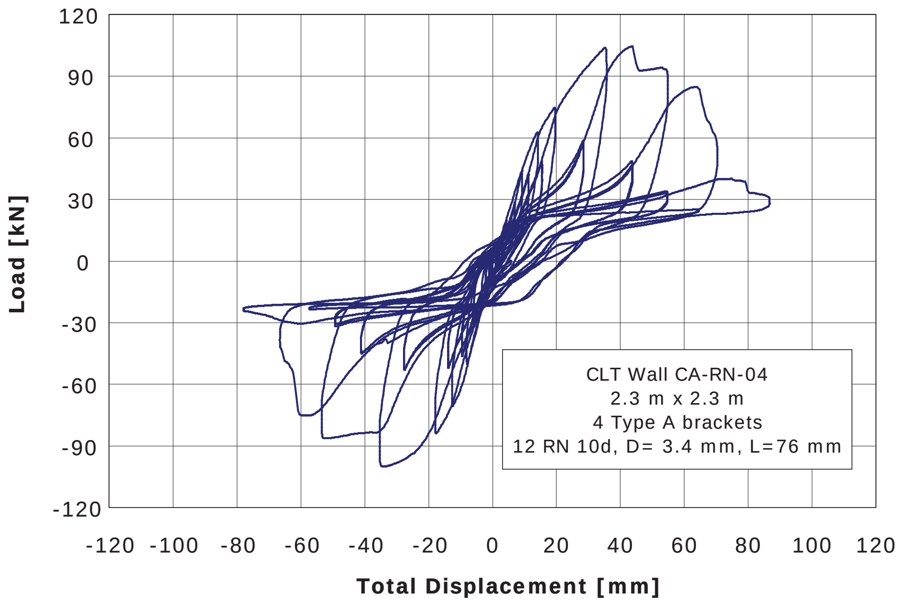


Figure 9
Hysteretic behaviour for wall 04 with ring nails

a) b)

Figure 10
Failure modes of the bracket connections at late stages of testing for: a) wall 02 with spiral nails; and b) wall 04 with annular ring nails

The walls with screws (05 and 06) reached similar maximum loads as the walls with nails. The load carrying capacity for CLT walls with screws (Figures 11 and 12), however, dropped a little bit faster at higher deformation levels than in the case of walls with nails.

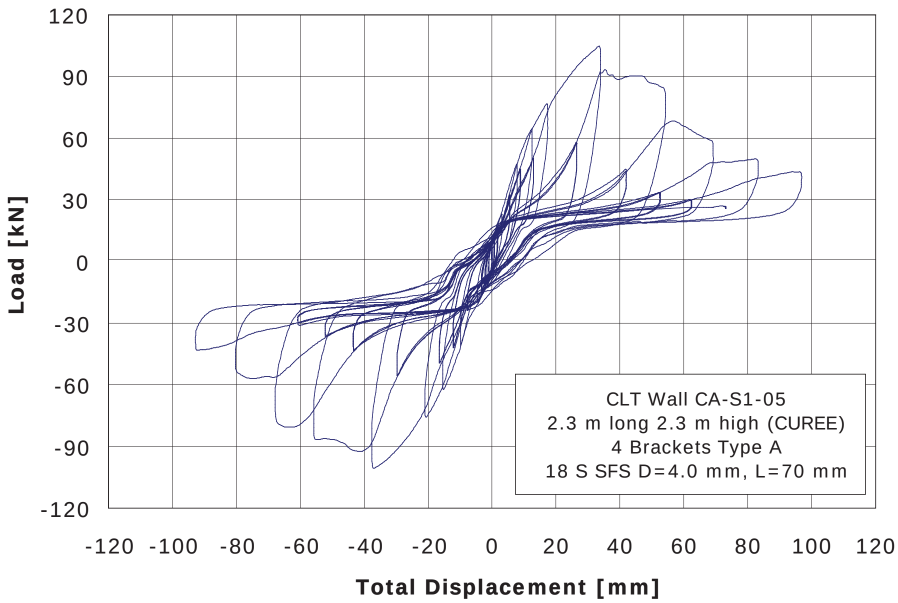


Figure 11
Hysteretic behaviour for wall 05 using 18 screws with D=4.0 mm and L=70 mm

CLT wall panel with hold-downs (wall 08A) showed one of the highest stiffness for a wall with a length of 2.3 m, its stiffness being 81% higher than wall 03 with 18 spiral nails per bracket. This CLT wall also showed relatively high ductility capacity (Figure 13). The behaviour of one corner of wall 08A during testing is shown in Figure 14.

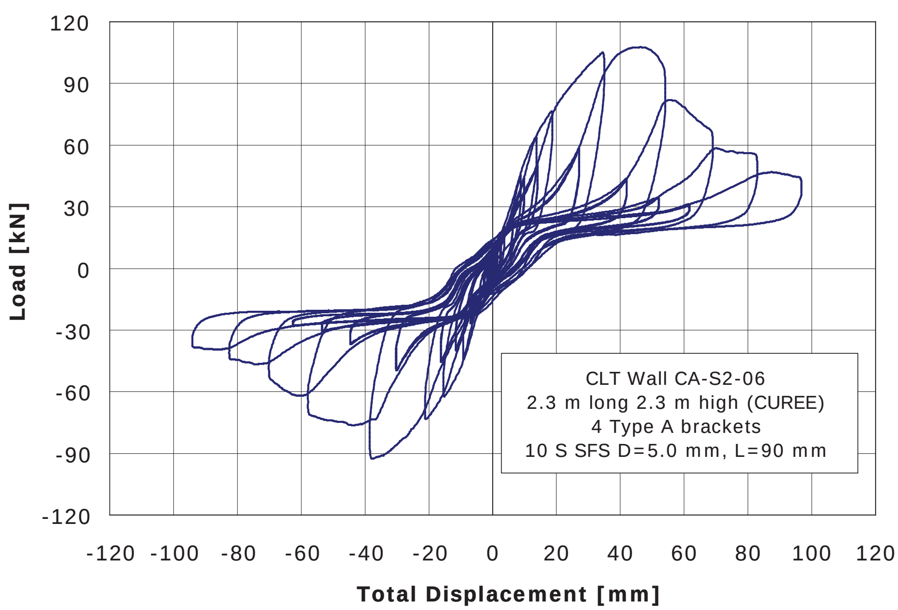


Figure 12
Hysteretic behaviour for wall 06 using 10 screws with D=5.0 mm and L=90 mm

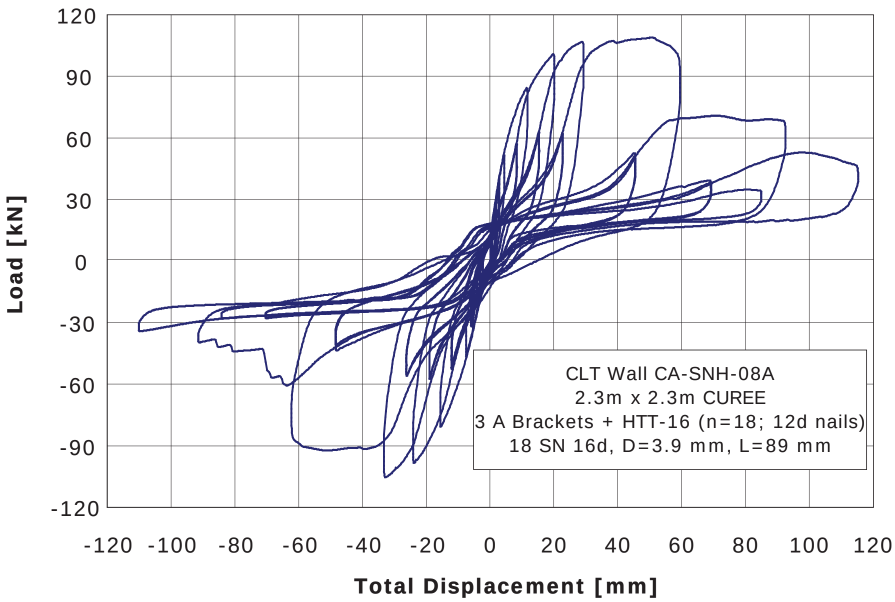


Figure 13
Hysteretic behaviour for wall 08A using brackets and hold-downs

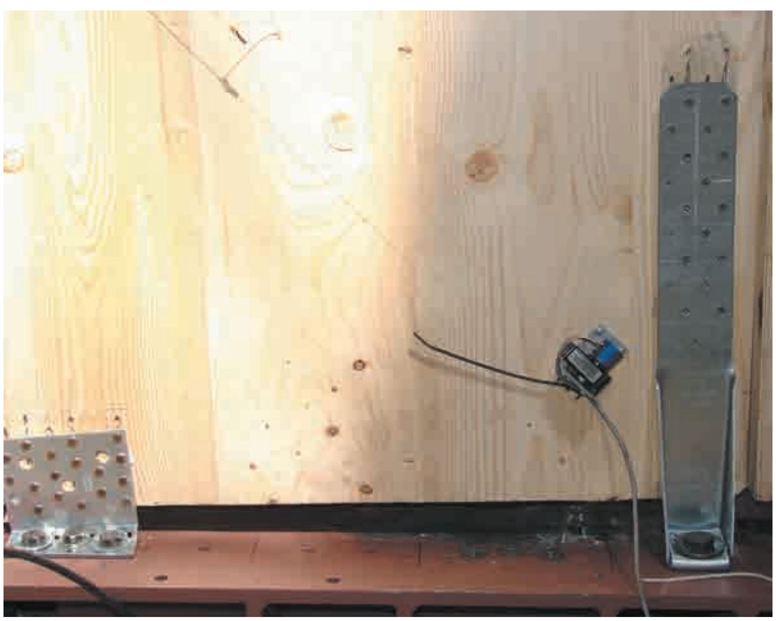


Figure 14
Behaviour of one corner of wall 08A during testing

Although timber rivets were developed to be used with glulam, they have recently been used in many other engineered wood products that have strands or veneers aligned in one direction. During this research program, an attempt was made to use rivets in CLT for the first time, despite the fact that when driven with their flat side along the grains in the outer layers, the rivets will be oriented across the grain in the middle layer. CLT wall 10A with ten rivets per bracket exhibited a higher stiffness than any wall tested in configuration I and was close second for a wall with a length of 2.3 m, with its stiffness being 220% higher than wall 03 with 18 spiral nails per bracket. Rivets were also able to carry more load per single fastener than any other fasteners used in the program. In addition, the wall was able to attain a relatively high ductility level. The hysteresis loop for wall 10A with timber rivets is shown in Figure 15.

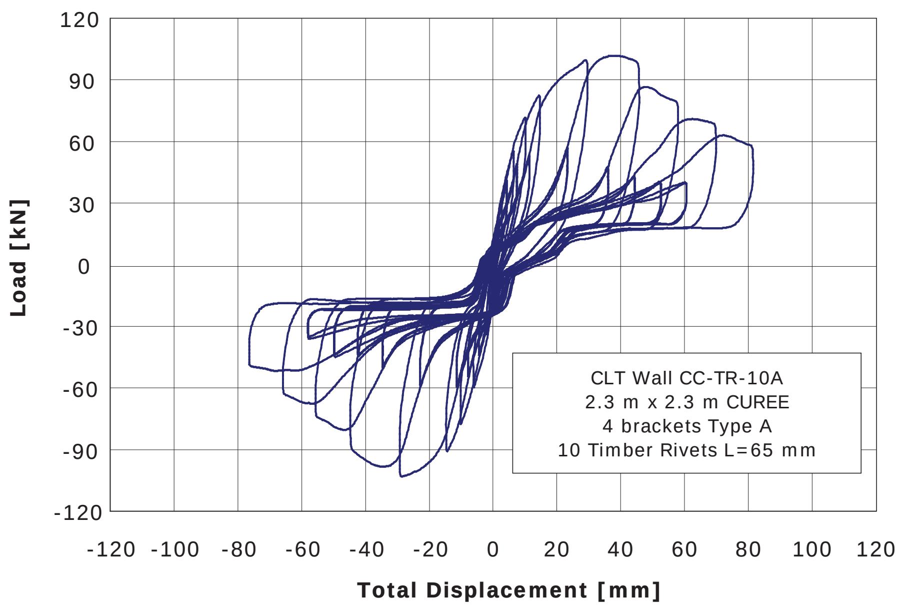


Figure 15

Hysteretic behaviour for wall 10A using timber rivets

By introducing a step joint in the wall, i.e. creating a wall made of two separate panels, the behaviour of the wall was not only influenced by the types of fasteners in the bottom brackets, but also by the type of fasteners used in the step joint. These walls (11 and 12) showed stiffness reduced by 32% and 22% respectively, and a slightly reduced strength, with respect to the reference wall 03. Both walls shifted the occurrence of the yield load F_y and ultimate load F_u to higher deflection levels, while only wall 12 was able to show an increase in its ultimate deflection.

Figure 16
Behaviour of wall 12 using two panels during testing

Wall 11 with WT-T screws in the step joint showed ultimate load reduced by 19%, while wall 12 with regular 5 x 90 mm screws showed a reduction of only 5%. In addition, wall 11 showed higher reduction of ductility compared to the reference wall 03, while the ductility for wall 12 was only slightly lower than that of the reference wall. Based on the results, in the case of multi-panel walls with step joints, the use of regular screws is recommended in high seismic zones. A photo of wall 12 during the testing is shown in Figure 16, while the behaviour of walls 11 and 12 are shown in Figures 17 and 18, respectively.

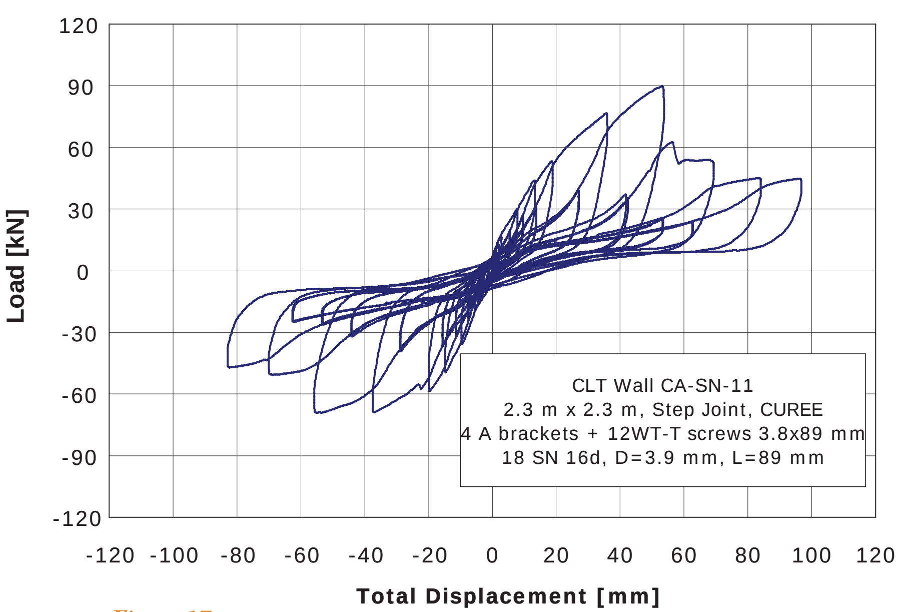


Figure 17
Hysteretic behaviour for wall 11 with 3.8 x 89 mm WT-T screws used in the step joints

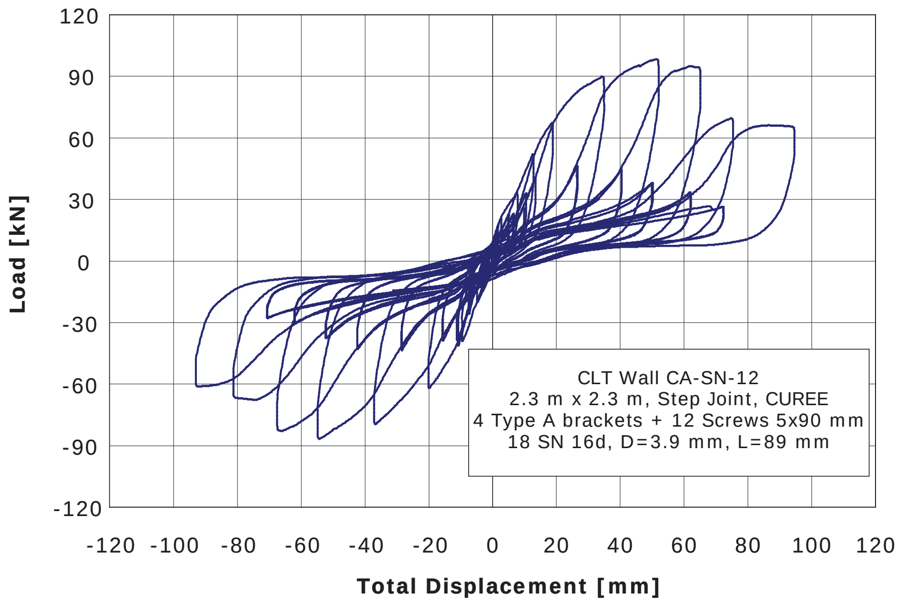


Figure 18
Hysteretic behaviour for the two-panel wall 12 with regular 5 x 90 mm screws used in the step joints under CUREE cycling protocol

The presence of the step joints and the type of fasteners used to connect them was found to have more significance on the overall wall behaviour as the length of the wall increased. For example, results from walls 14 and 16, which had lengths of 3.45 m, showed a significant change in stiffness and strength for wall 16 with step joints (Figure 20) compared to wall 14, which had no step joints (Figure 19). The step joints enabled wall 16 to carry a significant portion of the maximum load at higher deformation levels, but at a considerable (25%) reduction in maximum strength.

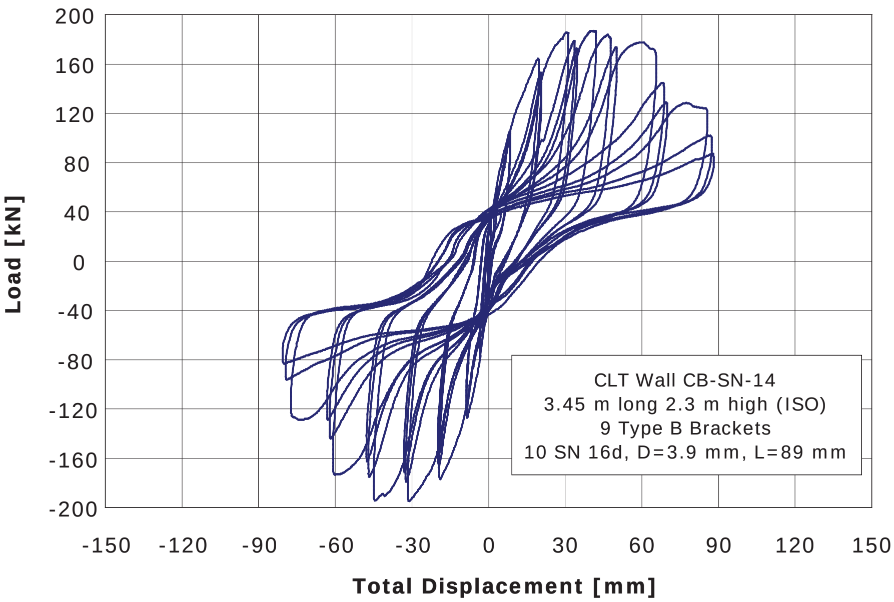


Figure 19
Hysteretic behaviour for wall 14 consisting of one 3.45 m long panel

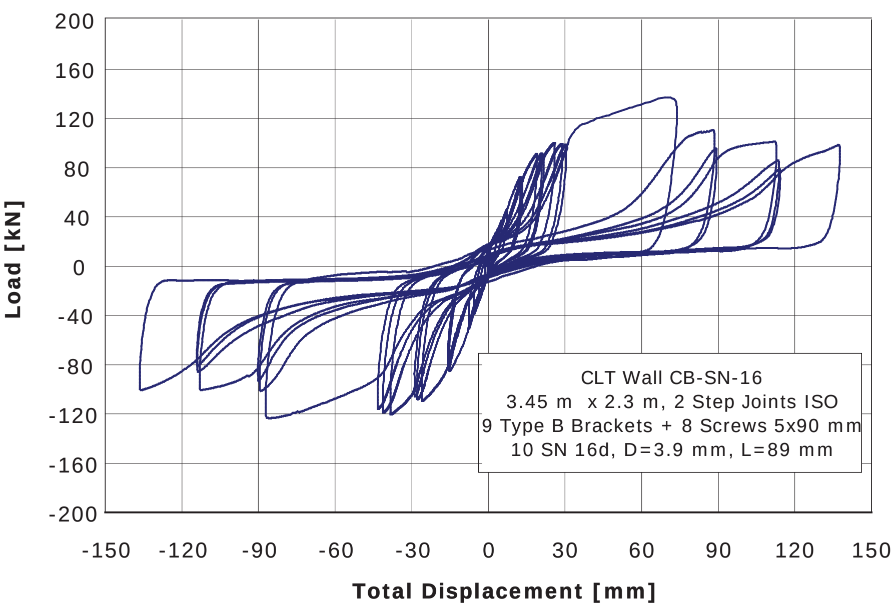


Figure 20
Hysteretic behaviour for the three-panel wall 16 where panels were connected with regular 5 x 90 mm screws

It is a well known fact that the protocol used for cyclic testing of wood-based connections or structural assemblies has an influence on the test results. By comparing results for walls 12 and 12A (Figures 18 and 21), it can be seen that the choice of the protocol had very little influence on the stiffness of the wall, the yield deflection (both determined using the EEEP method), and the maximum load (Table 4). However, there was significant difference in the deflection at which the maximum load was reached (41 mm with ISO vs. 53 mm with CUREE), and in the ultimate deflection, which was 72 mm using the CUREE protocol vs. 57 mm using the ISO protocol. These findings stress the importance of having specimens tested under both protocols so that a comparative assessment of the wall performances can be made.

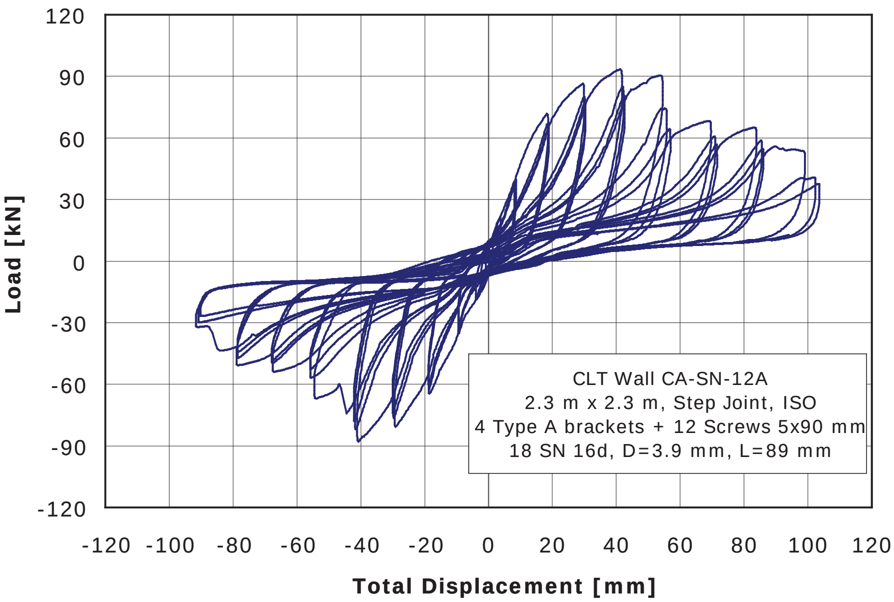


Figure 21
Hysteretic behaviour for the two-panel wall 12A tested under ISO cycling protocol

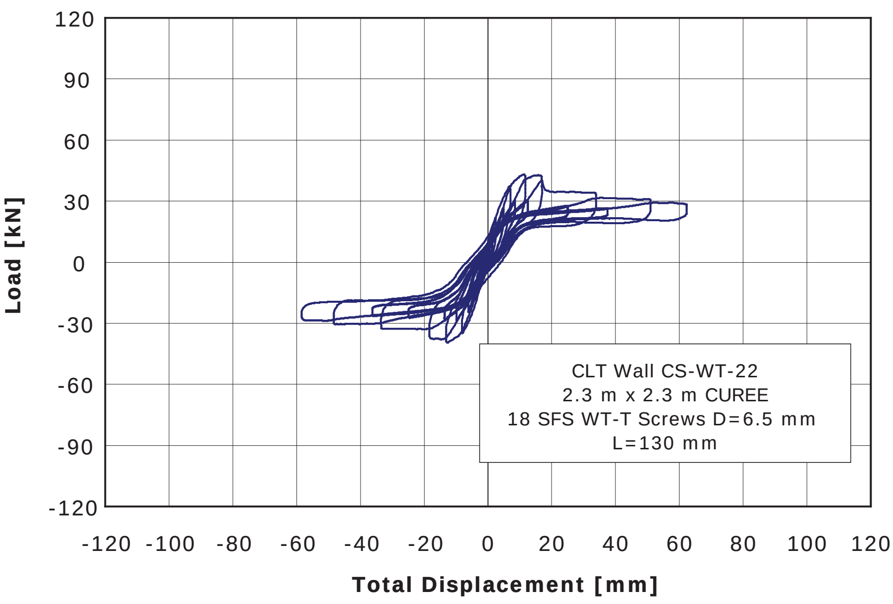


Figure 22
Hysteretic behaviour for wall 22 with WT-T screws

Specimens 22 (Figure 22) and 22B that were connected to the base CLT panel with WT-T type screws placed at 45° showed lower resistance than any single-storey wall in the program. Grouping the screws at the ends of the panels (wall 22B) created a hold-down effect and helped increase the wall capacity by about 30% compared to that of wall 22. Based on the test results, use of screws at an angle as a primary connector for wall-to-floor connections is not recommended for structures in seismic regions due to reduced capability for energy dissipation (Figure 22) and the sudden pull-out failure of screws in tension.

The behaviour of the tall walls' specimens with riveted connections was highly influenced by the number of rivets used in each bracket. Although the number and spacing of rivets in the brackets for walls 24, 25 and 26 were chosen to satisfy the rivet yielding failure mode according to existing Canadian code specifications for sawn lumber and glulam, they did not yield but experienced fastener pull-out combined with a wood shear plug failure mode (Figure 23a). By increasing the spacing between the rivets in wall 27, the failure mode was changed to the desired rivet yielding mode (Figure 23b).

a)

b)

Figure 23
Bracket failure modes for a) wall 25 with 40 L=65 mm rivets; and b) wall 27 with 20 L=90 mm rivets

Results from tests on two-storey assemblies (walls 28, 29B and 29C) showed that most of the deformation was concentrated at the connections at the bottom of the first storey wall. For example, for the East side of wall 29B, the maximum uplift between the floor slab and the upper storey was 4.2 mm, while it was around 60 mm at the bottom. No significant crushing of the slab or sliding at the top floor was observed. Figure 24 shows the deflection of wall 29C at the top of both storeys at various stages of the testing; the nearly linear lines indicating that the deformation came mostly from rotation at the base. Also, the shape of the lines is very similar, indicating that the deformation in the inelastic range came from rotation at the base and not shear deformation or rotation at the intermediate floor level.

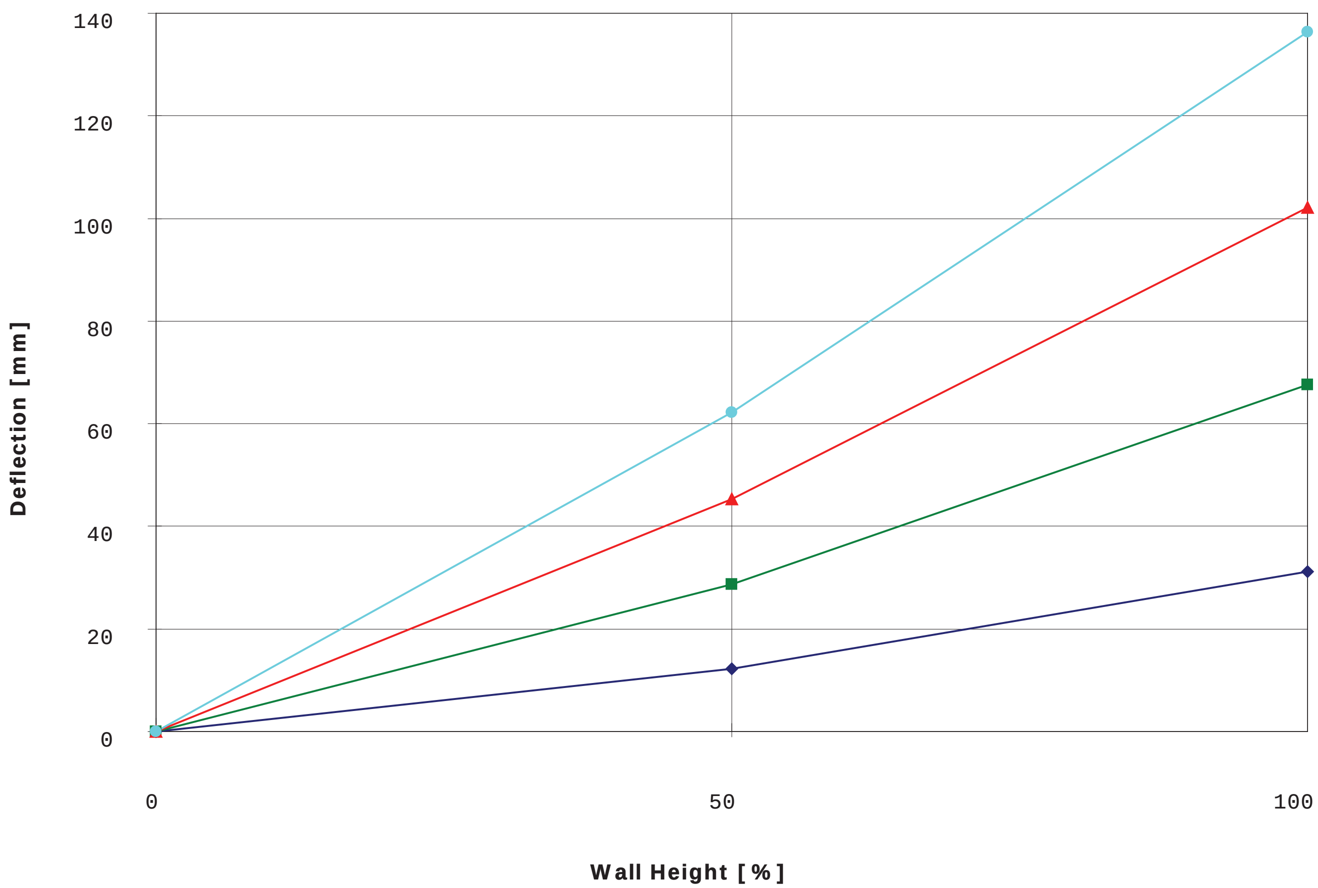


Figure 24
Deflection of wall 29C at the top of both storeys at various stages of testing

6 SEISMIC FORCE MODIFICATION FACTORS (R-FACTORS) FOR CLT STRUCTURES

The force modification factors (R-factors) in building codes in North America account for the capability of the structure to undergo ductile nonlinear response, which dissipates energy and increases the building period. This allows the structure to be designed for seismic forces smaller than the forces that would be generated if the structure remained elastic, without increasing the displacements from the seismic loads. Different R-factors are assigned to different types of structural systems reflecting their seismic performance during past earthquakes, and the ability to undergo nonlinear response with limited loss of strength as the structure goes through several cycles of motion.

In the 2005 edition of the National Building Code of Canada (NBCC, 2005), the elastic seismic load is reduced by two types of R-factors, an R_o-factor that is related to the over-strength of the system and an R_d-factor that is related to the ductility of the structure. In the major model codes used in the United States, that is the International Building Code (IBC, 2006) and the ASCE7 (ASCE7-05), there is only one R-factor, called the response modification coefficient, which reduces the seismic design force. Eurocode 8, which is the European model seismic code, also uses only one factor, the *q*-factor, for reduction of the seismic design force. Although every model code should be considered as a separate calibrated entity as the loading code is different in the different regions, it is useful to compare the product of R_dR_o in Canada to the R-factor in the US, and to the *q*-factor in Europe for the same seismic hazard probability.

6.1 Methods for Determining the R-factors

Often, there is little theoretical or experimental background given in the codes for determining the numerical values of the R-factors. Consequently, the process of assignment of R-factors requires considerable individual judgment. Most of the current values for the R-factors in the building codes are based on past seismic performances of the structural system and some results from non-linear time history dynamic analyses, if available.

6.1.1 European Approaches

In Europe, a common method to verify the current *q*-factors for various wood-based structural systems uses results from incremental non-linear dynamic analyses. The procedure can be summarized as follows. A building is designed according to the model code, where the structural system to be evaluated is the main Seismic Force Resisting System (SFRS) of the structure. The SFRS of the building is designed using a *q*-value equal to unity. A non-linear analytical model of the building is developed using a suitable structural analysis computer program, where the properties of the SFRS components in the model are obtained from testing. The “near collapse” condition for the model is defined, which is usually related to an ultimate deformation of the main members in the SFRS. The analytical model is then subjected to a series of earthquake records with a gradual increase of the Peak Ground Acceleration (PGA). For every earthquake record, the PGA of the record which produces the yield condition is then determined (in two different ways as explained below), as is the PGA at which the structure reaches the "near collapse" condition. The *q*-factor for any earthquake can then be determined using the acceleration-based approach or the base shear approach. Both approaches are described below.

- In the acceleration-based approach, the *q*-factor is calculated as a ratio of the acceleration that caused the "near collapse" condition (PGA_u) and the design acceleration in the model code (PGA_{code}) for the location for which the building was designed.
- In the base shear approach, the *q*-factor is calculated as the ratio of the base shear force obtained from an elastic analysis to the base shear force at the "near collapse" state of the structure for every input ground motion. This method takes into consideration the influence of the input ground motion on the elastic response of the structure.

In both methods, depending on the number of earthquake records used, the *q*-factor can be determined as a probabilistic relation between the numbers of records that cause near collapse to those that do not. More details on these approaches are given in Section 6.2.3.

6.1.2 Equivalency Approach

This approach is mostly used in the USA and is based on several publications by the International Code Council Evaluation Service (ICC-ES). According to the ICC-ES acceptance criteria document AC130 (ICC-ES, AC 130, 2009), assignment of an R-factor for new prefabricated wood shear-resisting wall assemblies can be made by showing equivalency in their seismic performance criteria (maximum load, ductility, storey drift, etc.) obtained from quasi-static cycle tests, compared to the same properties already observed from tests on lumber-based nailed shear walls. This document applies to prefabricated wood shear wall assemblies in which a wood-based sheathing or structural composite lumber (SCL) material is the primary mechanism resisting in-plane shear loads, and wood-based material studs are designed as the gravity load resisting elements. In a similar way, the AC 322 document (ICC-ES, AC 322, 2009) specifies acceptance criteria for prefabricated cold-formed, steel lateral force resisting assemblies.

6.1.3 The FEMA P695 Procedure

As design codes around the world have improved over the last several decades in how they address the seismic design, one of the results was an expansion of code-approved seismic force resisting systems. This was especially evident in the USA where, in the most recent IBC, there are more than 80 individual structural systems. Each of these systems has individual system response coefficients (R-factors, the Ω_o factor and the C_d coefficient) somewhat arbitrarily assigned in IBC, based largely on judgment and qualitative comparison with the known response capabilities of other systems. Many of these recently defined structural systems have never been subjected to significant levels of earthquake ground shaking and the potential response characteristics and ability to meet the design performance objectives are untested and unknown.

For these reasons, the Federal Emergency Management Agency (FEMA) and the Applied Technology Council (ATC) in the USA, under project ATC-63, have developed the FEMA P695 document (FEMA, 2009). For the first time, this document contains a procedural methodology where the inelastic response characteristics and performance of typical structural systems could be quantified, and the adequacy of the structural system provisions to meet the design performance objectives could be verified. The methodology directly accounts for the potential variations in structural configuration of buildings, the variations in ground motion to which these structures may be subjected, and the available laboratory data on the behavioral characteristics of structural elements. The developed procedure established for the first time consistent and rational evaluation of building system performance and the response parameters (R, C_d, Ω_0) used in current building codes in the USA. The primary application of the procedure is for the seismic evaluation of new structural systems, so they have equivalent earthquake performance for the maximum considered earthquake. It can be anticipated that this methodology will ultimately be used by the model building codes and standards to set minimum acceptable design criteria for standard code-approved systems, and to provide guidance in the selection of appropriate design criteria for new systems.

The drawbacks of the FEMA P695 procedure are that it is quite complex, very time consuming and therefore very expensive. A large number of non-linear dynamic analyses are required on a number of different building models with different configurations. In addition, the types of analyses required are sophisticated, and may be out of reach for average design engineers, especially in the area of timber design. FEMA is also working on a new procedure (FEMA P-795) entitled "Quantification of Building System Performance and Response Parameters - Component Equivalency Methodology" that is much easier to use and can be applied to introduce CLT as a substituting element in a structural system that is already implemented in the code.

6.2 Estimate of Values for R-factors in CLT Structures

In this section, an estimate will be made of the values of the force modification factors R_d and R_o for CLT structures in NBCC. The estimate will be based on results from experimental testing and analytical investigation of the seismic performance of CLT wall panels and structures in Europe mentioned in Section 2 of this chapter, along with the in-house experimental results presented. For more precise determination of the R-factors for CLT structures, either a series of incremental dynamic analyses or the complex approach in the FEMA P695 guidelines should be used.

CLT wall panels behave almost as rigid bodies when subjected to lateral seismic loads. Although there was some slight shear deformation measured in the panels during the in-house testing, most of the panel deflection occurred as a result of the deformation in the connections (brackets) connecting the walls to the foundation. Where nails were used to connect the CLT wall panels to the steel brackets, a ductile failure mode of the nailed connections was observed in all cases. Therefore, the ductile behaviour of the nailed connections completely influenced the behaviour of the entire wall, and will thus have a large influence on the behaviour of a CLT structure.

6.2.1 Behaviour Comparison with Structural Systems Already in NBCC

Although braced timber frames and portal moment-resisting frames in heavy timber are completely different structural systems compared to the CLT construction, they have some common points, as the performance of all these systems is mainly influenced by the behaviour of the connections. In braced frames, it is the connection between the brace and the rest of the frame, while in portal moment frames it is the connection between the column and the beam that governs the structural behaviour. These two structural systems have already been assigned an R_d-factor of 2.0 in NBCC, when designed with moderately ductile connections. Based on the research results from tests on braced timber frames and portal moment frames (Popovski et al., 2002, 2004 and 2008), performance of CLT panels with ductile connections (such as nails or slender screws) is equivalent if not better than that of these two systems.

In addition, although it is a platform type of structural system, CLT construction is far less susceptible to develop soft storey mechanisms than many other structural systems of the same type. Since the nonlinear behaviour (and the potential damage) is localized in the bracket connection areas only, the panels that are also the vertical load carrying elements are virtually left intact in place, and well connected to the floor panels, even after a "near collapse" state is reached. Also, in CLT construction, all walls in one storey contribute to the lateral and gravity resistance, thus providing a degree of redundancy.

The R_o-factor for structures with CLT wall panels in NBCC can be determined according to Mitchel et al. (2003). Based on the type of connections used, an R_o value of 1.5, which is currently used for most connections in heavy timber construction designed according to the CSA O86, is considered to be a reasonable conservative estimate.

6.2.2 Equivalency Approach (AC130)

According to the ICC-ES acceptance criteria document AC130 (ICC-ES, AC130, 2009), assigning of an R-factor for a new wood shear wall assembly in the USA that can be used in conjunction with wood-frame shear walls can be made by showing equivalency of the seismic performance of the new wall assembly in terms of maximum load, ductility, and storey drift obtained from quasi-static cyclic tests, with respect to the properties of lumber-based nailed shear walls, that are already implemented in the code. The equivalency criteria apply to prefabricated wall assemblies consisting of wood-based framing (dimension lumber or structural composite lumber (SCL) material) and a wood-based sheathing nailed to the framing. In a similar way, the AC322 document (ICC-ES, AC322, 2009) specifies acceptance criteria for prefabricated cold-formed, steel lateral force resisting assemblies.

Although CLT wall panels as a system differ from wood-frame shear walls, an effort will be made here to use the equivalency criteria given in AC130 in assessing the seismic behaviour of CLT panels, since the criteria are performance-based. For example, the AC130 criteria specify that, for a new shear wall assembly to have the same seismic design response coefficients ($R=6.5$, $C_d=4.0$, $\Omega_0=3.0$) used in IBC for regular shear walls, the assembly shall have the response characteristics listed below.

1. The lower bound on the ratio of the displacement at the post-peak load to the displacement at the assigned Allowable Stress Design (ASD) load level shall be equal or greater than 11 as shown in equation [1]:

$$\frac{\Delta_u}{\Delta_{ASD}} \geq 11 \qquad [1]$$

where:

Δ_{ASD} = the displacement at the ASD load level developed according to IBC or UBC;
Δ_U = the ultimate displacement taken from the backbone curve corresponding to an absolute load having no more than 20 percent strength degradation of the post-peak load data point as given in ASTM E 2126.

2. The minimum post-peak displacement shall be in accordance with the requirements of equation [2]:

$$\Delta_u \geq 0.028 \cdot H \qquad [2]$$

where H is the height of the panel element.

3. The ratio of the maximum load P_{max} obtained from the backbone curve of the panel to the assigned ASD load P_{ASD} shall be in accordance with the requirements of equation [3]:

$$2.5 \leq \frac{P_{\max}}{P_{ASD}} \leq 5.0 \qquad [3]$$

We can use the requirements shown in equations [1] to [3] to assess the performance of CLT panels according the AC130 criteria. In order to do that, we have to make the assumption that the future design values (lateral resistances) for CLT panels will have the same "safety margin" as those of regular wood-frame shear walls according to CSA O86. In such a case, we can assume that the design values for lateral loads for CLT panels can be derived in the same way as if they were determined for wood-frame shear walls. The specified strengths for shear walls in Canada were soft converted from the Allowable Stress Design (ASD) values of the Uniform Building Code (UBC) in the USA. The ASD values in UBC were derived using the average maximum load obtained from monotonic pushover tests divided by a safety factor of 2.8, or the average maximum load from cyclic tests divided by a safety factor of 2.5. In this case, since the cyclic test results were used, we will use the second safety factor. In addition, to be compatible with AC130 criteria, only single-storey walls tested under the CUREE protocol will be used for the analyses. Also, walls that used WT-T screws will be excluded as they showed undesirable failure modes. Finally, the influence of the vertical load is assumed not to have significant effect on wall performance, as AC130 criteria do not require the presence of a vertical load during the testing of the elements.

Main response parameters related to the AC130 criteria obtained from the envelopes of single-storey cyclic tests on CLT wall panels are shown in Tables 5, 6 and 7. Values in Table 5 are derived from the envelope curve of the hysteresis curve in the first quadrant (initial loading quadrant), while the values in Table 6 are derived from the envelope curve of the hysteresis curve in the third quadrant. The average values for all parameters based on both envelope curves are given in Table 7. In all tables, P_{max} is the maximum load, P_{ASD} is the load that would be an equivalent to ASD design level (determined as $P_{max}/2.5$), Δ_{ASD} is the displacement at P_{ASD}, and Δ_u is the ultimate displacement (displacement at which the load has dropped to 80% of the maximum).

Table 5

AC130 related properties of selected CLT walls based on the initial load cycle envelope

Wall	P_{ASD} [kN]	Δ_{ASD} [mm]	P_{max} [kN]	Δ_u [mm]	Δ_u [% drift]	Ductility Δ_u/Δ_{ASD}
00	35.8	5.6	89.4	69.1	3.0	12.3
02	37.4	8.6	93.4	76.2	3.3	8.9
03	41.8	6.1	104.5	62.9	2.7	10.3
04	41.8	8.6	104.6	63.9	2.8	7.4
05	42.4	6.2	106.1	43.6	1.9	7.0
06	43.0	8.2	107.6	53.9	2.3	6.6
08A	43.5	3.2	108.8	61.5	2.7	19.2
10A	41.2	3.8	103.1	45.3	2.0	11.9
12	39.3	9.0	98.2	70.2	3.1	7.8
14	74.7	4.5	186.9	69.9	3.0	15.5
16	54.8	8.7	137.0	101.9	4.4	11.7
20	64.6	6.7	161.5	71.6	3.1	10.7
21	23.0	3.9	57.5	84.5	3.7	21.7
23	29.0	4.4	72.5	78.9	3.4	17.9
Average for all CLT panels above				**68.1**	**3.0**	**12.1**
Minimum for all CLT panels above				**43.6**	**1.9**	**6.6**

As can be seen from the results above, most walls can satisfy the performance levels required by the AC130 criteria if the initial load cycle envelope properties are used. The number will be lower if the CLT wall properties developed on the average of two envelope curves are used. Although AC130 criteria do not deal with sets of different walls, one can always look at the average values of the entire set of CLT walls. In such a case, the average values for the entire set of CLT walls can satisfy the criteria (Table 7). The average ductility ratio (as defined in AC130) is 11.8, which is greater than the required minimum of 11, and the average ultimate storey drift is 3.0%, which is greater than the required 2.8%. The minimum value for the maximum drift was about 2% while the minimum ductility was approximately 6. The average values further improve if we take into consideration only the set of CLT walls with nailed connections (excluding walls 5 and 6 that used screws).

Table 6
AC130 related properties of selected CLT walls based on the secondary load cycle envelope

Wall	P_{ASD} [kN]	Δ_{ASD} [mm]	P_{max} [kN]	Δ_u [mm]	Δ_u [% drift]	Ductility Δ_u / Δ_{ASD}
00	35.4	9.9	88.5	64.1	2.8	6.5
02	34.9	8.3	87.2	66.8	2.9	8.0
03	36.7	8.9	91.6	64.4	2.8	7.2
04	40.0	6.3	100.0	55.2	2.4	8.8
05	39.7	9.8	99.3	63.8	2.8	6.5
06	37.0	7.9	92.6	46.4	2.0	5.9
08A	42.1	6.6	105.3	54.1	2.4	8.2
10A	40.7	2.5	101.7	52.7	2.3	21.1
12	34.7	8.1	86.7	73.7	3.2	9.1
14	78.0	3.3	195.0	65.5	2.8	19.8
16	49.4	7.6	123.5	112.3	4.9	14.8
20	57.1	10.5	142.7	69.4	3.0	6.6
21	20.3	3.3	50.8	85.3	3.7	25.9
23	28.7	6.7	71.9	80.8	3.5	12.1
Average for all CLT panels above				**68.2**	**3.1**	**11.5**
Minimum for all CLT panels above				**46.4**	**2.0**	**5.9**

Table 7
Average AC130 related properties of selected CLT walls

Wall	P_{ASD} [kN]	Δ_{ASD} [mm]	P_{max} [kN]	Δ_u [mm]	Δ_u [% drift]	Ductility Δ_u / Δ_{ASD}
00	35.6	7.8	88.9	66.6	2.9	9.4
02	36.1	8.5	90.3	71.5	3.1	8.5
03	39.2	7.5	98.1	63.6	2.8	8.8
04	40.9	7.5	102.3	59.6	2.6	8.1
05	41.1	8.0	102.7	53.7	2.3	6.8
06	40.0	8.1	100.1	50.1	2.2	6.2
08A	42.8	4.9	107.1	57.8	2.5	13.7
10A	41.0	3.2	102.4	49.0	2.1	16.5
12	37.0	8.6	92.5	72.0	3.1	8.5
14	76.4	3.9	190.9	67.7	2.9	17.7
16	52.1	8.2	130.2	107.1	4.7	13.2
20	60.8	8.6	152.1	70.5	3.1	8.7
21	21.6	3.6	54.1	84.9	3.7	23.8
23	28.9	5.6	72.2	79.8	3.5	15.0
Average for all CLT panels above				**68.1**	**3.0**	**11.8**
Minimum for all CLT panels above				**49.0**	**2.1**	**6.2**

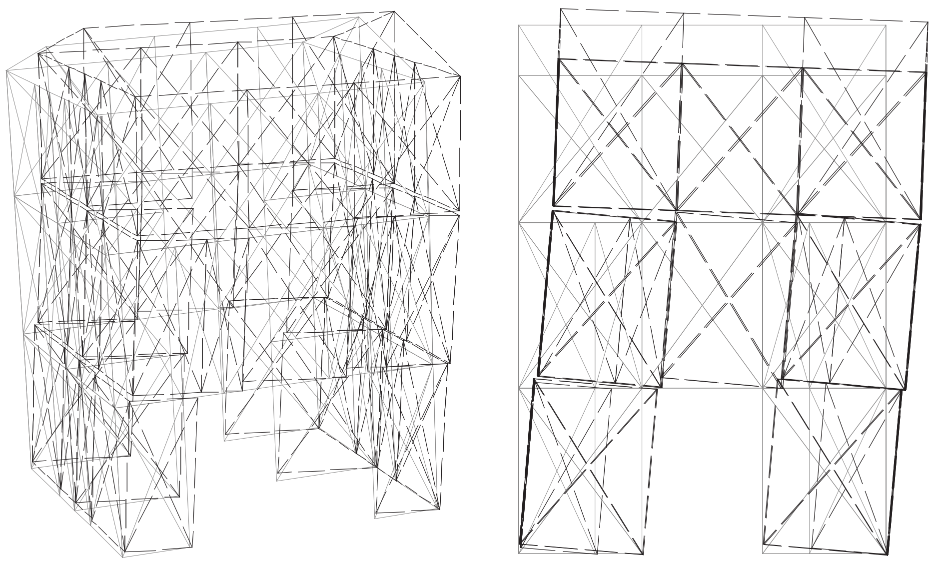
Based on the test results and the performance-based AC130 acceptance criteria, some of the individual CLT walls tested (and the average of the entire wall set) can qualify as new structural wall elements and can share the same seismic response parameters with regular wood-frame shear walls in the USA, which means using an R-factor of 6.5. This value would correspond to having a product of $R_d R_o$ in Canada of 5.1 (R_d factor of 3.0 and R_o factor of 1.7), which is currently used in the NBCC for wood-frame shear walls. However, at this early stage of system acceptance, the authors are of the opinion that a bit more conservative factors should be assigned for CLT wall systems.

Based on the results of the AC130 exercise, it is recommended that a conservative estimate of force modification factors for CLT as a structural system be an R_o factor of 1.5 and an R_d factor of 2.0.

6.2.3 The European Experience

The basis of the European approaches for quantifying the *q*-factor in Eurocode 8 (EN, 2004) are presented in Section 6.1.1 of this chapter. Some of the findings using these approaches are discussed in this section.

As mentioned in Section 2, shaking table tests on a 3-storey house were conducted in the laboratories of the NIED in Tsukuba, Japan as part of the SOFIE project. An analytical model of the 3-storey house was developed using the DRAIN 3-DX computer program, and the model was verified against the observed behaviour during the shaking table tests. Using the verified analytical model (Figure 25), a number of non-linear time-history dynamic analyses were conducted using eight different earthquake records. Based on the results from the analytical studies, an evaluation of the behaviour factor q for seismic design according to Eurocode 8 was conducted (Ceccotti, 2008). In this paper, the behaviour factor q was defined as the ratio between the PGA that caused near collapse of the structure (the analytical model) vs. the design PGA. The near collapse peak ground acceleration (labeled as $PGA_{u,eff}$) was taken as the acceleration that caused uplift of 25.5 mm at one or more hold-down positions in the walls, while the design PGA (labeled as $PGA_{u,code}$) was taken equal to 0.35.



(a) (b)

Figure 25
Deformed shapes of the analytical model under Nocera Umbra Earthquake record at PGA of 1.2 g (Ceccotti, 2008)

The behaviour factor obtained from all eight analyses using this PGA-based approach is shown in Table 8 (Ceccotti, 2008). As shown, for seven out of eight earthquakes, the *q*-factor was greater than 3.0 and in two cases even greater than 4.0, with an average of 3.4. Although the results presented here only apply to one 3-storey CLT structure with a given configuration and initial period, some observations can still be made. It seems that a *q*-factor of 3.0 is a reasonable estimate for the CLT structure evaluated that it is representative of a typical 3-storey building that uses screws in the brackets and nails in the hold-downs.

Table 8
Earthquake records and calculated *q*-factors from the analyses (Ceccotti, 2008)

Earthquake	Station and Component	$PGA_{u,eff}$	Calculated *q*-factor
Kobe	JMA, N-S	1.15	3.28
El Centro	Imperial Valley, N-S	1.20	3.43
Nocera Umbra	Nocera, E-W	1.60	4.57
Northridge	Newhall, E-W	0.88	2.51
Joshua	Landers, N-S	1.09	3.11
Loma Prieta	Corralitos, E-W	1.05	3.00
Mexico City	E-W	1.23	3.51
Kocaeli	Yapi Kredi, N-S	1.43	4.09
Average *q*-factor			**3.44**

Further refinement of the PGA-based approach used in Ceccotti (2008) can be found in Pozza et al. (2009). The differences in this base shear-based approach with respect to the PGA-based approach are:

- The *q*-factor is defined as a ratio of the base shear of the building during a linear elastic response and the base shear at near collapse level for each different record. This is a more common assumption as it defines the *q*-factor close to the terms of the well-known equivalent displacement rule;
- Since the response of the building subjected to different earthquakes is also dependent on its initial period of vibration, buildings with three different initial periods were used in the analyses. One building was the same as the 3-storey model tested during the SOFIE project, while the other two were with the same geometry, but just different mass. Although still far from the complexity of the FEMA P695 requirements, with the additions of several buildings with different periods in the analyses, this approach moves closer to them.

In this study, the authors also used different analytical models to represent the behaviour of the CLT elements in a building. The models were verified against the test results from the shaking table tests on the 3-storey building that was part of the SOFIE project. They also used five artificially generated earthquakes that meet the spectrum compatibility requirements for the regions with highest seismicity in Italy. The results from these analyses and the comparison of this method (the base shear approach) with respect to the one used by Ceccotti (2008) (the PGA approach) are shown in Table 9.

Table 9
The *q*-factor obtained using two different approaches, the PGA approach, and the base shear approach (results for the building analysed in Ceccotti (2008) are shown in colour)

	PGA Approach			Base Shear Approach		
Building Period	T=0.16 s	T=0.21 s	T=0.26 s	T=0.16 s	T=0.21 s	T=0.26 s
Earthquake 1	4.07	4.57	4.29	2.75	2.88	2.90
Earthquake 2	3.14	2.86	3.02	3.14	3.21	3.52
Earthquake 3	4.29	4.48	4.13	3.14	3.21	3.52
Earthquake 4	3.07	3.33	3.14	2.91	3.19	3.29
Earthquake 5	4.41	4.23	4.37	3.30	2.96	3.33
Average	3.80	3.89	3.79	3.05	3.09	3.31
Total average	**3.83**			**3.15**		

As can be seen in Table 9, the *q*-factor values calculated according to the base shear approach have a lower variability than those calculated according to the PGA-based approach. The base shear approach also showed lower values for the calculated *q*-factor that was on average 21% lower than using the PGA approach. The average values, however, showed again that a *q*-factor of 3.0 is acceptable for use in the seismic design of CLT structures in Europe.

A straightforward comparison of the suggested *q*-factor for CLT in Europe from both studies to the force modification factors in NBCC would correspond to a combination of $R_d R_o = 3.0$. Several things should be noted, however, which may have an influence on such a statement. First, unlike in the USA and Canada, Eurocode 8 (EN, 2004) still uses design ground motions with probability of exceedance of 10% in 50 years (earthquake return period of 475 years) and that may have an effect on the results. Second, both approaches presented here used elastically designed structures as reference structures for determining the *q*-factor, which usually leads to more conservative values of the *q*-factor. An approach that is suggested for future research should include analytical models of structures already designed with a certain *q*-factor and conduct incremental non-linear dynamic analyses using a set of earthquake ground motions with increasing PGA or pseudo-spectral acceleration (PSA) values.

7 BASICS OF CAPACITY DESIGN FOR CLT STRUCTURES

Although the primary focus of this document is the overall seismic performance of CLT structures and not the detailed process of their seismic design, it is important to just briefly address the fundamentals of seismic design for CLT structures. It is suggested that the seismic design of CLT structures follows the capacity design principles that are of major importance in seismic design in general. This design approach is based on the simple understanding of the way a structure sustains large deformations under severe earthquakes. By choosing certain modes of deformation, we can ensure that the brittle elements have the capacity to remain intact, while inelastic deformations occur in selected ductile elements. These "dissipative zones" act as dampers to control the force level in the structure. In steel structures, the members are typically designed to yield before the connections. For example, in space frame systems, beam failure mechanisms are preferred since they provide sufficient structural ductility without creating an undesirable mechanism of collapse. In timber structures, however, the failure of wood members in tension or bending is not favourable because of its brittle characteristics, and all failures should occur in the connections.

Consequently, it is suggested that all non-linear deformations and energy dissipation in the case of CLT structures should occur in the connections (e.g., brackets) that connect the wall to the floor panels, in the hold-down connections, if used, and in the vertical step joints in the walls, if present and if so chosen. All other connections shall be designed to remain linear elastic, with a strength that is slightly higher than the force induced on each of them when neighboring dissipative zones reach their probable strength. All connections used for energy dissipation in CLT structures should be designed to fail in fastener yielding mode. No wood failure modes in these connections should be allowed.

Using this strategy, the connections in horizontal step joints between floor panels (No. 2 in Figure 26) should have sufficient over-strength and adequate stiffness to allow for the diaphragm to act as a single unit. Similarly, connections tying up the floor panels to the walls below (No. 3 in Figure 26) should also be over-designed, and be one of the strongest connection elements in the structure. If vertical step joints are present in the walls (No. 4 in Figure 26), thus dividing the walls into several wall segments, the step joint connections can be designed as yielding elements (dissipative zones) that will yield simultaneously with the steel bracket connections (or hold-downs, if present) subjected to uplift at both ends of each wall segment. Yielding of the bracket connections at panel ends should be followed by yielding of the rest of the bracket connections connecting the walls to the floor. One can always use another design approach by over-designing the connections in the step joints, which will result in the entire wall being able to act as a single panel. In this case, wall uplift will start to occur at both ends of the wall during the seismic response, and the potential benefits of the step joints as energy dissipating zones will be lost.

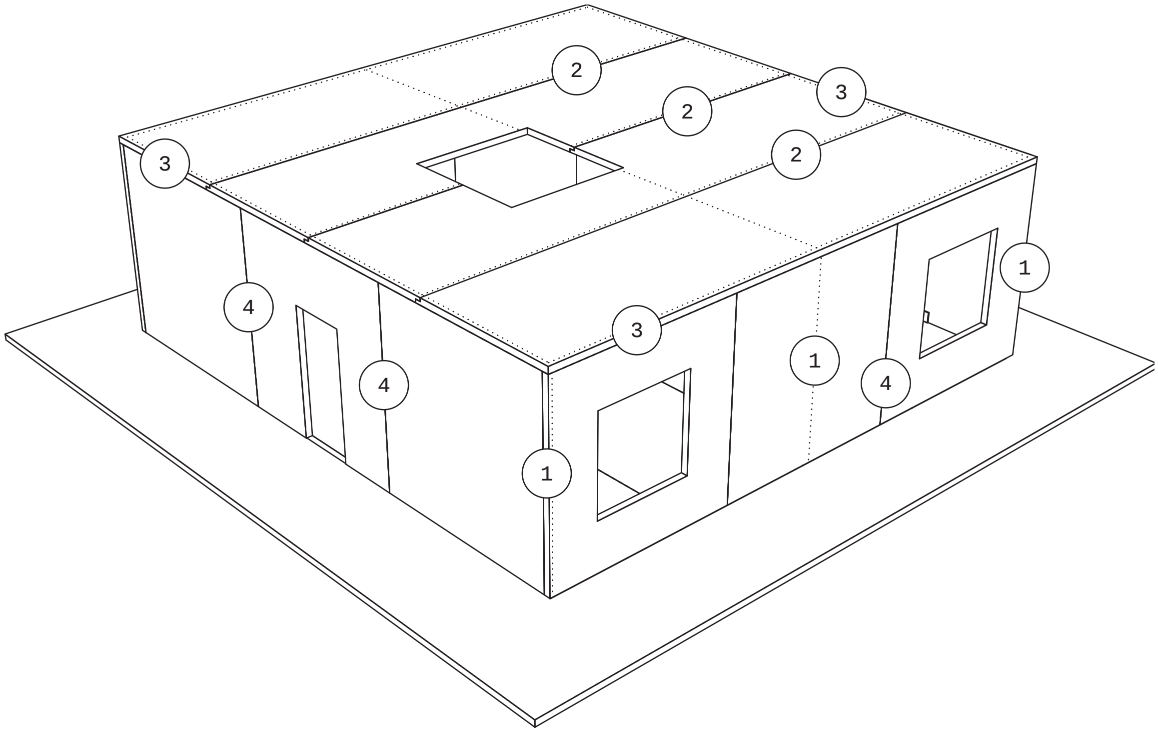


Figure 26
Typical storey of a CLT structure with various connections between the panels
(drawing courtesy of Dr. Ceccotti)

The vertical joints between perpendicular walls (No.1 in Figure 26), may or may not be included as dissipative joints. The effect of the perpendicular walls on the seismic performance of CLT walls has not yet been investigated in depth. Until these effects are fully known and quantified, it is suggested that vertical joints between perpendicular walls be over-designed. This approach also slightly simplifies the seismic design procedure and gives the structure additional level of robustness and safety.

Fasteners should be randomly placed in the available space in the steel brackets and hold-downs with the maximum fastener spacing possible. Larger fastener spacing will help avoid load concentration in a small area of the CLT panel.

8 CONCLUSIONS

Cross-laminated timber (CLT) is an innovative wood product that is gaining popularity in residential and non-residential applications in Europe. European experience shows that this system can be competitive, particularly in mid-rise and high-rise buildings. Although CLT has rarely been used in North America, it may be used as a viable wood-based structural solution for the shift towards sustainable densification of urban and suburban centers in Canada and the USA. In order to gain much needed wide acceptance and popularity, CLT as a structural system needs to be implemented in the North American codes arena.

Based on the literature review of the research work conducted around the world and the results from a series of quasi-static tests on CLT wall panels conducted at FPInnovations' laboratory in Vancouver, CLT wall panels can be an effective lateral load resisting system. They can have adequate seismic performance when nails or slender screws are used with steel brackets to connect the walls to the floors below. The use of hold-downs with nails on each end of the walls tends to further improve their seismic performance. Use of diagonally placed long screws to connect CLT walls to the floor below is not recommended in high seismic zones due to less ductile wall behaviour and sudden failure mechanism. Use of step joints in longer walls can be an effective solution not only to reduce the wall stiffness and thus reduce the seismic input load, but also to improve the wall deformation capabilities. Timber rivets in smaller groups with custom made brackets were found to be effective connectors for CLT wall panels. Further research in this field is needed to further clarify the use of timber rivets in CLT.

Although CLT construction is a platform type of structural system, it is far less susceptible to the soft storey mechanism than many other structural systems of the same type. Since the nonlinear behaviour (and the potential damage) is localized to the hold-down and bracket connection areas only, the panels that are also the vertical load carrying elements are virtually left intact in place, and well connected to the floor panels, even after a "near collapse" state is reached. In addition, all walls in one storey of a CLT construction contribute to the lateral and gravity resistance, thus providing a degree of redundancy and a system effect.

A preliminary evaluation of the force modification factors (R-factors) for the seismic design of structures according to the National Building Code of Canada (NBCC) was also performed. Based on the experimental and analytical research work conducted in this field in Europe and at FPInnovations, the performance comparison to already existing systems in NBCC and on the equivalency performance criteria given in AC130, values of 2.0 for the R_d factor and of 1.5 for the R_o are recommended as conservative estimates for CLT structures that use ductile connections such as nails and slender screws.

9
REFERENCES

American Society for Testing and Materials (ASTM). 2009. *Standard test methods for cyclic (reversed) load test for shear resistance of vertical elements of the lateral force resisting systems for buildings.* ASTM E 2126. West Conshohocken, Pennsylvania: ASTM. 15 p.

American Society of Civil Engineers (ASCE). 2005. *Minimum design loads for buildings and other structures.* ASCE 7-05. Reston, Virginia: ASCE. 424 p.

Canadian Standards Association (CSA). 2009. *Engineering design in wood (limit states design)*. CSA O86-09. Rexdale, Ontario: CSA. 222 p.

Ceccotti A. 2008. New technologies for construction of medium-rise buildings in seismic regions: The XLAM case. *Structural Engineering International (SEI)* 18(2):156-165.

Ceccotti A., and M. Follesa. 2006. Seismic behaviour of multi-storey XLAM buildings. In COST E29: *International Workshop on Earthquake Engineering on Timber Structures, November 9-10, 2006, Coimbra, Portugal*, 81-95.

Ceccotti A., M. Follesa, N. Kawai, M. P. Lauriola, C. Minowa, C. Sandhaas, and M. Yasumura. 2006. Which seismic behaviour factor for multi-storey buildings made of cross-laminated wooden panels? In *Proceedings of the 39th CIB W18 Meeting, Firenze, Italy*, paper 39-15-4.

Ceccotti A., M. Follesa, M. P. Lauriola, and C. Sandhaas. 2006. SOFIE Project: Test results on the lateral resistance of cross-laminated wooden panels. Paper presented at the First European Conference on Earthquake Engineering and Seismicity, Geneva, Switzerland.

Ceccotti A., and E. Karacabeyli. 2002. Validation of seismic design parameters for wood-frame shearwall systems. *Canadian Journal of Civil Engineering* 29:484-498.

Dujic B., S. Aicher, and R. Zarnic. 2006. Testing of wooden wall panels applying realistic boundary conditions. Paper presented at the 9th World Conference on Timber Engineering, Portland, Oregon.

_____________. 2005. Investigation on in-plane loaded wooden elements: Influence of loading and boundary conditions. *Otto-Graf-Journal* 16:259-272.

Dujic B., V. Hristovski, M. Stojmanovska, and R. Zarnic. 2006. Experimental investigation of massive wooden wall panel systems subjected to seismic excitation. Paper presented at the First European Conference on Earthquake Engineering and Seismicity, Geneva, Switzerland.

Dujic B., S. Klobcar, and R. Zarnic. 2006. Influence of openings on shear capacity of massive cross-laminated wooden walls. In COST E29: *International Workshop on Earthquake Engineering on Timber Structures, November 9-10, 2006, Coimbra, Portugal*, 105-118.

____________. 2007. Influence of openings on shear capacity of wooden walls. In *Proceedings of the 40th CIB-W18 Meeting, Bled, Slovenia*, paper 40-15-6.

____________. 2008. Shear capacity of cross-laminated wooden walls. Paper presented at the 10th World Conference on Timber Engineering, Myazaki, Japan.

Dujic B., J. Pucelj, and R. Zarnic. 2004. Testing of racking behavior of massive wooden wall panels. In *Proceedings of the 37th CIB-W18 Meeting, Edinburgh, Scotland*, paper 37-15-2.

Dujic B., and R. Zarnic. 2006. Study of lateral resistance of massive X-lam wooden wall system subjected to horizontal loads. In *COST E29: International Workshop on Earthquake Engineering on Timber Structures, November 9-10, 2006, Coimbra, Portugal*, 97-104.

European Committee for Standardization (CEN). 2005. *Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings*. BS EN 1998-1:2004. London: British Standards Institution (BSI). 232 p.

Federal Emergency Management Agency (FEMA). 2009. *Quantification of building seismic performance factors*. FEMA P695. Washington, D.C.: FEMA. 421 p.

____________. 2010. Quantification of building system performance and response parameters: Component equivalency methodology. Draft report FEMA P795. Washington, D.C.: FEMA.

Hristovski V., and M. Stojmanovska. 2005. Experimental and analytical evaluation of the racking strength of massive wooden wall panels: Preliminary project phase, EE21C, Skopje-Ohrid, Macedonia.

ICC Evaluation Service (ICC-ES). 2009. *Acceptance criteria for prefabricated, cold-formed, steel lateral-force-resisting vertical assemblies.* AC 322. Whittier, California: ICC-ES. 6 p.

ICC Evaluation Service (ICC-ES). 2009. *Acceptance criteria for prefabricated wood shear panels*. AC 130. Whittier, California: ICC-ES. 8 p.

International Code Council (ICC). 2006. *International building code.* Country Club Hills, Illinois: ICC. 664 p.

Lauriola M. P., and C. Sandhaas. 2006. Quasi-static and pseudo-dynamic tests on XLAM walls and buildings. In *COST E29: International Workshop on Earthquake Engineering on Timber Structures, November 9-10, 2006, Coimbra, Portugal*, 119-133.

Mitchell, D., R. Tremblay, E. Karacabeyli, P. Paultre, M. Saatcioglu, and D. L. Anderson. 2003. Seismic force modification factors for the proposed 2005 edition of the National Building Code of Canada. *Canadian Journal of Civil Engineering* 30:308–327.

National Research Council (NRC). 2005. *National building code of Canada, 2005*. Ottawa: Canada. National Research Council. 2 v.

Popovski, M., and E. Karacabeyli. 2004. Seismic performance of riveted connections in heavy timber construction. Paper presented at the 13th World Conference on Earthquake Engineering, Vancouver, BC, Canada.

Popovski, M., and E. Karacabeyli. 2008. Force modification factors and capacity design procedures for braced timber frames. Paper presented at the 14th World Conference on Earthquake Engineering, Beijing, China.

Popovski, M., H. G. L. Prion, and E. Karacabeyli. 2002. Seismic performance of connections in heavy timber construction. *Canadian Journal of Civil Engineering* 29:389-399.

Pozza, L., R. Scotta, and R. Vitaliani. 2009. A non linear numerical model for the assessment of the seismic behaviour and ductility factor of X-lam timber structures. In *Proceeding of the International Symposium on Timber Structures, June 25-27, 2009, Istanbul, Turkey*, 151-162.

Schädle, P., and H. J. Blaß. 2010. Earthquake behaviour of modern timber construction systems. Paper presented at the 11th World Conference on Timber Engineering, Riva del Garda, Italy.

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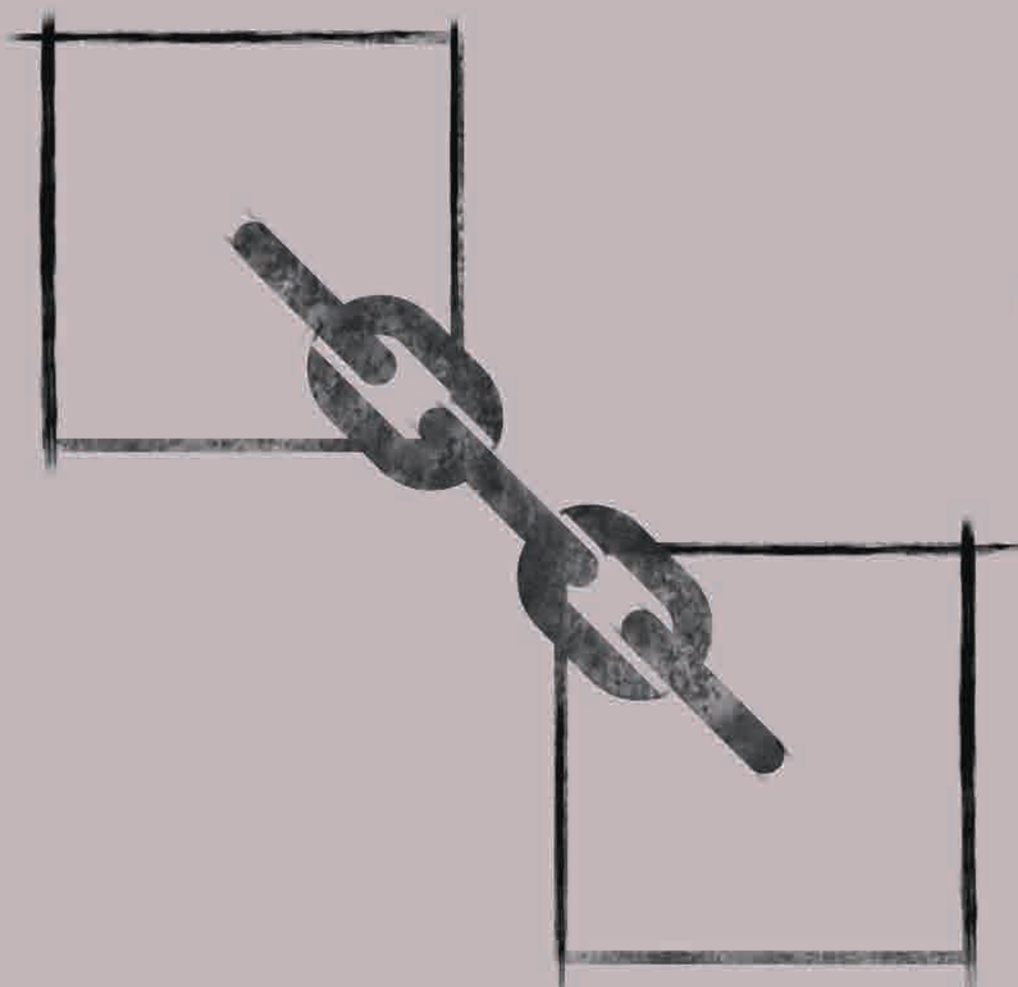

Connections

Connections in cross-laminated timber buildings

CHAPTER 5

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ABSTRACT

The light weight of cross-laminated timber (CLT) products combined with the high level of prefabrication involved, in addition to the need to provide wood-based alternative products and systems to steel and concrete, have significantly contributed to the development of CLT products and systems, especially in mid-rise buildings (5 to 9 storeys). While this product is well-established in Europe, work on the implementation of CLT products and systems has just begun in Canada and the USA. The structural efficiency of the floor system acting as a diaphragm and that of walls in resisting lateral loads depends on the efficiency of the fastening systems and connection details used to interconnect individual panels and assemblies. Long self-tapping screws are typically recommended by CLT manufacturers and are commonly used for connecting panels to panels in floors and floor-to-wall assemblies. However, there are other types of traditional and innovative fasteners and fastening systems that can be used in CLT assemblies.

This chapter focuses on a few connector systems that reflect present-day practices, some being conventional, others being proprietary. Given the recent introduction of CLT into the construction market, it is expected that new connection types will be developed in time. Issues associated with connection design specific to CLT assemblies are presented. The European design approach is presented and the applicability of CSA O86-09 design provisions for traditional fasteners in CLT such as bolts, dowels, nails and wood screws are reviewed and design guidelines are provided.

The information given in this chapter is aimed at Canadian designers, a group which has expressed a strong interest in specifying CLT products for non-residential and multi-storey applications. However, further studies are needed to assist designers in the development of Canadian engineering design specifications and procedures consistent with Canadian material design standards and the National Building Code of Canada. The technical information will also be used to facilitate code acceptance of CLT products in North America.

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1 CROSS-LAMINATED TIMBER IN CONSTRUCTION

Use of cross-laminated timber (CLT) panels in building construction has increased over the last few years. Several buildings have already been erected around the world using CLT panels, which is a good testimony to the many advantages that this product offers to the construction industry. The light weight and high quality of prefabrication of CLT result in quick erection times, especially in mid-rise construction (5 to 9 storeys). While this product is well-established in Europe, work on the implementation of CLT products and systems has just begun in Canada and the USA.

The structural efficiency of the floor system acting as a diaphragm and that of walls in resisting lateral loads depends on the efficiency of the fastening systems and connection details used to connect individual panels and assemblies. This chapter focuses on the design of connections for CLT construction based on current practices.

Figure 1
Typical CLT building with various components and connections

2 COMMON STRUCTURAL SYSTEMS IN CLT

There are several ways to design and construct CLT buildings. They all differ in the way the load-carrying panels/elements are arranged, the way the panels are connected and the type of wood and non wood-based materials used (such as the use of hybrid systems of construction).

The most common forms of construction systems in CLT are:

1. Platform construction, where the floor panels rest directly on top of wall panels, forming a platform for subsequent floors (Figure 2a). This is a typical North American light frame form of construction, except that CLT panels are used instead of stud wall systems with top and bottom plates. This is probably the most commonly used type of structural system in Europe for CLT assemblies, especially for multi-storey buildings. This includes buildings constructed exclusively with CLT or mixing CLT with other types of wood-based products (e.g., CLT and glulam), or CLT with non wood-based systems. There are several advantages to this system:

    - it simplifies the erection of upper storeys;
    - simple connection systems can be used; and
    - the load path is usually well-defined.

2. Balloon construction, a type of structural system where the walls continue for a few storeys with intermediate floor assemblies attached to those walls. Due to the limitations in the length of the CLT panels and other design and construction issues, this system is often used in low-rise, commercial or industrial buildings. Connections are usually more complex in this form of construction. Balloon construction is generally less common compared to platform construction. As with platform construction, mixed CLT and other types of wood-based and non wood-based products could also be used in the balloon type of systems.

(a) (b)

Figure 2
Different types of CLT construction systems: (a) platform construction; (b) mixed CLT walls and light-frame roof

3
INTRODUCTION TO CONNECTIONS IN CLT ASSEMBLIES – OVERVIEW

3.1 General

Connections in heavy timber construction, including those built with CLT, play an essential role in providing strength, stiffness, stability and ductility to the structure; consequently, they require careful attention by designers. Post-disaster surveys following strong earthquakes and hurricanes have shown that among other reasons, structural failures often occur due to inadequately designed or improperly fabricated connections. The interruption of continuity in the timber structure caused by the presence of connections may result in a decrease in the overall strength and stiffness of the structure (i.e. if not properly designed) which in turn implies an increase in the cross-section of the assembled timber elements.

When structural members are attached with fasteners or some other types of metal hardware, such joints are referred to as "mechanical connections". Typically, large fastener spacing and end and edge distances are required in most mechanical connections to avoid splitting and shear failures that are brittle in nature. The efficient design and fabrication of connections often determines the level of success of timber buildings when competing with other types of structural applications such as steel or concrete. This is particularly important for multi-storey heavy timber structures and hybrid buildings, where CLT is used alone or could be used in combination with steel or concrete.

The use of CLT panels enables a high degree of prefabrication at the plant. This facilitates the use of CNC technology to profile the panel for installation, at the plant, of conventional and sophisticated connection systems with a high degree of accuracy and efficiency. The dimensional stability of CLT products due to the use of kiln dried (KD) source material is better for connection 'stability' prior to installation and ensures good accuracy at installation.

In this section, a very brief overview of connection types is provided. More detailed information is provided in Section 4.

3.2 Connection Systems Commonly Used in CLT Assemblies

Currently, there is a wide variety of fasteners and many different types of joint details that can be used to establish roof/wall, wall/floor, and inter-storey connections in CLT assemblies or to connect CLT panels to other wood-based elements, or to concrete or steel in hybrid construction. While long self-tapping screws are typically recommended by CLT manufacturers and are commonly used for connecting panels to panels in floors and floor-to-wall assemblies, traditional dowel-type fasteners such as wood screws, nails, lag screws, rivets, bolts and dowels can also be effectively used in connecting panel elements. Other types of traditional fasteners, including bearing type fasteners such as split rings and shear plates, and tooth plates, may have some potential; however, their use is expected to be limited to applications where high loads are involved. Some interesting innovative connection systems are finding their way to the CLT construction market. These include glued-in rods, Geka connectors, the KNAPP® system and other systems that adopt similar concepts. Such systems have good potential for use in CLT applications, especially those that employ a high degree of prefabrication using CNC machining technology. Fortunately, major CLT panel and glulam manufacturing facilities are equipped with CNC technology which could facilitate the rapid adoption of such connection systems. The choice of the type of connection to use depends largely on the type of assemblies to be connected (i.e. panel-to-panel, floor-to-wall, etc.), panel configurations, and the type of structural system used in the building.

The following sections provide some basic information on the most commonly used types of mechanical fasteners in CLT assemblies. Detailed applications of these fasteners are presented in Section 4.

3.2.1 Wood and Self-Tapping Screws

Wood and self-tapping screws are extensively used in Europe for the assembly of CLT panels (Figure 3). The ease of installation and the high lateral and withdrawal capacity of such screws make them quite popular among designers and builders as they can take combined axial and lateral loads. Wood and self-tapping screws come in a variety of sizes and specific features. Self-tapping screws come in diameters that range from 4 mm to 12 mm and are available in lengths up to 600 mm (TEMTIS 2008). They do not require predrilling in most cases, unlike traditional wood or lag screws which require predrilled holes, the size of which depend on the density of the wood-based materials they are driven into and the diameter of the screws. The design capacity of screws in CLT must account for gaps in unglued cross-plies and other artificially sawn grooves common in CLT fabrication.

Figure 3
Self-tapping screws used in CLT connections

3.2.2 Nails and Glulam Rivets

Nails and glulam rivets are not as commonly used in the assembly of CLT panels as wood screws. Nails with specific surface features such as grooves, helically threaded nails and glulam rivets are mostly used with perforated metal plates and brackets and installed on the surface/plane of the panel (Figure 4). Most timber design standards do not allow the design of nailed connections in the end grain of wood-based products for withdrawal forces. Therefore, surface types of fasteners such as nails should not be driven in the edge of CLT panels (i.e. in end grain) to resist withdrawal forces. For lateral resistance, however, an end grain factor is usually applied to account for the reduction in the lateral resistance of nails driven in the end grain in most timber design standards, including CSA O86-09 (CSA 2009).

Figure 4
Power driven nails used in combination with perforated metal plates

3.2.3 Bolts and Dowels

Bolts and dowels are very common in heavy timber construction. They can also be used in the assembly of CLT panels, especially for lateral loading. If installed in the narrow face (on edge), care must be taken during the design, especially in CLT panels with unglued edges between the individual planks in a layer. This could eventually compromise the lateral resistance since there is a potential that such fasteners are driven in the gaps.

3.2.4 Bearing Type of Fasteners

While bearing-type fasteners such as split rings and shear plates are commonly used in connections of glulam, heavy sawn timber and structural composite lumber (SCL) such as parallel strand lumber (PSL), they are not widely used for the assembly of CLT panels. Bearing-type connections can be used in certain locations depending on the position of the fasteners with respect to the CLT layers and the type of service load. One drawback would be that panels require profiling at the plant prior to delivery.

3.2.5 Innovative Types of Fasteners

A new generation of fasteners such as glued-in rods, Geka connectors, the KNAPP® system and others are becoming increasingly popular in the assembly of mainstream heavy timber construction. This is driven by recent developments in CNC technology, wood materials and the desire for a high level of prefabrication to reduce assembly time and cost.

With respect to CLT, glued-in rods in particular can be used for connections subjected to high longitudinal and transverse loads and to reduce the splitting potential (TEMTIS 2008). More details on these next generation connection systems and their suitability for connecting CLT panels and assemblies are discussed in Section 4.

4
CONNECTIONS IN CLT ASSEMBLIES – DETAILS

This section is focused on providing detailed information and schematics on traditional and innovative types of connection systems typically used in establishing connections between CLT panels, and those between walls and foundations and walls and floors. Figure 5 shows details of the various locations of such connections in a multi-storey CLT building. While most of the commonly used types of fasteners and those with some potential for use in CLT assemblies are described below, the list is not comprehensive. Other types of innovative [alternative, proprietary, modern, privately-developed] fasteners, not mentioned under this section, could also be used if found suitable.

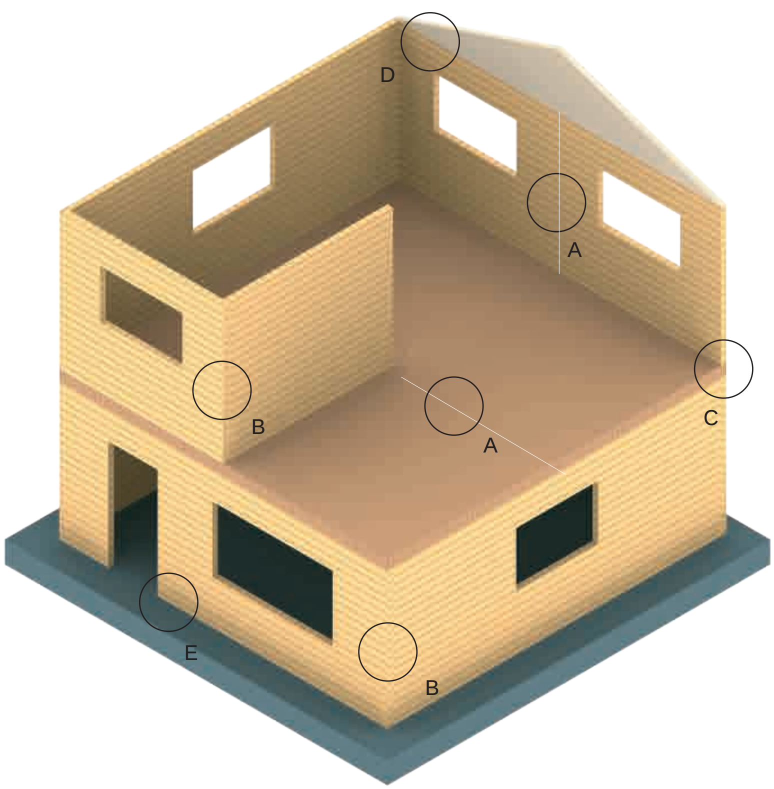


Figure 5
Typical 2-storey CLT building showing various connections between floor and wall panels

4.1 Panel-to-Panel Connections (Detail A)

This is the fundamental form of connection that is typically used to form wall and floor assemblies. It is used to connect panels along their longitudinal edges. Due to production and transport limitations related to the size of the panel that can be delivered to building sites, panel-to-panel connections are established mostly on site. Connection details must be easy to assemble and should facilitate quick fabrication. The panel-to-panel connection facilitates the transfer of forces through the wall or floor assembly. For example, when panel-to-panel connections are used in wall assembly, the connection must be designed to resist in-plane shear and out-of-plane bending. When the connection is used in floor assemblies acting as diaphragms, however, the connection must be capable of transferring in-plane diaphragm forces in principle, and maintain the integrity of the diaphragms and the overall, lateral load resisting system. Several possible panel-to-panel connection details are described below.

4.1.1 Internal Spline

A single wooden spline/strip made of lumber or SCL such as LVL could be used to form this connection. Profiling of the panel at the plant is necessary prior to delivery. Connection between the spline and the two panel edges could be established using self-tapping screws, wood screws or nails. One advantage of this detail is that it provides double-shear connection; however, it requires more accurate profiling and could be challenging in terms of fitting the different parts together on site. There are also other advantages regarding resistance to normal or out-of-plane loading. Structural adhesive could also be applied to the different parts in addition to the mechanical fasteners to provide more rigidity to the connection, if needed.

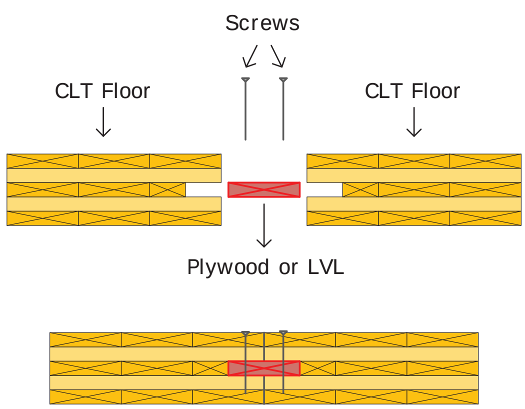


Figure 6
Internal spline

4.1.2 Single Surface Spline

This is a rather simple connection detail that can be established quickly on site. Panel edges are profiled to take a strip/spline of lumber or SCL such as LVL or X-ply LVL. Self-tapping screws, long wood screws or nails could be used for making the connection on site. Due to the single-shear connection involved, this connection detail is typically inferior to the internal spline described above. Structural adhesive could also be used in this type of connection detail.

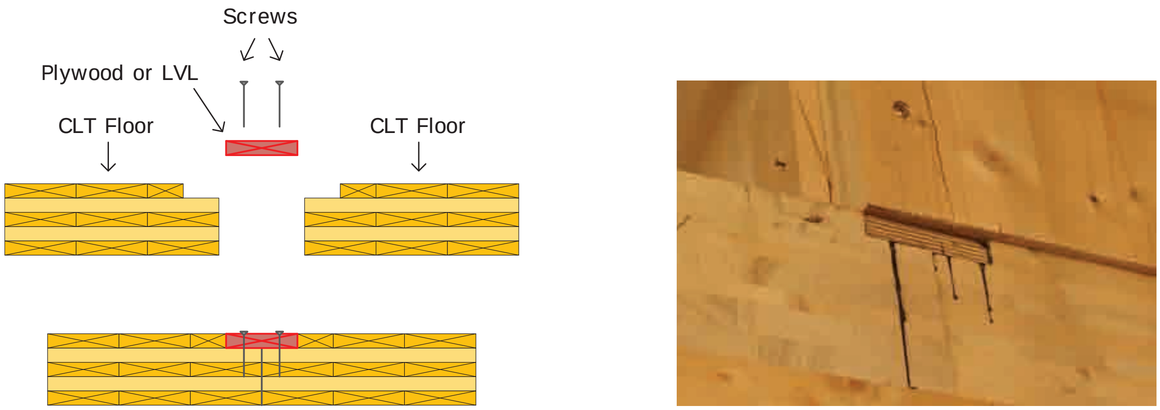


Figure 7
Single surface spline

4.1.3 Double Surface Spline

This connection detail is similar to that of the single surface spline described above, except that a double spline is used here to increase the connection strength and stiffness. Since two sets of screws are used which results in doubling the number of shear planes resisting the load, a better resistance can be achieved using this detail. However, this connection requires more machining and more time could be needed for erection since there is a need to attach the two splines from both sides of the panels during the insertion of fasteners, doubling the time needed for driving screws or nails. According to TEMTIS (2008), if SCL is used as the splines, then the joint could be designed to resist moment for out-of-plane loading. Structural adhesives could be used to enhance the strength and stiffness.

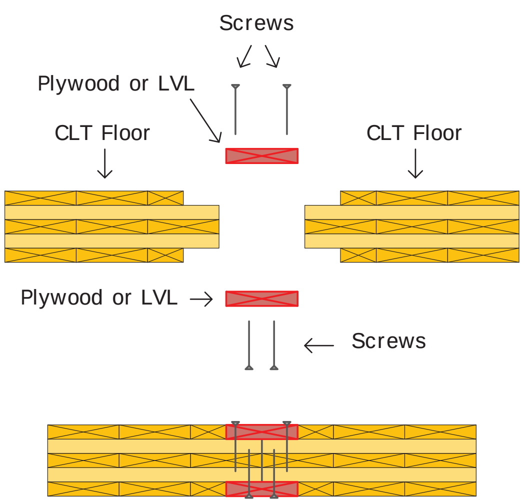


Figure 8
Double surface spline

4.1.4 Half-Lapped Joint

This connection detail involves milling a half-lapped joint at the plant and is commonly used for panel-to-panel connections in wall and floor assemblies (Figure 9). Long self-tapping screws are usually used to connect the panel edges. The joint can carry normal and transverse loads but is not considered to be a moment resisting connection (TEMTIS 2008). While this is a very simple connection detail that facilitates quick assembly of CLT elements, there is a risk of splitting of the cross-section due to concentration of tension perpendicular to grain stresses in the notched area. This is particularly pronounced for cases where uneven loading on the floor elements occur (TEMTIS 2008).

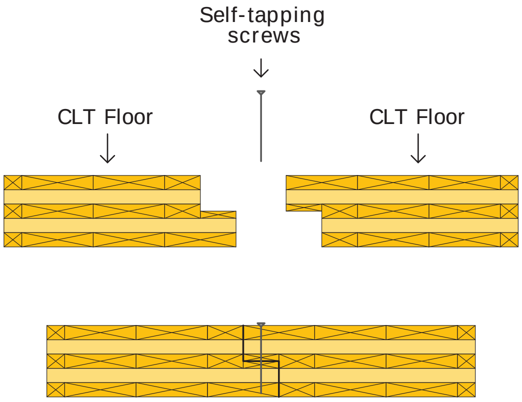


Figure 9
Details of half-lapped joints

4.1.5 Tube Connection System

This is an innovative type of connection system that has been developed and studied in Austria by G. Traetta (2007). This system incorporates a profiled steel tube with holes (Figure 10). Panel elements arrive on site with glued-in or screwed rods driven in the plane of the two panels to be connected and with holes machined in the panels at certain locations along the edges where the metal tubes could be placed. The tube connector is inserted at those locations along the panel elements and the system is tightened on site using metal nuts.

Tests have been carried out at the Building Research Center in Graz, Austria to evaluate the capacity of this innovative system (Traetta 2007). Usually no edge profiling along the panel is needed if this connection system as it principally relies on the pullout resistance of the screwed or glued-in rods.

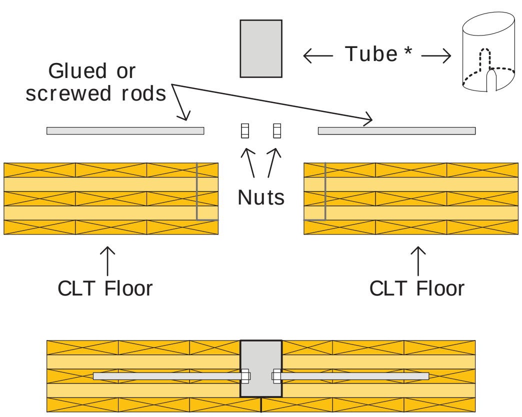


Figure 10
Details of the tube connection system

4.1.6 Alternative Systems

Certain innovative connection systems have good potential for use in CLT panel assemblies. One example of those is a German connection system called KNAPP®, which is used in prefabricated closed wood-based panels. The system facilitates quick erection as it involves a male/female type of attachment (Figure 11). It is mainly used for panel-to-panel connections along the panel longitudinal edges. KNAPP® brackets are usually attached to CLT panel elements using wood screws. They provide resistance in the plane and out of the plane of the panel, in addition to uplift resistance. The system provides resistance to in-plane and out-of-plane forces, including uplift. The KNAPP® system is equipped with a self locking mechanism that enables the wall to be tightly locked to the adjacent wall. While it might be relatively complicated to install or dismantle this system in complex plans with several intersecting wall segments, it does facilitate an easy and quick installation process.

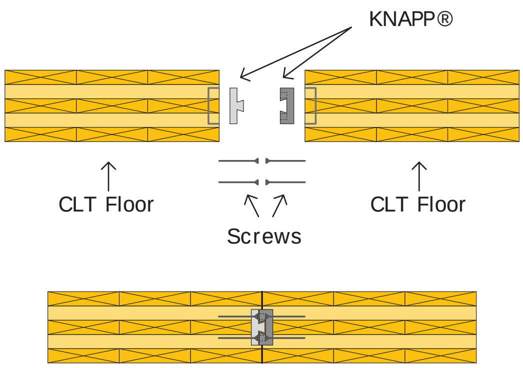


Figure 11
KNAPP® connection system

4.2 Wall-to-Wall Connections (Detail B)

This section covers connection details for connecting walls to walls positioned at right angles (wall junction in the transverse direction). Such connection details include interior partitions to exterior walls or simple exterior corner walls. Walls connected in the same plane of the panels were covered previously under panel-to-panel connections (Detail A). Most of the connection details described below are commonly used in the assembly of CLT walls. However, a few of these involve the use of innovative types of connection systems or details with some potential for use in such applications. The same connection systems adopted for connecting exterior walls in the transverse direction could be used for establishing connection between internal walls.

4.2.1 Self-Tapping Screws

Several systems have been adopted to establish connection between walls at right angles (wall junction). The simplest form of connection relies mainly on self-tapping screws to connect the walls together (Figure 12 and 14). There are some concerns however related to this direct form of connection due to the fact that the screws are driven in the narrow side of panels, in particular, if screws are installed in the end grain of the cross layers. While this may not be critical for small loads, it may not be suitable for walls subjected to high wind and seismic loads. Self-tapping screws could also be driven at an angle to avoid direct installation of screws in the narrow side of the panel (on edge) which would optimize the performance of the connection (i.e. toe screwing).

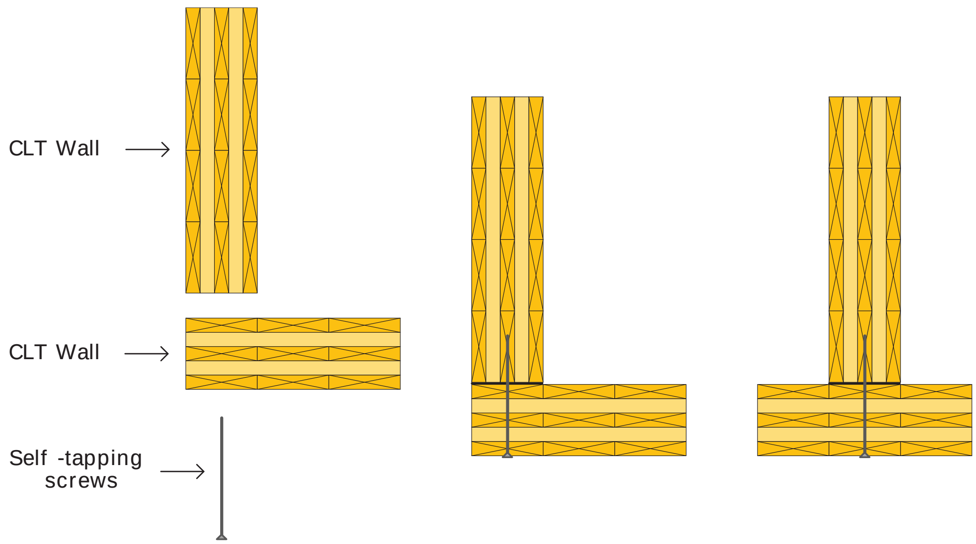


Figure 12
Self-tapping screws from the exterior

Figure 13
Installation of self-tapping screws from the exterior

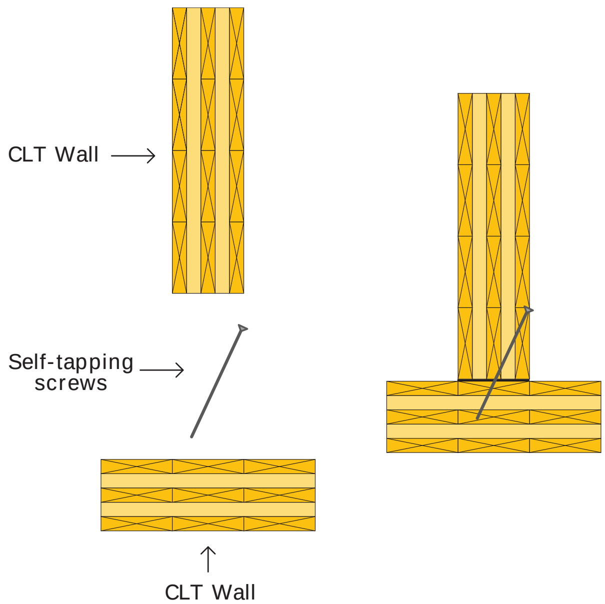


Figure 14
Self-tapping screws driven at an angle (toe screwing)

4.2.2 Wooden Profiles

Concealed wooden profiles (keys) could also be used in a similar way, with self-tapping screws or traditional wood screws. The advantage of this system over the direct use of self-tapping screws is the possibility of enhancing the connection resistance by driving more wood screws to connect the profiled panel to the central wood profile/key which is in turn, screwed to the transverse wall (Figure 15).

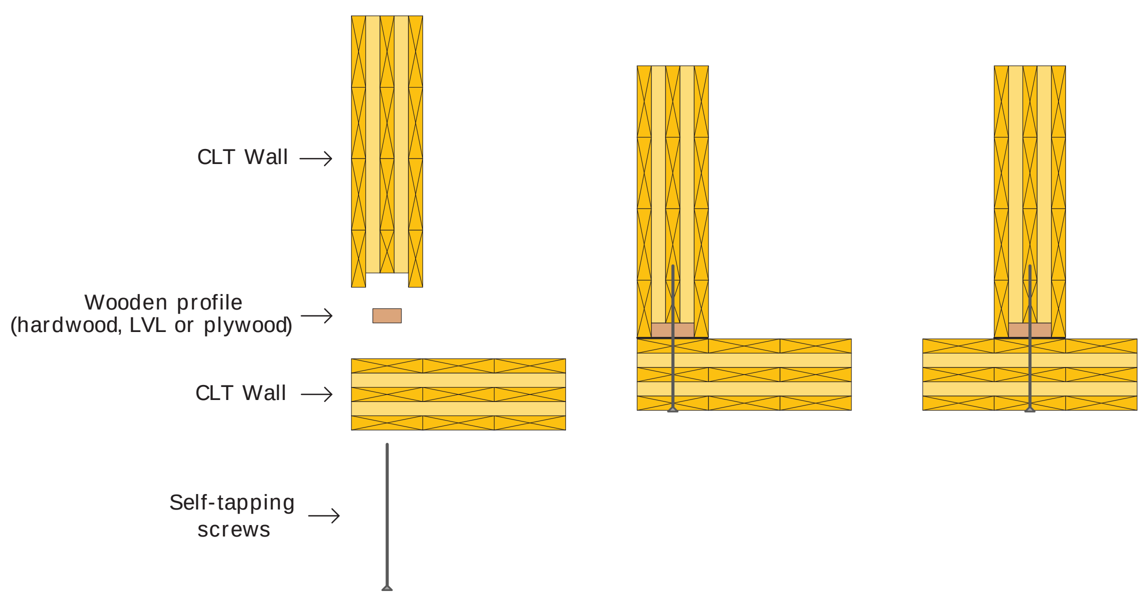


Figure 15
Concealed wooden profile

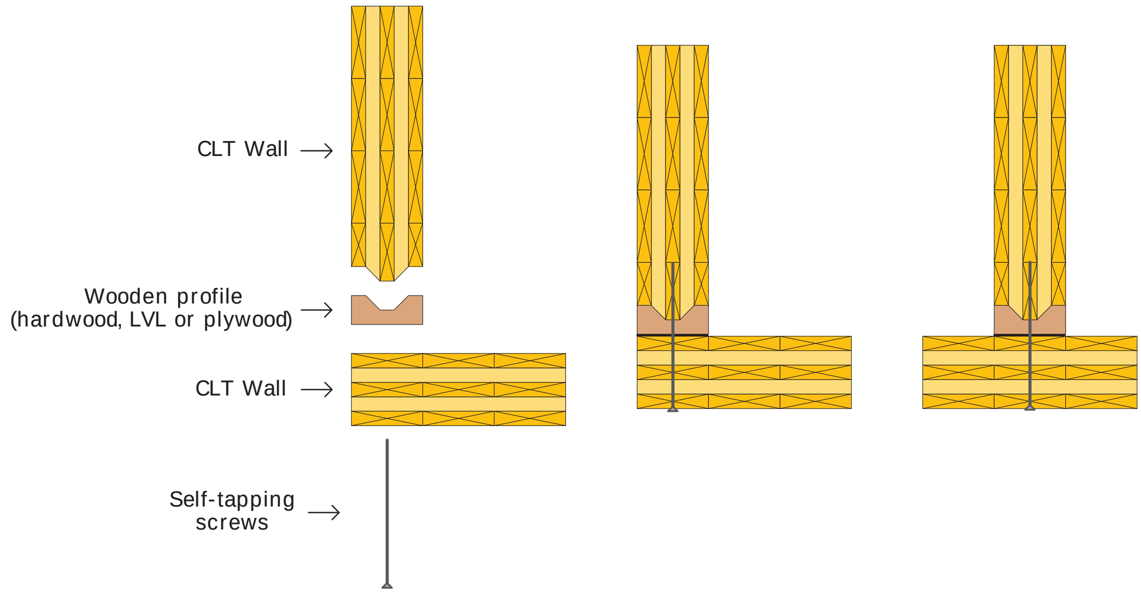


Figure 16
Edge protecting wooden profile

Other types of wooden profiles such as the one shown in Figure 16 could also be used to provide some form of reinforcement to the panel connected edges. Those are mainly made of hardwood or SCL. They are glued and screwed to the panel edge as mentioned earlier.

4.2.3 Metal Brackets

Another simple form of connecting walls in the transverse direction is the use of metal brackets with self-tapping screws, nails or even glulam rivets (Figure 17). While this connection is one of the simplest and most efficient types of connection in terms of strength resulting from fastening in the direction perpendicular to the plane of the panels, architects normally do not prefer this system as the metal plates are exposed and have less fire resistance compared to concealed connection systems. Some designers may choose to hide plates by profiling the wall panel at the locations of those brackets (recessing) then cover the metal hardware with finishing materials or simply, wood caps.

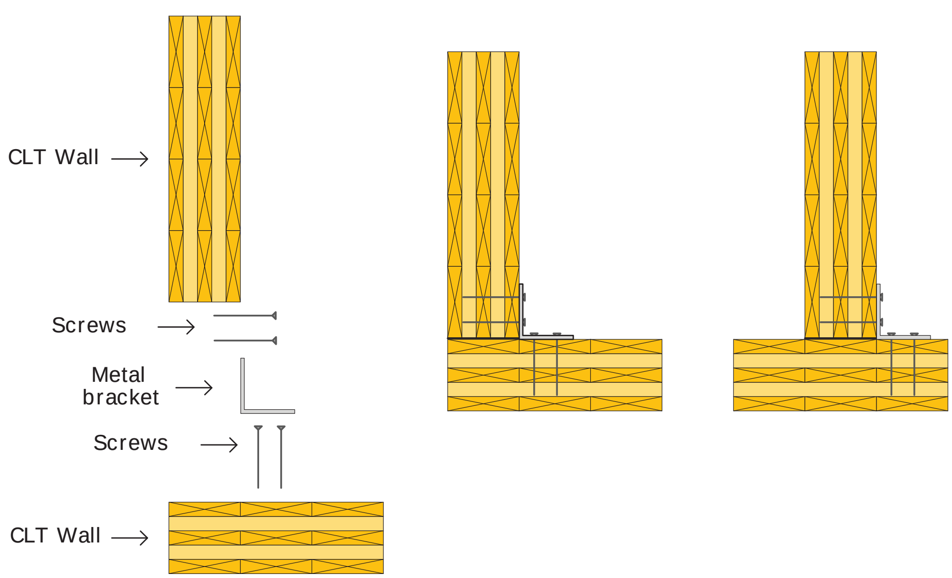


Figure 17
Interior metal bracket

4.2.4 Alternative Systems

Several alternative connection systems could be used for connecting CLT wall to wall. One interesting system involves the use of a dovetail type metal bracket to establish the connection between the wall panels (Figure 18). Several forms of a male/female type of connection can be designed to resist in-plane and out-of-plane loads. The metal brackets are attached to the wood using regular wood screws or self-tapping screws. They can be continuous along the edge of the panel/wall or a few of short length brackets can be installed along the panel/wall edge. The panel simply slides into place, which speeds the erection of the walls on site. Alternative systems such as hook joint and KNAPP® systems are based on the same principle (Figures 18 to 20). Wood screws are typically used to connect the metal components to CLT wall panels. It should be noted that dovetail systems require clearance/tolerance to facilitate the site installation. Measures should be taken to ensure that wall panels are firmly tied up.

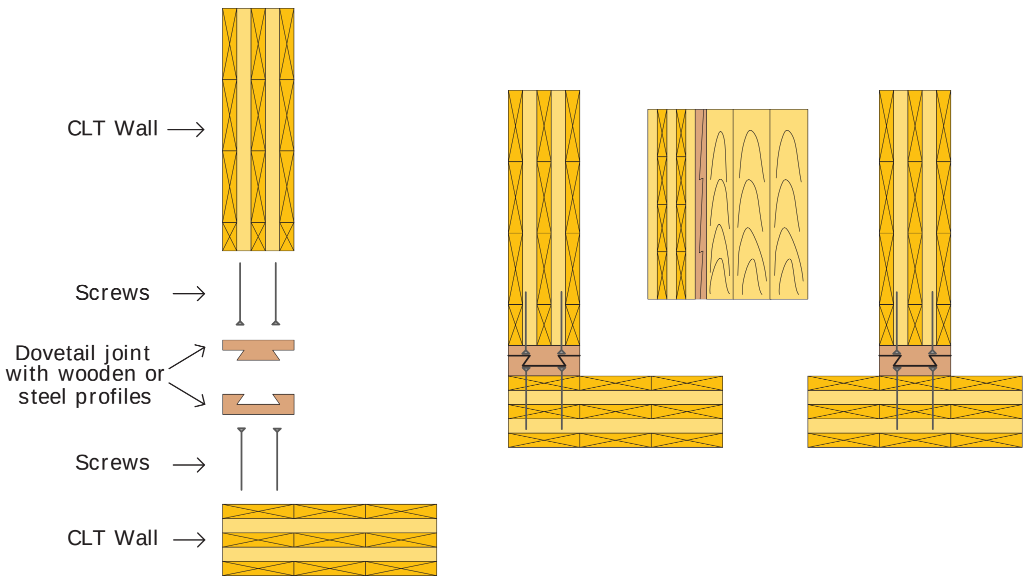


Figure 18
Details of dovetail joint

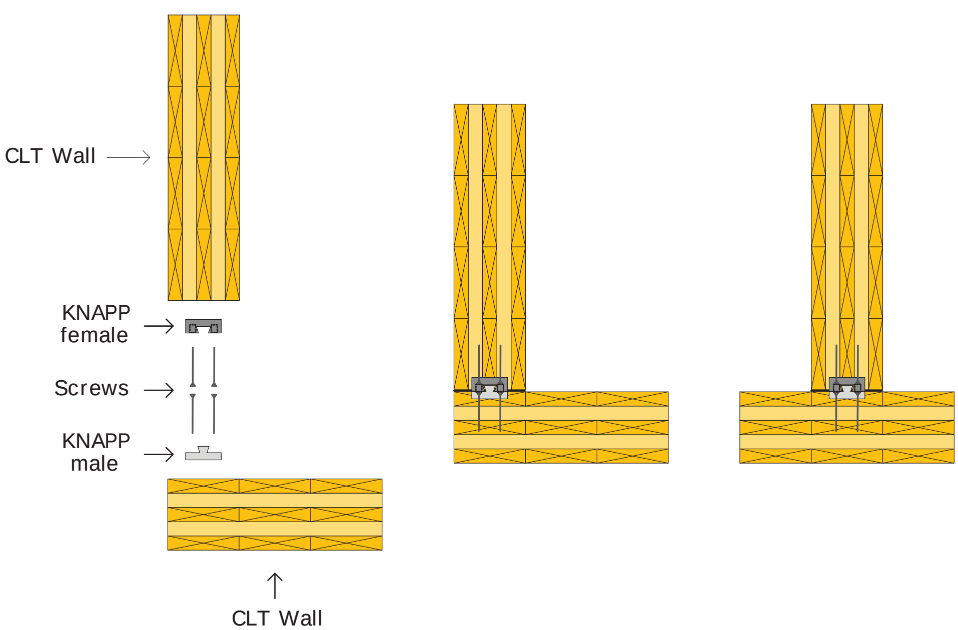


Figure 19
KNAPP® system

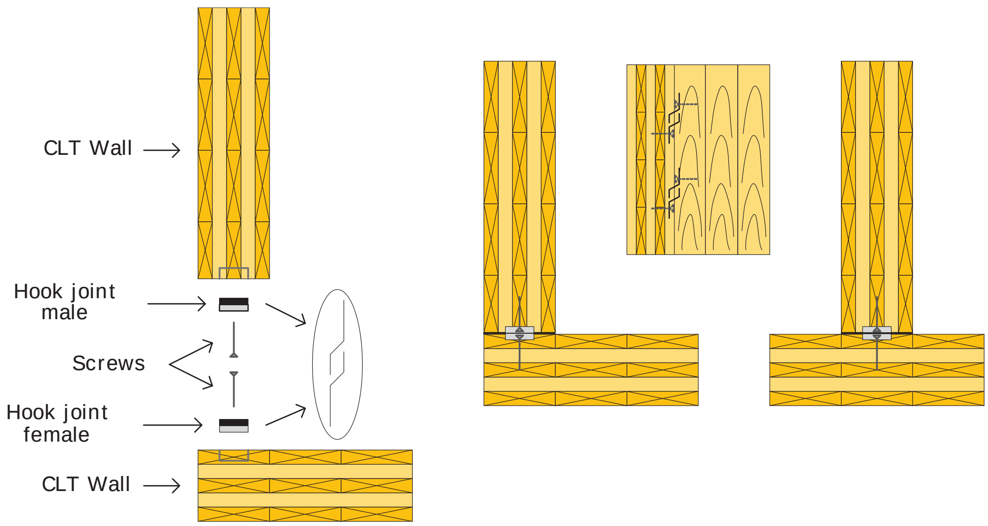


Figure 20
Hook joint

4.2.5 Concealed Metal Plates

Concealed metal plates can also be used to establish wall-to-wall connection in the transverse direction. Metal plate thickness could range from 6 mm up to 12 mm. As discussed above, while this system has considerable advantages over exposed plates and brackets, especially when it comes to fire performance, the system requires precise profiling at the plant using CNC technology (Figure 21). Proprietary self-drilling dowels that can penetrate through wood and steel such as those produced by SFS Intec (shown in Figure 22) can be used.

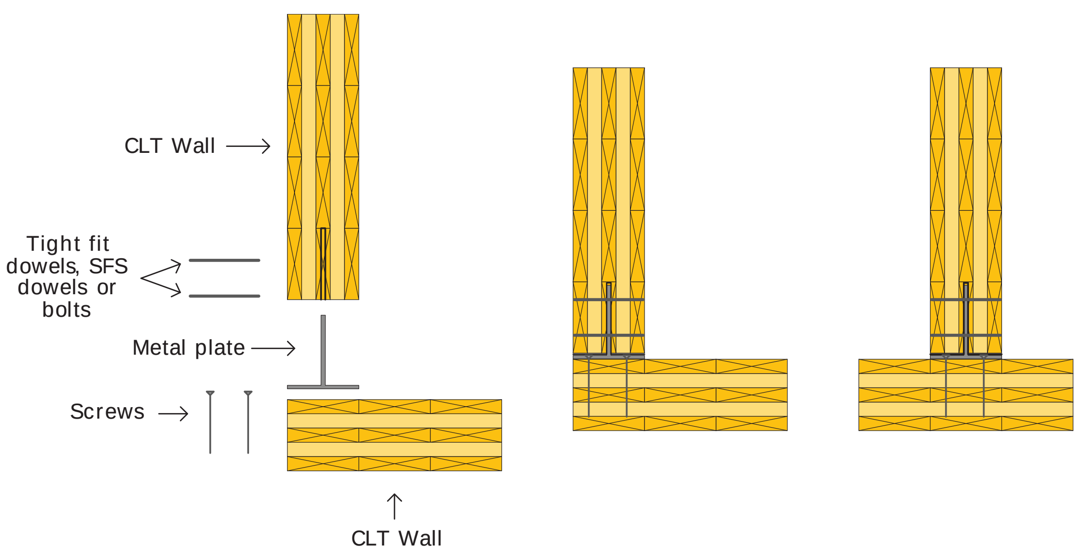


Figure 21
Concealed metal plate

Figure 22
Self-drilling dowel for use through steel and wood

4.3 Wall-to-Floor Connections (Detail C)

Several possibilities exist when it comes to connecting walls to the floors above or connecting walls on the upper storeys to floors, depending on the form of structural systems (i.e. platform vs. balloon), availability of fasteners and the degree of prefabrication.

4.3.1 Platform Construction

4.3.1.1 Self-Tapping Screws

For connecting a floor or a roof to walls below, the simplest method is to use long self-tapping screws driven from the CLT floor directly into the narrow side of the wall edge, as shown in Figure 23. Self-tapping screws could also be driven at an angle to maximize the fastening capacity in the panel edge. The same principle could be applied for connecting walls above to floors below, where self-tapping screws are driven at an angle in the wall near the junction with the floor. Depending on the angle and the length of the screws, the self-tapping screws could reach the bottom walls, further reinforcing the connection between the upper and lower walls and the floor.

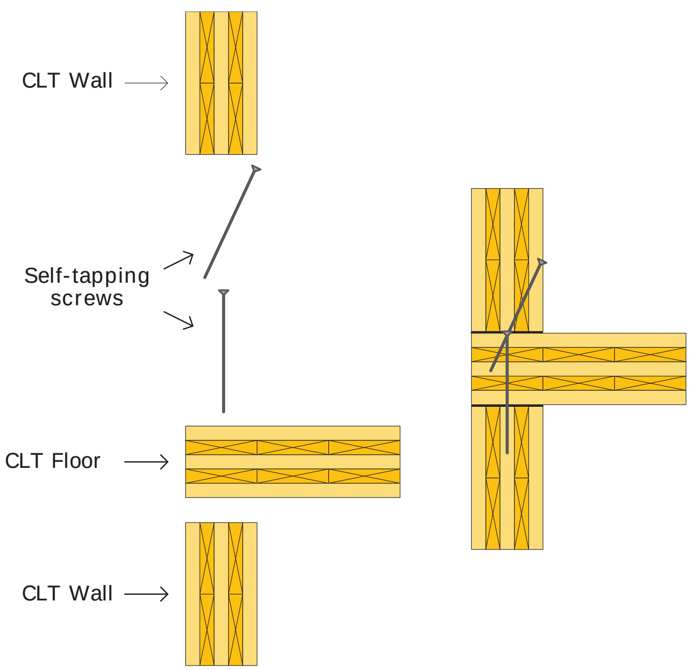


Figure 23
Self-tapping screws

4.3.1.2 Metal Brackets

Metal brackets are commonly used to connect floors to walls above and below. They are also used for connecting roofs to walls. Nails, glulam rivets and wood screws could be used to attach the metal brackets to the CLT panels (Figures 24 and 25).

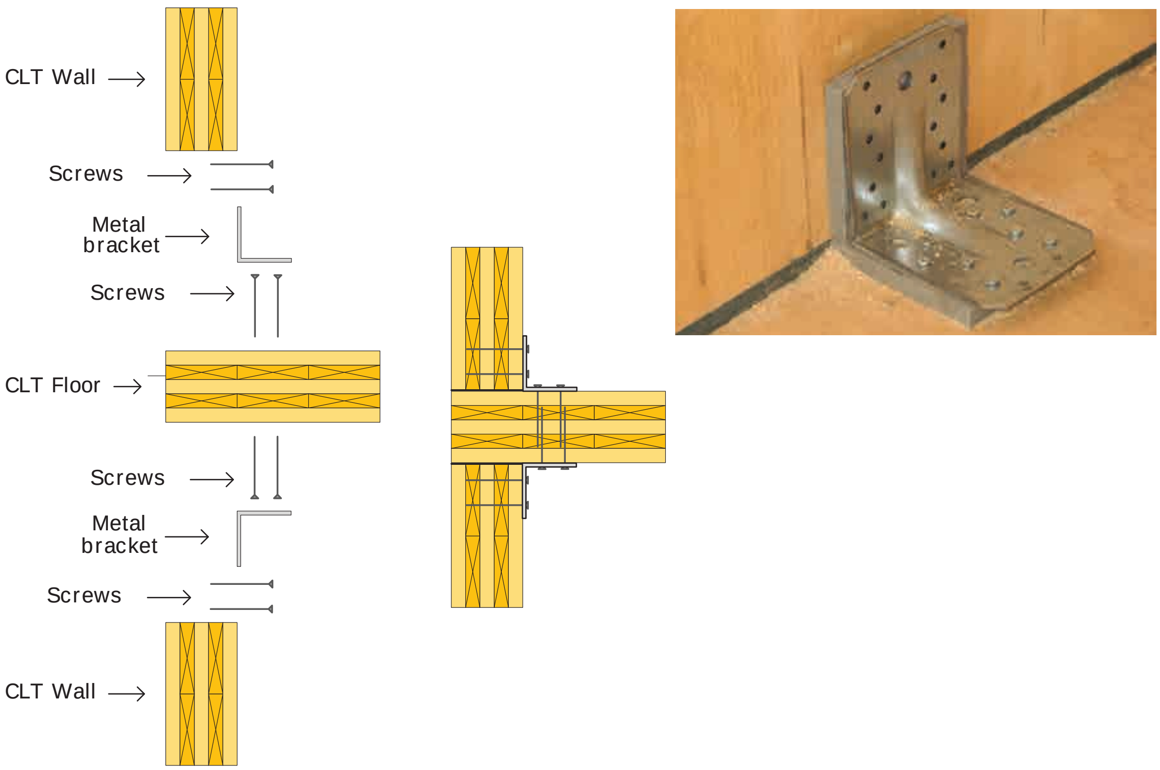


Figure 24
Metal brackets

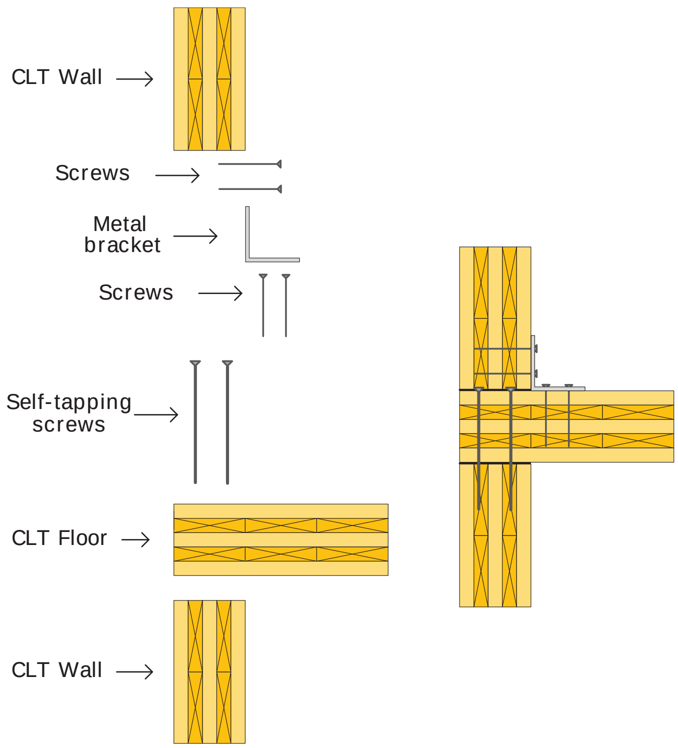


Figure 25
Metal bracket and self-tapping screws

4.3.1.3 Alternative Innovative Systems

This section covers the whole family of innovative fastening systems described above which includes: KNAPP® system, metal shaft connection system with dowels, threaded rod/screw connection system, glued-in rod, wooden profiles and dovetail connection system (Figures 26 to 30). Some of those systems, such as KNAPP®, have a self-locking mechanism to resist against uplift.

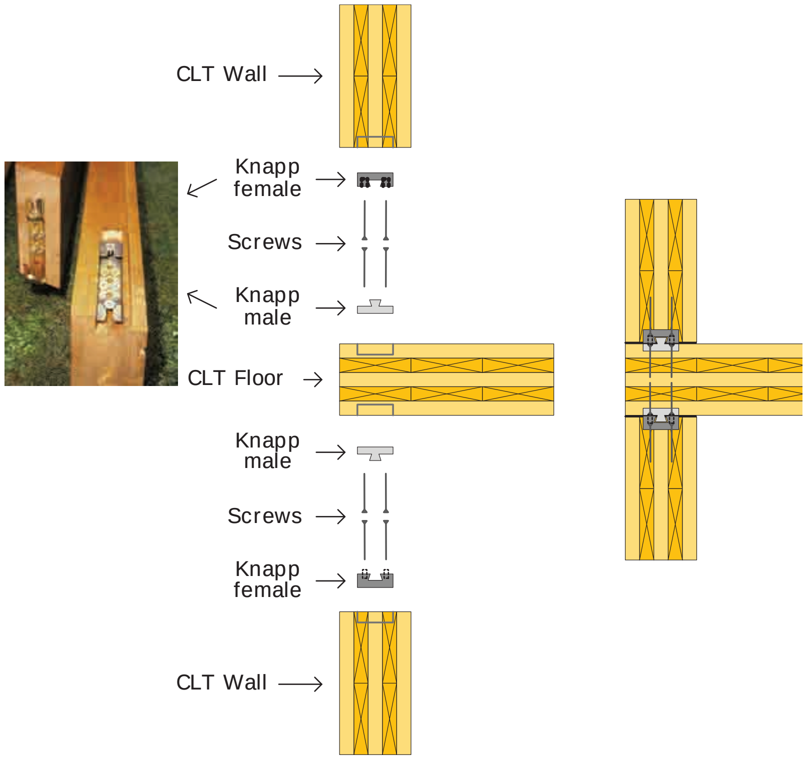


Figure 26
KNAPP® system

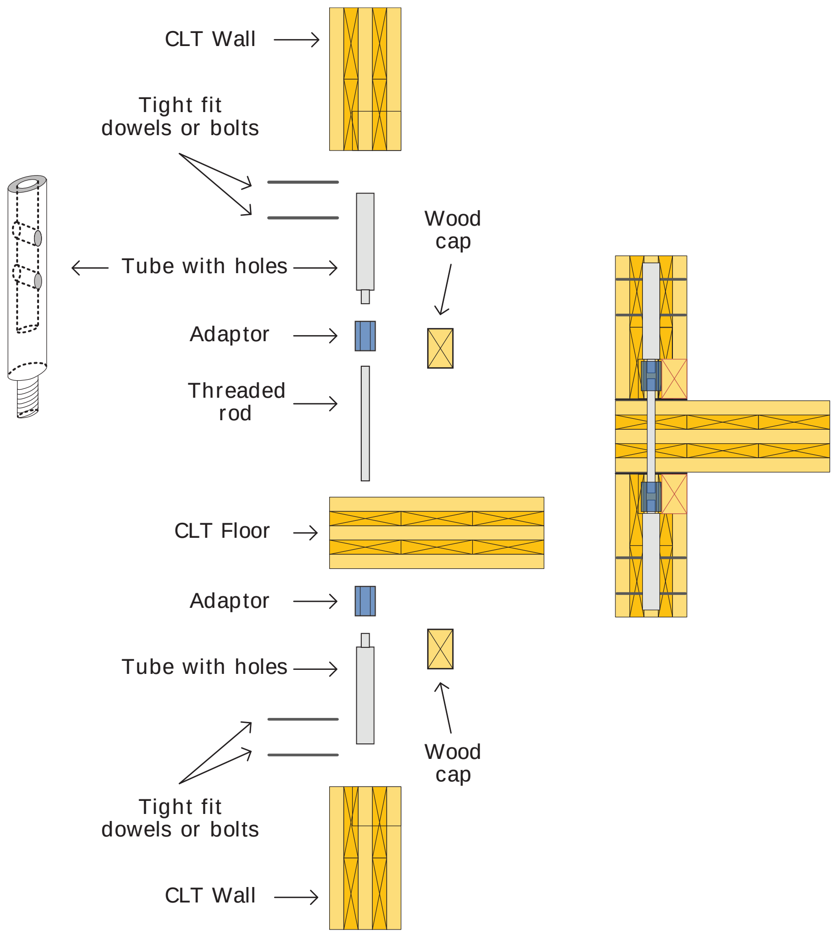


Figure 27
Metal shaft connection details

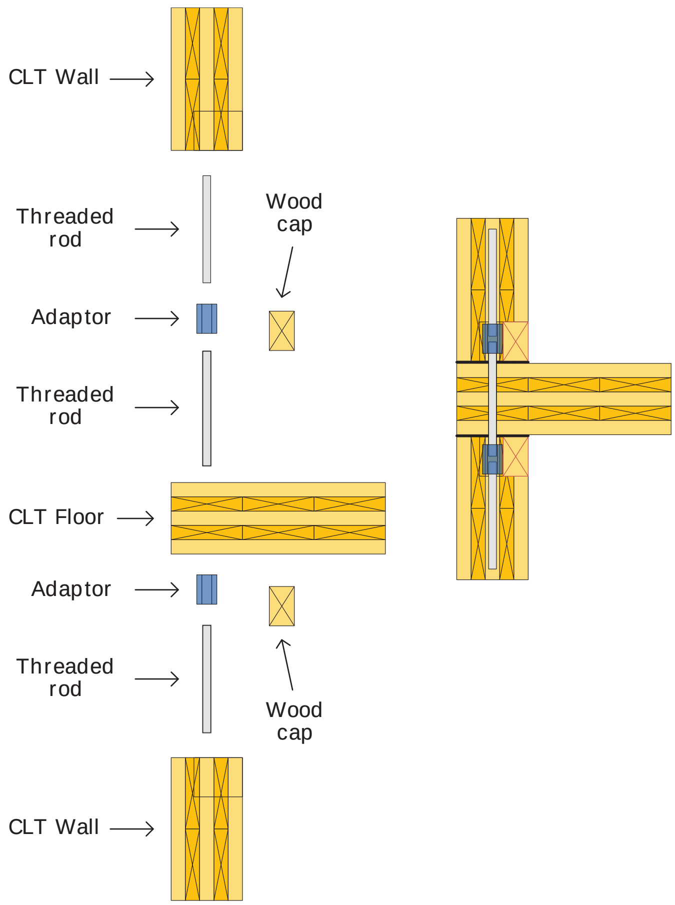


Figure 28
Threaded rod/screw connection system

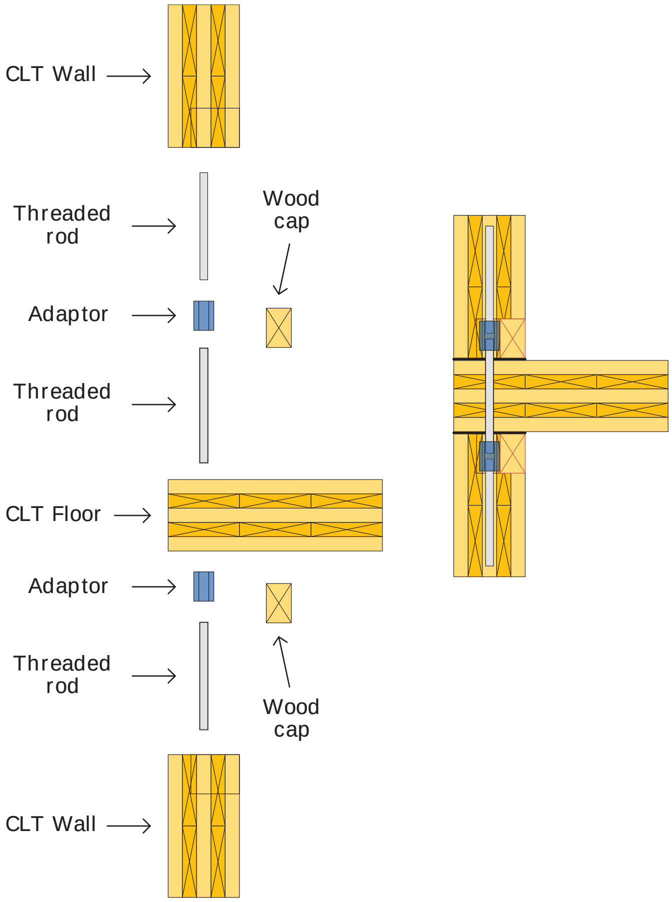


Figure 29
Glued-in rod and edge protecting wooden profile

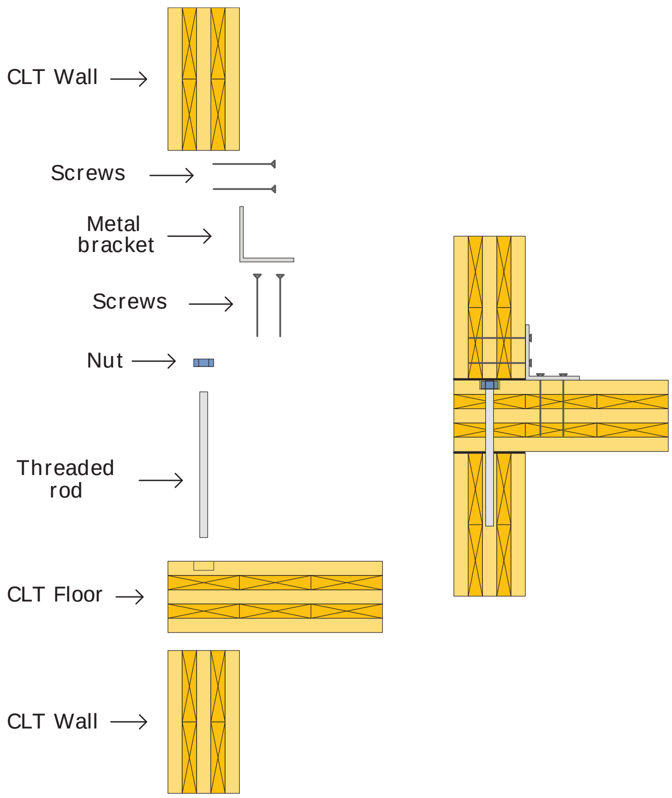


Figure 30
Metal bracket and threaded rod

4.3.1.4 Concealed Metal Plates

Concealed metal plates could also be used to establish wall-to-floor connections (Figure 31). As previously discussed, while this system has considerable advantages over exposed plates and brackets, especially when it comes to fire performance, the system requires precise profiling at the plant using CNC technology.

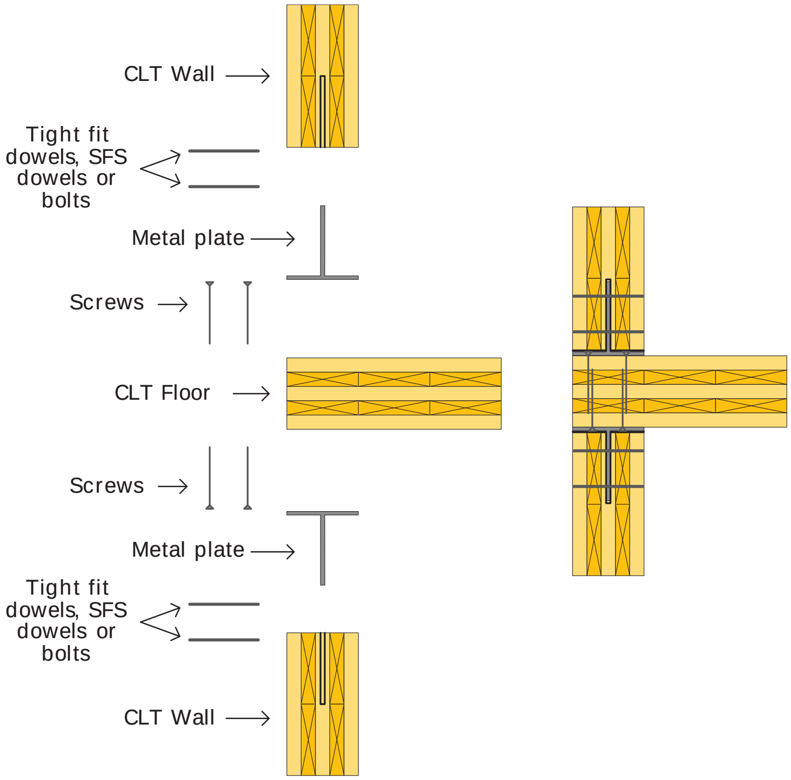


Figure 31
Concealed metal plates

4.3.2 Balcony Details

4.3.2.1 Balcony in Cantilever

For situations where a balcony is designed by extending the floor/roof panel to form a cantilever (Figure 32), the connection between the wall supporting the balcony below and the floor panel can be established using self-tapping screws or metal brackets. In this case, the panels should be installed with the principal axis (parallel to the grain of the outer layers) extending outward and forming the balcony. Self-tapping screws driven at an angle are preferred for improved performance compared to driving screws perpendicular to the plane of one panel into the edge of the other (i.e. the wall panel) (Figure 34b). If a parapet wall on top of the balcony is built, a typical connection detail using self-tapping screws or metal brackets could also be used (Figures 33 and 34). However, caution should be exercised when adopting this system in design as a cantilever due to potential issues related to water infiltration.

Figure 32
Balcony in cantilever

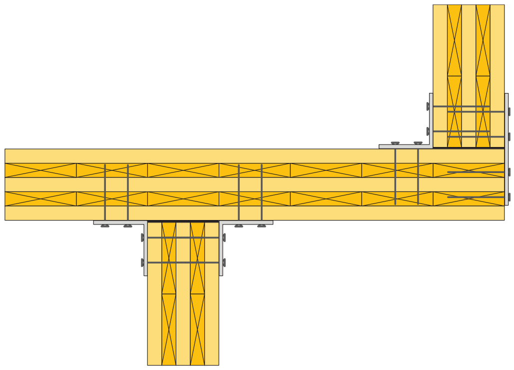
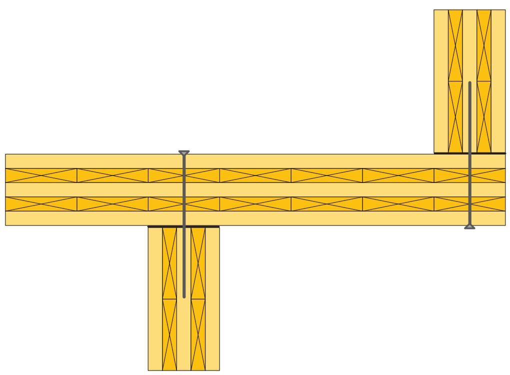
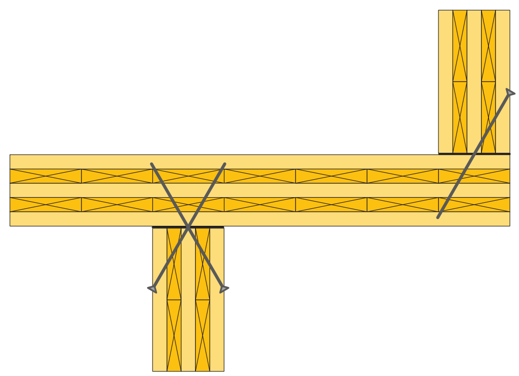


Figure 33
Metal brackets adopted for design of balcony



(a)



(b)

Figure 34
Self-tapping screws used in balcony design

4.3.2.2 Supported Balcony

In some cases, the balcony can be designed to be attached to the main CLT structure using simple fastening systems that allow for easy installation and dismantling (i.e. in case of any potential modification to the configuration of the building in the future) (Figure 35). Several buildings in Europe have been constructed with this type of balcony system. A combination of metal plates and hinges are usually employed to secure the balcony structure/box to the main structure as can be seen in Figure 36. Usually, the balcony is attached to the main building at four (4) points. The connection system is equipped with metal brackets which are attached to the CLT floors (top and bottom floors as can be seen in Figure 36) using self-tapping screws or lag screws. The balcony could be totally prefabricated as a box on the ground, at the construction site, lifted up and then secured to the building at each location/level. Other types of metal attachments could also be used if found proper. The gap between the building and the balcony needs to be closed with cladding materials either as part of the whole building envelope or separately, depending on the end use. Flashing should be installed to divert rain water away from the wall to avoid water accumulation.

For design of the balcony itself, different types of fastening systems could be used. Self-tapping screws alone or a combination of self-tapping screws and metal brackets could be used to attach the floor and roof to the walls.

A variety of other balcony designs could be adopted. One simple concept involves designing the balcony as part of the CLT structure (i.e. built-in). This concept has been used in the design of the Murray Grove building in London, UK, where several corner balconies were introduced as part of the main structure floor plan (Figure 37, left side). This is perhaps the simplest form of creating balconies. Other concepts involve designing and constructing an external structural system (e.g. posts) to support the extremity of the balcony, while the other side of the balcony is supported by the structure itself. This is also common in certain low-rise projects that have been built recently in Europe.

Figure 35
Balcony supported by the main structure

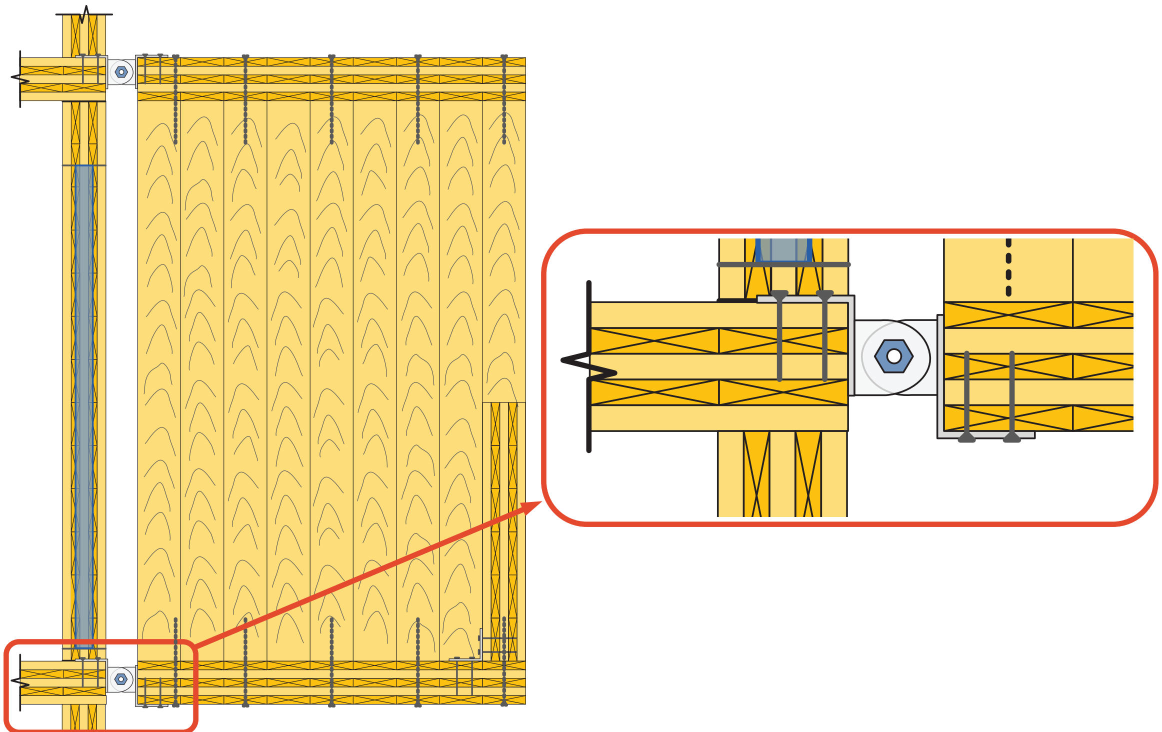


Figure 36
Balcony attached to the platform construction

Figure 37
Examples of European CLT projects with built-in balconies

4.3.3 Balloon Construction

The dominant type of structural form in CLT construction in Europe is the platform type of system due to its simplicity in design and erection. However, in non-residential construction, including farm and industrial buildings, it is common to use tall walls with a mezzanine, which is an intermediate floor between the main floors of a building. Mezzanine floors are often located between the ground floor and the first floor but it is not unusual to have a mezzanine in the upper floors of a building.

To connect a typical CLT floor to a continuous CLT tall wall for such applications, several attachment options exist. The simplest attachment detail includes the use of a wooden ledger to provide a continuous bearing support to the CLT floor panels (Figure 38). The ledger is usually made of SCL such as LVL, LSL or PSL.

CLT ledger could also be used. Another type of attachment is established with the use of metal brackets similar to the one shown in Figure 39 (a and b). Attachment of SCL ledger or metal brackets to the CLT wall and floor panels is established through the use of self-tapping screws, lag screws, nails or glulam rivets.

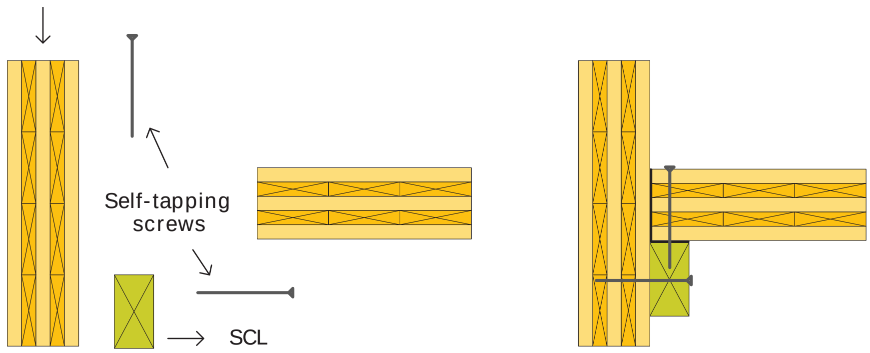
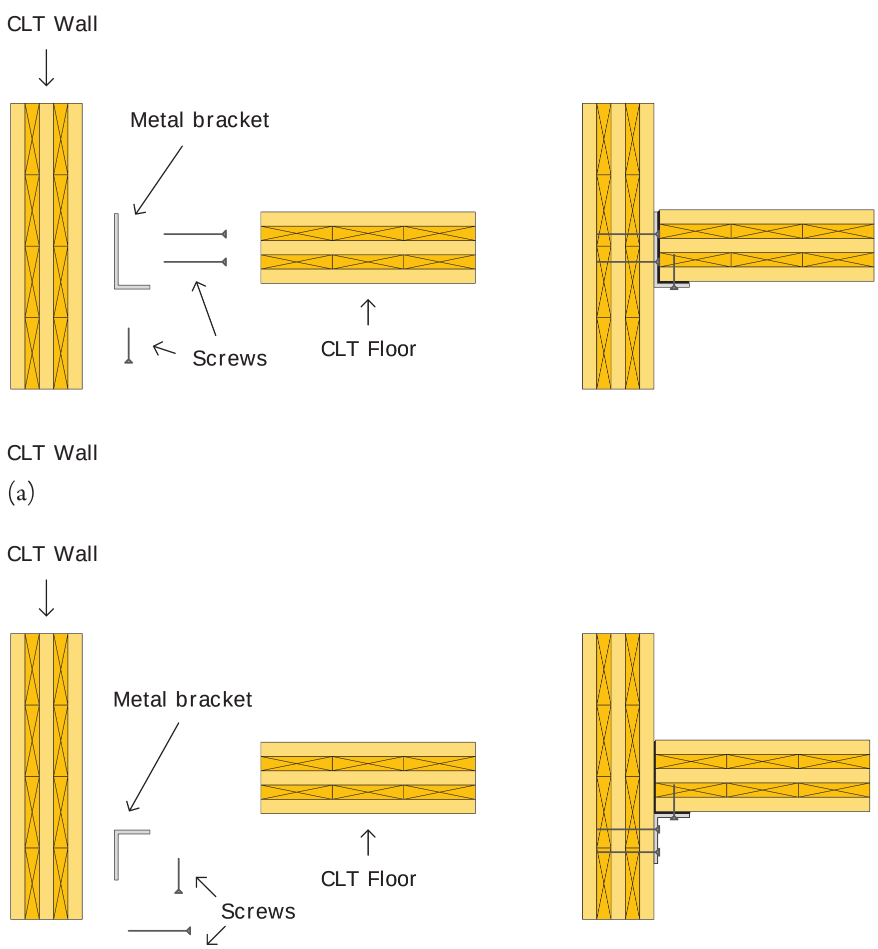


Figure 38
SCL components for bearing support (adapted from TRADA 2009)



(b)

Figure 39
Metal bracket for bearing support (adapted from TRADA 2009)

4.4 Wall-to-Roof Connections (Detail D)

For walls to sloping or flat roof connections, the same type of connection as for attaching floors to walls is used (Figure 40). Self-tapping screws and metal brackets are the most commonly used fastening systems in this application (Figures 41 and 42).

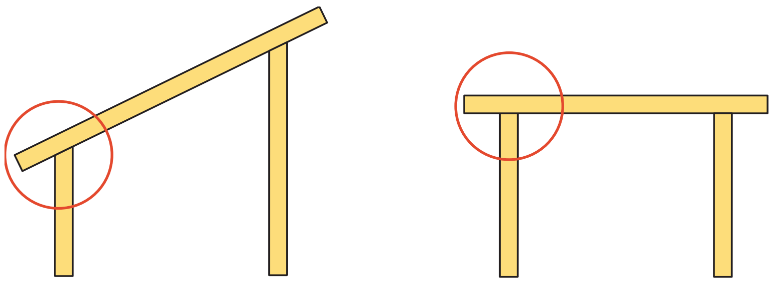
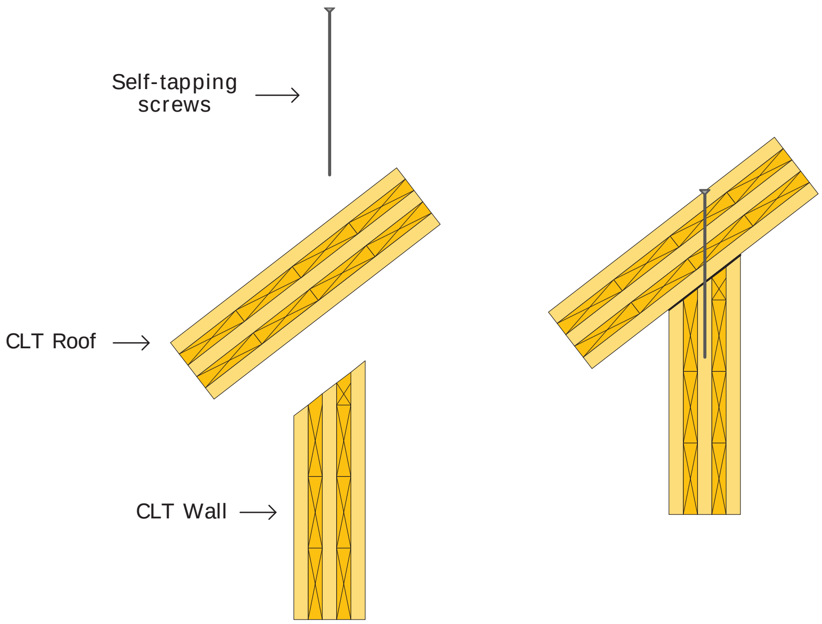
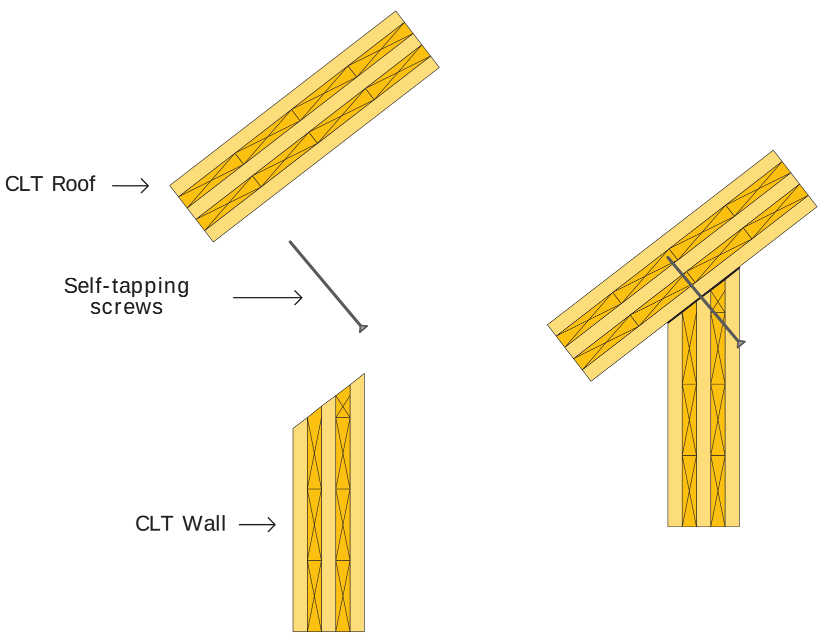


Figure 40
Possible roof-to-wall joints configurations



(a)



(b)

Figure 41
Self-tapping screws

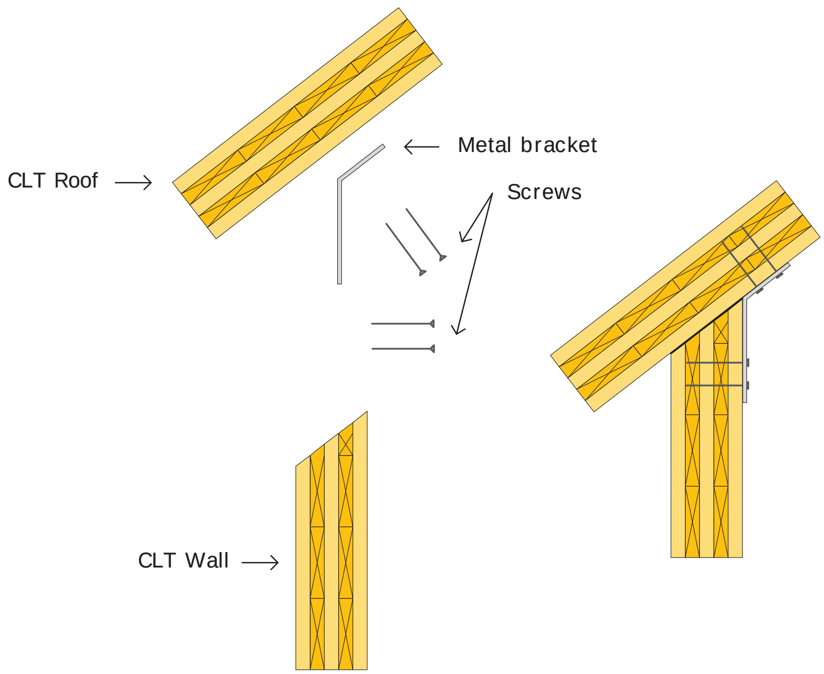


Figure 42
Metal bracket

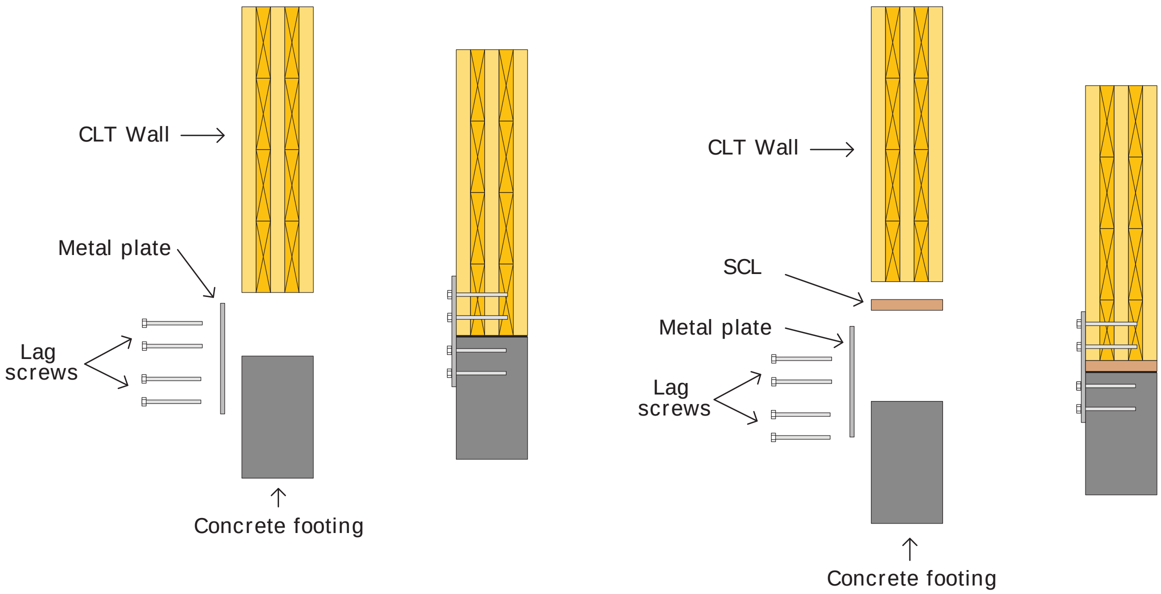
4.5 Wall-to-Foundation Connections (Detail E)

4.5.1 Visible/Exposed Plates

In connecting CLT wall panels to concrete foundations (common for the first storey in a CLT building, with concrete footing or with multi-storey CLT building with the first storey made of concrete) or to steel beams, several fastening systems are available to establish such a connection. Exterior metal plates and brackets are commonly used in such applications as there is a variety of such metal hardware readily available on the market. Exposed steel plates, similar to those shown in Figure 43, are probably the most commonly used in Europe due to their simplicity in terms of installation. When connections are established from outside, then a typical metal plate is used (Figure 43). However, when access is provided from inside the building and where a concrete slab exists, metal brackets such as those shown in Figures 44 and 45 are used. Lag screws or powder-actuated fasteners can be used to connect the metal plate to the concrete footing/slab, while lag screws or self-drilling screws are used to connect the plate to the CLT panel.

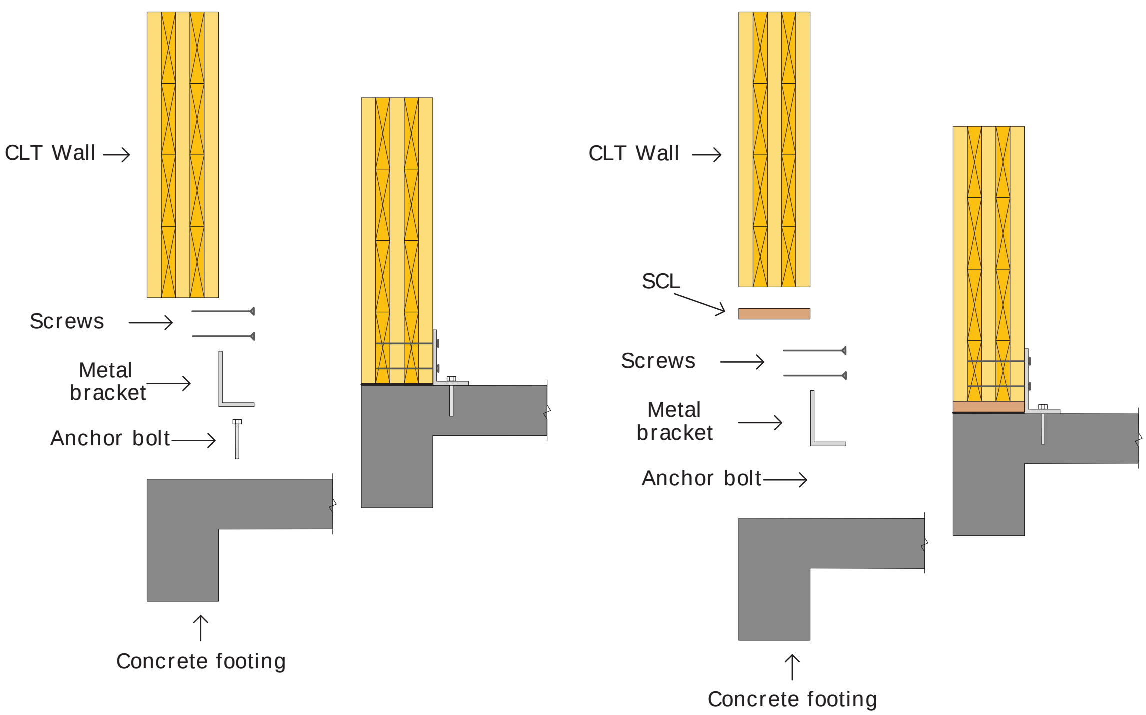
Typically, metal plates or brackets are placed at a 1219 mm interval. But that all depends on the level of load the connection is supposed to resist and its ductility. Different types of metal plates or brackets can be used as shown in Figures 43 and 44, depending on whether the CLT panel is attached to a concrete wall/footing or a slab and whether the plate is attached from the outside or the inside of the wall panel.

To protect wood and improve the durability of CLT panels, a SCL sill plate [or bottom plate] such as that shown in Figures 43b and 44b is installed between the concrete foundation and the CLT panels. This also simplifies assembly.



(a) (b)

Figure 43
Exterior metal plate



(a) (b)

Figure 44
Metal brackets

Figure 45
Metal brackets installed on site

4.5.2 Concealed Hardware

To achieve better fire performance and improve aesthetics, designers prefer to conceal connection systems. Hidden metal plates similar to those shown in Figure 46 can be used, but they require some machining to produce the grooves in the CLT panel to conceal the metal plates. Tight dowels or bolts could be used to attach the plates to the CLT panel. However, precise CNC machining is required in some cases. Some innovative types of fasteners that can be drilled through metal and wood (e.g. WF series of dowels from SFSIntec do not require any predrilling or CNC machining) or other types of screws that can penetrate through both materials can also be used for this purpose.

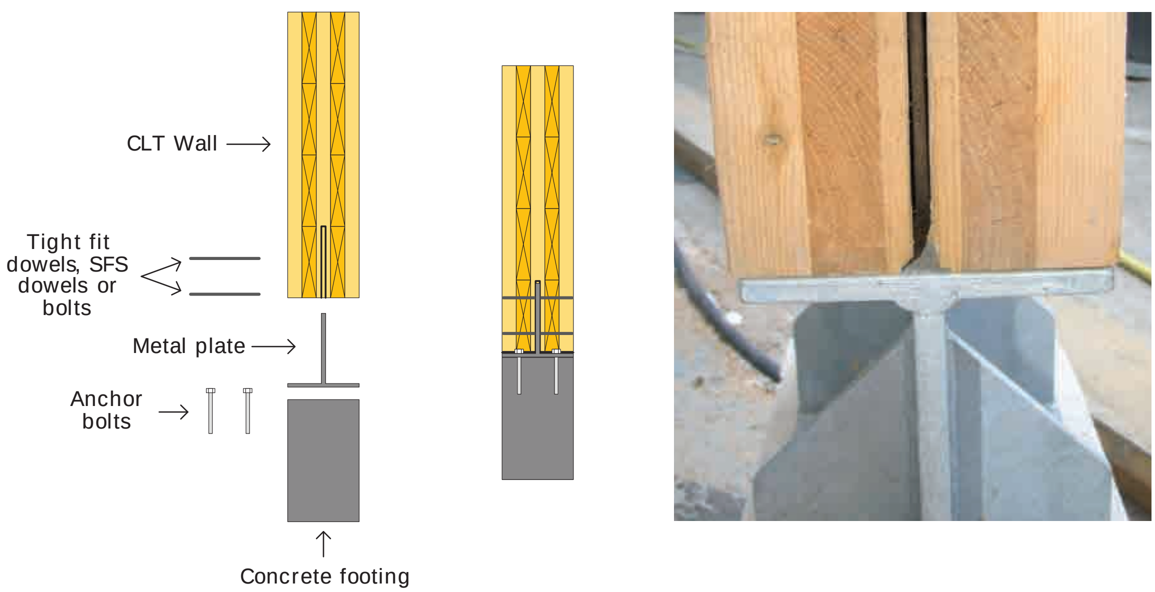


Figure 46
Concealed metal plates

4.5.3 Metal Shafts

Another option for connecting CLT wall panels to concrete foundations is to use a hollow small diameter metal tube/shaft with threaded ends (Figure 47). Holes are predrilled in the edge (narrow side) of the panel element to accommodate the metal shaft, which is fixed inside the panel using small diameter dowels or bolts. Epoxy could also be used to attach the metal shafts to the panel in the plant. The panels arrive at the construction site already equipped with the shafts to minimize work on site. Threaded anchor bolts cast in the concrete foundations are connected to the shaft's threaded end using a nut adaptor. Usually, a small access hole in the panel is drilled to enable connection between the adaptor and the threaded anchor bolt. A wooden cap is used to cover the access hole and the shafts, making this a completely concealed, fire protected connection. The actual detail depends on the magnitude of design service loads that the wall panel will resist and the panel configurations (such as window and door openings).

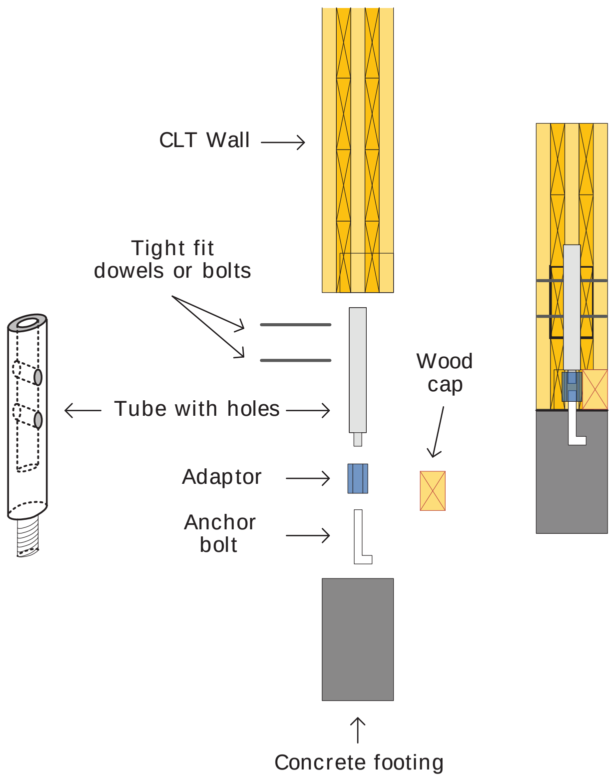


Figure 47
Metal shaft connection details

4.5.4 Threaded Rod/Screw

Like the metal shaft connection system, this system utilizes long threaded rods/screws similar to what is being used for transverse reinforcement of large glulam beams/arches against tension stresses perpendicular to the grain. One particular threaded rod/screw produced by SFSIntec, called "Wood Bar", is suitable for this application (Figure 48). The long threaded rod is screwed in the end grain of the panel element. The panels arrive on site equipped with an adaptor. The installation process is similar to that described for the metal shaft connection system.

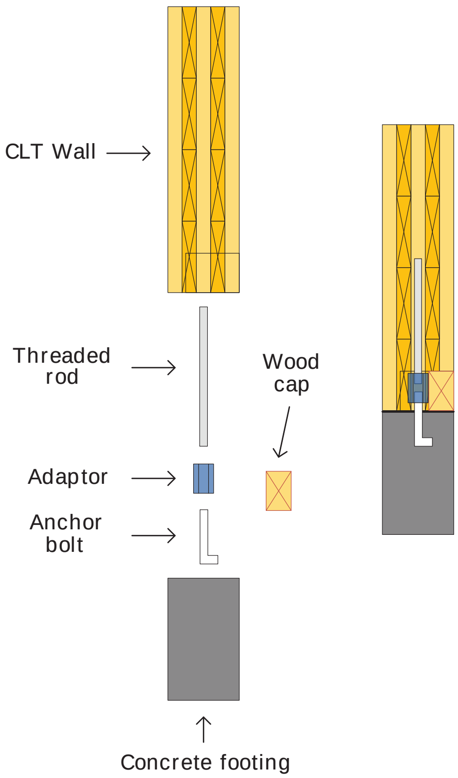


Figure 48
Threaded rod/screw connection system

4.5.5 Wooden Profiles

Wooden profiles are commonly used in connecting structural insulated panels (SIP) and other types of prefabricated wood-framed walls. It is important that such wooden profiles are fabricated from high density and stable materials. Engineered wood products or hardwood can generally be used for this purpose. The major advantage of this system is the ease of assembly. The wooden profiles are typically attached to CLT panels with wood screws or self-tapping screws. Structural adhesives are also used, sometimes in combination with mechanical fasteners since the wooden profile is installed in the plant. They are often used in combination with metal plates or brackets to improve the lateral load resistance as can be seen in Figure 49. CNC machining is needed at the CLT plant to produce the profiles in the panels. The use of wooden profiles is not limited only to wall to foundation applications. They can also be used for wall-to-wall or floor-to-wall connections. The wooden profiles could take several forms, as shown in Figure 49, to provide additional protection and reinforcement to the bottom edge of the panel.

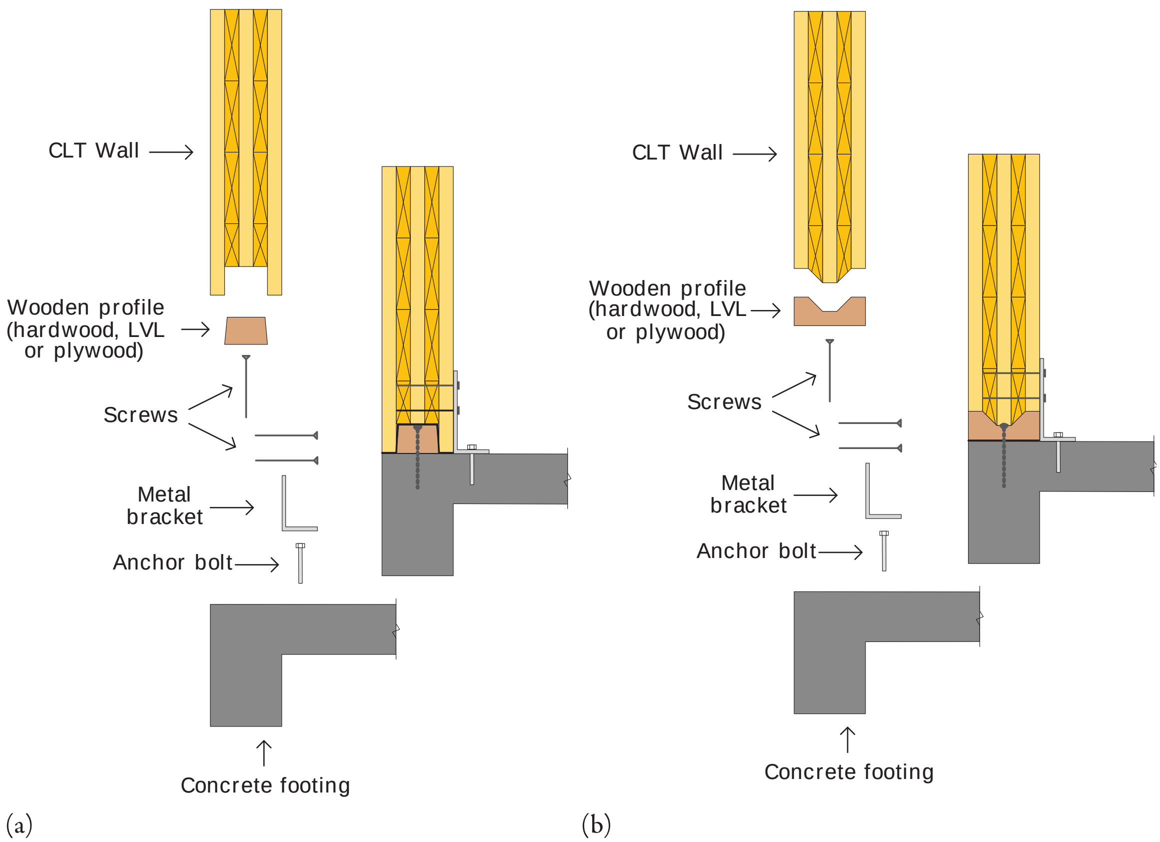


Figure 49
Concealed (a) and exposed (b) wooden profiles

4.5.6 Alternative System

While this system is more suited for use in wall-to-wall connections, it may also be suitable for wall-to-foundation connections. The connection between the concrete foundation and KNAPP® bracket could be established through lag screws or powder-actuated nails (Figure 50). It would be preferable to use galvanized components to prevent corrosion as a result of water condensation at the interface with concrete.

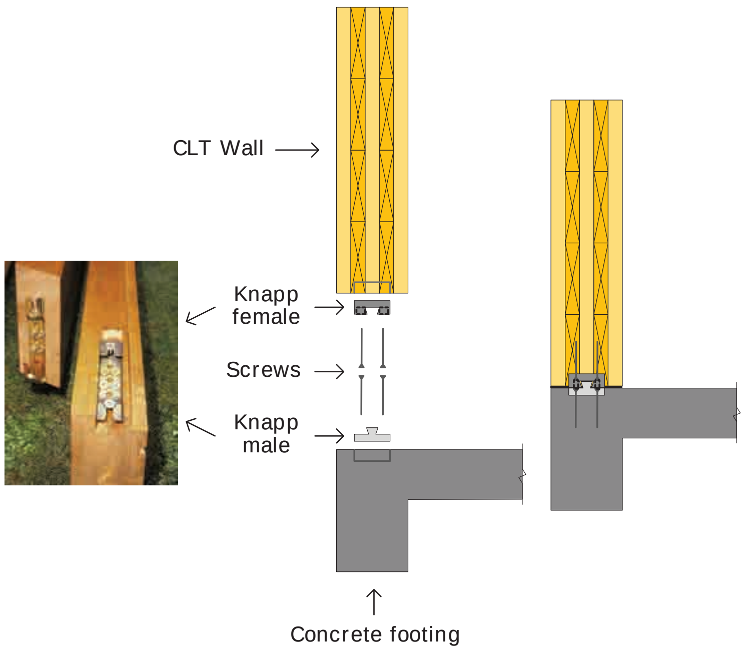


Figure 50
KNAPP® Gigant system

5 CONNECTIONS IN MIXED HYBRID CLT CONSTRUCTION – DETAILS

Mixed systems using CLT with other types of wood-based materials such as glued-laminated timber (glulam) are common. Those mixed systems are becoming increasingly popular in Europe as a way to optimize the overall design by capitalizing on the positive attributes of the various products. Mixing CLT with other types of construction materials such as concrete and masonry or mixing different types of structural forms is also common.

5.1 Mixed CLT with Other Wood-Based Materials and Systems

In CLT assemblies, mixing different wood-based materials and structural systems is done in such a way to optimize the design and to meet certain performance requirements. Therefore, it is not unusual to combine CLT wall assemblies with joisted floor systems using glulam, wood I-joists, metal plated wood trusses or other types of engineered wood elements as the main floor support system, with either wood-based decking such as wood boards or structural panels. The following provides a brief summary of potential structural forms where CLT and other types of wood-based materials could be combined. Connection systems between those different materials are described.

5.1.1 Platform Construction

For platform-type construction, the main structural supporting elements of the floor system rest on top of the walls below. In mixed construction where walls are made of CLT panels, typical joisted floor system is placed on top of those walls as can be seen in Figures 51 and 52.

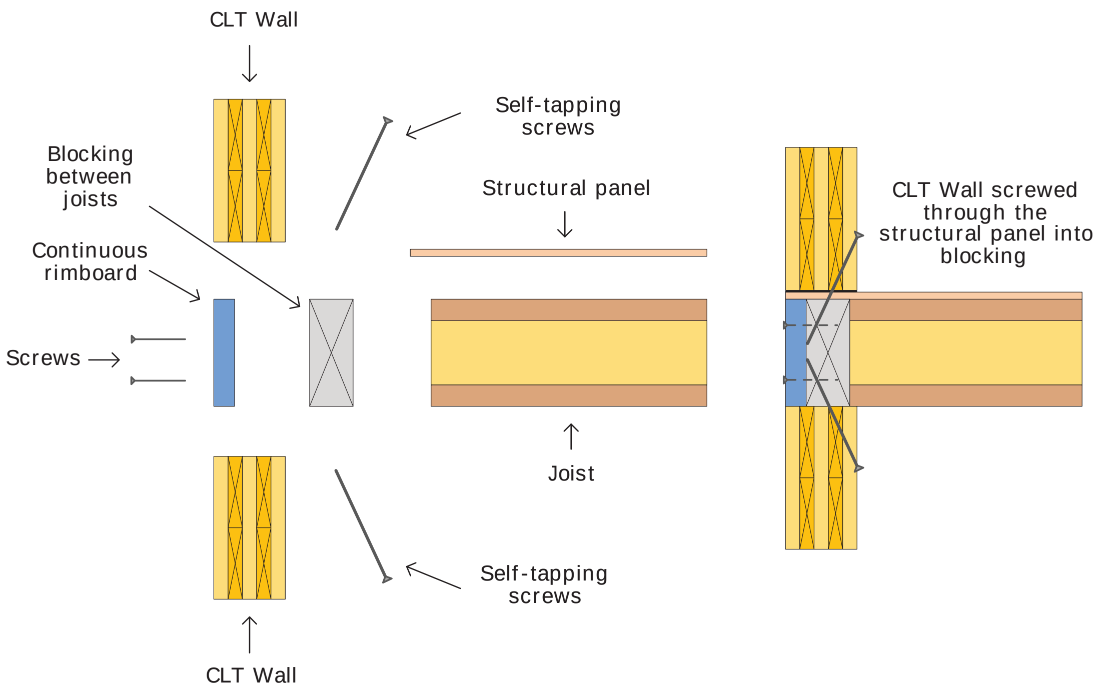


Figure 51
CLT Wall – I-joist (adapted from TRADA, 2009)

A combination of rimboard and blocking elements made of SCL such as PSL, LVL or LSL between joists is generally used to ensure transfer of vertical loads from storeys above to the CLT wall below. Differential shrinkage is not an issue here as next storey CLT walls are resting completely on the rimboard and the blocking elements. Typical solid sawn lumber or SCL such as wood I-joists could be used as the main structural systems supporting the subfloor. In the case of wood floor trusses, it is necessary to provide wood-based blocking to prevent localized crushing of truss top chords and to have a uniform stress distribution along the wall perimeter (Figure 52). The wood blocking should be made of SCL for better deformation properties and for dimensional stability.

Connection between walls above and below can be established using self-tapping screws driven at an angle or through one of the alternative methods of fastening described above.

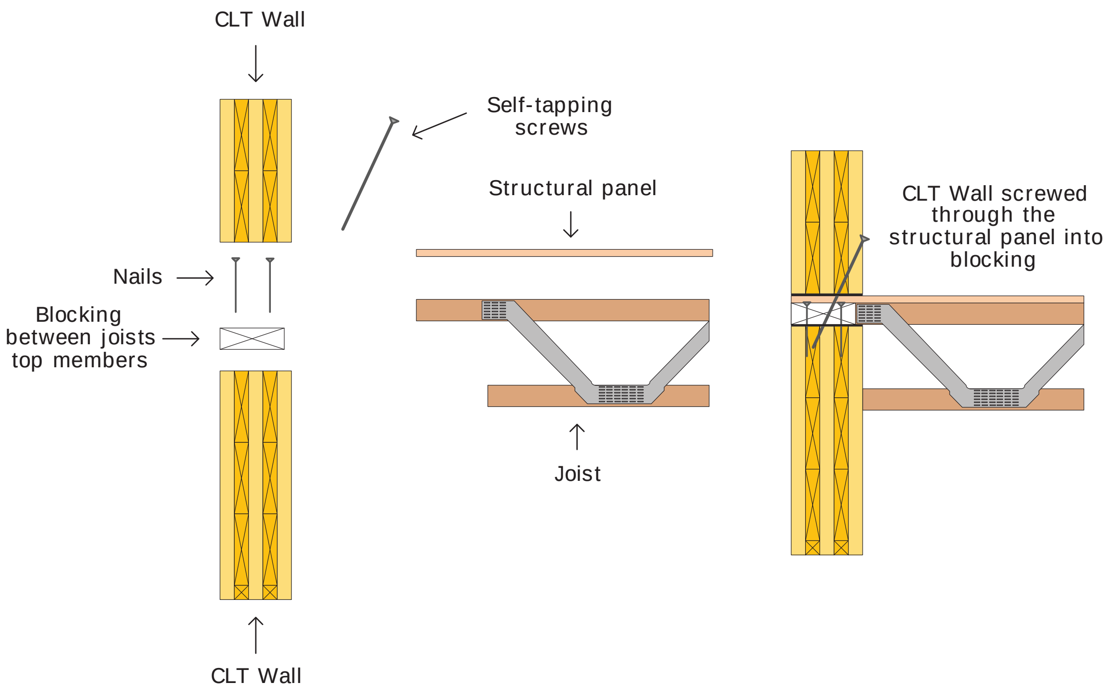


Figure 52
CLT Wall – Metal plated floor truss (adapted from TRADA 2009)

5.1.2 Balloon Construction

Mixed CLT construction could also be used in buildings with a balloon structural form. In this type of construction, the joisted floor system which incorporates a variety of joist products such as sawn lumber, wood I-joists, and SCL can be attached to the CLT walls using traditional metal hangers commonly used in light-frame and heavy post-and-beam timber construction (Figures 53). The wall panels are continuous at the connection between the floor system and the wall and it provides support to the floor system.

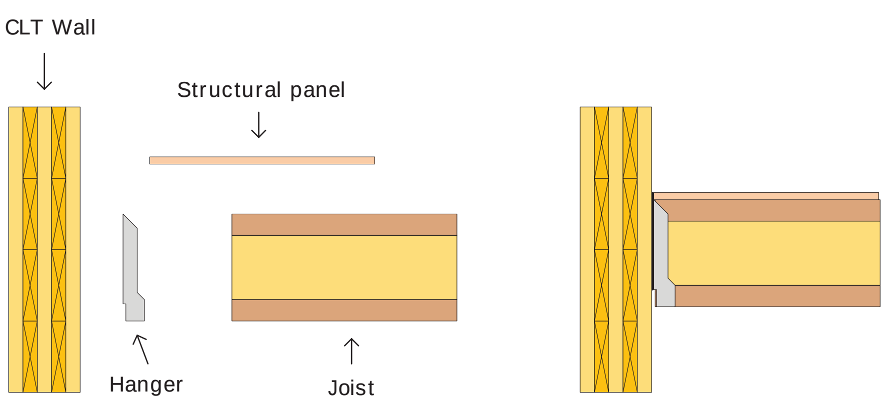


Figure 53
CLT Wall – I-joist (adapted from TRADA 2009)

6 DESIGNING CONNECTIONS IN CROSS-LAMINATED TIMBER

6.1 Why Connections in CLT are Different than Those in Solid Timber or Glulam

CLT is usually made of laminated lumber boards along the strong axis of the panel and crosswise. The cross lamination and the built-up nature of the panel, in addition to certain unique panel features such as edge-gluing (or lack of it) and the presence of grooves sawn into the boards to relieve drying stresses, further complicate the determination of the fastening capacity in CLT compared to traditional sawn solid lumber or SCL. Panels from some manufacturers are produced with gaps between the longitudinal boards as big as 6 mm.

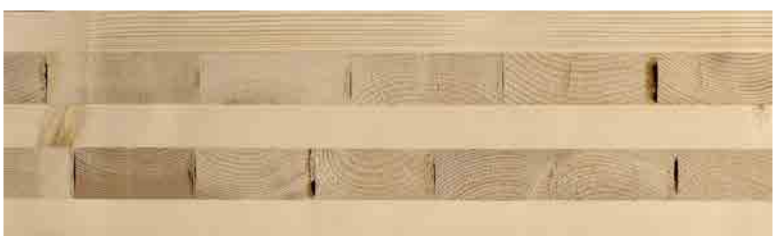


Figure 54
CLT panel section with gaps and grooves sawn in the timber to relieve shrinkage stresses

It is well established that the loading direction relative to the grain direction of wood affects the fastening capacity when relatively large diameter fasteners (> 6 mm diameter) such as bolts, lag screws and large diameter long self-drilling screws are used. The embedment strength of slender fasteners in wood such as nails and small wood screws is less sensitive to grain direction. Timber design standards such as CSA O86-09 (CSA 2009) specifies different embedment formulae for connections in timber loaded, either in the direction parallel or perpendicular to grain for bolts and dowels exceeding 6 mm diameter. CLT manufacturers in Europe are well aware of the fastening issues and rigorous testing programs were established to develop the fastening capacity in their products for different dowel-type fasteners. Ultimately, embedment formulae specific to CLT panels need to be developed in order to establish the lateral load resistance for fasteners such as screws, bolts and lag screws, taking into account the nature of lamination, lay-up, species, edge-gluing or lack of it, and other panel specific features. Similarly, the withdrawal resistance of fasteners such as screws and nails from the face and edges of the panel needs to be developed.

While yielding failure modes in accordance with European Yield Model (EYM) are the dominant type of failure for slender type of fasteners in CLT (Figure 55), there is a potential for developing brittle failure modes in CLT such as row shear, group tear-out, tension or splitting (Figure 56). However, it is less likely that such brittle failure modes will develop with fasteners driven perpendicular to the plane of the panel. But with fasteners driven in end grain, it is possible to trigger splitting due to tension stresses perpendicular to the grain in small thickness panels when fasteners are loaded in shear. Therefore, there is a need to establish the conditions where brittle failure modes may occur with large diameter fasteners used with CLT. According to tests conducted by Uibel and Blass (2006) in Europe with dowels and screws loaded perpendicular to the plane of the panel, the connections exhibited considerable ductility. Even when plug shear or splitting occurred in the outer layers, the load remained at the same level or showed a localized marginal drop. This could be attributed to the reinforcement effect provided by cross lamination in CLT. However, this finding is limited to the tested configurations.

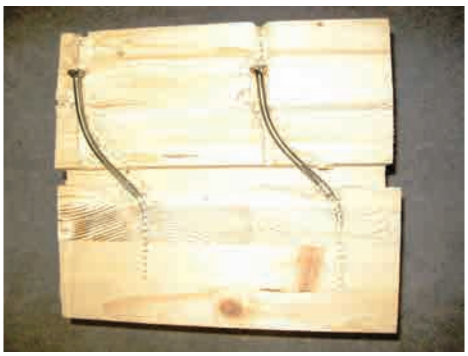


Figure 55
Ductile failure modes experienced during testing of self-tapping screws in CLT half-lapped connections

6.2 Current European Design Approach for Connections in CLT

Extensive research has been conducted in Europe to evaluate the fastening capacity of different types of fasteners in CLT. Comprehensive research on the fastening capacity of CLT connections was conducted by Uibel and Blass (2006, 2007). The shear capacity of traditional fasteners in CLT was studied by the authors with the intent of developing a calculation methodology to establish the load carrying capacity of connections with dowel-type fasteners in the direction perpendicular to the CLT panel and on their narrow side (i.e. edge joints). Embedment tests were conducted using different types of CLT products and dowel-type fasteners. Empirical models expressed as a function of the fastener diameter, wood density and loading angle relative to the grain direction of the surface lamina were developed based on test results to establish the embedment strength under lateral loading. Different models were developed for each of the different types of dowel-type fasteners (i.e. nails, screws and dowels). Once the embedment strength properties were established, the load carrying capacity in accordance to Johansen's yield model (EYM) could be determined. However, the validity of these models was limited to a maximum thickness of lamina and thickness ratio of the longitudinal and cross layers (Uibel and Blass, 2006).

Withdrawal strength of self-tapping screws, typically used in connecting CLT panels perpendicular to the plane of the panel or in the panel edges, was also investigated by Uibel and Blass (2007). The withdrawal resistance was derived from tests using self-tapping screws with diameters ranging from 6 mm to 12 mm. The location of the screws was selected in such a way to have them installed at the joint between two boards within a lamina, or between one lamina and another. The derived withdrawal resistance was expressed as a function of the screw diameter, wood density and the screw point side length of penetration. It is important to note that when the withdrawal capacity of a fastener is determined in the narrow side of panel, the input characteristic density value should be that of the lamina/ply in which the fastener is driven, not that of the whole panel. Validation tests were performed and a reasonable correlation was found between tests and predictions. The tests were also used to rationalize the required spacing and end and edge distances. However, a more generalized and simplified approach using the overall panel density would be recommended. Long-term lateral and withdrawal tests using self-tapping screws in end joints are being conducted by the authors to determine the long-term behaviour under changing environmental conditions. However, no results have been published yet.

6.3 Could CSA O86 Design Provisions be used for Design of Connections in CLT?

6.3.1 Current Design Philosophy for Dowel-Type Fasteners in CSA O86-09

The design methods for timber connections should be able to capture all potential failure mechanisms that can occur and be able to assign strength and deformation capacities to any of these failure modes. Fortunately, recent editions of the Canadian timber design code "Engineering Design in Wood" (CSA O86-09) provides a design methodology for bolted and dowelled connections that gives designers control over the type of failure mode the connections will experience at the design stage. The designer needs to verify both the yielding (ductile) and the brittle capacities of the connection, and the minimum of the two controls the design value. The yielding failure modes are based on Johansen's Yield Equation Model (sometimes referred to as the European Yield Model, EYM), where ductile failure modes could occur due to the crushing of the wood in bearing and/or yielding of the fastener. Typical brittle failure modes in heavy timber construction include: row shear, group tear-out, tension at the reduced section (i.e. where bolt holes are drilled), and splitting for loading perpendicular to grain (Figure 56). Detailed information on these types of brittle failure in bolted and dowel-type timber connections can be found in Quenneville and Mohammad (2000).

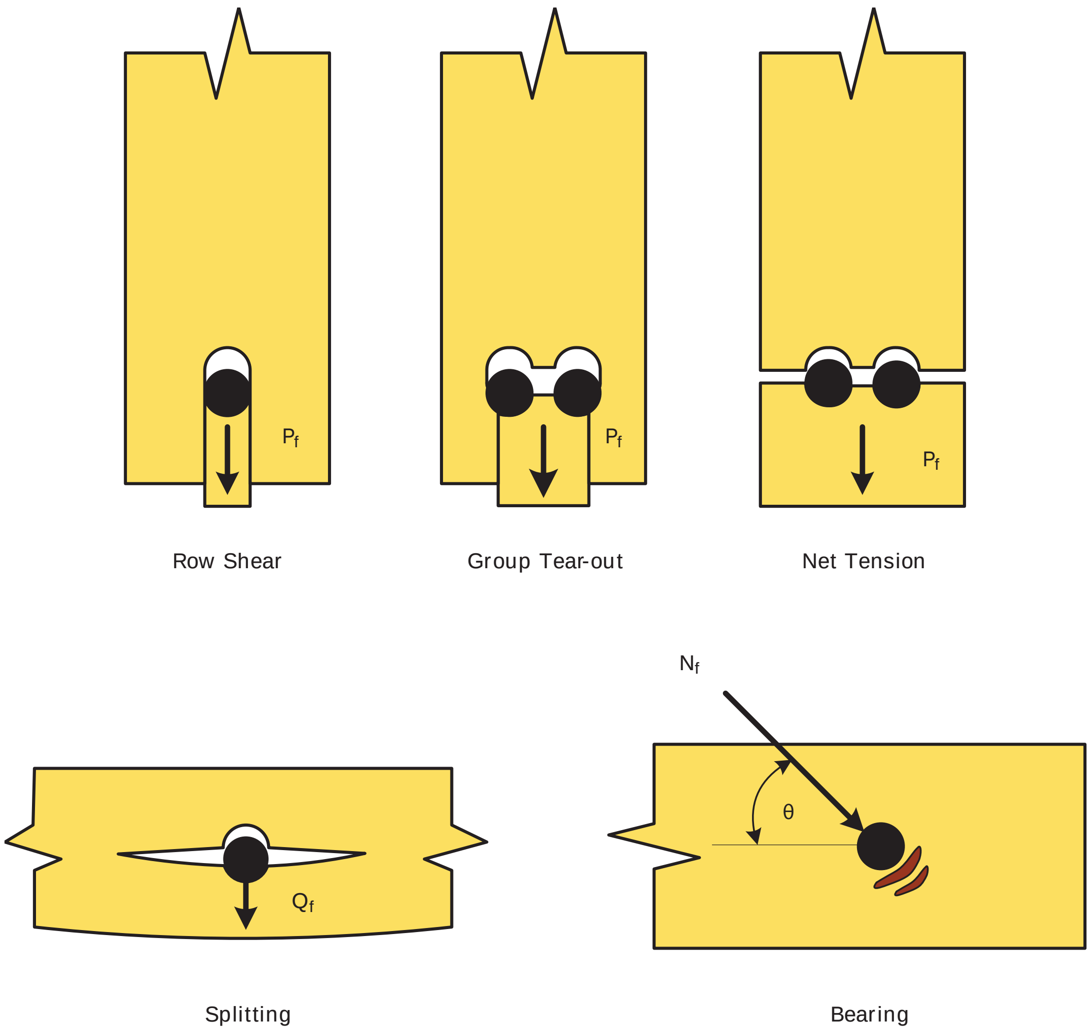


Figure 56
Possible failure modes in traditional solid timber or glued laminated timber

Generally, the type of failure mode that a timber connection with a dowel-type fastener could experience depends on several parameters including:

- connection geometry (loaded and unloaded end and edge distances, row and bolt spacing, type of connection);
- wood member thicknesses;
- fastener diameter and yield strength;
- wood basic mechanical and physical properties; and
- loading direction relative to grain orientation.

Ductile failure modes in CSA O86-09 are expressed as a function of the embedment strength of the mechanical fastener or dowel in the side or main wood-based member and in the steel side plates, the yielding strength of the fastener, members thicknesses and fastener diameter. Embedment formulae based on extensive research by European and North American researchers were developed for the different types of wood-based materials and loading directions relative to grain. Embedment strength formulae for wood-based connection members in CSA O86-09 are usually given as a function of wood-based material density and fastener diameter. Most of design provisions in timber design standards such as the National Design Specification (NDS) (AF&PA 2006) for timber construction in the USA and Eurocode 5 (EN 2004) adopt the European Yield Model concept for the design of dowel-type connections in timber. One set of embedment equations are typically given for slender fasteners such as nails and wood screws for both loading directions (i.e. parallel and perpendicular to grain). However, for large diameter fasteners, two sets of embedment equations are provided.

Transition from one failure mode to another at the design stage could be achieved through the choice that the designer makes regarding one or a combination of the above parameters. For example, smaller loaded end and edge distances and spacing between fasteners in a row and between rows will most likely trigger brittle failure modes. Therefore, if designers would like to maximize the connection ductility, it is important to maximize loaded end and edge distances and fasteners spacing and/or to use a large slenderness ratio if possible. The type of brittle failure mode (such as row shear or group tear-out) for a connection with multiple rows is mostly determined by the row spacing and the spacing of fasteners in a row. Smaller row spacing will result in a situation where group tear-out capacity will govern. However, larger row spacing will increase the group tear-out capacity and trigger a row shear failure mode. The designers can modify their connection configuration to give the desired balance between ductility, and capacity.

6.4 Application of Current CSA O86-09 Design Provisions to Connections in CLT

Similar to some modern SCL products such as LSL, PSL and LVL with cross layers, which have fully or partially cross-aligned wafers or strands that can overcome the traditional problems associated with splitting of the wood, CLT has a more favourable ability to resist splitting in simple lap joint applications due to the cross lamination. Therefore, it is generally expected that higher capacity for splitting could be achieved in CLT compared to solid timber.

Within the context of CSA O86, if the embedment strength properties of dowel-type fasteners are established in CLT in the direction perpendicular to the plane of the panel and in the narrow side (edge), then it would be possible to evaluate the ductile lateral capacity in this product following current design provisions in CSA O86-09. Yield model equations as given in CSA O86-09 would be applicable. However, due to the grain orientation relative to the load, it would be necessary to incorporate the proper embedment strength properties/equations for parallel and perpendicular layers in those equations. While a single set of embedment equations will suffice for slender type dowel fasteners (≤ 6 mm) for both parallel and cross layers, separate embedment equations will be needed for large diameter dowel type fasteners. In CSA O86-09, two sets of embedment equations are given for large fasteners loaded parallel or perpendicular to the loading direction. The modified yield equations will take care of the layer orientation and the relative thickness of the layers. A calculation procedure/model has been proposed by Uibel and Blass (2006) to determine the load carrying capacity of dowels in a steel-to-solid-wood-panel connection with an inner steel plate. While direct substitution for embedment properties equations derived for the parallel and cross layers can be made for ductile failure modes (i.e. failure modes a, b and c in CSA O86-09) that involve crushing of wood, the failure modes where plastic hinges are developed require further analysis (e.g. failure modes d, e, f and g).

Connection configuration and geometry that governs the ductile capacities of connections in CLT for dowel-type fasteners need to be established as well. This includes end and edge distances, fastener type, row spacing and slenderness ratio. Depending on the type of dowel fasteners, it is expected that, as a minimum, the current minimum requirements for end and edge distances and for fasteners and row spacing in solid sawn lumber and glulam, as given in CSA O86-09, could be applicable to CLT for the relevant dowel-type fasteners such as nails, wood screws, lag bolts and lag screws. Attention should be given, however, to specific CLT panel features that could affect the connection capacity such as gaps and grooves, which may reduce the embedment strength due to localized weaknesses consecutive to those fabrication features, as discussed above.

Currently, long self-tapping screws (commonly used in Europe for the assembly of CLT panels) with diameters greater than 6 mm are not covered under the current design provisions in CSA O86-09. However, the ductile lateral resistance of bolts, lag screws, wood screws (up to 6 mm diameter), nails and rivets in CLT can be designed following the existing provisions, provided the appropriate embedment strength properties of such fasteners in CLT are established. Laterally loaded fasteners that do not bear on the full cross-section of the CLT have a potential for brittle failure. For example, a group of connectors at the end of a face ply that is not edge-glued will need to rely on transferring the tension force into the CLT panel by rolling shear (Figure 57). As discussed above, design provisions for brittle failure modes in CLT are beyond the scope of this chapter. Until recently, no studies that focus on the brittle behaviour of fasteners in CLT have been conducted. This is a potential research topic in the future.

Figure 57
Possible brittle failure mode in CLT connections with glulam rivets

6.4.1 Design for the Lateral Load Resistance of Bolts and Dowels in CLT

Although bolts and dowels are not as commonly used in CLT assemblies compared to assemblies made with glulam or other wood-based products, there is still a need to provide some guidance to designers who may choose these types of fasteners for connections in CLT. This section is focused mainly on the design for the ductile lateral resistance of bolts and dowels in the current Canadian timber design standard (CSA O86-09).

6.4.1.1 Embedment of Doweled and Bolted Connections Perpendicular to the Plane of CLT Panel

Two embedment models were developed by Uibel and Blass using a multiple regression analysis on 438 test results for dowels installed perpendicular to the plane of the panel and loaded at different directions with respect to the panel strong axis and at different positions of the fastener in the plane of the CLT panel. The first model shown in equation [1] is quite general and is independent of the type of lay-up of the panel. The model is expressed as a function of the fastener diameter, overall wood density of the panel and loading direction with respect to the strong axis of the panel (i.e. grain direction of the surface layers of the CLT panel).

$$f_{h,pred} = \frac{0.035\,(1-0.015\,d)\,\rho^{1.16}}{1.1\sin^2\alpha + \cos^2\alpha} \quad (N/mm^2) \qquad [1]$$

where,

$f_{h,pred}$ = predicted embedment strength (N/mm^2)
d = fastener diameter (mm)
ρ = average density of main member, based on dry weight and volume basis (kg/m^3)
α = angle between load and grain direction of the outer layer

The second model shown in equation [2], however, is panel build-up specific and has the following form:

$$f_{h,pred} = 0.037\,(1-0.016\,d)\,\rho^{1.16}\left[\frac{\sum_{i=1}^{n} t_{0,i}}{t\,(1.2\sin^2\alpha + \cos^2\alpha)} + \frac{\sum_{j=1}^{n-1} t_{90,j}}{t\,(1.2\cos^2\alpha + \sin^2\alpha)}\right] \quad (N/mm^2) \qquad [2]$$

where,

$f_{h,pred}$ = predicted embedment strength (N/mm^2)
d = fastener diameter (mm)
ρ = average density of main member, based on dry weight and volume basis (kg/m^3)
α = angle between load and grain direction of the outer layer
$t_{0,i}$; $t_{90,i}$ = thickness of each layer (i.e. with t_0 being the thickness of individual layers orientated parallel to the outer layers and t_{90} the thickness of transverse layers) (mm)
t = panel thickness (mm)

The validity of the two models, however, is limited to the maximum thickness of a single layer not exceeding 40 mm and the ratio of the thicknesses of the longitudinal and cross laminate being between 0.95 and 2.1. Designers should be cautious when using these models.

The proposed equation by Uibel and Blass (2006) to establish the characteristic embedment strength of dowels in CLT on the basis of equation [1] is given below in equation [3]:

$$f_{h,k} = \frac{0.031\,(1-0.015\,d)\,\rho_k^{1.16}}{1.1\sin^2\alpha + \cos^2\alpha} \quad (N/mm^2) \qquad [3]$$

where,

$f_{h,k}$ = characteristic embedment strength (N/mm^2)
d = dowel diameter (mm)
ρ_k = characteristic density of cross laminated timber panels, based on dry weight and volume basis (kg/m^3)
α = angle between load and grain direction of the outer layer

This embedment equation is approximately equivalent to the embedment equations given in CSA O86-09, except that a duration of load factor of 0.8 needs to be applied to convert from short-term to standard-term duration to be in line with the CSA O86 design procedure. The 0.8 factor is typically applied for all wood-based products in CSA O86-09, including glulam. There could be a need to validate this factor for connections in CLT. Current service conditions factor, duration of load factor and treatment factors (K_{SF}, K_D and K_T) as given in clauses 4.3.2, 10.2.1.5 and 10.2.1.7 of CSA O86-09, may be used provided that some conservatism is taken into account due to the lack of research to support the adoption of those factors for CLT. Once the specified embedment equations are established for bolts and dowels in CLT, then the unit lateral yielding resistance of each type of fastener can be calculated as per CSA O86-09.

6.4.1.2 Embedment of Doweled and Bolted Connections in the Narrow Side (On Edge)

For situations where bolts or dowels are installed in the narrow side of the CLT panel (e.g., corner connection between wall panels at right angles as shown in Figure 58), the equation proposed by Uibel and Blass (2007) for calculating the characteristic embedment strength of dowels and bolts can be used. As with equations [1] to [3], the new expression is empirical and was developed based on a large number of tests using multiple regression analysis. Over 100 embedment tests for dowels installed in different positions and loaded either parallel or perpendicular to the grain of the lamina were used in deriving the proposed equation. The equation is expressed as a function of the dowel diameter and density of the relevant layer(s) in which the dowel is driven, as shown in equation [4]:

$$f_{h,k} = 0.0435\,(1-0.017\,d)\,\rho_{ply,k}^{0.91} \quad (N/mm^2) \qquad [4]$$

where,

$f_{h,k}$ = characteristic embedment strength (N/mm²)

d = fastener diameter (mm)

$\rho_{ply,k}$ = characteristic density of relevant layers, based on dry weight and volume basis (kg/m³)

It should be noted that, if the panel is made from materials of uniform density, then the overall density of the panel in the vicinity of the dowel could be used in equation [4] for simplicity.

Figure 58
Opened connection with dowels in cross-laminated timber (courtesy of Uibel and Blass, 2007)

6.4.2 Lateral Load Resistance of Screws and Nails in CLT

6.4.2.1 Embedment of Nails and Screws Perpendicular to the Plane of CLT Panel

The new design provisions for nails and wood screws in CSA O86-09 provide a methodology to calculate the lateral resistance based on the specified embedment properties of nails and wood screws in wood-based products. Once the specified embedment strength is known, then the unit lateral capacity of the connections in CLT can be calculated.

Characteristic embedment equations for nails of 4.2 mm and screws up to 12 mm in diameter were developed by Uibel and Blass (2006) with fasteners installed in the direction perpendicular to the plane of the panel. The equation is specific to the panel lay-up as it is expressed as a function of the density of the layer in which the fastener is placed, as shown in equation [5]:

$$f_{h,k} = 0.112\, d^{-0.5}\ \rho_k^{1.05} \quad (N/mm^2) \qquad [5]$$

where,

$f_{h,k}$ = characteristic embedment strength (N/mm²)
d = fastener diameter (mm)
ρ_k = characteristic density of cross-laminated timber panels, based on dry weight and volume basis (kg/m³)

A simplified form could be used as a substitute for equation [5] if a uniform density is used in the analysis. The validity of this equation, however, is limited to CLT panels with layers of 7 mm thickness or less (Uibel and Blass, 2006). More work is needed to develop a more generalized expression for the determination of embedment properties for CLT panels made of thicker lamina. Note that the proposed characteristic embedment equation is independent of the loading direction with respect to the grain orientation of the layers.

6.4.2.2 Embedment of Nails and Screws in the Narrow Side of CLT Panels (On Edge)

Embedment equations to calculate the embedment strength of screws and nails on the narrow side of CLT panels were also developed in Europe. Equation [6] below has been proposed by Uibel and Blass (2007):

$$f_{h,k} = 0.862\, d^{-0.5}\ \rho_{ply,k}^{0.56} \quad (N/mm^2) \qquad [6]$$

where,

$f_{h,k}$ = characteristic embedment strength (N/mm²)
d = fastener diameter (mm)
$\rho_{ply,k}$ = characteristic density of relevant layers, based on dry weight and volume basis (kg/m³)

6.4.3 Design for the Withdrawal Resistance of Screws in CLT

Withdrawal resistance tests of self-tapping screws in CLT driven perpendicular to the plane of the panel and on edge (in the narrow side) were conducted in Europe, with screws driven at different locations (Uibel and Blass, 2007). Screws were placed at different positions to capture the effect of gaps (i.e. screws driven in gaps or away from gaps). Based on tests results, equations were developed and proposed for the calculation of the characteristic withdrawal resistance of self-tapping screws in CLT, which has the following form:

$$R_{ax,s,k} = \frac{0.35\, d^{0.8}\ l_{ef}^{\ 0.9}\ \rho_k^{\ 0.75}}{1.5 \cos^2 \varepsilon + \sin^2 \varepsilon} \quad (N) \qquad [7]$$

where:

$R_{ax,s,k}$ = characteristic withdrawal capacity (N)
d = fastener diameter (mm)
l_{ef} = effective point-side penetration length (i.e. length of the threaded part minus one screw diameter) (mm)
ρ_k = characteristic density of CLT panel (whole cross-section) for fasteners driven perpendicular to the plane of the panel or density of relevant layers for fasteners driven on edge (kg/m³)
ε = angle between screw axis and CLT grain direction (equals to 90° in the plane of the panel or 0° in joints on the narrow side–i.e. edge joints)

It should be mentioned, however, that the expression given in equation [7] is limited to self-tapping screws and valid only when the characteristic withdrawal strength in solid wood exceeds the following:

$$f_{ax,k} = 80\ \rho_k^{\ 2}\ 10^{-6} \quad (N/mm^2) \qquad [8]$$

where:

$f_{ax,k}$ = characteristic withdrawal strength (N/mm^2)
ρ_k = characteristic density of solid wood, based on dry weight and volume basis (kg/m^3)

This requirement needs to be verified and modifications are expected in order to develop a more generalized expression.

6.4.4 Placement of Fasteners in Joints

Minimum requirements are given in CSA O86-09 for loaded end and edge distances, fastener spacing in a row and spacing between rows of fasteners for a variety of traditional fasteners such as bolts, lag screws, nails, wood screws and glulam rivets in solid sawn timber and glulam. While these requirements could be applied conservatively to fasteners driven or placed in the direction perpendicular to the plane of the CLT panel (as discussed above), they may not necessarily be applicable to fasteners placed in the narrow side (on edge) of the panel. Generally, spacing and end distances are less critical for fasteners placed perpendicular to the plane of the CLT panel due to cross laminations which tend to reinforce the section (as discussed above).

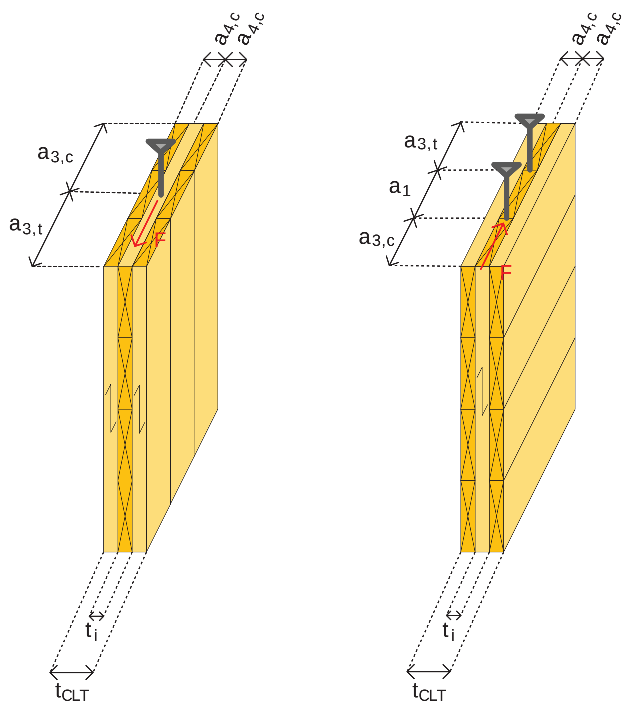


Figure 59
Recommended end and edge distances and spacing for dowel-type fasteners (adapted from Uibel and Blass, 2007)

Table 1
Recommended end and edge distances for dowel-type fasteners (adapted from Uibel and Blass, 2007)

Spacings	Type of fastener	
	Self-tapping screws	Dowels
a_1	10 d	4 d
a_2	3 d	4 d
$a_{3,t}$	12 d	5 d
$a_{3,c}$	7 d	3 d
$a_{4,c}$	5 d	3 d

Realizing the importance of investigating the required end distances and spacing for fasteners driven or placed on edge, European researchers have developed some minimum requirements for placement of mainly self-tapping screws and dowels in CLT panels. This was done to avoid premature splitting and ensure that full bearing capacity of the dowels in the CLT is achieved. This is critical for CLT panels when they are connected at right angles (e.g. floor-to-wall or wall-to-wall corner connections) and fasteners are driven in the narrow side (on edge) of one panel. In such situations, the fastener may tend to force fibres or plies apart across the panel thickness due to excessive tension perpendicular to grain stresses. This could trigger premature splitting in the vicinity of the fastener, thereby weakening the connection. Recommended end and edge distances and spacing for self-tapping screws and dowels placed on edge in wall panels are given in Figure 59, based on European research.

6.4.5 Detailing of Connections in CLT

In detailing and optimizing connections in CLT, it is important to consider not only the strength and stiffness performance of the connection system, but other performance attributes such as fire, sound insulation, air tightness, durability and vibration. Typically, sealant and other types of membranes are used to provide air tightness and improve sound insulation at the interfaces between the floor and wall plates (Figure 60). Shrinkage and swelling in CLT due to seasonal changes in the ambient environmental conditions need to be taken into account when designing connections. This is particularly important when other sealant products and membranes are incorporated as that might compromise the effectiveness of such products. Differential movement between CLT and other wood-based products or materials (in case of mixed materials and systems) need to be taken into account at the design and detailing stages due to potential shrinkage-induced stress that could undermine the connection capacity in CLT. Future versions of this chapter will provide more details and guidelines related to detailing.

Figure 60
Acoustic membrane inserted between walls and floors to provide air tightness (in exterior walls) and improve sound insulation

7 CONCLUSION

Connections in timber construction, including those built with CLT, play an important role in maintaining the integrity of the timber structure and in providing strength, stiffness, stability and ductility. Consequently, they require detailed attention by designers.

Traditional and innovative connection systems have been used in CLT assemblies in Europe. Several types of such connection systems for connecting CLT panels to panels, walls to walls and walls to floors are described in detail in this chapter. They are mostly based on the European experience since there is currently no CLT production in Canada or North America at the time of writing this chapter.

Researchers in Europe have developed design procedures for traditional connections in CLT, including dowels, wood screws and nails which are commonly used in Europe for designing CLT assemblies. The proposed design procedure deals only with ductile failure modes to determine the lateral load resistance of such connections. Expressions were developed for the calculation of characteristic embedment properties of each type of fastener, depending on its location with respect to the plane of the panel (perpendicular to or on edge). The expressions were verified and results seem to correspond well with predictions. European Yield Model (EYM) equations for ductile failure modes as given in Eurocode 5 were adopted for design using CLT fastener embedment equations.

Information on the applicability of the proposed design approach from Europe to traditional connection systems in CLT are presented in this chapter. It is believed that once the embedment properties of such fasteners in CLT are established, it will be possible to apply the current ductile design provisions in CSA O86-09. Due to the reinforcing effect of cross lamination in CLT, it is speculated that current minimum geometric requirements given in CSA O86-09 for dowels, screws and nails in solid timber or glulam are applicable to CLT. However, designers need to be cautious about this as further verification is required, considering the specific features of each panel (no generic CLT panels have been produced yet in Canada). Brittle failure modes also need to be taken into account and have not been investigated yet. Further work is needed to verify possible brittle failure modes associated with each type of fasteners in CLT connections.

8 REFERENCES

American Forest & Paper Association (AF&PA). 2006. *National design specification (NDS) for wood construction with commentary and supplement, 2005 edition*. Washington, DC: AF&PA. 174 p.

Augustin, M., ed. 2008. Timber structures. Handbook 1 of *Educational materials for designing and testing of timber structures: TEMTIS*. Leonardo da Vinci Pilot Project No. CZ/06/B/F/PP/168007. Ostrava, Czech Republic: VSB - Technical University of Ostrava. 250 p.

Canadian Standard Association (CSA). 2009. *Engineering design in wood (limit states design)*. CSA O86-09. Rexdale, ON: CSA. 222 p.

European Committee for Standardization. 2004. *Eurocode 5: Design of timber structures. Part 1-1: General – Common rules and rules for buildings*. EN 1995-1-1. Brussels: CEN. 124 p.

Quenneville, J.H.P., and M. Mohammad. 2000. On the failure modes and strength of steel-wood-steel bolted timber connections loaded parallel-to-grain. *Canadian Journal of Civil Engineering* 27:761-773.

TRADA. 2009. *Cross laminated timber: Structural principles*. High Wycombe, UK: TRADA. 8 p.

Traetta, G. 2007. Connection techniques for CLT elements. Paper presented at the TEMTIS Austrian Country Seminar: Cross Laminated Timber, Graz, Austria.

Uibel, T., and H. J. Blass. 2006. Load carrying capacity of joints with dowel type fasteners in solid wood panels. Paper presented at the 39th meeting of the Working Commission W18–Timber Structures, International Council for Research and Innovation in Building and Construction, Florence, Italy, August 2006.

_________. 2007. Edge joints with dowel type fasteners in cross laminated timber. Paper presented at the 40th meeting of the Working Commission W18–Timber Structures, International Council for Research and Innovation in Building and Construction, Bled, Slovenia, August 2007.

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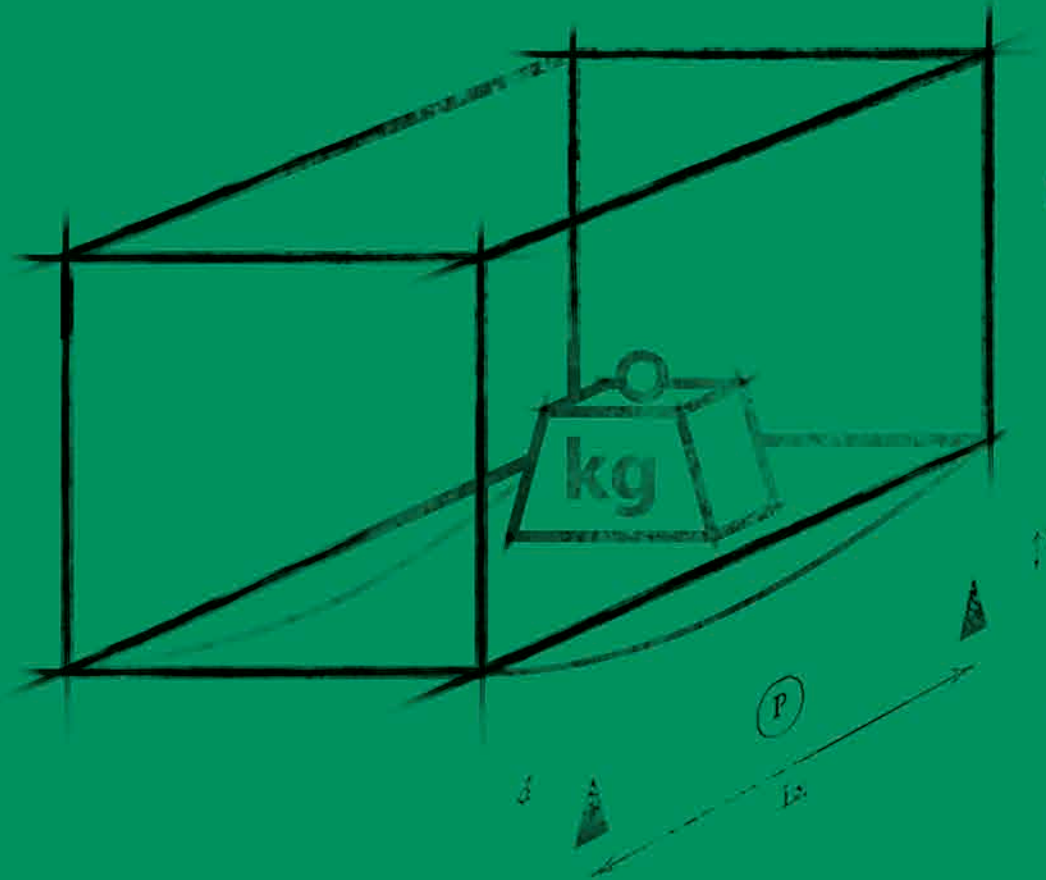

CHAPTER 6

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ABSTRACT

Cross-laminated timber (CLT) products are used as load-carrying slab and wall elements in structural systems, thus load duration and creep behaviour are critical characteristics that should be taken into account in design. Given the nature of CLT with orthogonal arrangement of layers and either mechanically fastened with nails or wood dowels, or bonded with structural adhesive, CLT is more prone to time-dependent deformations under load (creep) than other engineered wood products such as glued-laminated timber.

Time-dependent behaviour of structural wood products is accounted for in design standards by providing load duration factors to adjust specified strengths. Since the Canadian Standard on Engineering Design in Wood (CSA O86-09) does not deal with CLT, it does not provide load duration and service condition factors. Until this can be rectified, two options are proposed for adopters of CLT systems in Canada. These include not only load duration and service factors, but also an approach to accounting for creep in CLT structural elements. The proposed recommendations are in line with the specifications in CSA O86-09 and Canadian National Building Code.

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1 OVERVIEW

This chapter aims to describe how the duration of load[1] and creep[2] effects of wood are taken into account in design of wood structures, when the design is carried out in accordance with the current Canadian and European Timber Design Standards. Moreover, since CLT is not covered by the Canadian Standard on Engineering Design in Wood, the intent is to recommend a suitable approach for Canadian users of CLT at this time to account for the duration of load and creep effects in the design of CLT.

[1]Load duration is defined as the duration of continuing application of a load or a series of periods of intermittent applications of the same load type (CSA O86-09, 2009).

[2]Creep is defined as an increase in deformation of a material in time under constant loading.

2

DURATION OF LOAD AND CREEP EFFECTS IN CANADIAN CODES AND STANDARDS

The current United States standard for the evaluation of duration of load and creep effects (ASTM D 6815, 2002) is a pass/fail procedure and, at the moment, does not provide a method for calculation of duration of load or creep factors. This standard was developed for the evaluation of engineered wood products, but it would not be practical to carry out ASTM D 6815 tests on full-size CLT specimens as it cannot lead to duration of load and creep factors specific to CLT. The Canadian Standard on Engineering Design in Wood (CSA O86-09, 2009) takes into account duration of load categories (that account for the dependency of wood on duration of applied load); however, it does not include the effect of service class on the duration of applied load (that allows for sensitivity of wood to moisture content variations and its consequent effect on creep and, typically, referred to as the mechano-sorptive effect of wood).

2.1 Load Duration Factor in CSA O86-09

A load duration factor, K_D, is specified in Clause 4.3.2 of CSA O86-09 for three load categories: short term, standard term, and long term loading. The short term category allows up to seven days of continuous or cumulative loading throughout the life of the structure, while the long term category implies more or less continuous loading during the intended life of the structure. Duration of load in the standard term (e.g. snow and occupancy loads) falls between that of short term and long term. The capacity design values in CSA O86-09 are given for standard term load duration. The load duration factors are given in Table 1.

Table 1
Load duration factor, K_D (Table 4.3.2.2, CSA O86-09)

Duration of Loading	K_D
Short term	1.15
Standard term	1.00
Long term	0.65

Clause 4.3.2.3 of CSA O86-09 provides an equation for calculating the duration of load factor when the specified long term load, P_L, is greater than the specified standard term load, P_S. In this case, the long term load factor may be used or the load duration factor may be calculated as follows:

$$K_D = 1.0 - 0.50 \log (P_L/P_S) \geq 0.65 \qquad [1]$$

where,

P_L = specified long term load

P_S = specified standard term load based on S and L loads acting alone or in combination = S, L, S+0.5L, or 0.5S+L, determined using importance factors equal to 1.0

The load duration factors, as indicated above, are used to adjust the specified strength of lumber, wood-based products including glued-laminated timber and plywood, and connections capacity. The specified strength of a structural element is multiplied by a load duration factor according to Clause 4.3.2 of CSA O86-09.

2.2 Service Condition Factor in CSA O86-09

CSA O86-09 defines dry service as a climatic condition in which the average equilibrium moisture content over a year is 15% or less and does not exceed 19%. To deal with service conditions other than dry, CSA O86-09 provides service condition factors, K_S. The strength specified by CSA O86-09 for the product is multiplied by the appropriate service condition factor. Service condition factors for glued-laminated timber are shown in Table 2, while those for plywood are given in Table 3.

Table 2

Service condition factor, K_S, for glued-laminated timber (Table 6.4.2, CSA O86-09)

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 3

Service condition factor, K_S, for plywood (Table 7.4.2, CSA O86-09)

Property to be Modified	Plywood	
	Dry Service Conditions	Wet Service Conditions
Specified strength capacity	1.00	0.80
Specified stiffness and rigidity capacities	1.00	0.85

The service condition factors, as indicated above, are used to adjust the specified strength of lumber and wood-based products. The specified strength of a product used in wet service conditions is multiplied by the appropriate service condition factor tabulated in CSA O86-09.

3

DURATION OF LOAD AND CREEP EFFECTS IN EUROPEAN CODES AND STANDARDS

The current European approach takes into account the duration of load and creep effects by introducing load duration classes associated with accumulated duration of load. The load duration and creep factors take into account duration of load classes and service classes, and they are product specific. The main factors affecting creep of solid wood-based products include the magnitude, type and duration of load, moisture content and temperature. Interactions occur among all factors, but only the combined effects of load duration and moisture content are taken into account in the design rules specified in Eurocode 5 – Design of Timber Structures, EN 1995-1-1 (Eurocode 5, 2004), which provides load duration classes and modification factors for service classes that are used in the design of structures. Load duration classes are shown in Table 4, while service classes are shown in Table 5.

Table 4

Load duration classes (Table 2.1, EN 1995-1-1)

Load Duration Class	Accumulated Duration of Load
Permanent	> 10 years
Long term†	6 months - 10 years
Medium term	1 week - 6 months
Short term	< 1 week
Instantaneous	N/A

Note: †Standard term for load duration factor in CSA O86-09 exceeds 7 days but it is less than almost continuous loading throughout the life of the structure.

Table 5
Service classes (Clause 2.3.1.3, EN 1995-1-1)

Service Class	Climatic Condition
Service class 1	Moisture content (MC) of material @ 20°C and > 65% relative humidity (RH) for a few weeks per year (softwood timber MC < 12%; panels MC < 8%)
Service class 2	Moisture content (MC) of material @ 20°C and > 85% relative humidity (RH) for a few weeks per year (softwood timber MC < 20%; panels MC < 15%)
Service class 3	Condition leading to higher MC than Service class 2 (softwood timber MC > 20%; panels MC > 15%)

Note: CSA O86-09 defines dry service conditions as climatic conditions at which MC of solid wood is less than 19% per year (equilibrium MC ≤ 15%). Wet service conditions correspond to all conditions other than dry.

3.1 Strength Modification Factor in EN 1995-1-1

Product-specific strength modification factors, k_{mod}, for service classes and load duration classes are given in Table 6. Note that design strength and capacity values are based on tests to failure in 5±2 minutes, and they are similar for glued-laminated timber and plywood.

Table 6
Strength modification factor, k_{mod} (Table 3.1, EN 1995-1-1)

Material / Load Duration Class	Service Class 1	Service Class 2	Service Class 3
Glued-Laminated Timber			
Permanent	0.60	0.60	0.50
Long term	0.70	0.70	0.55
Medium term	0.80	0.80	0.65
Short term	0.90	0.90	0.70
Instantaneous	1.10	1.10	0.90
Plywood[1]			
Permanent	0.60	0.60	0.50
Long term	0.70	0.70	0.55
Medium term	0.80	0.80	0.65
Short term	0.90	0.90	0.70
Instantaneous	1.10	1.10	0.90

Notes:
[1] Plywood classified in accordance to Part 1, Part 2 and Part 3 of EN 636 may be used under Service Class 1; plywood classified in accordance to Part 2 and Part 3 of EN 636 may be used under Service Class 2; and plywood classified in accordance to Part 3 of EN 636 may be used under Service Class 3. Additional information about the three plywood categories is given in Table 7.

3.2 Deformation Modification Factor in EN 1995-1-1

Deformation factor or creep factor, k_{def}, takes into account creep deformation for the relevant service classes, and is shown in Table 7.

Table 7
Deformation modification factor, k_{def} (Table 3.2, EN 1995-1-1)

Material (Standard)	Service Class 1	Service Class 2	Service Class 3
Solid timber[1] (EN 14081-1)	0.60	0.80	2.00
Glued-laminated timber (EN 14080)	0.60	0.80	2.00
Plywood (EN 636[2])			
Part 1	0.80	-	-
Part 2	0.80	1.00	-
Part 3	0.80	1.00	2.50

Notes:
[1] k_{def} is to be increased by 1.00 for timber near saturation point which is likely to dry out under load;
[2] The 1997 edition of EN 636 classified plywood in the following three categories:
Part 1: Plywood manufactured for use in DRY conditions = interior applications with no risk of wetting, defined in hazard class 1, with a moisture content (MC) corresponding to environmental conditions of 20°C and 65% RH (12% MC or less).
Part 2: Plywood manufactured for use in HUMID conditions = protected exterior applications as defined in hazard class 2, with a MC corresponding to environmental conditions of 20°C and 90% RH (20% MC or less).
Part 3: Plywood manufactured for use in EXTERIOR conditions = unprotected external applications, as defined in hazard class 3, where the MC will frequently be above 20%.
The latest version of EN 636 (2003) integrates the three separate parts for plywood for use in dry conditions (EN 636-1:1997), humid conditions (EN 636-2:1997) and exterior conditions (EN 636-3:1997), and supersedes the 1997 editions.

4

DURATION OF LOAD AND CREEP EFFECTS OF CLT

4.1 European Approach

Material properties including duration of load and creep factors for CLT are not specified in Eurocode 5 because of the proprietary nature of these products in Europe. However, CLT is covered in some national building codes such as DIN 1052 (Design of Timber Structures, Germany) and SIA 265 (Timber section of the Swiss Building Code). Engineers in Europe use allowable design values indicated in product catalogues which are made available by the CLT manufacturers to design CLT structures, and obtain special approvals from local building officials.

Research conducted at Graz University of Technology in Austria concluded that long term behaviour of CLT products is more likely comparable with that of other cross-laminated wood-based products (such as plywood) as opposed to products laminated unidirectionally (such as glued-laminated timber) (Jöbstl and Schickhofer, 2007). The authors reported 30%-40% larger creep values for CLT compared to glued-laminated timber after one year loading in bending, which is attributed to crosswise layers in CLT. Using the deformation factor obtained for 5-layer CLT, the authors derived the deformation factors for CLT products ranging from 3-layer to 19-layer, and recommended using the deformation factor for plywood for CLT with more than 9 layers, and increase the deformation factor for plywood by 10% for CLT with 7 layers or less.

In Eurocode 5, the final deformation is calculated for the quasi-permanent[3] combination of actions. Assuming a linear relationship between the loads and the corresponding deformations, the final deformation (u_{fin}) may be calculated as a sum of the final deformation due to permanent loads ($u_{fin,P}$), the final deformation due to the main live loads ($u_{fin,Q1}$), and the final deformation due to accompanying live loads ($u_{fin,Qi}$) (Clause 2.2.3(5) of EN 1995-1-1).

$$u_{fin,P} = u_{inst,P}\,(1 + k_{def}) \qquad \text{-- for permanent loads, P} \qquad [2]$$

$$u_{fin,Q,1} = u_{inst,Q,1}\,(1 + \psi_{2,1} k_{def}) \qquad \text{-- for main live loads, } Q_1 \qquad [3]$$

$$u_{fin,Q,i} = u_{inst,Q,i}\,(\psi_{0,i} + \psi_{2,i} k_{def}) \qquad \text{-- for accompanying live loads, } Q_i\ (i>1) \qquad [4]$$

$u_{inst,P}$, $u_{inst,Q,1}$, $u_{inst,Q,i}$ = instantaneous deformations for loads P, Q_1, Q_i, respectively

where,

$\psi_{2,1}$, $\psi_{2,i}$ = factors for the quasi-permanent value of live loads;

$\psi_{0,i}$ = factors for the combination value of live loads;

k_{def} = deformation factor.

[3] Quasi-permanent combination is used mainly to take into account long term effects.

4.2 Options for a Canadian Approach

Since CSA O86-09 does not deal with CLT, it does not provide load duration and service condition factors. Until this can be rectified, two options are proposed for adopters of CLT systems in Canada. These include not only load duration and service factors, but also an approach to accounting for creep in CLT structural elements.

1) Option I (Consistent with the format in the Canadian Standard on Engineering Design in Wood Standard)

   a) **Load Duration Factor**: Use the load duration factor, K_D, specified in Clause 4.3.2 of CSA O86-09.

   b) **Service Condition Factor**: Use the service condition factor, K_S = 1.0 for dry service conditions. For humid service (protected exterior conditions), use the service condition factor, K_S, for glued-laminated timber shown in Table 2.

   c) **Creep Factor**: The current load duration factor, K_D, and service condition factor, K_S, specified in CSA O86-09, do not account for creep that may occur in CLT. In this option, the recommendation is to determine the elastic deflection due to total load, with a 25% reduction in the rolling shear modulus, G, for cross laminations when calculating shear stiffness, GA_{eff}, of a CLT slab element. Moreover, it is recommended to determine the permanent deformation due to long term loads, with a 50% reduction in the rolling shear modulus, G, for cross laminations when calculating shear stiffness, GA_{eff}, of a CLT slab element. The proposed reductions in the rolling shear modulus are taken due to 30%-40% higher creep values for CLT compared to glued-laminated timber for one year of sustained loading, as reported by Jöbstl and Schickhofer (2007).

2) Option II (Consistent with the format in the European Timber Design Standard)

   a) **Load Duration and Service Condition Factors**: Instead of using K_D and K_S factors, adopt k_{mod} factors from Eurocode 5 for Service Classes 1 and 2 (given in Table 6).

   b) **Creep Factor**: According to the research conducted in Europe described in Section 4.1, CLT deflects more than glued-laminated timber in part due to the orientation of the cross-laminations and the fact that wood creeps more in the direction perpendicular to the grain. A 10% increase in k_{def} factors for plywood (given in Table 7) is reflected in the calculation of k_{def} factors for CLT (given in Table 8) to account for the differences between CLT and glued-laminated timber test results obtained by Jöbstl and Schickhofer (2007). One year constant load duration can be assumed sufficient to account for the cumulative damage effects due to occupancy and snow loads.

Table 8
Deformation modification factor, k_{def}, adjusted to CLT (based on recommendations of Jöbstl and Schickhofer – 2007)

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

For long term loads, however, a further increase of k_{def} or reduction of deformation limits is recommended.

A parametric study will be carried out on CLT slabs subjected to various load configurations and spans to verify these proposals. The findings will be reflected in future editions of this chapter.

5

MODIFICATION FACTORS FOR CONNECTIONS USED IN CLT BUILDINGS

Load duration and time-dependent slip behaviour of connections also affect the performance of a CLT system. CSA O86-09 specifies the same load duration factors, K_D, for fastenings as shown in Table 1. Service condition factors for fastenings, K_{SF}, are also tabulated in the CSA standard (Table 10.2.1.5 in CSA O86-09). It is important to note that service condition factors for fastenings are different than those for lumber or for glued-laminated timber seasoned at moisture content of 15% or less, and above 15%. The CSA standard also specifies service creep factors, K_m, for nails and spike joints for the calculation of the lateral deformation in wood-to-wood joints (Table A.10.9.3.2 in CSA O86-09). Work is currently underway to revise the current K_{SF} factors for connections in CSA to reflect the newly developed design methodology for bolts and dowels which has been adopted in CSA O86-09. Additional information on connections with CLT is given in Chapter 5, *Connections in Cross-Laminated Timber Buildings*.

6 PRODUCT-SPECIFIC PARAMETERS THAT MAY AFFECT DURATION OF LOAD AND CREEP EFFECTS OF CLT

6.1 Adhesives

A structural adhesive is not expected to creep in service. Canadian standards for evaluation of adhesives for structural application have built in tests for assessing creep under various loads and service conditions. The proposed CLT manufacturers and product qualification standard specifies that adhesives for CLT manufacturing have to pass the minimum requirements of CSA O112.10, Standard for Evaluation of Adhesives for Structural Wood Products for Limited Moisture Exposure (CSA O112.10, 2008). The CSA O112.10 standard requires that creep tests are carried out at specific conditions: environment "A" (7 days at 20°C and 95% RH), environment "B1" (7 days at 70°C and ambient RH), and environment "B2" (2 hours at 180°C) while loaded at 2.5 MPa, 2.5 MPa, and 2.1 MPa, respectively. Adhesives passing the minimum requirements of the CSA O112.10 would show negligible creep in the bond line, which is considered insignificant relative to the creep that occurs in CLT products due to the orientation of crosswise laminations.

6.2 Edge-Gluing and Width-to-Thickness Ratio

CLT products without edge-glued laminations may have lower load-carrying capacities than those with edge-glued laminations due to lower rolling shear modulus. However, no research results have been published to show any correlation between rolling shear modulus of edge-glued and non-edge-glued laminations and its effect on load carrying capacity of the CLT element.

Parameters affecting rolling shear properties include: lamination width, direction of annual rings in boards, earlywood to latewood ratios, adhesive type, panel pressure during manufacturing, and type of loading. A true value of rolling shear modulus is difficult to obtain due to very low shear deflections measured during the tests, which makes the calculation of rolling shear modulus very sensitive to experimental error. In Europe, a rolling shear modulus of 50 MPa is used for CLT design; this value was obtained for spruce with an oven-dry density of 460 kg/m^3 (Aicher and Dill-Langer, 2000; Aicher et al., 2001). Typically, rolling shear modulus for spruce ranges from 40 MPa to 80 MPa (Fellmoser and Blass, 2004).

Preliminary observation suggests a decrease in rolling shear modulus with decreasing width-to-thickness ratio of boards in the cross layer. A minimum width-to-thickness ratio of 4:1 is suggested for lumber to ensure good contact during pressing and adequate rolling shear strength (Schickhofer et al., 2009). The draft European standard for CLT recommends further verification through testing when the minimum width-to-thickness ratio of lumber is less than 4:1 (prEN, 2010). For these reasons, it is recommended that rolling shear strength and modulus are verified by testing when using cross laminations with a width-to-thickness ratio of less than 3.5. Research is ongoing to develop appropriate testing methods for assessing rolling shear strength of CLT, and to quantify the width-to-thickness effect.

6.3 Release Grooves

CLT products manufactured with release grooves are likely to have lower load-carrying capacities than those without release grooves due to the lower rolling shear modulus of cross laminations caused by the release grooves. Some manufacturers in Europe mill release grooves into lumber in cross laminations to minimize the effect of cupping. The depth of grooves may take up to 90% of the lumber thickness (prEN, 2010). Failure of CLT loaded in bending is typically initiated in the cross layers by rotation of the cross layers and "rolling" of the earlywood zones in lumber (Augustin, 2008). The grooves are weak zones in the cross section, which is significantly reduced at the grooves and prone to failure under high loads generating narrower strips of lumber that are further likely to "roll" under load leading to high deformations and ultimately failure. Since the release grooves are considered unbonded edges, it is recommended that rolling shear strength and modulus are verified by testing when using cross laminations with release grooves.

6.4 Nails or Wooden Dowels in Non-adhesively Bonded CLT Products

Mechanically fastened CLT is outside the scope of the CLT Handbook and the design provisions given in Chapter 3, *Structural Design of Cross-Laminated Timber Elements*, do not cover such products. In Europe, some manufacturers are using aluminum nails or wooden dowels to vertically connect wood layers in CLT. These CLT products are not glued-laminated, and may deflect and creep significantly more than adhesively-bonded CLT. Researchers at University of British Columbia have found four times larger deflections for nailed CLT specimens compared to glued CLT specimens for the same specimen thickness (Chen and Lam, 2008). The deflection range was due to different nailing schedules of the CLT layers. These products may be more suitable for wall applications but the load duration and creep factors recommended in this document are not applicable to non-adhesively bonded CLT products.

7

PROPOSED CANADIAN APPROACH FOR USING LOAD DURATION AND CREEP FACTORS IN CLT DESIGN

7.1 Ultimate Limit State

The recommended approach for Canadian users of CLT at this time is to follow Option I of Section 4.2. Accept the existent load duration factors specified in the CSA O86-09 (Section 4.2, Option I (a)), with the understanding that these factors may not conservatively account for shear perpendicular to the grain (rolling shear) effects; however, the design of CLT used as floor and roof elements is usually governed by deflection, and deflection falls under serviceability state design.

Use the service condition factors for glued-laminated timber specified in CSA O86-09 and given in Table 2 (Section 4.2, Option I (b)).

Verification of shear and bending out-of-plane strengths is explained in detail in Chapter 3.

7.2 Serviceability Limit State

7.2.1 Deflection

Check the elastic deflection and permanent deformation for CLT slab elements as per Section 4.2, Option I(c). The proposed recommendation is to check the elastic deflection due to total load, calculated with a 25% reduction in the rolling shear modulus, G, for cross laminations, as to not exceed 1/180 of the span, in accordance with Clause 4.5.2 of CSA O86-09. Moreover, it is proposed to check the deformation due to the long term loads, calculated with a 50% reduction in the rolling shear modulus, G, for cross laminations, as to not exceed 1/360 of the span, in accordance with Clause 4.5.3 of CSA O86-09. The proposed recommendations are in line with the maximum deflection limits prescribed in Table D-1 of NBCC (NBCC, 2005). Note that a limit of 1/180 of the span will control immediate deflection under total load, while a limit of 1/360 of the span will control permanent deflection under long term load.

7.2.2 Floor Vibration

Check maximum floor vibrations for CLT slab elements. A design method for controlling vibrations in CLT floors is provided in Chapter 7.

8 REFERENCES

Aicher, S., and G. Dill-Langer. 2000. Basic considerations to rolling shear modulus in wooden boards. *Otto-Graf-Journal* 11:157-165.

Aicher, S., G. Dill-Langer, and L. Hofflin. 2001. Effect of polar anisotropy of wood loaded perpendicular to grain. *Journal of Materials in Civil Engineering* 13 (1):2-9.

American Society for Testing and Materials (ASTM). 2002. *Standard specification for evaluation of duration of load and creep effects of wood and wood-based products.* ASTM D 6815-02a. West Conshohocken, PA: ASTM. 11 p.

Augustin, M., ed. 2008. Wood based panels (in particular cross laminated timber (CLT)). In Timber structures. Handbook 1 of *Educational materials for designing and testing of timber structures: TEMTIS*. Leonardo da Vinci Pilot Project No. CZ/06/B/F/PP/168007, 63-99.

Canadian Standards Association (CSA). 2008. *Evaluation of adhesives for structural wood products (limited moisture exposure)*. CSA O112.10-08. Rexdale, ON: CSA. 60 p.

______. 2009. *Engineering design in wood (limit states design)*. CSA O86-09. Rexdale, ON: CSA. 252 p.

Chen, J. Y., and F. Lam. 2008. Development of thick laminated MPB wood plates, prepared for Forestry Innovation Investment. Report MDP 08 – 0066B. Vancouver, BC: University of British Columbia. 18 p.

European Committee for Standardization (CEN). 2004. *Eurocode 5: Design of timber structures. Part 1-1: General – Common rules and rules for buildings*. EN 1995-1-1. Brussels: CEN. 124 p.

______. 2010. Timber structures – Cross-laminated timber – Requirements. Draft European Standard prEN xxxxx. Working Document WI 124.128. Brussels: CEN. Committee CEN/TC 124. 94 p.

Fellmoser, P., and H. J. Blaß. 2004. Influence of rolling shear modulus on strength and stiffness of structural bonded elements. In *CIB-W18 Meeting 37, Edinburgh, United Kingdom*, paper 37-6-5.

Jöbstl, R. A., T. Moosbrugger, T. Bogensperger, and G. Schickhofer. 2006. A contribution to the design and system effect of cross-laminated timber (CLT). In *CIB-W18 Meeting 39, Florence, Italy*, paper 39-12-4.

Jöbstl, R.A., and G. Schickhofer. 2007. Comparative examination of creep of GLT – and CLT – slabs in bending. In *CIB-W18 Meeting 40, Bled, Slovenia*, paper 40-12-3.

National Research Council of Canada (NRCC). Canadian Commission on Building and Fire Codes. 2006. *User's guide – NBC 2005 structural commentaries (Part 4 of Division B)*. Ottawa, ON: NRCC. 1 v.

Schickhofer, G. 2010. Cross-laminated timber (CLT) in Europe: From conception to implementation. Presentation made at the CLT Seminar, University of British Columbia, March 2010.

Schickhofer, G., T. Bogensperger, T. Moosbrugger, R. A. Jöbstl, M. Augustin, A. Thiel, G. Traetta, et al. 2009. *BSPhandbuch: Holz-Massivbauweise in Brettsperrholz.* Graz, Austria: Technische Universität Graz, Institute für Holzbau und Holztechnologie. 353 p.

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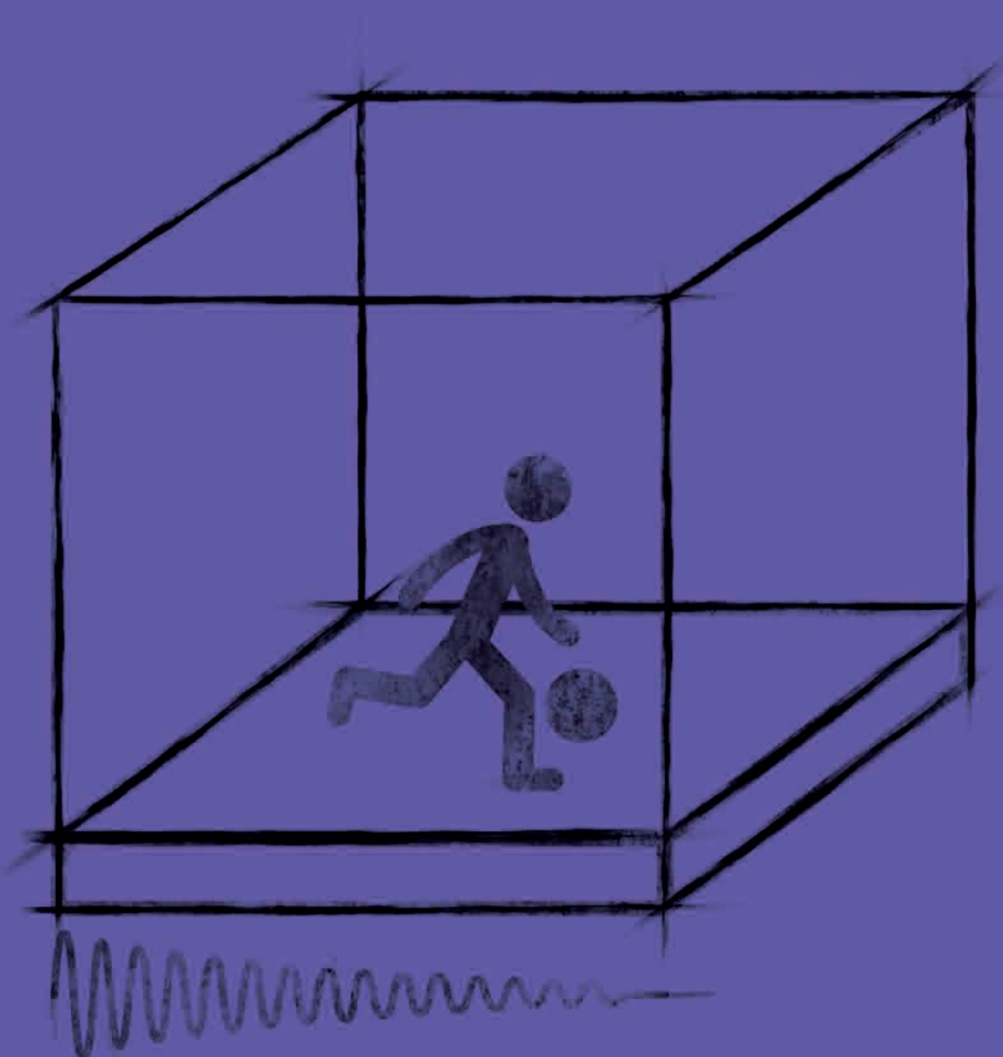

vibration

Vibration performance of cross-laminated timber floors

CHAPTER 7

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ABSTRACT

Cross-laminated timber (CLT) is proving to be a promising solution for wood to compete in building sectors where steel and concrete have traditionally predominated. Studies at FPInnovations found that bare CLT floor systems differ from traditional lightweight wood joisted floors with typical mass around 20 kg/m^2 and fundamental natural frequency above 15 Hz, and heavy concrete slab floors with a mass above 200 kg/m^2 and fundamental natural frequency below 9 Hz. Based on FPInnovations' test results, bare CLT floors were found to have mass varying from approximately 30 kg/m^2 to 150 kg/m^2, and a fundamental natural frequency above 9 Hz. Due to these special properties, the existing standard vibration controlled design methods for lightweight and heavy floors may not be applicable for CLT floors. Manufacturers recommend to use the uniformly distribution load (UDL) deflection method for CLT floor control vibrations by limiting the static deflections of the CLT panels under UDL. Using this approach, the success in avoiding excessive vibrations in CLT floors relies mostly on the engineer's judgement. A new design methodology is needed to determine the vibration controlled spans for CLT floors.

SINTEF's extensive CLT floor vibration field study found that FPInnovations' new design method using 1 kN static deflection and fundamental natural frequency as design parameters, predicted bare CLT floor vibration performance that matched well with occupants' expectations. This criterion was originally developed for wood joisted floors. The new design method is a modified version of the original FPInnovations design method for bare CLT floors based on bare CLT floor test data at FPInnovations. The new design method included the new form of the design criterion using calculated 1 kN static deflection and fundamental natural frequency for bare CLT floors as the criterion parameters, in addition to the new equations to calculate the 1 kN static deflection and fundamental natural frequency. A simple form to directly calculate the vibration controlled spans from CLT stiffness and density was derived from the new design method. Verification showed that the proposed design method predicted well the vibration performance of bare CLT floors studied at FPInnovations with the subjective ratings of the floor vibration performance. The impact study showed that the vibration controlled spans of bare CLT floors predicted by this new design method were almost the same as the spans determined by the CLTdesigner software that was developed in Austria. Working examples were given to demonstrate the procedure of using the simple form of the new design method to calculate the vibration controlled spans of CLT floors.

It is concluded that the proposed design methodology to determine vibration controlled maximum spans of bare CLT floors is promising. It is simple as it only uses the design values of CLT mechanical properties, is user-friendly, and reliable.

Wide acceptance of the proposed design method relies on the use and evaluation of the method by manufacturers of products and designers. FPInnovations is open to feedbacks and ready to evolve the design method according to the needs of the producers and designers. From the vibration control point of view, the low damping ratio (about 1% critical damping ratio) can be a weakness of bare CLT floors. Any measures for increasing the damping ratio through CLT product design and CLT floor construction will enhance vibration performance of bare CLT floors. The current form of the design method applies to CLT floors without heavy topping. A study of the effect of heavy topping on vibration performance of CLT floors is under way.

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CLT FLOORS

Cross-laminated timber (CLT) is proving to be a promising solution for wood to compete in building sectors where steel and concrete have traditionally predominated. It has higher stiffness/strength to mass ratio than cold-formed steel, reinforced concrete and masonry. Moreover, CLT building components are prefabricated. The prefabrication accelerates the construction process and makes the construction efficient in terms of time, labour, and materials. Another advantage for the use of CLT is the environmental benefits of wood, which makes CLT more environmentally friendly than other building materials such as steel and concrete. Figure 1 shows the cross-section of a CLT floor.

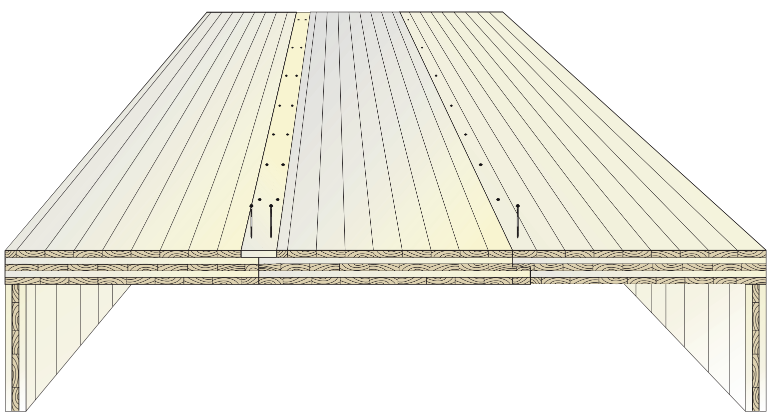

Figure 1
Cross-section of a bare CLT floor

2

UNIQUE FEATURES OF CLT FLOORS – SPECIAL VIBRATION BEHAVIOUR

Laboratory and field tests on CLT floors (Gagnon and Hu, 2007) have found that the vibration behaviour of CLT floors is different from lightweight wood joisted floors and heavy concrete slab floors. Below are some explanations for such differences.

2.1 Construction

Conventional lightweight wood joisted floors are usually built with joists spaced no more than 600 mm o.c. with a wood subfloor of 15.5 mm or 18 mm thick depending on the joist spacing (Figure 2); conversely, CLT floors have no joists and are solid (Figure 1). The appearance of CLT plates is similar to concrete slabs.

Furthermore, in comparison with joisted floors having the same span and equivalent vibration performance, CLT floors are less deep than conventional lightweight joisted floors. For example, a 6.5 m span floor can usually be built using 0.23 m thick CLT panels. If the same floor is built using conventional wood joists, then at least 0.3 m deep joists are needed.

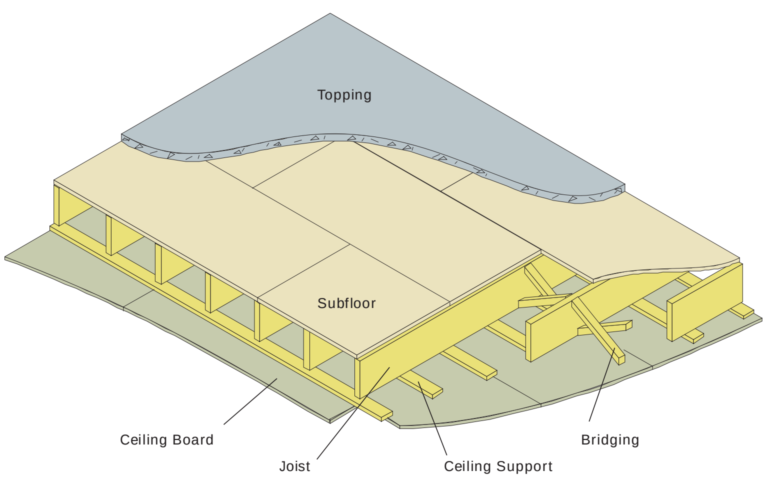

Figure 2
Conventional lightweight wood floor built with joists and subfloor

2.2 Dead Load

CLT floors are heavier than conventional joisted wood floors and lighter than concrete slab floors. Currently, thickness of the CLT panels on the market varies from 60 mm to 320 mm. For floor applications, the minimum thickness will be about 100 mm. Therefore, the area mass of CLT floors varies from about 50 kg/m^2 to 150 kg/m^2. The conventional wood joisted floor systems have an area mass of about 20 kg/m^2 for base floors and about 110 kg/m^2 for base floors with a 38 mm thick normal weight concrete topping. The concrete slab floors normally have an area mass above 200 kg/m^2.

2.3 Fundamental Natural Frequency

Due to the specific mass to stiffness characteristic of CLT floors, their vibrations exhibit unique behaviour indicated by the fundamental natural frequency. The lower boundary of the measured fundamental natural frequencies for satisfactory bare CLT floors tested in our laboratory was 10 Hz, while 15 Hz is usually measured for bare conventional wood joisted floors and 10 Hz for bare joisted floors with a concrete topping. Concrete slab floors normally have a fundamental natural frequency below 8 Hz. The higher the fundamental natural frequency is, the easier it is to control the vibrations of floors.

2.4 Damping

The measured modal damping ratios of bare CLT floor specimens tested in our laboratory were about 1% of the critical damping ratio. The conventional wood joisted floor systems normally have damping ratios around 3%. Low damping results in vibrations in CLT floors indicate longer persistence and are more annoying to occupants than that in conventional lightweight wood joisted floors. The higher the damping, the easier it is to control vibrations. Damping is determined by the material and the construction details including structural and non-structural elements, supporting systems, etc. Ungar (1992) provides a detailed discussion on structural damping and its sources.

Table 1
Summary of bare CLT floor dynamic characteristics

Damping	About 1%
Area Mass	About 50-150 kg/m^2
Fundamental Natural Frequency	Above 9 Hz

3

REVIEW OF THE FEASIBILITY OF THE APPLICATION OF THE EXISTING DESIGN METHODS FOR CLT FLOORS

3.1 Uniformly Distributed Load (UDL) Deflection Method

The uniformly distributed load (UDL) deflection method attempts to control vibrations by limiting the static deflections of the CLT panels under a uniform design load. For example, some CLT manufacturers recommend limiting the total UDL deflection to L/400. The problem with the UDL deflection method is that it allows the longer span floors having larger deflection than short span floors. For example, if we use the L/400 UDL deflection method to determine the floor vibration controlled spans, then it means that the L/400 limits allow a 3 m span floor to have a 7.5 mm deflection; consequently, it allows a 6 m span floor to have a 15 mm deflection. Is it rational? This can explain why we often found that the long span floors usually had poor vibration performance when the floor spans were determined using this method.

Therefore, if rationally using this method to avoid excessive vibrations in CLT floors, the design engineer needs a good judgment to select proper UDL deflection limits accordingly to the spans. A standardized design method is then needed for CLT floor vibration controlled design so that all CLT floors can be economically designed with satisfactory in-service performance.

3.2 Conventional Design Methods for Wood and Steel-Concrete Floors

The 2005 National Building Code of Canada (NBCC, 2005) recommends limits for static deflections of lightweight lumber joisted floors under 1 kN static load. It was shown that this method is only applicable for wood joisted floors without topping, i.e. floors having an area mass less than 30 kg/m^2 (Hu and Gagnon, 2009).

A design method was developed by Murray et al (1997) for heavy steel-concrete floors having fundamental natural frequency below 9 Hz and is proposed in the Steel Design Guide. This method limits the peak accelerations of floors to control the vibrations of heavy floors. Table 2 summarizes the scope of these common design methods. As revealed in the table, the scopes of the existing design methods do not cover CLT floors.

Table 2
Summary of common floor design methods for wood and steel-concrete floors and their scope

Design Method	2005 National Building Code of Canada	Non-Existence	Murray *et al.* (1997) for Steel-Concrete
Floor construction	Lightweight joisted floors without topping	1. Lightweight joisted floors with topping 2. CLT	Heavy steel-concrete
Floor mass character (kg/m^2)	15-30	30-150	> 150
Floor frequency characteristic (Hz)	> 15	> 9	< 9

3.3 FPInnovations' Proposed Design Method for Joisted Wood Floors

FPInnovations and UNB developed a design method to control vibrations in a broad range of wood joisted floor systems with an area mass varying from 15 kg/m^2 to 150 kg/m^2 and for fundamental natural frequency above 9 Hz (Hu, 2007). The design method used 1 kN static deflection and fundamental natural frequency as design parameters so that the floor stiffness and mass were accounted.

SINTEF (Homb, 2008) has conducted extensive field and laboratory studies on the vibration performance of CLT floors. SINTEF found that the FPInnovations' design criterion predicted the vibration performance of CLT field floors that matched well the occupants' expectation as illustrated in Figure 3. Each symbol in the figure represents a CLT field floor. If the symbol is below the curve, it means that the CLT floor is accepted to the criterion. SINTEF's field study has shown that the occupants were generally satisfied with the vibration performance of the CLT field floors studied.

SINTEF's study confirmed that FPInnovations' new design criterion is applicable to CLT floors. But the equations for calculation of the 1 kN static deflection and fundamental natural frequency of CLT floors needed to be developed. The equations in FPInnovations' design method were originally derived from the conventional wood joisted floors (Chui, 2002) based on ribbed plate theory, not for non-joisted slab floors like CLT floors. Meanwhile, the form of the criterion shown in Figure 3 also needed to be calibrated to the new equations to achieve a new design criterion for CLT floors. The next section provides details on the new proposed design method including the new design criterion and new calculation equations.

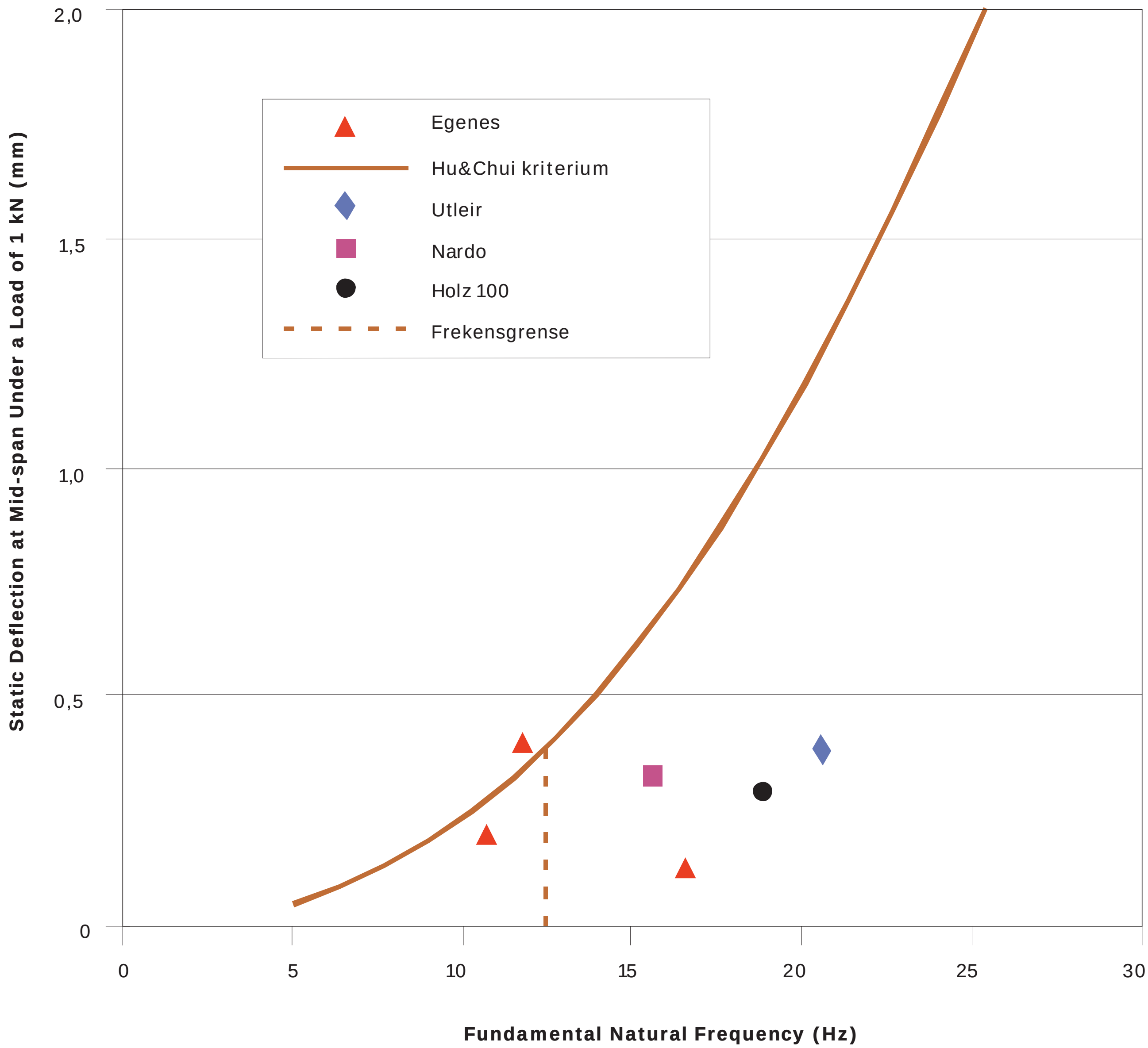

Figure 3
Comparison of FPInnovations' design criterion (Hu & Chui criterion) with the vibration performance of CLT floors studied at SINTEF (Byggforsk, Norway) (Homb, 2008)

4

PROPOSED DESIGN METHOD FOR CLT FLOORS

4.1 Scope and Limitations

At this point, the proposed new design method to control vibrations of CLT floors is for:

1. Bare floors with finishing, partitions and furniture, but without heavy topping;
2. Vibrations induced by normal walking;
3. Well-supported floors;
4. Well-connected CLT panels;
5. Inclusion of the self weight of CLT panels only; not live load.

However, because of the mechanics-based feature, it is possible to expand its scope and to include other construction details. A study has been planned to extend the scope in order to include various types of toppings and ceilings, and other floor design options, including heavy topping.

4.2 Expected Features

The proposed design method is focused on target features, which include, among others:

1. Simple for hand calculation;
2. User-friendly;
3. Mechanics-based using the design values for CLT panels available in producer's specifications;
4. Reliable to prevent CLT floors from excessive vibrations induced by normal walking.

4.3 Design Criterion

The design criterion is expressed in equation [1].

$$\frac{f}{d^{0.7}} \geq 13.0 \quad \text{Or} \quad d \leq \frac{f^{1.43}}{39} \qquad [1]$$

Where:
f = fundamental natural frequency calculated using equation [2] in Hz
d = 1 kN static deflection calculated using equation [3] in mm

4.4 Equations for Calculating the Criterion Parameters

The fundamental natural frequency can be obtained as:

$$f = \frac{3.142}{2l^2}\sqrt{\frac{EI_{eff}^{1m}}{\rho A}} \qquad [2]$$

Where:
f = fundamental natural frequency of 1 m CLT panel simply supported in Hz
l = CLT floor maximum span in meter
EI_{eff}^{1m} = effective apparent stiffness in the span direction for 1 m wide panel in N-m^2
ρ = density of CLT in kg/m^3
A = area of cross-section of 1 m wide CLT panel, i.e. thickness x 1 m wide in m^2

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

Where:
d = static deflection at mid-span of the 1 m wide simply supported CLT panel under 1 kN load in mm
P = 1000 N

4.5 Simple Form of Design Method

Inserting equations [2] and [3] into the design criterion, i.e. Eq. [1], we obtain the simple form of the design method expressed by equation [4].

$$l \leq \frac{1}{9.15}\frac{(EI_{eff}^{1m})^{0.293}}{(\rho A)^{0.123}} \qquad [4]$$

Using equation [4], we can determine the vibration controlled spans for CLT floors directly from the effective apparent stiffness in the span direction, density and cross-section area of 1 m wide CLT panels.

4.6 Verification

The design method was verified using FPInnovations' tests data obtained from a limited laboratory study on floors built with CLT panels having three thicknesses: 140 mm, 182 mm and 230 mm. In these tests, the performance of each floor was rated by a group of participants using the rating scale and procedure developed at FPInnovations back to 1970's (Onysko and Bellosillo, 1978), evolved in the 1990's (Hu, 1997), and recently simplified and reported by Hu and Gagnon (2010). Figure 4 shows one CLT floor built in laboratory for the vibration tests and subjective evaluation.

The static deflection under 1 kN load and fundamental natural frequency of each floor were calculated using equations [3] and [2], respectively. This allowed the calculation of the performance parameter using equation [1].

The comparison was also plotted in Figure 5. In the graph, each symbol represents a CLT floor while the curve is the design criterion defined by equation [1]. If the symbol is below the curve, it means the floor vibration performance is satisfactory and vice visa. The plot clearly demonstrates the reliability of the proposed design method for CLT floors.

Figure 4
CLT floor built in laboratory for the vibration tests and subjective evaluation

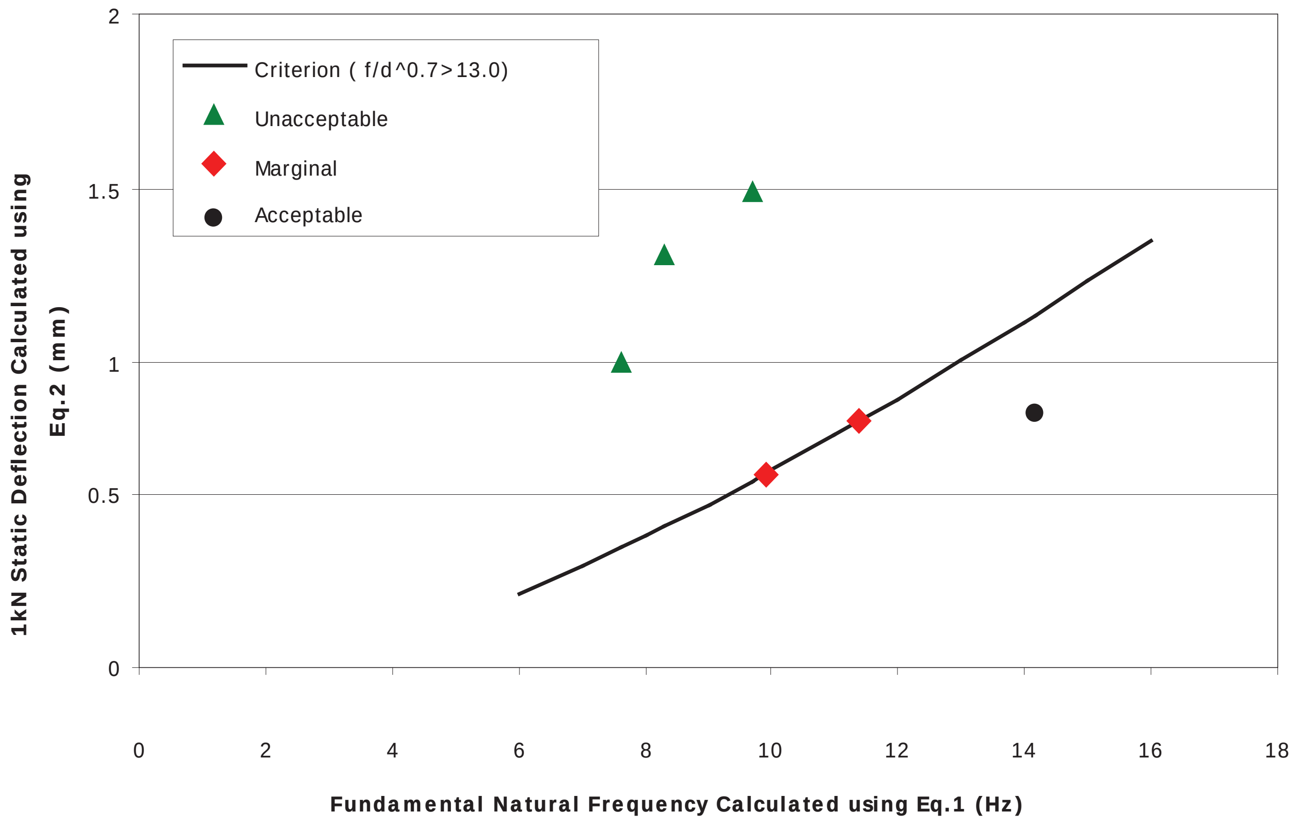

Figure 5
Predicted CLT floor vibration performance by the proposed design method vs. subjective rating by participants

4.7 Impact Study

4.7.1 Comparing Proposed Design Method with UDL Deflection Method

The vibration controlled CLT floor spans determined using the proposed design method were used to derive the equivalent UDL deflection limits using products from KLH (Austria) as an example. The total design load is 3.9 kN/m^2, which consists of 1.5 kN/m^2 dead load and 2.4 kN/m^2 live load. Producers' recommendation for the UDL deflection limit is L/400.

Table 3
Vibration controlled CLT floor with maximum spans determined using the new design method vs. UDL deflection criterion

Type of CLT	Thickness (mm)	Vibration Controlled Max. Span, L (m)	Equivalent UDL Criterion
5-layer (5s)	140	4.75	L/417
5-layer (5s)	182	5.50	L/497
7-layer (7ss)	230	7.00	L/606

As shown in Table 3, according to the proposed design method, more stringent UDL deflection limits should be imposed for longer span floors. This is more rational than the traditional UDL limits using a fixed ratio such as L/400 for all spans.

4.7.2 Comparing CLT Floor Spans Determined using the Proposed Design Method with Spans Determined using the CLTdesigner Software (Schickhofer, 2010)

The vibration controlled CLT floor spans determined using the proposed design method were compared with the spans determined using CLTdesigner, a software developed at the University of Graz in Austria (Schickhofer, 2010). Table 4 provides the comparison.

Table 4
Vibration controlled CLT floor spans determined using the new design method vs. spans determined using the CLTdesigner software

CLT Thickness (mm)	FPInnovations' Design Method Proposed Span (m)	CLTdesigner Proposed Span for 1% Damping and No-topping Floors (Schickhofer, 2010) (m)
100	3.58	3.53
120	3.76	3.75
140	4.50	4.43
160	4.80	4.76
180	5.16	5.14
200	5.68	5.67
220	5.84	5.89
240	6.09	6.17

As shown in Table 4, the vibration controlled spans of bare CLT floors predicted by the proposed design method are almost the same as the spans determined using the CLTdesigner software.

5

WORKING EXAMPLES FOR THE NEW DESIGN METHOD

Examples are given below to calculate the vibration controlled spans of two CLT floors using the simple form of the new design method given in Eq. [4].

Example 1: The design properties of the CLT panel specified by the producer are listed below (KLH, 2008).

- Type = 7ss
- Thickness = 0.23 m
- Density = 480 kg/m^3
- Width = 1.0 m
- MOE = 12 GPa

Table 5
Specified effective apparent I (I_{eff})

Simple Span (m)	I_{eff} (cm^4)
2	45,979
4	74,100
6	84,238
8	88,534

Calculation of the vibration controlled span for the floor using this CLT panels follows the steps below.

Step 1: Curve-fit the data in Table 5 to obtain equation [5] to calculate Ieff from the span (Figure 6).

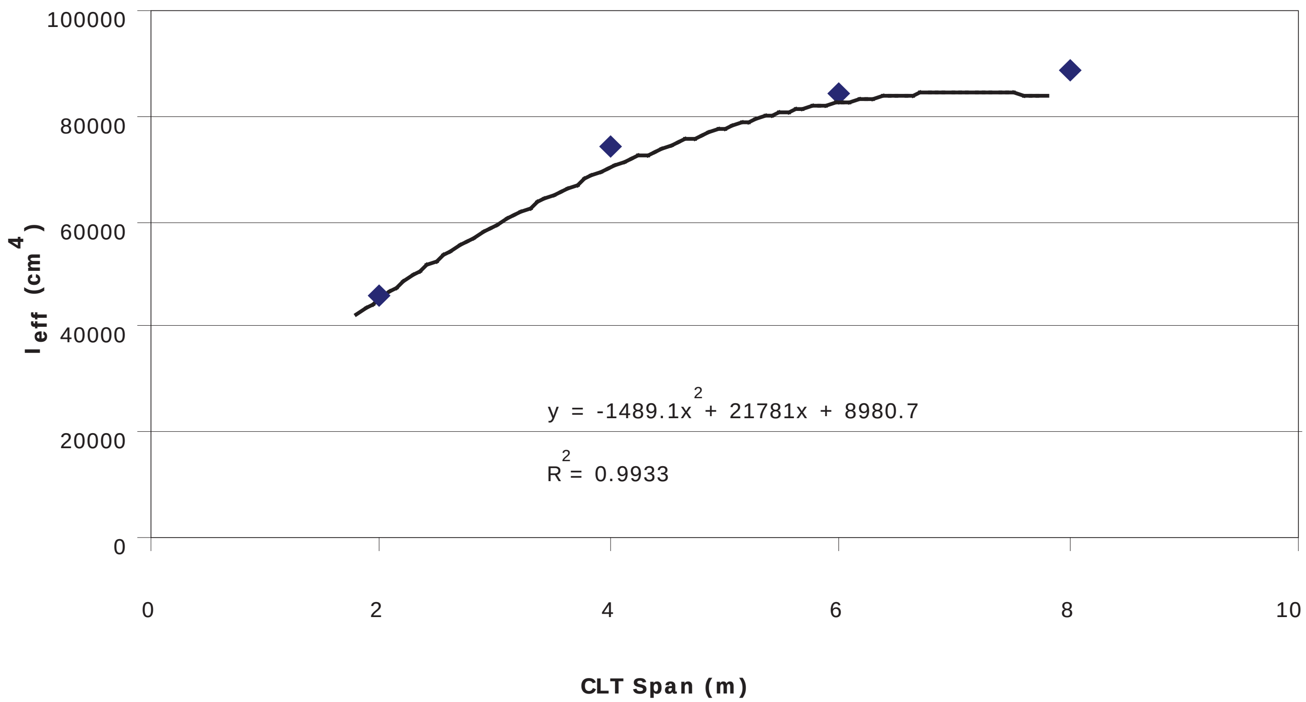

Figure 6
CLT I_{eff} in function of span of 7 ss, 0.23 m thick CLT

$$I_{eff} = (-1489.1l^2 + 21781l + 8980.7)/10^8 \qquad [5]$$

where l is the CLT span in meter and I_{eff} is the effective I in m^4 given by the producer.

Step 2: Calculate the first trial span, assuming that the span is 30 times the thickness of the CLT panel; this leads to the first trial span of 6.9 m.

Step 3: Insert the trial span of 6.9 m into Eq. [5] to calculate trial I_{eff}; this leads to:

$$I_{eff} = 0.00088374 \text{ m}^4$$

Step 4: Insert the value of trial I_{eff}, the design values of density, thickness, MOE and width of the CLT panel into Eq. [4] to calculate the vibration controlled maximum span limit; this leads to:

$$l \le \frac{1}{9.15}\frac{(12*10^9*0.00088374)^{0.293}}{(480*0.23*1.0)^{0.123}} = 7.01m$$

Step 5: If the trial span is less or larger than the vibration controlled span limit, then repeat steps 3 and 4 using a new trial span to determine the new vibration controlled maximum span limit.

Step 6: Repeat step 5 until the new trial span is almost equal to the new vibration controlled span limit. This can be easily performed if we implement the procedure into an Excel spreadsheet as shown in Table 6.

Table 6
Method to implement the calculation procedure for Example 1 into Excel

Thickness (m)	Trial Span (m)	I_{eff} Eq. [5] (m^4)	MOE (GPa)	Density (kg/m^3)	Span Limit Eq.[4] (m)
0.23	6.90	0.00088374	12	480	7.01
0.23	7.01	0.00088491	12	480	7.01
0.23	7.30	0.00088628	12	480	7.02

Finally, examining the iteration results shown in Table 6, we find that the trial span of 7.01 m is equal to the vibration controlled span limit; therefore, we can comfortably conclude that 7.01 m is the vibration controlled span for the CLT floor using the 7ss, 0.23 m thick CLT panels.

Above example shows the procedure to determine the vibration controlled spans for floors using CLT panels with the specified effective apparent bending stiffness (App. EI_{eff}). However, some producers do not provide the effective apparent bending stiffness, but rather specify the design values of effective true bending stiffness (True EI_{eff}) and effective shear stiffness (GA_{eff}). In this case, the effective apparent bending stiffness (App. EI_{eff}) can be determined using the following equation:

$$EI_{eff}^{1m} = \frac{1}{\dfrac{1}{True\ EI} + \dfrac{11.52}{GA * l^2}} \qquad [6]$$

Next example demonstrates the procedure to determine the vibration controlled spans for floors using CLT panels with the given true EI_{eff} and GA_{eff}.

Example 2:

Design values of the CLT panel properties are:

- Thickness = 0.14m
- Density = 500 kg/m^3
- Width = 1.0 m
- True EI_{eff} = 2.143x10^6 N-m^2
- GA_{eff} = 1.082x10^7 N

Calculation of the vibration controlled span for the floor using this CLT panel follows the steps below.

Step 1: Calculate the first trial span, assuming that the trial span is 30 times the thickness; this leads to the first trial span of 4.2 m.

Step 2: Insert the first trial span of 4.2 m into Eq. [6] to determine the trial effective apparent stiffness, EI_{eff}^{1m} from the design value of the true EI_{eff} and GA_{eff}; this leads to:

$$EI_{eff}^{1m} = 1.898\text{x}10^6 \text{ N-m}^2$$

Step 3: Insert the value of trial EI_{eff}^{1m} , the design values of density, thickness and width of the CLT panel into Eq. [4] to calculate the vibration controlled maximum span limit; this leads to:

$$l \leq \frac{1}{9.15} \frac{(1898000)^{0.293}}{(500 * 0.14 * 1.0)^{0.123}} = 4.48m$$

Step 4: If the trial span is less or larger than the vibration controlled span limit, then repeat steps 2 and 3 using a new trial span to determine the new vibration controlled maximum span limit.

Step 5: Repeat step 4 until the new trial span is almost equal to the new vibration controlled span limit. This can be easily performed if we implement the procedure into an Excel spreadsheet as shown in Table 7.

Table 7
Method to implement the calculation procedure for Example 2 into Excel

Thickness (m)	Trial Span (m)	True EI_{eff} ($\text{x}10^6$ N-m^2)	GA_{eff} ($\text{x}10^7$ N)	EI_{eff}^{1m} Eq. [6] ($\text{x}10^6$ N-m^2)	Density (kg/m^3)	Span Limit Eq.[4] (m)
0.14	4.20	2.143	1.082	1.900	500	4.48
0.14	4.48	2.143	1.082	1.924	500	4.50
0.14	4.50	2.143	1.082	1.926	500	4.50
0.14	4.60	2.143	1.082	1.934	500	4.50

Finally, examining the iteration results shown in Table 7, we find that the trial span of 4.5 m is equal to the vibration controlled span limit; therefore, we can comfortably conclude that the 4.5 m is the vibration controlled span for the CLT floor using the 0.14 m thick panels.

6 CONCLUSION

It is concluded that the proposed design method to determine vibration controlled maximum spans of bare CLT floors is promising. It is mechanics-based, utilizes the fundamental mechanical properties of CLT, is user-friendly, and reliable.

7 RECOMMENDATIONS

Wide acceptance of the proposed design method relies on its use and evaluation by designers and manufacturers. FPInnovations welcomes feedback on the proposed design method. From a vibration control point of view, the low damping ratio is one of the major weaknesses of bare CLT floors. Any measures for increasing the damping ratio through CLT product design and CLT floor construction detail will enhance the vibration performance of CLT floor systems.

8 REFERENCES

Chui, Y. H. 2002. Application of ribbed-plate theory to predict vibrational serviceability of timber floor systems. In *Proceedings of the 7th World Conference on Timber Engineering*, August 12-15, 2002, Shah Alam, Malaysia, paper no. 9.3.1, vol. 4, 87-93.

Gagnon, S., and L.J. Hu. 2007. Trip report : Sweden, Norway and France, November 1-11, 2007. Quebec: FPInnovations. 22 p.

Homb, A. 2008. *Vibrasjonsegenskaper til dekker av massivtre* (in Norwegian). Prosjektrapport 24. Oslo, Norway: SINTEF Byggforsk. 57 p.

Hu, L. J. 1997. Serviceability design criteria for commercial and multi-family floors. Canadian Forest Service Report No. 4. Quebec: Forintek Canada Corp. 9 p.

______. 2007. Design guide for wood-framed floor systems. Canadian Forest Service Report No. 32. Quebec: FPInnovations. 60 p. + appendices.

Hu, L. J., and S. Gagnon. 2009. Verification of 2005 NBCC maximum spans for concrete topped lumber joist floors. Canadian Forest Service Report No. 2. Quebec: FPInnovations. 20 p.

_________. 2010. Construction solutions for wood-based floors in hybrid building systems. Canadian Forest Service Report No. 1. Quebec: FPInnovations. 5 p.

KLH. 2008. Engineering. Version 01/2008. www.klh.cc (accessed July 23, 2009).

Holz.Bau Forschungs GmbH. 2010. CLTdesigner. Version 1.1.2. http://www.cltdesigner.at/webstart/testversion/cltdesignertestversion.jnlp (accessed July 1st, 2010).

Murray, T.M., D.E. Allen, and E.E. Ungar. 1997. *Floor vibrations due to human activity.* Steel Design Guide Series 11. Chicago: American Institute of Steel Construction and Canadian Institute of Steel Construction. 69 p.

National Research Council (NRC). 2005. *National building code of Canada, 2005*. Ottawa: Canada. National Research Council. 2 v.

Onysko, D., and S.B. Bellosillo. 1978. *Performance criteria for residential floors: Final report to Central Mortgage and Housing Corporation*. Grant no. 120-74. Ottawa: Canada. Eastern Forest Products Laboratory. 16 p.

Schickhofer, G. 2010. Comments on FPInnovations new design method for CLT floor vibration control. E-mail message to author, July 1st, 2010.

Ungar, E. E. 1992. Structural damping. In *Noise and vibration control engineering: Principles and applications*, edited by Leo L. Beranek and Istvan L. Vér, 451-481. New York: John Wiley and Sons.

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Fire

Fire performance of cross-laminated timber assemblies

CHAPTER 8

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ABSTRACT

Cross-laminated timber (CLT) panels have the potential to provide excellent fire resistance often comparable to typical massive assemblies of non-combustible construction. Due to the inherent nature of thick timber members to slowly char at a predictable rate, massive wood systems can maintain significant structural capacity for extended durations when exposed to fire.

In order to facilitate the acceptance of future code provisions for the design of CLT panels for fire resistance, a one-year research project was launched at FPInnovations in April 2010. The main objective of the project is to develop and validate a generic procedure to calculate the fire-resistance ratings of CLT wall and floor assemblies. A series of full-scale fire-resistance experiments is currently under way to allow a comparison between the fire-resistance rating measured during a standard fire-resistance test and that calculated using the proposed procedure. In light of the fact that the research project is just beginning, a simple but conservative design procedure is presented in this chapter, following the current state-of-the-art information from Europe and North America.

The Canadian Standard for Engineering Design in Wood (CSA O86) can be used to calculate the fire-resistance rating of CLT panels along with the same methodology that is currently used for calculating the fire-resistance ratings of glued-laminated timber and "heavy" timber in the United States, New Zealand and Europe. This method is called the reduced (or effective) cross-section method and allows the use of the design values that can be found in CSA O86. It is recommended that a qualified fire protection engineer undertake or oversee the design of CLT assemblies for fire resistance. The fire protection engineer should work closely with the structural engineer so that the implications of fire exposure to the structural design are considered.

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1 INTRODUCTION

Cross-laminated timber (CLT) panels have the potential to provide excellent fire resistance often comparable to typical massive assemblies of non-combustible construction. Due to the inherent nature of thick timber members to char slowly and at a predictable rate, massive wood systems can maintain significant structural resistance for extended durations when exposed to fire.

Building regulations require that key building assemblies exhibit sufficient fire resistance to allow time for occupants to escape and to minimize property losses. The intent is to compartmentalize the structure to prevent the spread of fire and smoke, and to ensure structural adequacy to prevent or delay collapse. The fire-resistance rating of a building assembly has traditionally been assessed by subjecting a replicate of the assembly to the standard fire-resistance test (CAN/ULC S101 in Canada, ASTM E119 in the USA and ISO 834 in most other countries). These three standards are all very similar as they require a wall or floor assembly to be exposed to a severe fire in which the temperature of fire gases increases over time following a specified temperature-time curve. The test standards also require the assembly to be loaded, and in North America, assemblies are typically loaded to their full design load based on strength (as opposed to serviceability criteria such as deflection). This ensures that the fire-resistance rating obtained for a particular assembly is appropriate for use in any building independently of the load conditions (assuming they satisfy the structural requirements).

The standard fire-resistance test has three failure criteria:

- Firstly, the structural criterion must be met: the assembly must support the applied load for the duration of the test.
- Secondly, the insulation criterion must be met: the assembly must prevent the temperature rise on the unexposed surface from being greater than 180°C at any location, or an average of 140°C measured at a number of locations.
- Lastly, the integrity criterion must be met: the assembly must prevent the passage of flame or gases hot enough to ignite a cotton pad.

The time at which the assembly can no longer satisfy these three criteria defines the assembly's fire-resistance rating.

The fire performance of solid wood assemblies is not new to the National Building Code of Canada (NBCC). The minimum thicknesses of both loadbearing and non-loadbearing solid wood walls and floors are specified in Section 2.4 of Appendix D of the NBCC for fire-resistance ratings of 30, 45, 60 and 90 minutes. The solid wood assemblies currently referred to in the NBCC consist of a single layer of lumber, often tongue-and-groove and nailed. CLT panel assemblies which use multiple layers of lumber glued together have the potential to provide better fire performance due to the backing of joints from one layer to the next.

In order to facilitate the acceptance of future code provisions for the design of CLT panels for fire resistance, a one-year research project was launched at FPInnovations in April 2010. The main objective of the project is to develop and validate a generic procedure to calculate the fire-resistance ratings of CLT wall and floor assemblies. A series of full-scale fire-resistance experiments is currently under way to allow a comparison between the fire-resistance rating measured during a standard fire-resistance test and that calculated using the proposed procedure. In light of the fact the research project is just beginning, a simple but conservative design procedure is presented in this chapter following the current state-of-the-art information from Europe and North America. Over the next year, this procedure will be refined with the intention of reducing the level of conservatism to ensure the product is used efficiently.

2

FIRE RESISTANCE

When designing CLT panel buildings, it is often necessary to determine the fire-resistance rating provided by the assembly to ensure its performance satisfies the building code requirements. In some instances, such as for non-loadbearing wall assemblies, only the separating function is necessary for defining the fire resistance; that is, the assembly must only meet the insulation and integrity criteria. In the case of loadbearing walls and all floor assemblies, the assembly must provide both the separating function as well as structural integrity for the duration of the fire-resistance rating. For this reason, the determination of fire-resistance rating has been split into requirements for separating fire resistance and structural fire resistance in this chapter.

Since the following calculation method is engineering based, there is no reason to limit the duration of fire resistance calculated. In fact, for exposures in excess of one hour, the actual rate of charring will likely be less than that assumed below, leading to increasingly conservative results for longer duration fire-resistance ratings.

2.1 Fire Separating Function

The separating function of CLT panel assemblies can easily be met provided some simple steps are taken. The most important is that the panels and joints between panels be effectively sealed to prevent air or hot gases from penetrating the assembly during fire exposure. This may be accomplished in a number of ways such as by edge-gluing at least one internal ply of the panel, by using an adhesive that foams sealing gaps between boards throughout the panel or by applying gypsum board.

Another important aspect of the assembly with respect to the separating function is the integrity of the joints between panels. The fire protection engineer must ensure that the joint details between the panels are sufficient so as not to reduce the fire resistance of the assembly. Similar to the point discussed above, it is important that the joints do not permit air or hot gases to penetrate between the adjoining panels. The use of splines, tongue-and-groove joints, or lap joints that are tightly fitted should provide sufficient fire resistance. It is also recommended, in the case of a lap joint, that either a bead of construction adhesive or a gasket material be used to ensure that the joint is sealed. The joint detail shown in Figure 1 is a half-lapped joint that has been tested in full-scale wall and floor fire-resistance tests and proven to have sufficient fire resistance.

For non-loadbearing wall assemblies, the fire-resistance rating can be calculated based on the charring rate. A conservative assumption would be to permit the char depth to reach within 25 mm of the unexposed side. This corresponds to a temperature on the unexposed side less than 50°C based on research reported by Janssens and White (1994). This depth will also ensure that the wall assembly will remain structurally sound so as to remain self supporting.

Provided the above fire separating criteria (insulation and integrity) are met, the fire-resistance rating for all loadbearing assemblies will be governed by the loss of structural resistance during fire exposure.

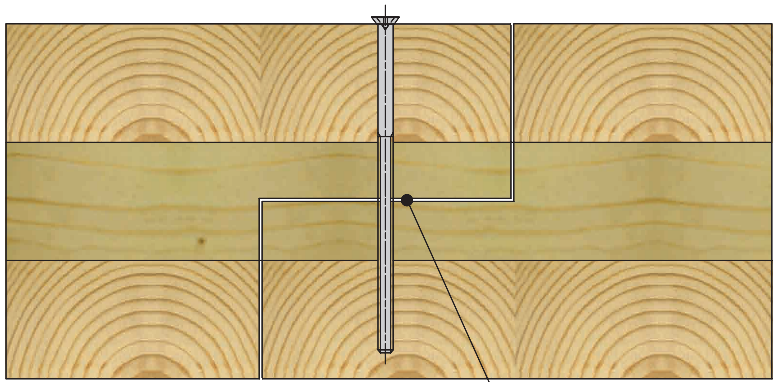

Figure 1
Cross-laminated timber panel joint details

2.2 Structural Fire Resistance

The following section proposes a calculation procedure for determining the fire-resistance rating of a CLT panel assembly. The Canadian Standard for Engineering Design in Wood (CSA O86) can be used to calculate the fire-resistance rating of CLT panels along with the same methodology that is currently used for calculating the fire-resistance ratings of glued-laminated timber and "heavy" timber in the United States, New Zealand and Europe. This method is called the reduced (or effective) cross-section method and allows the use of the design values found in the wood design standard CSA O86-09. The calculation procedure described below should be completed by a fire protection engineer familiar with wood design.

This calculation procedure applies only to CLT panel assemblies exposed to the standard fire-resistance test exposure. If an alternative exposure is chosen, then a heat transfer analysis may be needed in order to determine the appropriate charring rate. Research on the performance of CLT panel assemblies exposed to non-standard fire exposures is currently under way at Carleton University.

Calculation of the fire-resistance rating of CLT wall or floor assemblies is outlined in the following five steps. Figure 2 shows a CLT panel exposed to fire and some of the nomenclature used in calculating the fire-resistance rating. Note that, in calculating the moment resistance for floor panels or axial resistance for wall panels, only plies running in the direction of the applied stress shall be considered. The cross plies, while providing a fixed spacing between the longitudinal layers, are assumed to provide no structural contribution to the panels' moment or axial resistance.

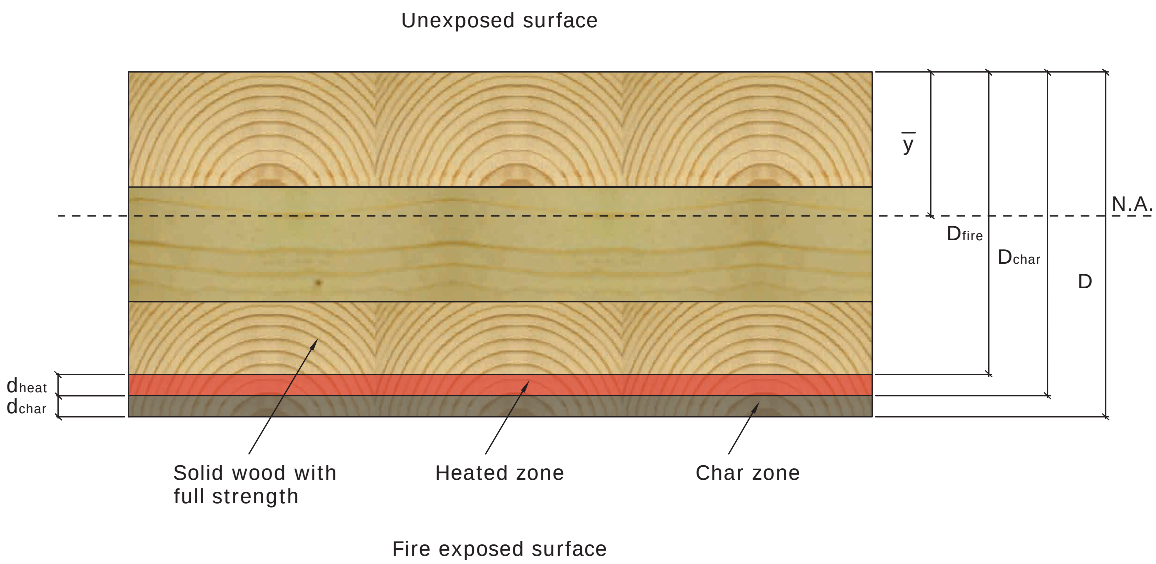

Figure 2
Terms used in calculating the fire-resistance rating of a cross-laminated timber panel exposed to fire from below

Step 1: Calculation of the char depth

Calculate the depth of char based on a fixed charring rate multiplied by the duration of exposure (i.e. the desired fire-resistance rating). The charring rate, β, can be taken as 0.65 mm/min as is specified in Eurocode 5 Part 1-2, Table 3.1 for one dimensional charring. The depth of char can be calculated as follows:

$$d_{char} = \beta t \qquad [1]$$

where:

d_{char} = depth of char in mm
β = char rate in mm/min
t = duration of fire exposure in min.

The remaining cross-section depth, D_{char}, is simply the original depth D minus the char depth:

$$D_{char} = D - d_{char} \qquad [2]$$

Step 2: Determination of effective residual cross-section

Using the remaining cross-section depth calculated in Step 1, subtract an additional thickness to account for the loss of strength in the heated zone beneath the char front. This reduction in depth, d_{heat}, shall be taken as 10 mm if the heated zone is in tension (i.e. floor assembly) and 16 mm if the zone is in compression (i.e. wall assembly) as suggested by Schmid et al. (2010). The cross-section depth remaining for design under fire conditions D_{fire} can be calculated as:

$$D_{fire} = D_{char} - d_{heat} \qquad [3]$$

Since the cross plies do not contribute to the structural capacity of the panel, if D_{fire} falls between plies that are parallel to the applied stress, then D_{fire} must be reduced to the edge of the nearest ply.

If the exposure time is less than 20 minutes, then the heated zone is to vary linearly from zero at time zero to the depth d_{heat} at 20 minutes. This is the same practice used in Eurocode 5 Part 1-2 for the heated zone used for glulam and heavy timber.

Step 3: Find location of neutral axis and moment of inertia of the residual cross-section

If the plies in the direction of the applied stress all consist of the same grade and species group and therefore have the same modulus of elasticity, then the following two equations can be used to calculate the distance of the neutral axis from the unexposed surface and the moment of inertia of the residual cross-section:

$$\bar{y} = \frac{\sum_i \tilde{y}_i \, D_i}{\sum_i D_i} \qquad [4]$$

where:

$\bar{y}$ = distance from the unexposed surface of the panel to the neutral axis in mm

$\tilde{y}_i$ = distance from the unexposed surface of the panel to the centroid of ply i in mm

D_i = remaining depth of ply i in mm.

The moment of inertia of the residual cross-section can be calculated as follows:

$$I_{eff} = \sum_i \frac{B \, D_i^{\,3}}{12} + \sum_i B \, D_i \, d_i^{\,2} \qquad [5]$$

where:

I_{eff} = effective moment of inertia for the cross-section in mm^4

B = unit width of the panel (typically 1000 mm)

d_i = distance from the neutral axis to the centroid of ply i in mm.

If the plies parallel to the direction of the applied stress do not all have the same modulus of elasticity, then the following equation must be used to calculate the location of the neutral axis:

$$\bar{y} = \frac{\sum_i \tilde{y}_i D_i \, E_i}{\sum_i D_i \, E_i} \qquad [6]$$

where:

E_i = modulus of elasticity of ply i in MPa.

Similarly, the effective stiffness can be calculated as:

$$EI_{eff} = \sum_i \frac{B\,D_i^{\,3}}{12} E_i + \sum_i B\,D_i\,d_i^{\,2} E_i \qquad [7]$$

where:

EI_{eff} = effective stiffness in N mm².

Step 4: Calculation of structural resistance

Using the reduced cross-section determined in Step 2 and ignoring any contribution to the strength provided by the plies perpendicular to the applied stress, calculate the member resistance using the design values specified in CSA O86. (Note that this is conservative as most codes allow for an increase in the strength values used from the fifth percentile strength values specified in the code. For example, Eurocode 5 Part 1-2 permits the use of the twentieth percentile strength values for the calculation of fire-resistance ratings). In calculating the resistance, it is common to use the resistance factor, $\varphi = 1.0$, when determining the fire-resistance rating of a member as specified in Eurocode 5 Part 1-2. The calculation of the bending moment resistance for floors and the axial resistance for walls has been split into Steps 4a and 4b respectively due to the different equations used.

Step 4a: Calculation of the moment resistance (floors)

The bending moment resistance of a CLT floor assembly can be calculated based on the factored bending strength of the wood and the effective section modulus. The moment resistance can be calculated using the procedure in CSA O86-09 under Clause 5.5.4.

Special considerations for calculating the resistance of CLT under fire conditions using Clause 5.5.4 of CSA O86-09 include:

– The resistance factor φ can be taken as 1.0 for fire design.
– The duration of load factor, K_D, can be taken as 1.15 for short term loading (see Table 4.3.2.2 in CSA O86-09). The short term duration of load factor is chosen based on the rationale that the fire exposure and resulting maximum stress condition in the reduced cross-section is an event that lasts in the order of a few minutes to a few hours. Short term loading is specified in CSA O86 as a load that is not expected to last more than seven days.
– The effective section modulus is calculated based on the moment of inertia of the plies running in the direction of the applied stress and the location of the neutral axis (NA).

$$S_{eff} = \frac{I_{eff}}{D_{fire} - \bar{y}} \qquad [8]$$

where:

S_{eff} = effective section modulus per unit width in mm³/m
I_{eff} = effective moment of inertia per unit width in mm⁴/m
$\bar{y}$ = distance from the unexposed surface of the CLT panel to the neutral axis in mm.

The above equation assumes that each of the plies considered in calculating the moment of inertia have the same modulus of elasticity. If this is not the case, then EI_{eff} must be calculated based on equation 7 and the effective section modulus calculated as

$$S_{eff} = \frac{EI_{eff}}{E\left(D_{fire} - \bar{y}\right)} \quad [9]$$

where:

EI_{eff} = effective stiffness per unit width in N mm²/m

E = modulus of elasticity of the ply that experiences the greatest tensile stress in MPa.

– The size factor K_{Zb} shall be taken as 1.0. Currently, a size factor specific to CLT has yet to be developed, and use of the size factor for sawn lumber which is greater than 1.0 may not be appropriate. Therefore, to be conservative, a size factor K_{Zb} = 1.0 is used for fire design. When a size factor is developed for CLT in bending, it should replace the conservative value of 1.0.
– The lateral stability factor for bending members K_L shall be taken as 1.0 for CLT panels.

Note that the residual cross-section, neutral axis, moment of inertia and section modulus are continually changing during fire exposure as the cross-section is being reduced. Therefore, in cases where fire resistance may be the controlling design factor, it is recommended that these calculations be completed in a spread sheet so the moment resistance can be calculated as a function of time.

An example showing the calculation of the bending moment resistance of a floor assembly is shown in Section 6.1 of this chapter.

Step 4b: Calculation of axial resistance (walls)

The axial resistance of a CLT wall assembly can be calculated based on the factored compressive strength of the wood, the cross-sectional area and the slenderness. The axial resistance can be calculated using the procedure in CSA O86-09 under Clause 5.5.6.

Special considerations for calculating the axial resistance of CLT under fire conditions using Clause 5.5.6 of CSA O86-09 include:

– The resistance factor φ can be taken as 1.0 for fire design.
– The duration of load factor, K_D, can be take as 1.15 for short-term loading (see Table 4.3.2.2 in CSA O86-09).
– The size factor, K_{Zc}, can be taken as 1.0. Currently, a size factor specific to CLT has yet to be developed, and use of the size factor for sawn lumber which is greater than 1.0 may not be appropriate. Therefore, to be conservative, a size factor K_{Zc} = 1.0 is used for fire design. When a size factor is developed for CLT in compression, it should replace the conservative value of 1.0.
– In order to calculate the slenderness factor, the slenderness ratio must be calculated using the following equation (as derived in Chapter 3, Equations 52 and 53).

$$C_c = \frac{H}{2\sqrt{3}\sqrt{I/A}} \qquad [10]$$

where:

C_c = slenderness ratio
H = height of the wall assembly in mm
I = moment of inertia in mm^4
A = remaining area of the plies in the direction of the applied stress in mm^2.

Note that the residual cross-section, neutral axis, moment of inertia and slenderness ratio are continually changing during fire exposure as the cross-section is being reduced. Therefore, in cases where fire resistance may be the controlling design factor, it is recommended that these calculations be completed in a spread sheet so the axial resistance can be calculated as a function of time.

An example showing the calculation of the axial resistance of a wall assembly is given in Section 6.2 of this chapter.

Step 5: Comparison of residual resistance to calculated load

Compare the calculated resistance of the CLT assembly to the appropriate load. The calculation of the load may be based on the specified load condition or the reduced load factors found in Commentary A, Section 25 of the National Building Code of Canada (NBCC) based on judgment by the fire protection engineer. Provided that the structural resistance of the remaining CLT panel cross-section is greater than the load, the fire-resistance rating of the panel will be greater than that of the duration used to determine the remaining cross-section.

3 ADDITIONAL CONSIDERATIONS

The following topics are additional considerations for the design of CLT panel assemblies/buildings for fire resistance.

3.1 Design of CLT Assemblies for Fire Resistance

It is recommended that a qualified fire protection engineer undertakes or oversees the design of CLT assemblies for fire resistance. The fire protection engineer should work closely with the structural engineer so that the implications of fire exposure to the structural design are considered.

3.2 Gypsum Board Protection

The above calculations are based on an unprotected CLT panel in standard fire exposure. If gypsum board is applied on the exposed side, a simple addition of 30 minutes to the fire-resistance rating can be made for one layer of 15.9 mm type X gypsum board and 60 minutes for two layers of 15.9 mm type X gypsum board. These additional times are based on experiments completed on beams in tension at the US Forest Products Laboratory by White (2009). Alternatively, the methodology outlined under Section 3.4.3 of Eurocode 5 Part 1-2 can be used provided that data on the protection to be used such as time to start of charring, charring rate during protection and time to failure of the protection are available. (Note: It is advisable that values for gypsum board are based on North American gypsum board products).

3.3 Adhesive

In assuming the charring rate is constant at 0.65 mm/min, it is assumed the CLT panel behaves like solid wood or glulam. Therefore, it is assumed the adhesive does not negatively affect the performance of the CLT panel when exposed to fire. The adhesive plays two main roles:

1. The adhesive must maintain the bond between wood members when heated to prevent failure at the bond line while the wood still has considerable strength. If a thermoplastic adhesive has been used in the manufacturing of the panel in either end joining or face bonding the plies and if the depth of char (d_{char}) determined in Step 1 results in the char front being less than 12 mm, then it is possible the strength of the adhesive in that bondline has been compromised. This 12 mm depth corresponds to a bondline temperature of 150°C based on research reported by Janssens and White (1994) and the 150°C corresponds to research reported by Craft et al. (2008). Since the proposed method (in Step 2) uses zero strength zones (d_{heat}) to account for the heated zone of 10 mm for floors and 16 mm for walls, the impact of a thermoplastic can be considered to be accounted for. However, if further research indicates that the depth of the zero strength zones can be decreased from the current values, then this effect should be considered.

2. The adhesive must be of sufficient strength above the temperatures associated with the charring of wood in order to ensure the charred layers of wood do not fall off since this char plays an important insulating function for the remaining cross-section. If not, then consideration should be given to the impact on the charring rate and the associated reduction in the fire-resistance rating of the assembly. A research paper by Frangi et al. (2009) provides guidance on calculating the charring rate when an adhesive fails to hold the char layer in place so that it can no longer protect the uncharred wood.

It should be noted that, while the product standards have not yet been finalized for CLT, it appears as though there will be some minimum level of heat durability required of the adhesives used in the manufacturing of CLT. The product standards are looking at referencing the adhesive specification ASTM D2559-10a Service Class A in the United States and CSA O112.10 in Canada, and adding ASTM D7247.

4
FLAMMABILITY OF CLT

The flammability of materials is regulated in the National Building Code of Canada (NBCC) in order to limit the contribution of room linings to fire growth within a compartment. In many cases, such as within residential suites and in many sprinklered buildings, the flame-spread rating is limited to 150. The flame-spread rating is measured based on the standard test method CAN/ULC-S102-07 in Canada and ASTM E84-10 in the USA.

The flammability of CLT panels is the same as that of the lumber with which it is manufactured. Under Section D-3.1 of Appendix D in the NBCC, lumber is assigned a flame-spread rating of not more than 150 provided the thickness is greater than 16 mm. This rating applies regardless of whether the lumber is unfinished or coated with paint or varnish less than 1.3 mm in thickness. Therefore, CLT may be left exposed anywhere the flame-spread rating is permitted to be 150 or less. It is quite probable that the actual flame-spread rating would be much lower due to the fact that the main species used in CLT (particularly in the exterior plies) are spruce or Douglas fir, which both tend to have a flame-spread rating in the order of 60 – 100[1].

[1]As summarized in Forintek Canada Corp. Technote TEC-49E: Surface Flammability of Building Materials

5 NOMENCLATURE

A	Cross-sectional area of the plies in the direction of the applied stress per unit width, mm^2/m
B	Unit width of CLT panel, mm (typically 1000 mm)
D	Original thickness of CLT panel, mm
D_i	Remaining depth of ply i, mm
D_{char}	Remaining cross-section thickness excluding charred thickness, mm
D_{fire}	Effective cross-section thickness used in calculating resistance of CLT assembly, mm
D_L	Specified dead load, kN/m^2
d_{char}	Depth of char, mm
d_{heat}	Depth of heated zone which is assumed to have zero strength, mm
d_i	Distance from the neutral axis to the centroid of ply i, mm
E	Modulus of elasticity, MPa
E_i	Modulus of elasticity of ply i, MPa
EI_{eff}	Effective stiffness, N mm^2
F_b	Factored bending strength taken from CSA O86, MPa
F_c	Factored compressive strength taken from CSA O86, MPa
f_b	Specified bending strength taken from CSA O86, MPa
K_C	Slenderness factor as specified in CSA O08 Clause 5.5.6.2.3
K_D	Load duration factor used to modify the specified strength (taken from CSA O86 Clause 4.3.2.2)
K_{Zc}	Size factor for compression (to be taken as 1.0 for CLT in fire)
I_{eff}	Moment of inertia of CLT floor assembly per unit width, mm^4/m
L_L	Specified live load, kN/m^2
l	Unsupported span of floor panel, mm
M_r	Moment resistance of CLT floor assembly per unit width, N mm/m
$m_{f(fire)}$	Factored moment under fire conditions, kNm/m
S_{eff}	Effective section modulus per unit width, mm^3/m
t	Duration of fire exposure (min)
$w_{f(fire)}$	Factored distributed load under fire conditions, kN/m^2
$\overline{y}$	Distance from the top surface of the assembly to the neutral axis of the CLT panel, mm
$\widetilde{y}_i$	Distance from the unexposed surface of the CLT panel to the centroid of the ply i, mm
φ	Resistance factor (taken as 1.0 for fire design)
β	Char rate under standard fire exposure (mm/min)

6

WORKING EXAMPLES

The following two examples are intended to provide clarification of the design procedure described above. The examples only consider the structural fire resistance of the assembly and do not cover the other aspects of design such as joint details or reductions based on adhesive performance.

6.1 CLT Floor Example

The following floor example follows the steps listed above for determining whether the structural fire resistance of a 5-ply CLT floor assembly meets the hypothetically required fire-resistance rating of 90 minutes. The floor assembly has the following specifications:

– Composed of a 5-ply CLT panel
– Longitudinal plies are 38 mm x 89 mm SPF 1650 F_b MSR
– Cross plies are 38 mm x 89 mm SPF No.3/Stud
– Floor span = 4730 mm
– Adhesive is a thermoset (phenol-resorcinol-formaldehyde)
– Dead load is 5.1 kPa and live load is 20.5 kPa (based on the design load and a live to dead load ratio of 4:1).

Factored load:

The load on the floor in the event of fire can be calculated using the reduced load factors found in Commentary A, Section 25 of the NBCC.

$$w_{f(fire)} = D_L + 0.5L_L = (5.1) + 0.5 \cdot (20.5) = 15.4 \text{ kN/m}^2$$

$$m_{f(fire)} = \frac{w_{f(fire)} \, l^2}{8} = \frac{(15.4) \cdot (4730)^2}{8} = 43.1 \text{ kNm/m}$$

Note that the load in kN/m^2 is simply the distributed load of kN/m for a unit width of floor spanning between supports.

Therefore, if the moment resistance calculated after a 90 minute exposure is greater than 43.1 kNm/m, then the fire-resistance rating will be greater than 90 minutes.

Calculation of the structural resistance after 90 minutes of standard fire exposure:

Step 1: Calculation of the char depth

The charring rate is taken as 0.65 mm/min.

$$d_{char} = \beta t = (0.65\text{ mm/min})\cdot(90\text{ min}) = 58.5\text{ mm}$$

Therefore, the remaining cross-section is calculated as:

$$D_{char} = D - d_{char} = (38\text{ mm} \times 5) - 58.5 = 131.5\text{ mm}$$

Step 2: Determination of effective residual cross-section

The heated zone for floors is taken as 10.5 mm after a minimum exposure of 20 minutes. The remaining cross-section can then be calculated as:

$$D_{fire} = D_{char} - d_{heat} = 131.5\text{ mm} - 10\text{ mm} = 121.5\text{ mm}$$

A summary of the cross-section details is provided in Figure 3.

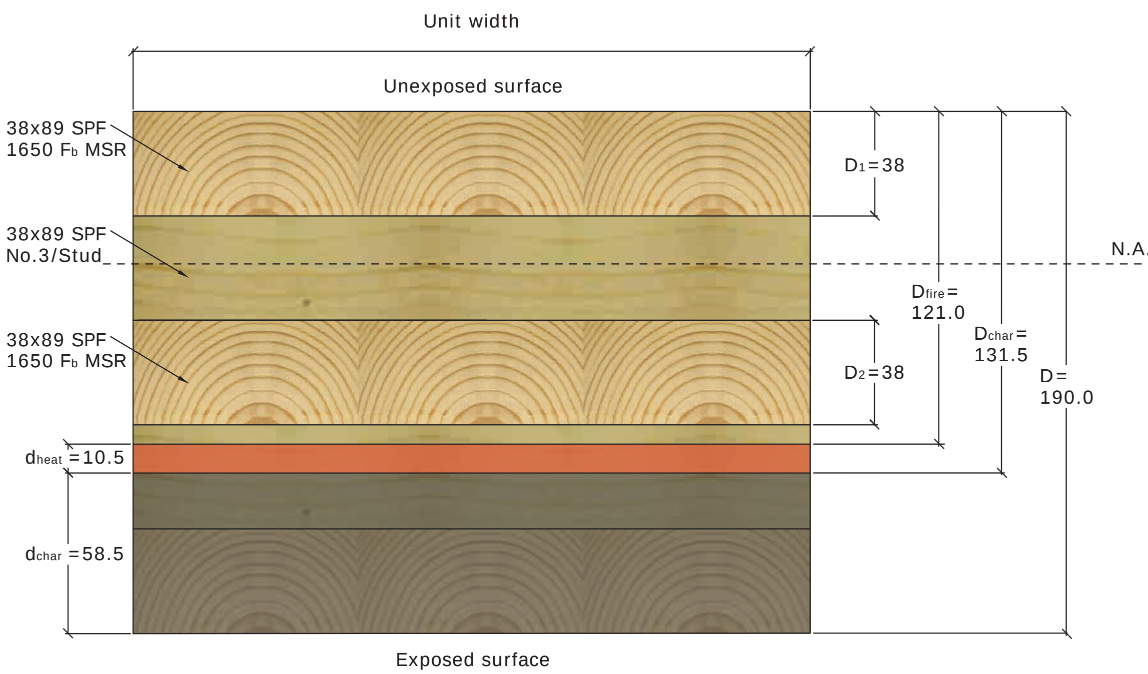

Figure 3
Cross-section of a 5-ply CLT floor assembly after 90 minutes of fire exposure

However, since the residual cross-section includes a portion of the cross ply at the bottom, the D_{fire} is reduced to the bottom of the middle ply. Therefore D_{fire} = 114 mm.

Step 3: Find location of neutral axis and moment of inertia of residual cross-section

The remaining plies in the direction of the applied stress include the middle ply and the unexposed ply. Therefore, the neutral axis can be calculated as:

$$\bar{y} = \frac{\sum_i \tilde{y}_i D_i}{\sum_i D_i} = \frac{\left[\left(\frac{38}{2}\right)\cdot(38) + \left(\left(\frac{38}{2}\right) + (2\times 38)\right)\cdot(38)\right]}{(38+38)} = 57.0 \text{ mm}$$

The reduced cross-section used to calculate the moment of inertia is shown in Figure 4.

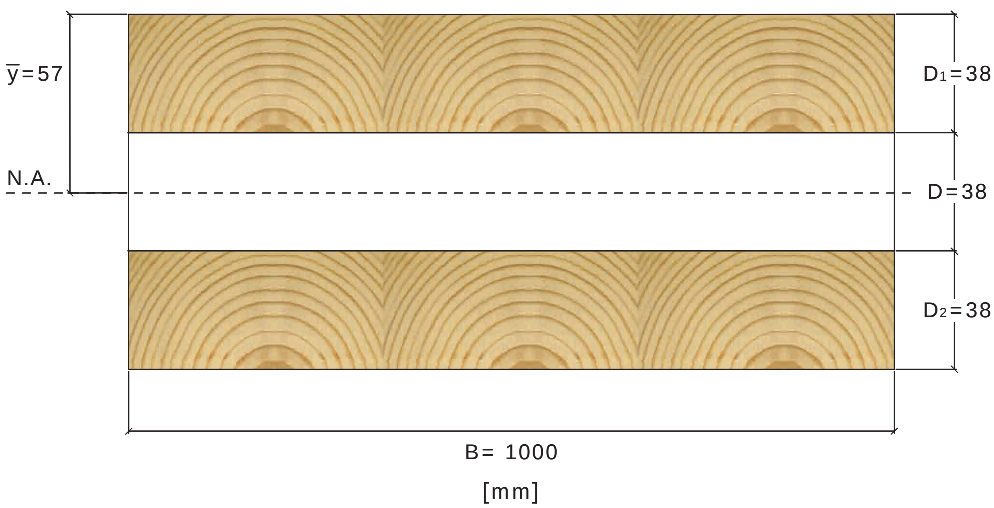

Figure 4
Reduced cross-section of a 5-ply CLT floor assembly after 90 minutes of fire exposure

The moment of inertia is calculated as:

$$I = \sum_i \frac{B D_i^3}{12} + \sum_i B D_i d_i^2 = \left[\frac{(1000)\cdot(38)^3}{12} + \frac{(1000)\cdot(38)^3}{12}\right] +$$

$$\left[(1000)\cdot(38)\cdot\left(57.0 - {}^{38}\!/_{2}\right)^2 + (1000)\cdot(38)\cdot\left(38+38+{}^{38}\!/_{2} - 57.0\right)^2\right] = 119\times 10^6 \text{ mm}^4$$

Step 4a: Calculation of the moment resistance (floors)

The moment resistance can be calculated based on Clause 5.5.4. of CSA O86-09.

$$M_r = \varphi F_b S K_{Zb} K_L$$

where:

φ = 1.0
F_b = $f_b(K_D K_H K_{Sc} K_T)$
f_b = 23.9 MPa (Table 5.3.2, CSA O86)

K_D = 1.15 (short-term duration of load under fire conditions – Table 4.3.2.2, CSA O86)
K_H = 1.0 (no load sharing – Cl. 5.4.4, CSA O86)
K_{Sc} = 1.0 (dry service condition – Table 5.4.2, CSA O86)
K_T = 1.0 (no treatment – Table 5.4.3, CSA O86)
F_b = 23.9 (1.15 x 1.0 x 1.0 x 1.0) = 27.5 MPa
K_{Zb} = 1.0
K_L = 1.0

$$S_{eff} = \frac{I}{D_{fire} - \bar{y}} = \frac{119 \times 10^6}{(114 - 57)} = 2.09 \times 10^6 \text{ mm}^3$$

then,

$$M_r = \varphi F_b S K_{Zb} K_L = (1.0) \cdot (27.5) \cdot (2.09 \times 10^6) \cdot (1.0) \cdot (1.0) = 57.5 \text{ kNm/m}$$

Conclusion:

After 90 minutes of exposure, the floor moment resistance is 57.5 kNm/m, which is greater than the load under fire conditions of 43.1 kNm/m determined above. Therefore, the structural performance of the floor meets the requirements of a 90 minute fire-resistance rating.

6.2 CLT Wall Example

The following wall example follows the steps listed above for determining whether the structural fire resistance of a 3-ply CLT wall assembly meets the hypothetically required fire-resistance rating of 45 minutes. The wall assembly has the following specifications:

- Composed of a 3-ply CLT panel
- The two exterior plies are 38 mm x 89 mm SPF 1650 F_b MSR
- Interior ply is 38 mm x 89 mm SPF No.3/Stud
- Wall height is 3048 mm
- One layer of 15.9 mm type X gypsum board on each face
- Adhesive is a thermoset (phenol-resorcinol-formaldehyde)
- Dead load is 93 kN and live load is 373 kN (based on the design load and a live to dead load ratio of 4:1)

Factored load:

The load on the wall in the event of fire can be calculated using the reduced load factors found in Commentary A, Section 25 of the NBCC.

$$P_{f(fire)} = D_L + 0.5L_L = (93) + 0.5(373) = 280 \text{ kN}$$

Therefore, if the axial resistance calculated after a 45 minute exposure is greater than 280 kN, then the fire-resistance rating will be greater than 45 minutes.

Calculation of the structural resistance after 45 minutes of standard fire exposure:

Step 1: Calculation of the char depth

Since one layer of 15.9 mm type X gypsum board adds 30 minutes to the fire-resistance rating, the char depth will be calculated given 15 minutes of unprotected exposure. The charring rate is taken as 0.65 mm/min.

$$d_{char} = \beta t = (0.65\text{mm/min}) \cdot (15\text{min}) = 9.8 \text{ mm}$$

Therefore, the remaining cross-section is calculated as:

$$D_{char} = D - d_{char} = (38\text{mm} \times 3) - 9.8 = 104.2 \text{ mm}$$

Step 2: Determination of effective residual cross-section

The heated zone for walls is taken as 16 mm after a minimum exposure of 20 minutes. Since the exposure in this example is 15 minutes, the heated zone is linearly interpolated.

$$d_{heat} = 16 \text{ mm} \cdot \left(\frac{15 \text{ min}}{20 \text{ min}} \right) = 12 \text{ mm}$$

The remaining cross-section can then be calculated as:

$$D_{fire} = D_{char} - d_{heat} = 104.2 \text{ mm} - 12 \text{ mm} = 92.2 \text{ mm}$$

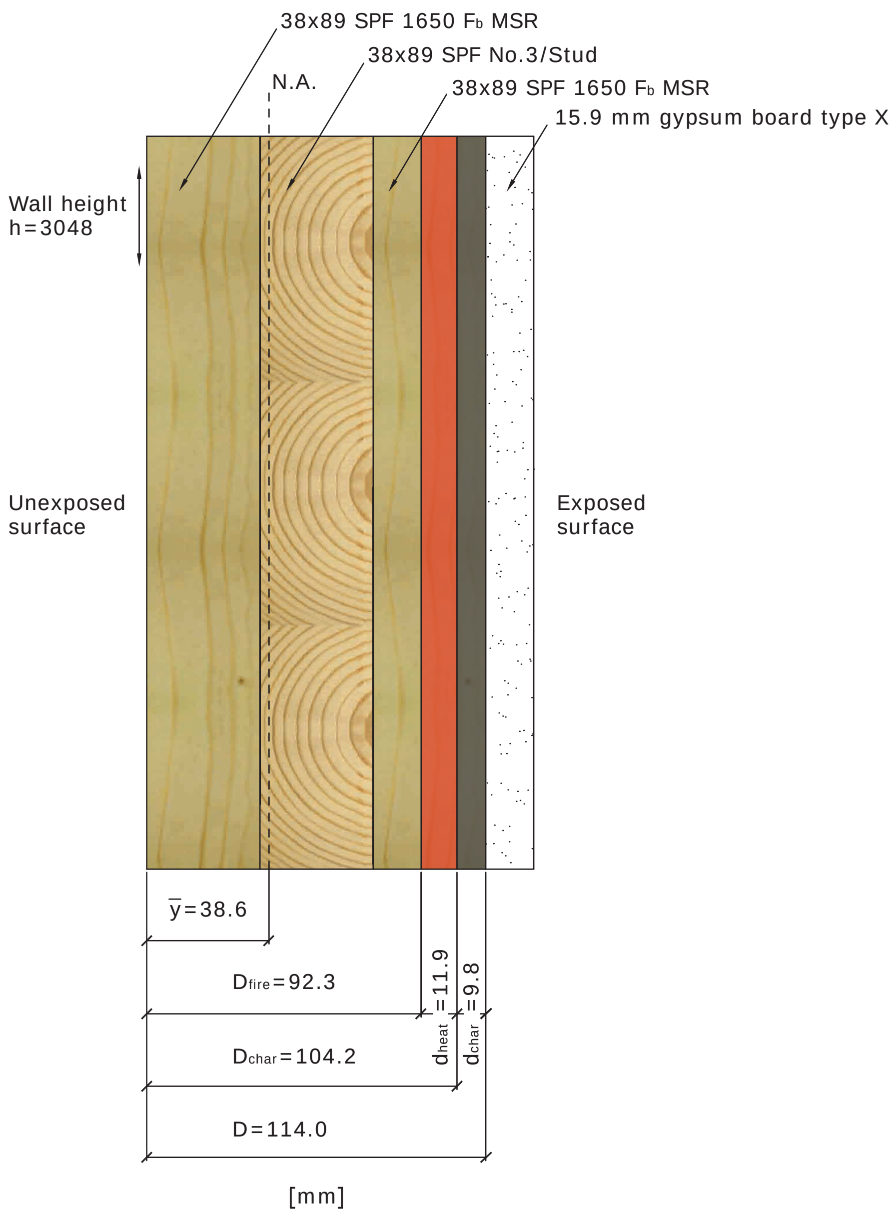

Figure 5
Cross-section of a 3-ply CLT wall assembly after 45 minutes of fire exposure

Step 3: Find location of neutral axis and moment of inertia of residual cross-section

The remaining thickness of the first ply exposed to fire is equal to the remaining thickness subtract the second and third layers (92.2 mm – 2(38 mm) = 16.2 mm).

$$\bar{y} = \frac{\sum_i \tilde{y}_i D_i}{\sum_i D_i} = \frac{\left[\left(\frac{38}{2}\right)\cdot(38)+\left(\left(\frac{16.2}{2}\right)+(2\times 38)\right)\cdot(16.2)\right]}{(38+16.2)} = 38.5\text{ mm}$$

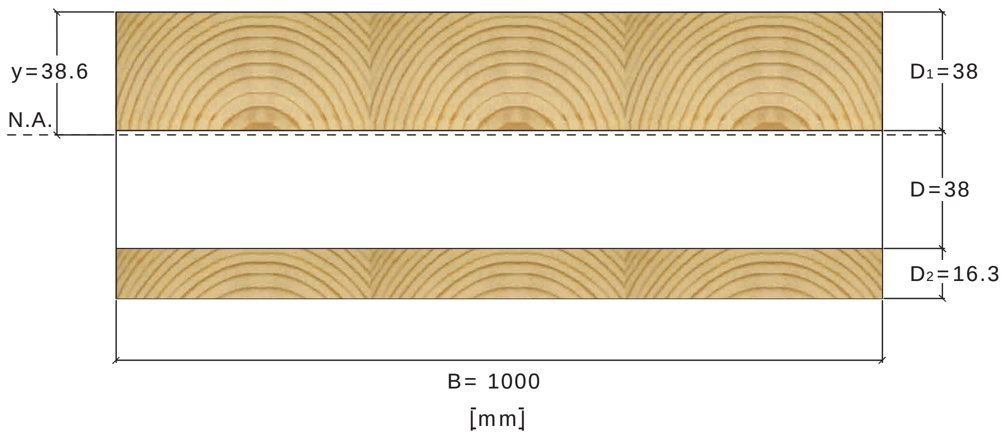

Figure 6
Reduced cross-section of a 3-ply CLT wall assembly after 45 minutes of fire exposure

The moment of inertia is calculated as:

$$I = \sum_i \frac{B D_i^3}{12} + \sum_i B D_i d_i^2 = \left[\frac{(1000)\cdot(38)^3}{12} + \frac{(1000)\cdot(16.2)^3}{12}\right] +$$

$$\left[(1000)\cdot(38)\cdot\left(38.5 - {}^{38}\!/_{2}\right)^2 + (1000)\cdot(16.2)\cdot\left(38+38+{}^{16.2}\!/_{2} - 38.5\right)^2\right] = 53.1\times 10^6\text{ mm}^4$$

Step 4b: Calculation of axial resistance (walls)

The axial resistance can be calculated from Clause 5.5.6 in CSA O86.

$$P_r = \varphi F_c A K_{Zc} K_C$$

where:

φ = 1.0
F_b = $f_b(K_D K_H K_{Sc} K_T)$
f_c = 18.1 MPa (Table 5.3.2, CSA O86)
K_D = 1.15 (short-term duration of load under fire conditions – Table 4.3.2.2, CSA O86)
K_H = 1.0 (no load sharing – Cl. 5.4.4, CSA O86)
K_{Sc} = 1.0 (dry service condition – Table 5.4.2, CSA O86)
K_T = 1.0 (no treatment – Table 5.4.3, CSA O86)
F_c = 18.1 (1.15 x 1.0 x 1.0 x 1.0) = 20.8 MPa
A = 1000 x (38 + 16.2) = 54.2 x 10^3 mm^2
K_{Zc} = 1.0

$$K_C = \left[1.0 + \frac{F_c K_{Zc} C_c^3}{35 E_{05} K_{SE} K_T}\right]^{-1}$$

where:

$$C_c = \frac{H}{2\sqrt{3}\sqrt{I/A}} = \frac{3048}{2\sqrt{3}\sqrt{\frac{53.1\times10^6}{(54.2\times10^3)}}} = 28.1$$

E_{05} = 0.82 E (Clause 5.5.6.2.3, CSA O86)
E = 10300 MPa (Table 5.3.2, CSA O86)
E_{05} = 0.82(10300) = 8446 MPa
K_{SE} = 1.0 (dry service condition – Table 5.4.2, CSA O86)

$$K_C = \left[1.0 + \frac{F_c K_{Zc} C_c^3}{35 E_{05} K_{SE} K_T}\right]^{-1} = \left[1.0 + \frac{(20.8)\cdot(1.0)\cdot(28.1)^3}{35(8446)\cdot(1.0)\cdot(1.0)}\right]^{-1} = 0.39$$

then,

$$P_r = \varphi F_c A K_{Zc} K_C = (1.0)\cdot(20.8)\cdot(54.2\times10^3)\cdot(1.0)\cdot(0.39) = 440\times10^3\ N$$

Conclusion:

After 45 minutes of exposure, the wall axial resistance is 440 kN, which is greater than the load under fire conditions of 280 kN determined above. Therefore, the structural performance of the wall meets the requirements of a 45 minute fire-resistance rating.

7

REFERENCES

American Society for Testing and Materials (ASTM). 2006. *Standard test method for evaluating the shear strength of adhesive bonds in laminated wood products at elevated temperatures*. ASTM D7247-06. West Conshohocken, PA: ASTM. 6 p.

______. 2010a. *Standard specification for adhesives for structural laminated wood products for use under exterior (wet use) exposure conditions*. ASTM D2559-10a. West Conshohocken, PA: ASTM. 19 p.

______. 2010b. *Standard test method for surface burning characteristics of building materials*. ASTM E84-10. West Conshohocken, PA: ASTM. 22 p.

______. 2010c. *Standard test methods for fire tests of building construction and materials*. ASTM E119-10a. West Conshohocken, PA: ASTM. 33 p.

British Standards Institution (BSI). 2004. *Eurocode 5: Design of timber structures. Part 1-2: General – Structural fire design.* BS EN 1995-1-2. London, UK: BSI. 76 p.

Canadian Standards Association (CSA). 2005. *Engineering design in wood*. CAN/CSA O86. Mississauga, ON: CSA. 197 p.

______. 2008. *Evaluation of adhesives for structural wood products (limited moisture exposure)*. CSA O112.10-08. Mississauga, ON: CSA. 60 p.

Craft, S., R. Desjardins, and L. Richardson. 2008. Development of small-scale evaluation methods for wood adhesives at elevated temperatures. Paper presented at the 10th World Conference on Timber Engineering, June 2-5, 2008, Miyazaki, Japan. 8 p.

Frangi, A., M. Fontana, E. Hugi, and R. Jobstl. 2009. Experimental analysis of cross-laminated timber panels in fire. *Fire Safety Journal* 44:1078-1087.

International Standards Organization (ISO). 1975. *Fire resistance tests – Elements of construction*. ISO 834. Geneva: ISO. 16 p.

Janssens, M. L., and R. H. White. 1994. Temperature profiles in wood members exposed to fire. *Fire and Materials* 18:263-265.

National Research Council of Canada (NRC). Institute for Research in Construction (IRC). 2005. *National building code of Canada, 2005*. Ottawa, ON: NRC. 2 v.

Schmid, J., J. Konig, and J. Kohler. 2010. Design model for fire exposed cross-laminated timber. In *Structures in Fire: Proceedings of the Sixth International Conference, June 2 4, 2010, Michigan State University, East Lansing, MI*, 511-519.

Underwriters' Laboratories of Canada (ULC). 2007a. *Standard method of test for surface burning characteristics of building materials and assemblies*. CAN/ULC S102-07. Scarborough, ON: ULC. 49 p.

______. 2007b. *Standard methods of fire endurance tests of building construction and materials*. CAN/ULC S101-07. Scarborough, ON: ULC. 85 p.

White, R. H. 2009. Fire resistance of wood members with directly applied protection. Paper presented at the Fire and Materials International Conference, January 26-28, 2009, San Francisco, USA. 12 p.

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CHAPTER 9

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ABSTRACT

Adequate levels of noise/sound control in multi-family buildings are mandatory requirements of building codes in Canada, the United States, Europe, and most developed Asian countries. In many jurisdictions, these requirements are as strictly enforced as those for structural sufficiency and fire safety. Much effort has been spent on evaluation of sound transmission class (STC) and impact sound insulation class (IIC) of floor and wall assemblies and on studies of flanking transmission in multi-family dwellings in Canada. However, little work has been done so far in Canada on the acoustic performance of CLT systems in construction.

This chapter focuses mainly on the development of CLT floor and wall assemblies made of cross-laminated timber elements capable of good acoustic performance in residential, multi-residential and non-residential buildings in Canada and the USA. Existing generic floor and wall assemblies used in Europe are also presented in this chapter, as well as examples of floor assemblies tested in laboratories that could be used in the North American market.

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OBJECTIVES

This chapter focuses mainly on the development of CLT floor and wall assemblies made of cross-laminated timber elements capable of good acoustic performance in residential, multi-residential and non-residential buildings in Canada and the USA. Existing generic floor and wall assemblies used in Europe are also presented in this chapter, as well as examples of floor and wall assemblies that could be used in the North American market.

2

CROSS-LAMINATED TIMBER PANEL – DEFINITION

Cross-laminated timber (CLT) panels consist of several layers of boards glued on faces with most of the layers stacked crosswise. The cross-section of CLT elements is generally characterized by at least three glued board layers with orthogonally alternating orientation of neighbored layers (Mestek et al., 2008). Most of the time, the narrow faces of the boards are not glued although sometimes board layers positioned in the longitudinal direction of the panel are edge-glued. Some manufacturers will also produce panels having the transverse planks edge-glued. Adjacent layers are placed perpendicular to each other while, for some configurations, two consecutive board layers may be placed in the same direction, giving a double layer (i.e., double longitudinal layers at the outer faces and additional double layers at the centre of the panel for some configurations). CLT products are usually fabricated with 3 to 11 board layers. Figure 1 illustrates a CLT panel configuration while Figure 2 shows examples of CLT panel cross-sections.

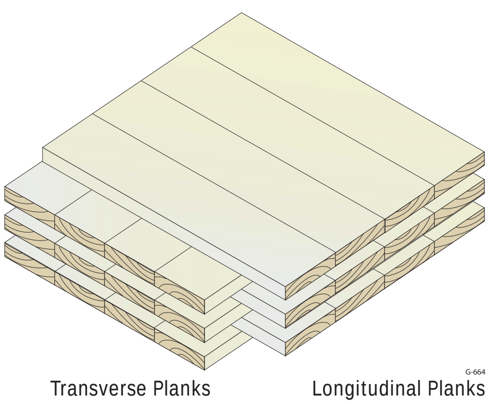


Figure 1
CLT panel configuration

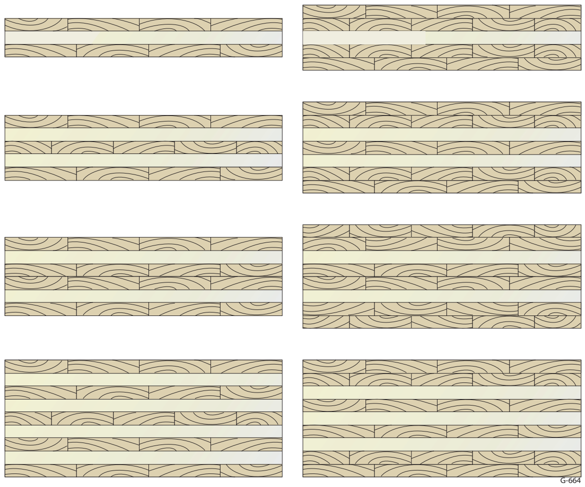


Figure 2
Examples of CLT panel cross-sections

3

BUILDING CODE REQUIREMENTS FOR ACOUSTIC PERFORMANCE

Adequate levels of noise/sound control in multi-family buildings are mandatory requirements of building codes in Canada, the United States, Europe, and most developed Asian countries. In many jurisdictions, these requirements are as strictly enforced as those for structural sufficiency and fire safety.

The National Building Code of Canada (NBCC 2005) states that a dwelling unit shall be separated from every other space in a building in which noise may be generated by construction providing a sound transmission class (STC) of at least 50 dB. This level of performance shall be of 55 dB near elevators or refuse chute (NRC 2005). NBCC 2005 has no specific requirement for impact sound insulation class (IIC), but provides a recommendation that bare floors (i.e. those without finishes such as vinyl, carpet, etc.) achieve an IIC of 55 dB or better. Therefore, an IIC of 55 dB or above is normally targeted. However, following a number of homeowner acoustic-comfort surveys, these minimum requirements given for STC or recommended for IIC are not satisfactory, mostly in multi-family buildings.

4 ACOUSTIC PERFORMANCE OF GENERIC WOOD-FRAME ASSEMBLIES

Sound transmission class (STC), impact sound insulation class (IIC) and fire resistance (FR) ratings for many of the generic construction assemblies traditionally used in construction of Canadian housing and small buildings have been published in the National Building Code of Canada (NBCC) since 1950. Architects, fire protection engineers and building officials make extensive use of the STC, IIC and FR ratings stated in the NBCC when designing and approving housing and small buildings in Canada. These ratings are also extensively used in the design of larger engineered structures. Wood-frame assemblies, more than any others, are designed and constructed in accordance with the STC, IIC and FR ratings listed in the NBCC.

The sound performance of a typical wood floor mainly depends on the construction details, including materials and thickness of the layers (e.g. finishing, topping, sub-floor, ceiling board, sound-absorbing material in the ceiling cavity), the attachment between layers, the size and spacing of the joists, and the spacing of the resilient channels it used. It can be found in the NBCC that the typical STC for generic wood-frame floor assemblies varies from about 30 dB to 70 dB and greatly depends on the construction details used. In the same logic, data provided for IIC for wood-frame floors range from about 20 dB to 50 dB.

In the case of wood-frame wall assemblies traditionally used in construction of Canadian housing and small buildings, only the sound transmission class is needed. It can be found in the NBCC that the typical STC of generic wood-frame wall assemblies varies from 30 dB to 65 dB and depends on the construction details, including materials and thickness of the layers (e.g. gypsum boards, sound absorbing material in the wall cavity), the spacing between the studs, and the spacing of resilient channels.

5 FLANKING TRANSMISSION IN BUILDING SYSTEMS

Flanking transmission may be defined as the airborne sound that reaches a building occupant by certain paths around or through an acoustical barrier between two living spaces (walls or floors). Flanking transmission can be particularly annoying in multi-family buildings. Adequate detailing shall be specified early in the design and construction phase of the building. Then, simply specifying a high performance generic floor or wall system will not guarantee an adequate STC. Different aspects of the floor or wall assemblies must be carefully considered such as windows, partition walls, light switches, telephone outlets and lighting fixtures, plumbing systems, etc. It should be noted that sound transmission class ratings made available for generic wood-frame assemblies (i.e. tested in laboratory) do not consider the flanking sound transmission because only one physical barrier (wall or floor) is tested in laboratory. This is why we recommend targeting a level of performance greater than the minimum requirements given in the codes and detailing adequately the assemblies to limit the flanking sound transmission.

6 CLT ASSEMBLIES AND ACOUSTICS

Much effort has been invested on evaluation of sound transmission class (STC) and impact sound insulation class (IIC) of floor and wall assemblies traditionally used in construction of Canadian housing and small buildings and on studies of flanking transmission in multi-family dwellings in Canada since 1950. Generic construction assemblies have been published in the NBCC. However, little work has been done in Canada on the acoustic performance of CLT systems.

6.1 Collaboration with the FCBA (France)

FPInnovations worked in close collaboration with the French Institute of Technology for Forest-Based and Furniture Sectors (FCBA) of Bordeaux, France. The FCBA has good experience with testing, predicting and enhancing sound transmission loss of wood construction. Different tests have already been done by the FCBA on wood construction and especially on CLT panel floors and walls.

The FCBA provided step-by-step guidance for FPInnovations' design of North American CLT floor systems, including the selection of materials. Intensive literature review has been performed and documented. A study period has been required to analyze, model, measure and validate the vibroacoustic behaviour of the new CLT panel design using Statistical Energy Analysis (SEA) and Experimental SEA approaches. Finally, tests have been performed at the FCBA in July 2009 on several floor assemblies that could be reproduced in North America. It should be noted that numerous floor systems constructed with CLT panels have been tested at the FCBA over the past several years for assessing the acoustic performance of such systems.

6.2 Existing CLT Floor Assemblies in Europe

In this section, several examples of common CLT floor assemblies available in European countries are presented. Sound transmission class (STC) and impact sound insulation class (IIC) are provided for each floor assembly.

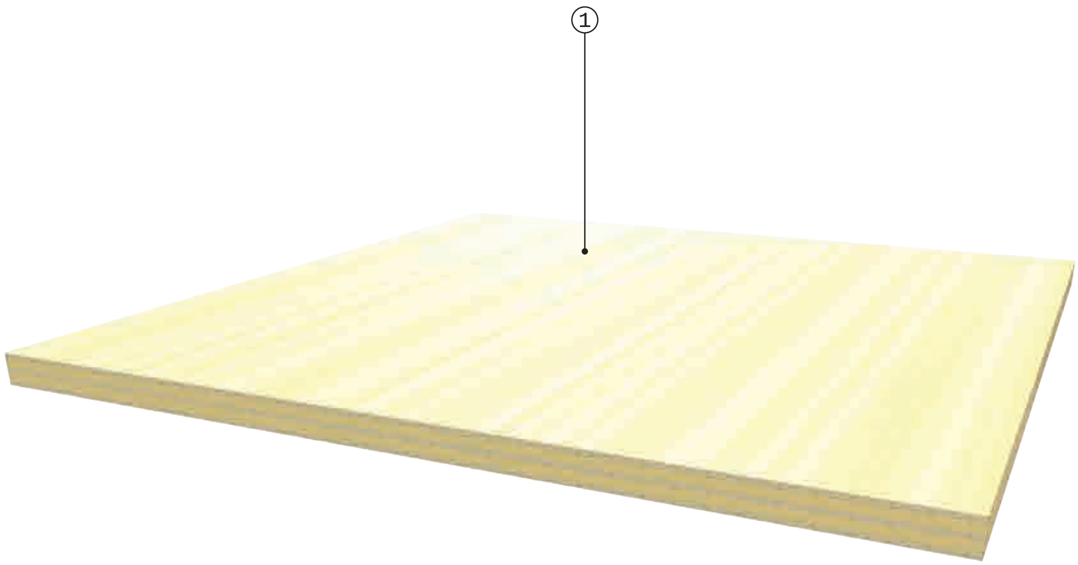
6.2.1 Floor Assemblies Tested by the FCBA in 2006

The following figures provide relevant information on different floor systems tested at the FCBA in 2006. The product used for the tests was a 5-layer CLT panel. The floor assemblies were manufactured using typical European construction materials.

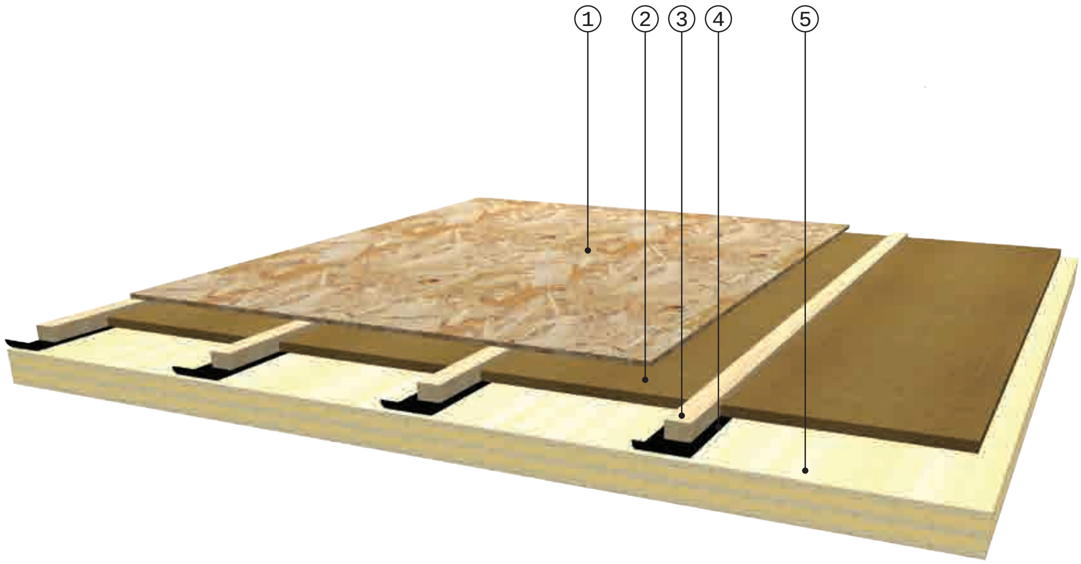
In the first series of five floor configurations, it can be observed that the sound insulation material was installed only on one side, i.e. on the top of or underneath the CLT floor. It can be seen that the CLT tested alone provides a STC of 39 dB and an IIC of 24 dB. The best sound insulation performance is achieved when a suspended ceiling is used. With this floor configuration, we may expect a STC of 64 dB and an IIC of 59 dB. When the floor is built with a subfloor, the maximum rating obtained for STC and IIC are 53 dB and 45 dB, respectively. Then, for approximately the same total floor thickness, STC and IIC ratings are much lower for the floor built with a subfloor compared to the floor built with a suspended ceiling.

Series 1

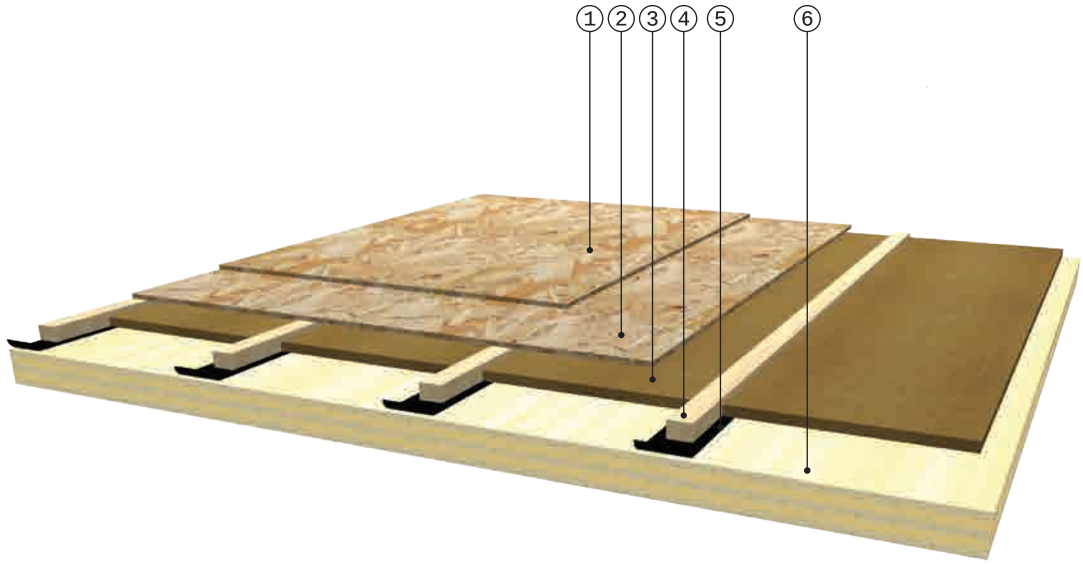
Results from tests performed at the FCBA in 2006 – CLT floor assemblies using sound insulation on the top or on the bottom



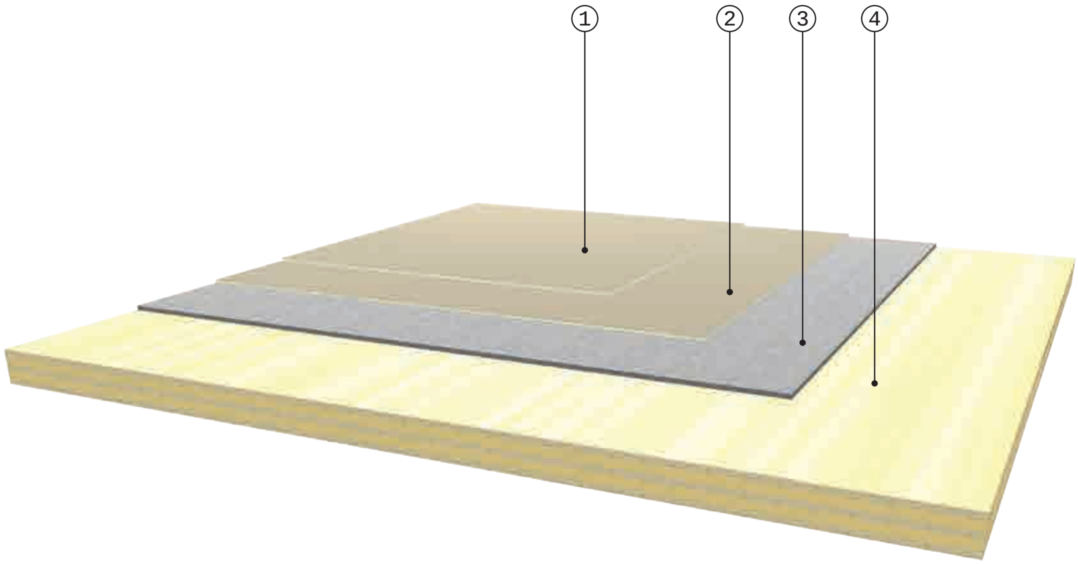
Floor Composition (F1.1)		Airborne (STC) dB	Impact (IIC) dB
1	5-layer CLT panel 146 mm	39	24



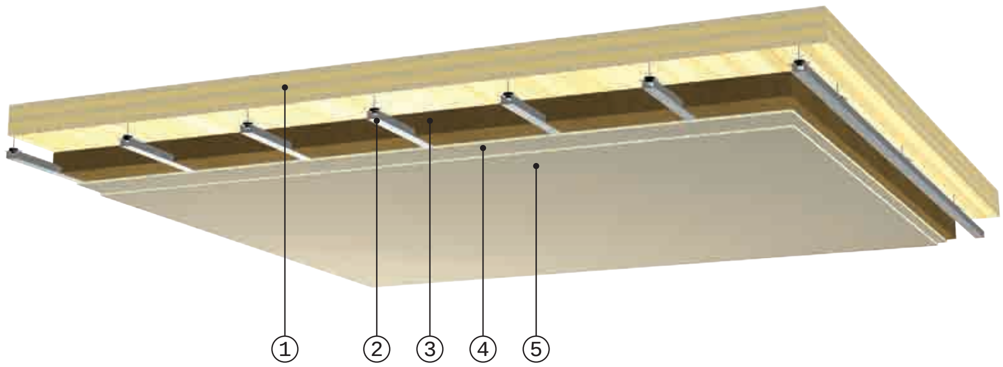
Floor Composition (F1.2)		Airborne (STC) dB	Impact (IIC) dB
1	Particleboard panel 22 mm	52	45
2	Sound insulation material (≈ 40 mm)		
3	Lumber sleepers		
4	REGUPOL underlayment		
5	5-layer CLT panel 146 mm		



Floor Composition (F1.3)		Airborne (STC) dB	Impact (IIC) dB
1	Particleboard panel 22 mm	53	45
2	Particleboard panel 22 mm		
3	Sound insulation material (≈ 40 mm)		
4	Lumber sleepers		
5	REGUPOL underlayment		
6	5-layer CLT panel 146 mm		



Floor Composition (F1.4)		Airborne (STC) dB	Impact (IIC) dB
1	Gypsum board 13 mm	≤ 45	32
2	Gypsum board 13 mm		
3	Dry topping 22 mm (Pellets PLACOSOL)		
4	5-layer CLT panel 146 mm		



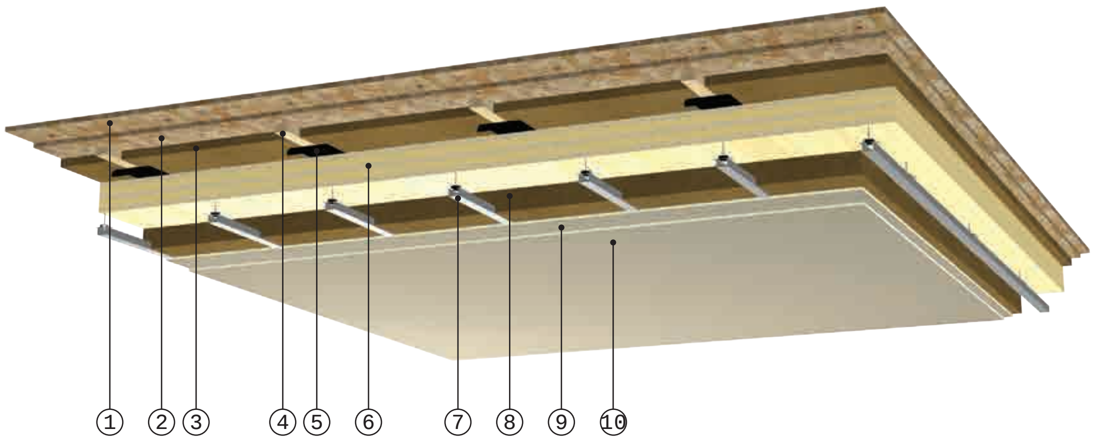
Floor Composition (F1.5)		Airborne (STC) dB	Impact (IIC) dB
1	5-layer CLT panel 146 mm	64	59
2	Resilient supports and rails (100 mm)		
3	Sound insulation material (100 mm)		
4	Gypsum board 13 mm		
5	Gypsum board 13 mm		

The next series provides CLT floor assemblies sound-insulated on both sides, i.e. top and bottom at the same time. The maximum STC is achieved with the first configuration, with 67 dB, while the maximum IIC is achieved with the fourth configuration, with 65 dB.

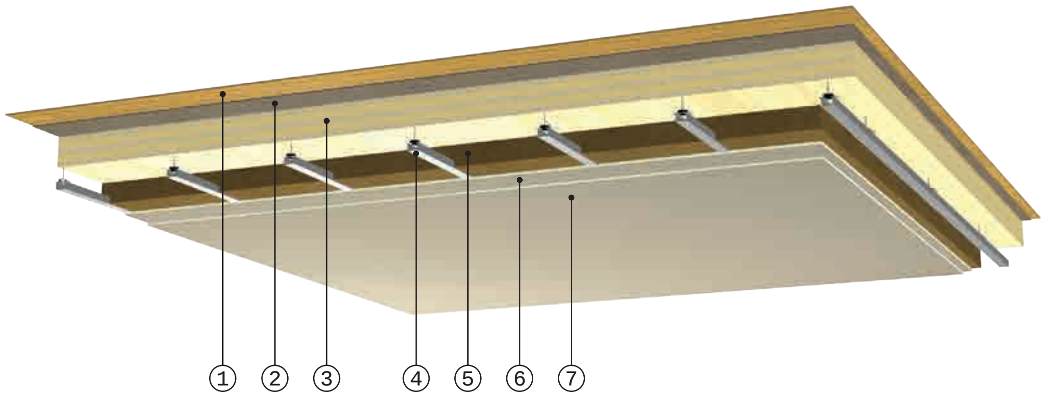
It may be observed that, even if the floor assembly is sound-insulated on both sides, there is no significant improvement for STC compared to the maximum STC obtained for the previous series above (STC = 64 dB). However, the IIC ratings have significantly increased, going from 59 dB to about 65 dB.

Series 2

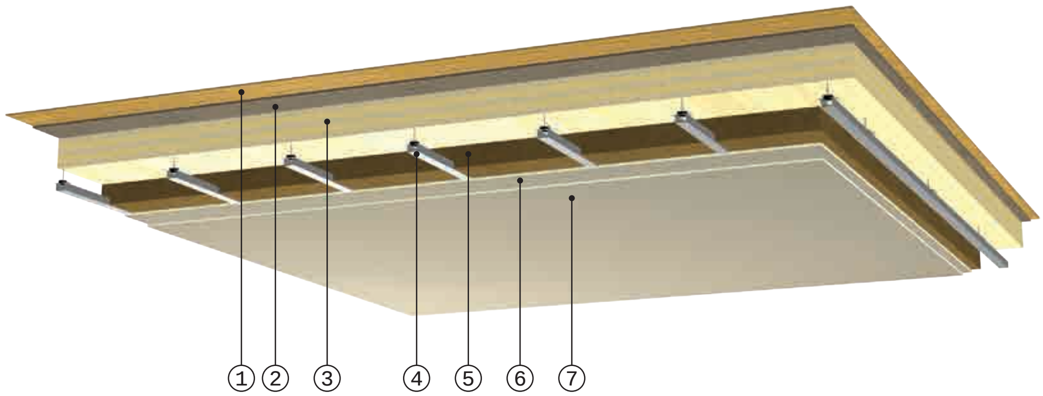
Results from tests performed at the FCBA in 2006 – CLT floor assemblies insulated on top and bottom



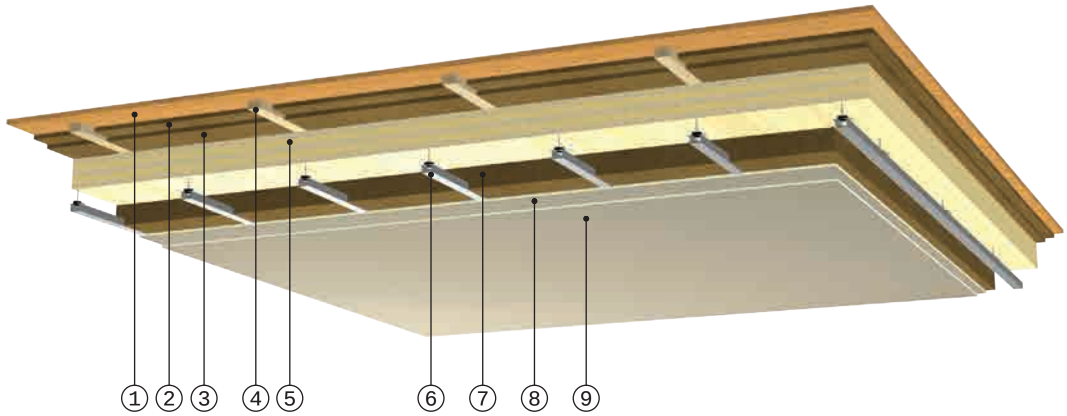
	Floor Composition (F2.1)	Airborn (STC) dB	Impact (IIC) dB
1	Particleboard panel 22 mm	67	≥ 62
2	Particleboard panel 22 mm		
3	Sound insulation material (≈ 40 mm)		
4	Lumber sleepers		
5	REGUPOL underlayment		
6	5-layer CLT panel 146 mm		
7	Resilient supports and rails (100 mm)		
8	Sound insulation material (100 mm)		
9	Gypsum board 13 mm		
10	Gypsum board 13 mm		



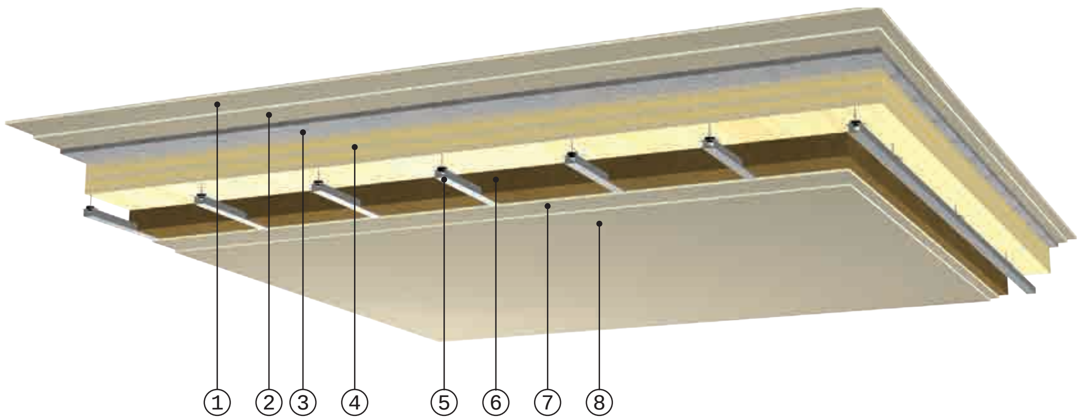
	Floor Composition (F2.2)	Airborne (STC) dB	Impact (IIC) dB
1	Laminate flooring 7 mm	62	≥ 63
2	Low-density fibre board 5 mm (PHALTEX)		
3	5-layer CLT panel 146 mm		
4	Resilient supports and rails (100 mm)		
5	Sound insulation material (100 mm)		
6	Gypsum board 13 mm		
7	Gypsum board 13 mm		



Floor Composition (F2.3)		Airborne (STC) dB	Impact (IIC) dB
1	Laminate flooring 7 mm	63	≥ 64
2	Low-density fibre board 10 mm (PHALTEX)		
3	5-layer CLT panel 146 mm		
4	Resilient supports and rails (100 mm)		
5	Sound insulation material (100 mm)		
6	Gypsum board 13 mm		
7	Gypsum board 13 mm		



Floor Composition (F2.4)		Airborne (STC) dB	Impact (IIC) dB
1	Floorboards nailed to sleepers	64	≥ 65
2	Low-density fibre board THERMISOREL 20 mm		
3	Low-density fibre board THERMISOREL 20 mm		
4	Lumber sleepers		
5	5-layer CLT panel 146 mm		
6	Resilient supports and rails (100 mm)		
7	Sound insulation material (100 mm)		
8	Gypsum board 13 mm		
9	Gypsum board 13 mm		



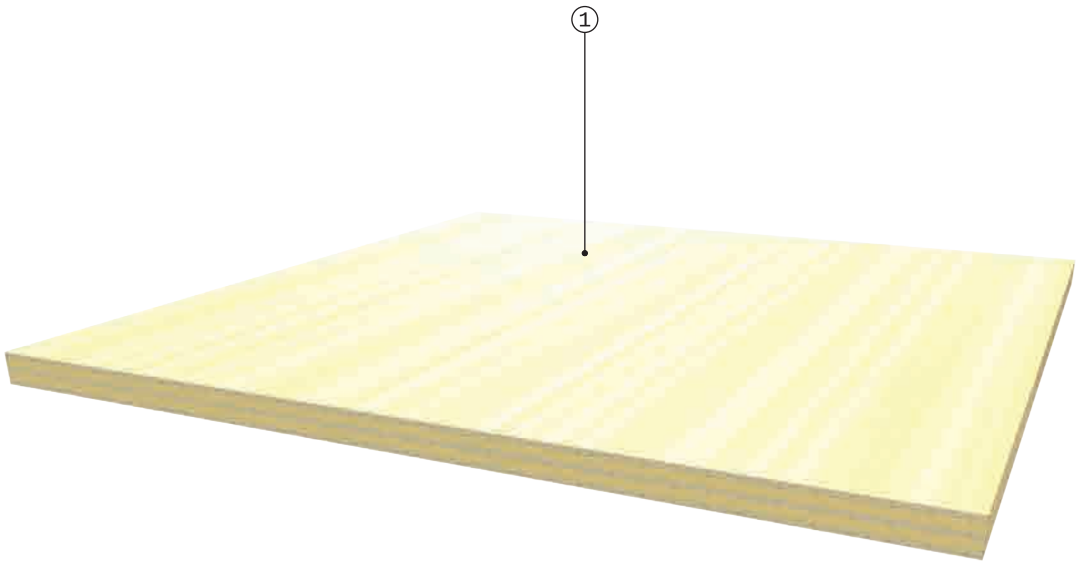
Floor Composition (F2.5)		Airborne (STC) dB	Impact (IIC) dB
1	Gypsum board 13 mm	63	≥ 63
2	Gypsum board 13 mm		
3	Dry topping 22 mm (Pellets PLACOSOL)		
4	5-layer CLT panel 146 mm		
5	Resilient supports and rails (100 mm)		
6	Sound insulation material (100 mm)		
7	Gypsum board 13 mm		
8	Gypsum board 13 mm		

6.2.2 Proprietary Floor Assemblies by a European CLT Panel Producer

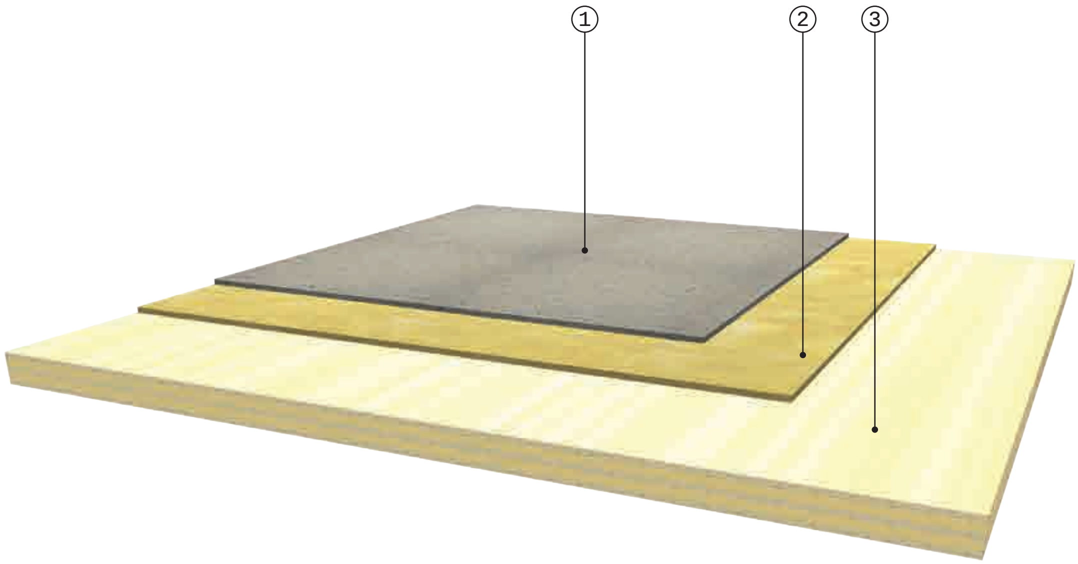
The following series illustrates the proprietary STC and IIC data published by a European CLT panel producer. It may be observed that the floor assembly is only sound-insulated on top, which allows the ceiling to be visible inside buildings. The maximum ratings are obtained for the last configuration, with a STC of 64 dB and an IIC of 72 dB. However, this floor assembly will be relatively heavy due to the use of concrete.

Series 3

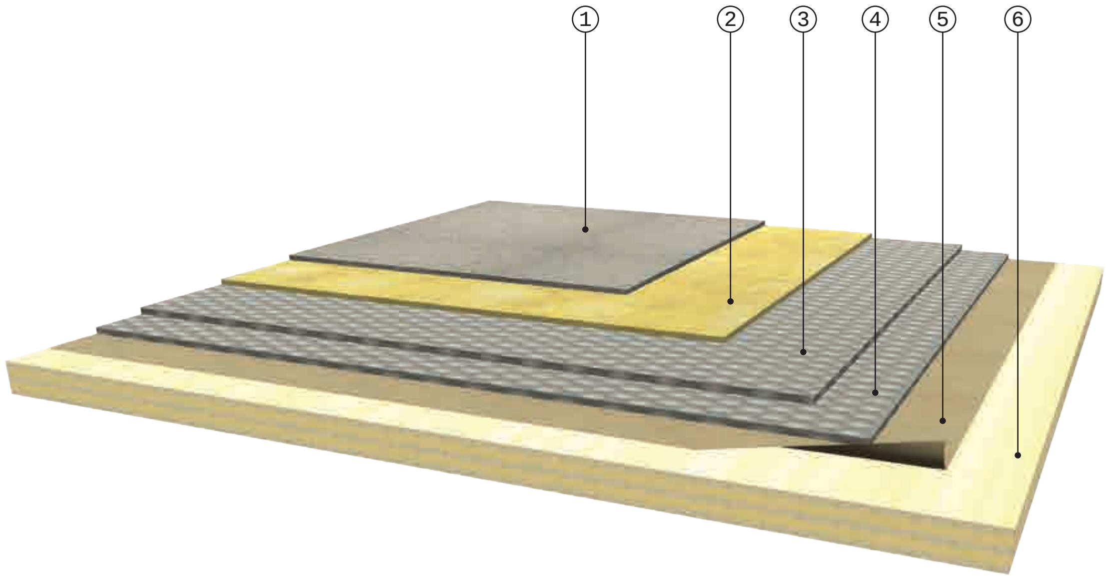
STC and IIC data from a European CLT Panel Producer



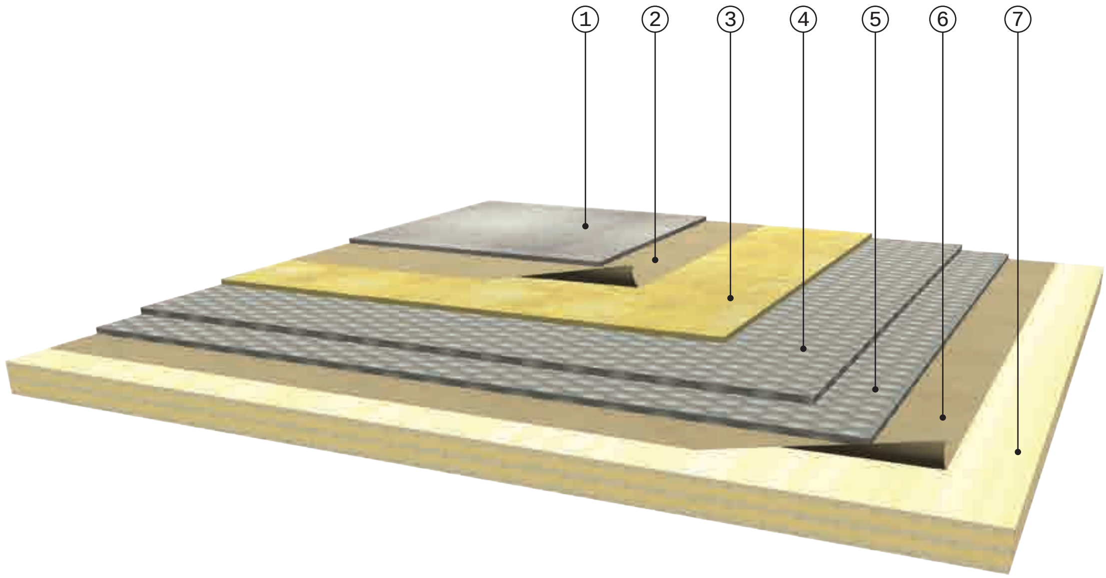
	Floor Composition (F3.1)	Airborne (STC) dB	Impact (IIC) dB
1	5-layer CLT panel 135 mm	≤ 39	≤ 23



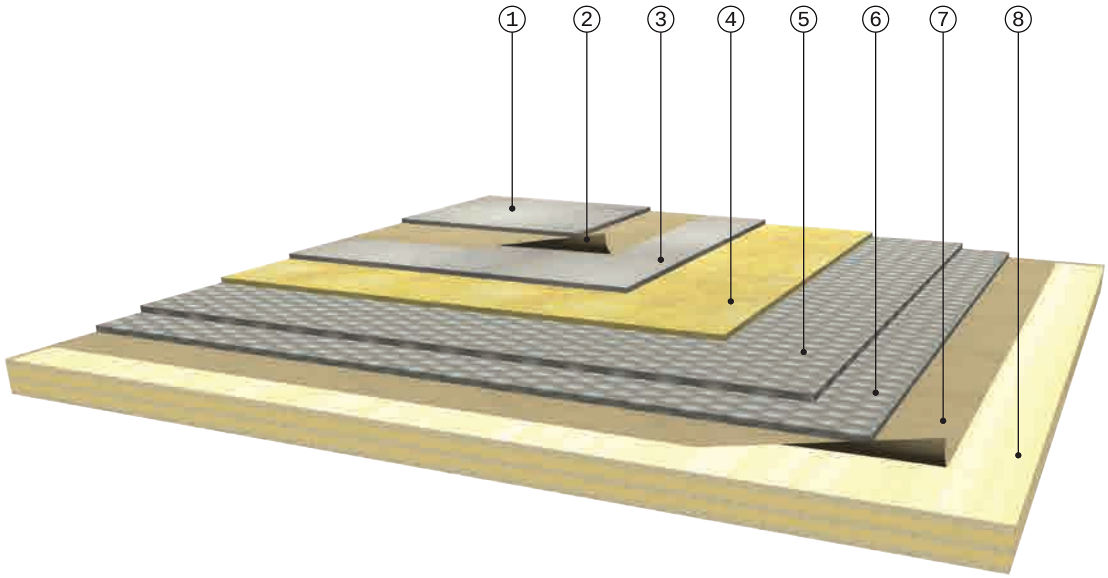
Floor Composition (F3.2)		Airborne (STC) dB	Impact (IIC) dB
1	Gypsum fibre board FERMACELL 25 mm	≤ 53	≤ 49
2	Sub-floor ISOVER EP3 20 mm		
3	5-layer CLT panel 135 mm		



Floor Composition (F3.3)		Airborne (STC) dB	Impact (IIC) dB
1	Gypsum fibre board FERMACELL 25 mm	≤ 62	≤ 59
2	Sub-floor ISOVER EP3 20 mm		
3	Honeycomb acoustic infill FERMACELL 30 mm		
4	Honeycomb acoustic infill FERMACELL 30 mm		
5	Kraft paper underlayment		
6	5-layer CLT panel 135 mm		



Floor Composition (F3.4)		Airborne (STC) dB	Impact (IIC) dB
1	Prefabricated concrete topping 20 mm		
2	Kraft paper underlayment		
3	Sub-floor ISOVER EP2 25 mm		
4	Honeycomb acoustic infill FERMACELL 30 mm	≤ 64	≤ 60
5	Honeycomb acoustic infill FERMACELL 30 mm		
6	Kraft paper underlayment		
7	5-layer CLT panel 135 mm		



Floor Composition (F3.5)		Airborne (STC) dB	Impact (IIC) dB
1	Prefabricated concrete topping 20 mm	≤ 64	≤ 72
2	Kraft paper underlayment		
3	Prefabricated concrete topping 20 mm		
4	Sub-floor ISOVER EP1 30 mm		
5	Honeycomb acoustic infill FERMACELL 30 mm		
6	Honeycomb acoustic infill FERMACELL 30 mm		
7	Kraft paper underlayment		
8	5-layer CLT panel 135 mm		

6.3 CLT Floor Assemblies Tested at the FCBA for FPInnovations

The work performed by the FCBA led to the development of different floor assemblies that could be used in the North American market. A total of six floor configurations were selected for testing. The next series gives the sound transmission class (STC) and impact sound insulation class (IIC) for each floor configuration. The CLT panel used for the tests was a 5-layer panel manufactured by KLH, from Austria.

6.3.1 Dry Topping

The first subfloor tested was built using gypsum fibre board glued to a rock fibre board made by FERMACELL (Figure 3). This subfloor was installed directly on the CLT floor. The second subfloor tested was a wood topping and was constructed using lumber sleepers screwed or not to the CLT floor. Low-density fibre boards from THERMISOREL were installed between the lumber sleepers and finally covered by OSB panels (Figure 4). Flooring underlayment from ROBERTS was also used in this configuration.

Figure 3
FERMACELL subfloor

Figure 4
Subfloor with THERMISOREL, lumber sleepers and OSB (wood topping)

6.3.2 Suspended Ceiling

The suspended ceiling system was provided by PAC International using resilient sound isolation clips RSIC-1 and furring channel. The cavities were filled with typical fibre glass insulation. Finally, two sheets of fire-rated gypsum board were screwed to the furring channels (Figure 5).

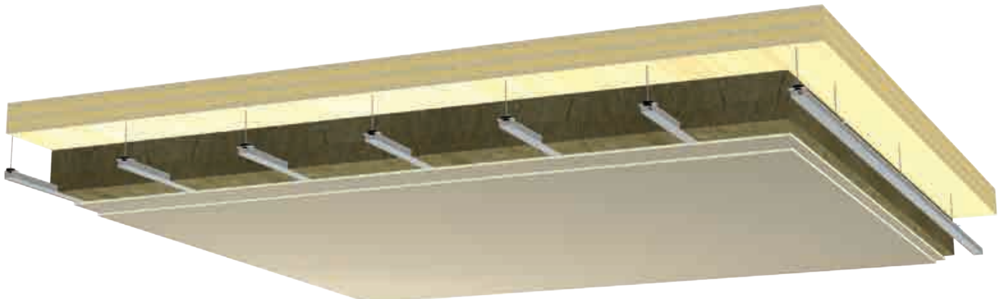


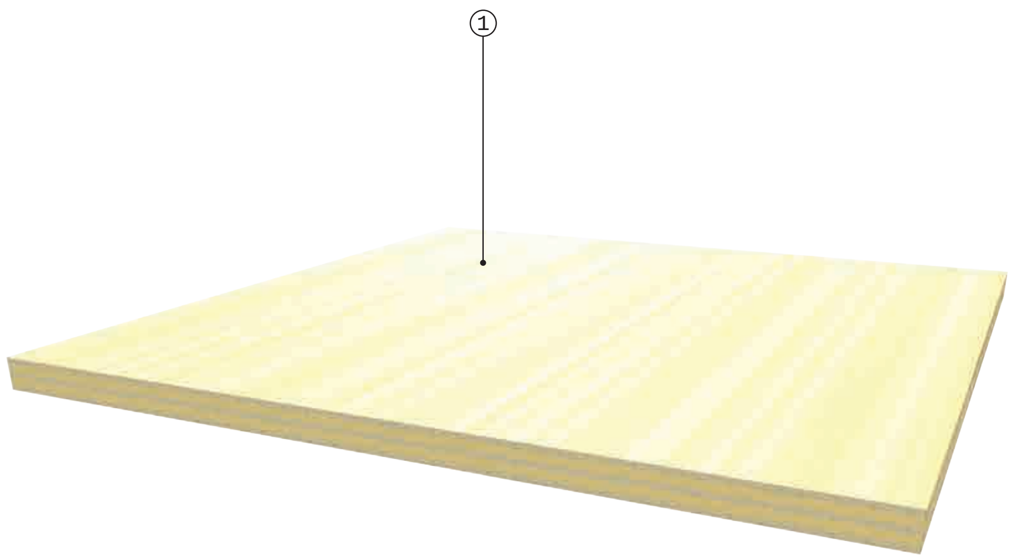
Figure 5
Suspended ceiling system

6.3.3 Results

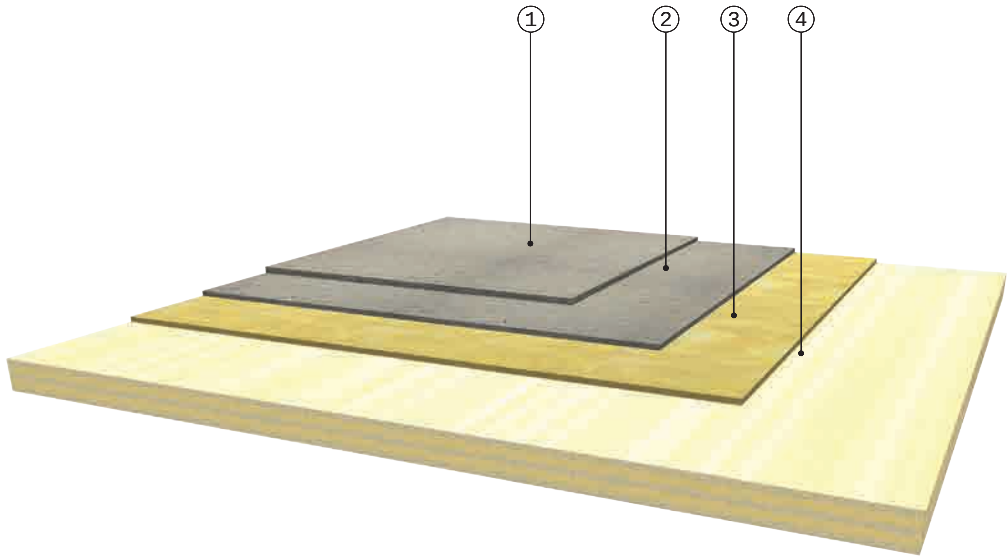
It can be seen in the next series that the maximum sound transmission class (STC) rating of 66 dB was obtained for the floor configuration made from a wood topping subfloor together with a suspended ceiling. For the same configuration, the IIC was relatively high, at 69 dB. It should be noted that the floor configuration using only the suspended ceiling provided very good STC and IIC ratings of 63 dB and 62 dB, respectively. These results would normally be sufficient in North American multi-family construction.

Series 4

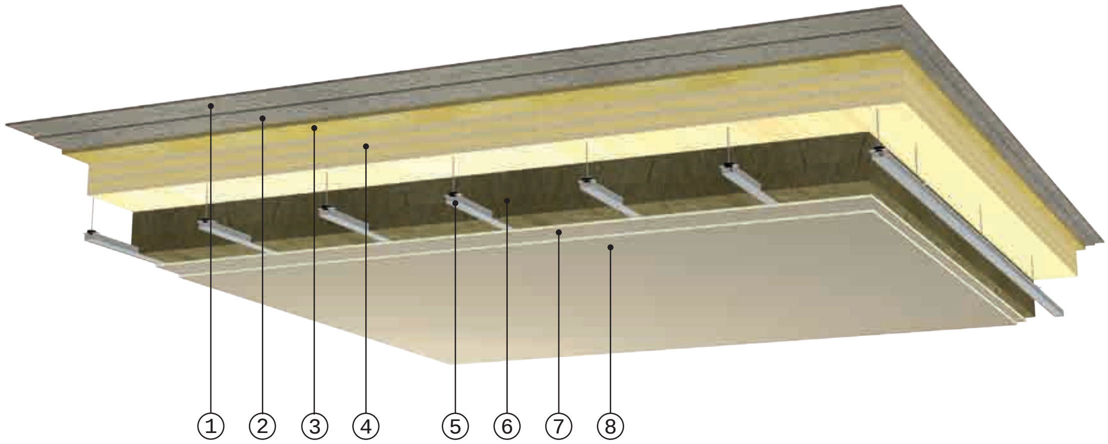
Results from tests performed for FPInnovations at the FCBA – July 2009



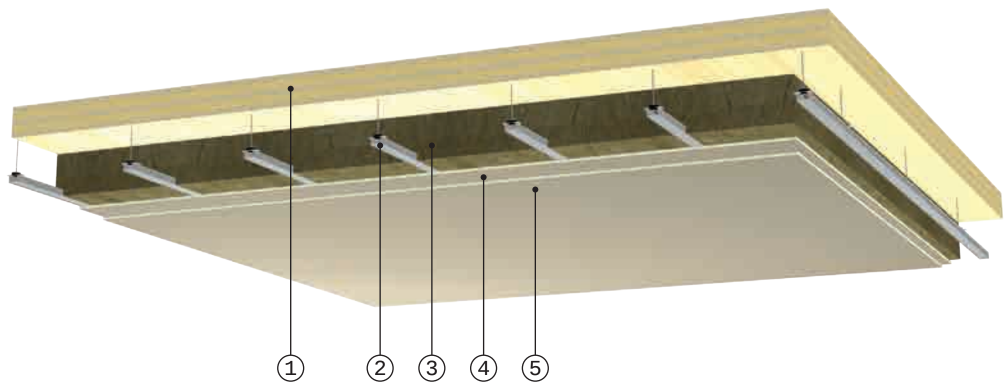
Floor Composition (F4.1)		Airborne (STC) dB	Impact (IIC) dB
1	5-layer CLT panel 146 mm	38	26



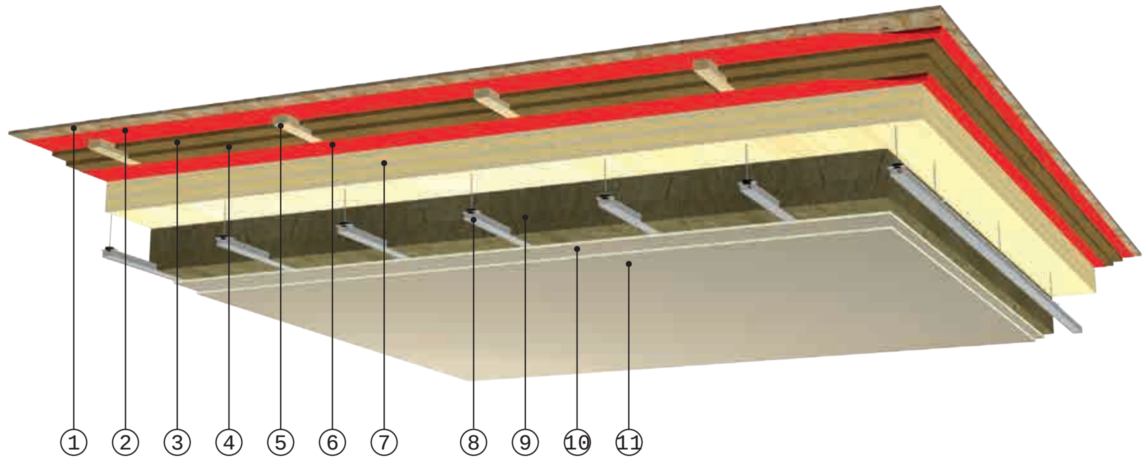
Floor Composition (F4.2)		Airborne (STC) dB	Impact (IIC) dB
1	Gypsum fibre board FERMACELL 10 mm	47	43
2	Gypsum fibre board FERMACELL 10 mm		
3	Rock fibre board 10 mm		
4	5-layer CLT panel 146 mm		



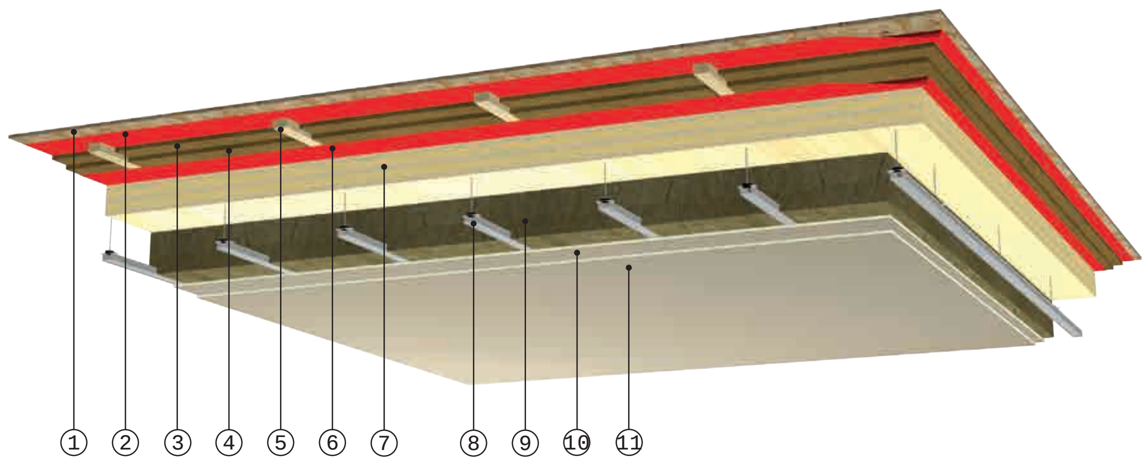
Floor Composition (F4.3)		Airborne (STC) dB	Impact (IIC) dB
1	Gypsum fibre board FERMACELL 10 mm	63	66
2	Gypsum fibre board FERMACELL 10 mm		
3	Rock fibre board 10 mm		
4	5-layer CLT panel 146 mm		
5	Resilient supports and rails (200 mm)		
6	Sound insulation material (fibre glass) (200 mm)		
7	Gypsum board 15 mm		
8	Gypsum board 15 mm		



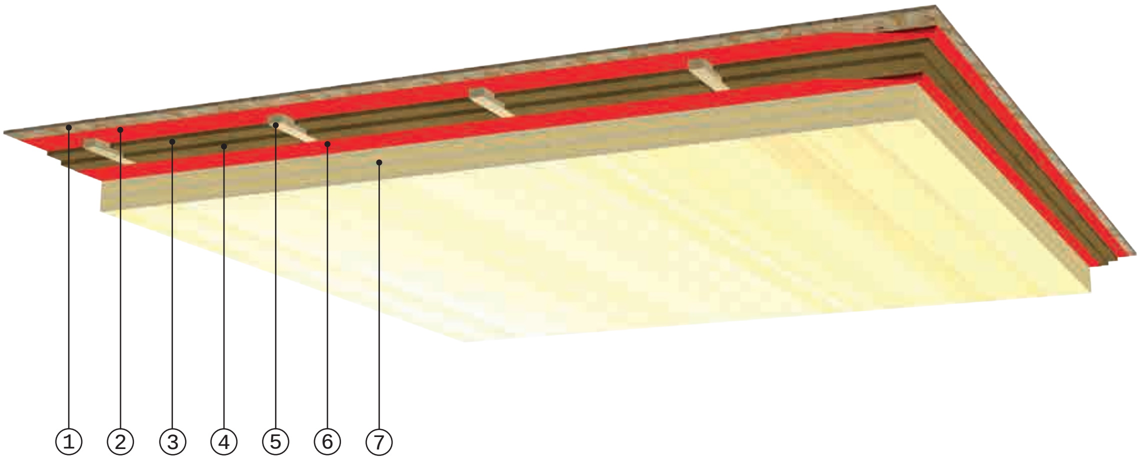
Floor Composition (F4.4)		Airborne (STC) dB	Impact (IIC) dB
1	5-layer CLT panel 146 mm	63	62
2	Resilient supports and rails (200 mm)		
3	Sound insulation material (fibre glass) (200 mm)		
4	Gypsum board 15 mm		
5	Gypsum board 15 mm		



Floor Composition (F4.5)		Airborne (STC) dB	Impact (IIC) dB
1	OSB panel 15 mm	66	69
2	Flooring underlayment ROBERTS		
3	Low-density fibre board THERMISOREL 20 mm		
4	Low-density fibre board THERMISOREL 20 mm		
5	Lumber sleepers 40 mm x 40 mm		
6	Flooring underlayment ROBERTS		
7	5-layer CLT panel 146 mm		
8	Resilient supports and rails (200 mm)		
9	Sound insulation material (fibre glass) (200 mm)		
10	Gypsum board 15 mm		
11	Gypsum board 15 mm		



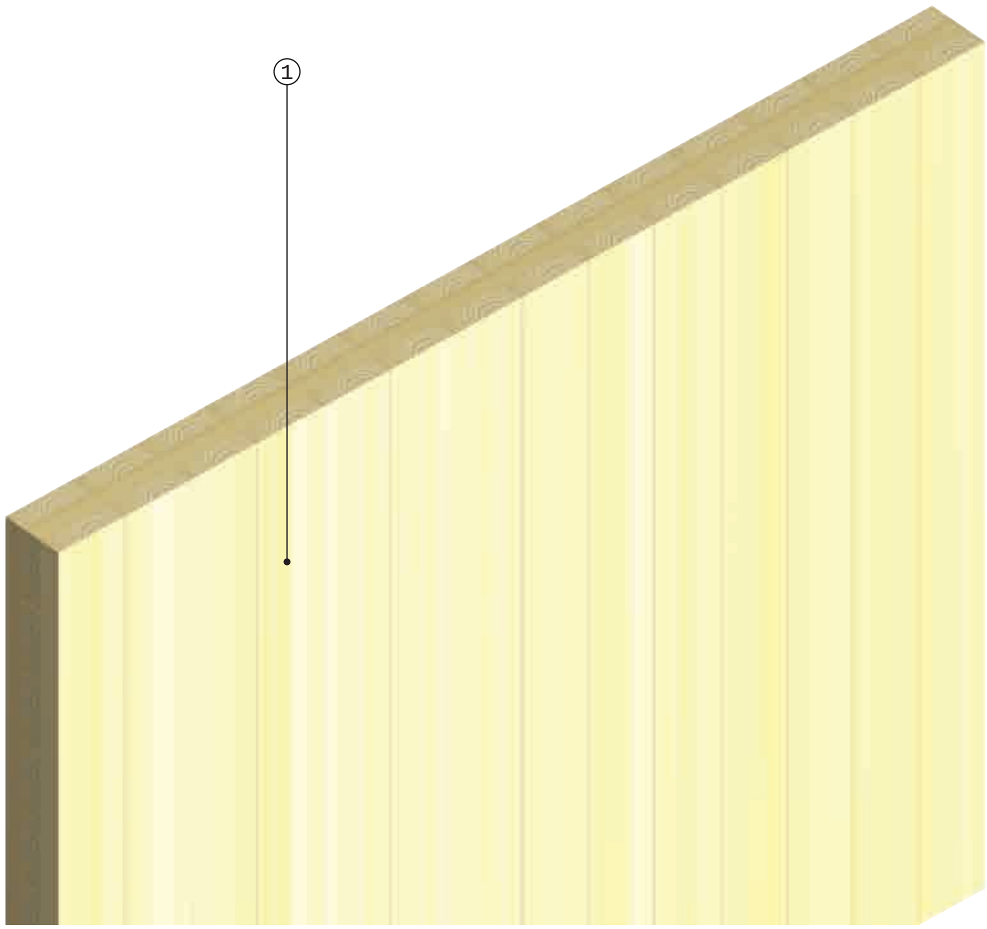
	Floor Composition (F4.6)	Airborne (STC) dB	Impact (IIC) dB
1	OSB panel 15 mm	62	62
2	Flooring underlayment ROBERTS		
3	Low-density fibre board THERMISOREIL 20 mm		
4	Low-density fibre board THERMISOREL 20 mm		
5	Lumber sleepers 40 mm x 40 mm screwed to CLT panel		
6	Flooring underlayment ROBERTS		
7	5-layer CLT panel 146 mm		
8	Resilient supports and rails (200 mm)		
9	Sound insulation material (fibre glass) (200 mm)		
10	Gypsum board 15 mm		
11	Gypsum board 15 mm		



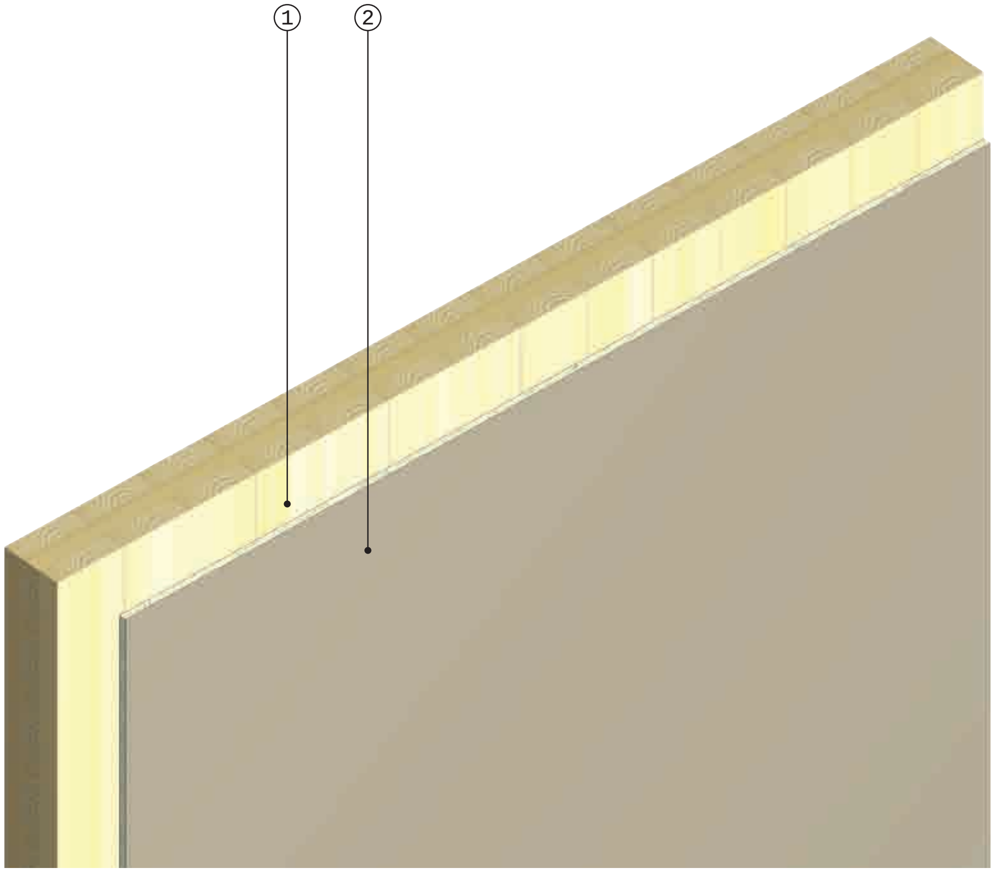
Floor Composition (F4.7)		Airborne (STC) dB	Impact (IIC) dB
1	OSB panel 15 mm	44	39
2	Flooring underlayment ROBERTS		
3	Low-density fibre board THERMISOREL 20 mm		
4	Low-density fibre board THERMISOREL 20 mm		
5	Lumber sleepers 40 mm x 40 mm screwed to CLT panel		
6	Flooring underlayment ROBERTS		
7	5-layer CLT panel 146 mm		

6.4 Existing CLT Wall Assemblies in Europe

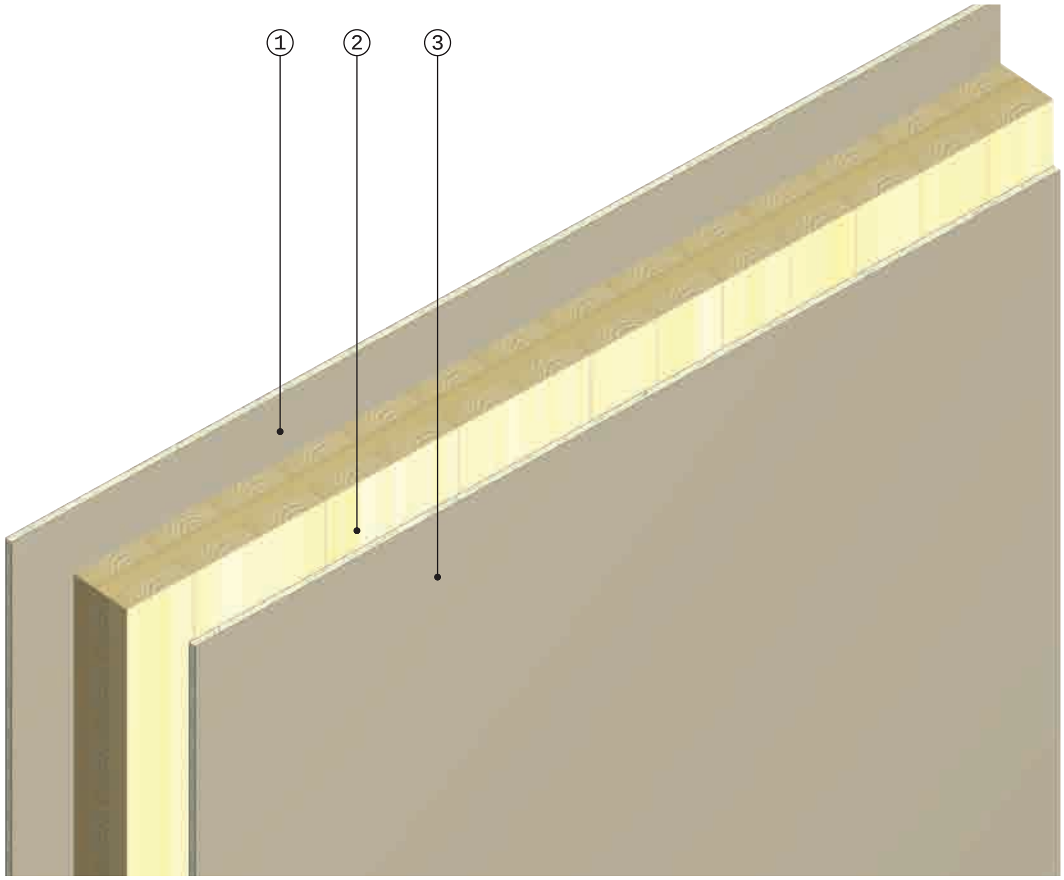
In this section, some examples of common CLT wall assemblies available in European countries are presented. Estimated sound transmission class (STC) is provided for each wall assembly.



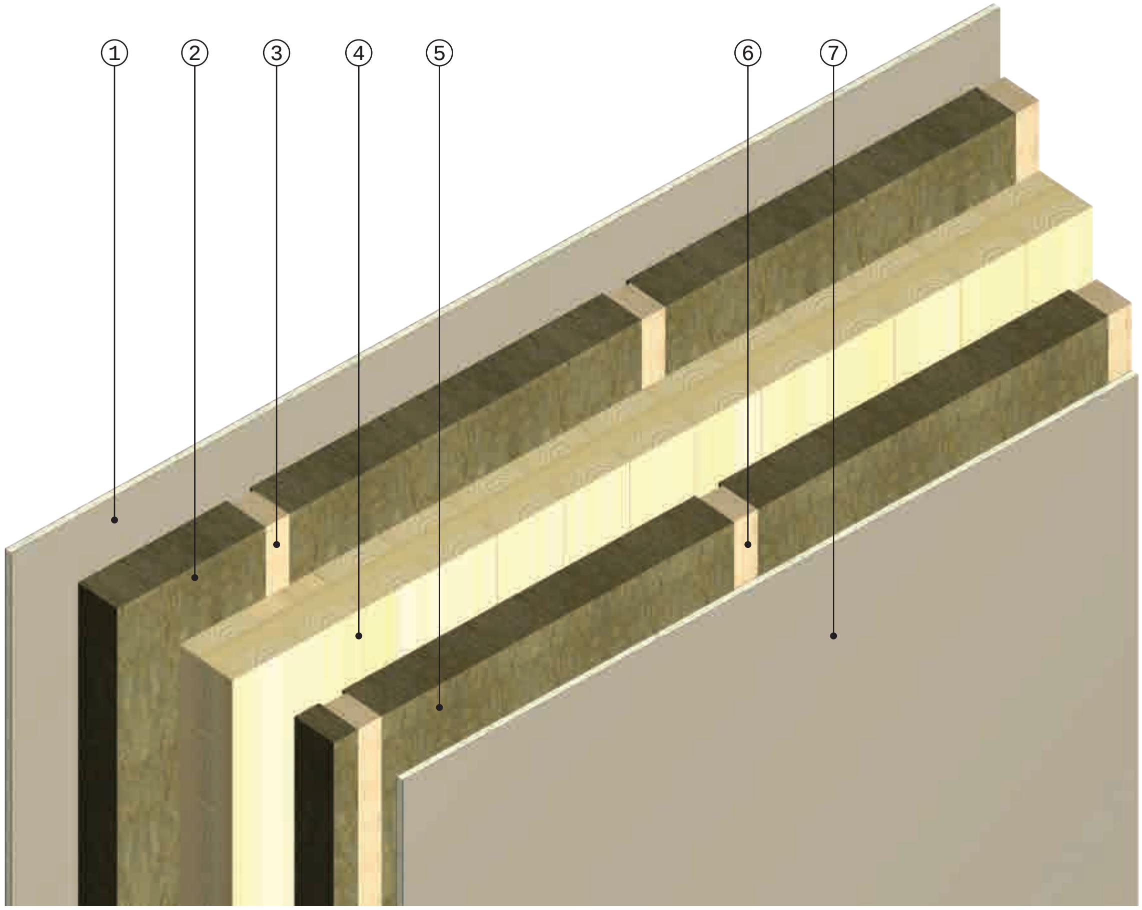
Wall Composition (W1)		Airborne (STC) dB
1	3-layer CLT panel (95 mm ~ 115 mm)	≤ 32 ~ 34



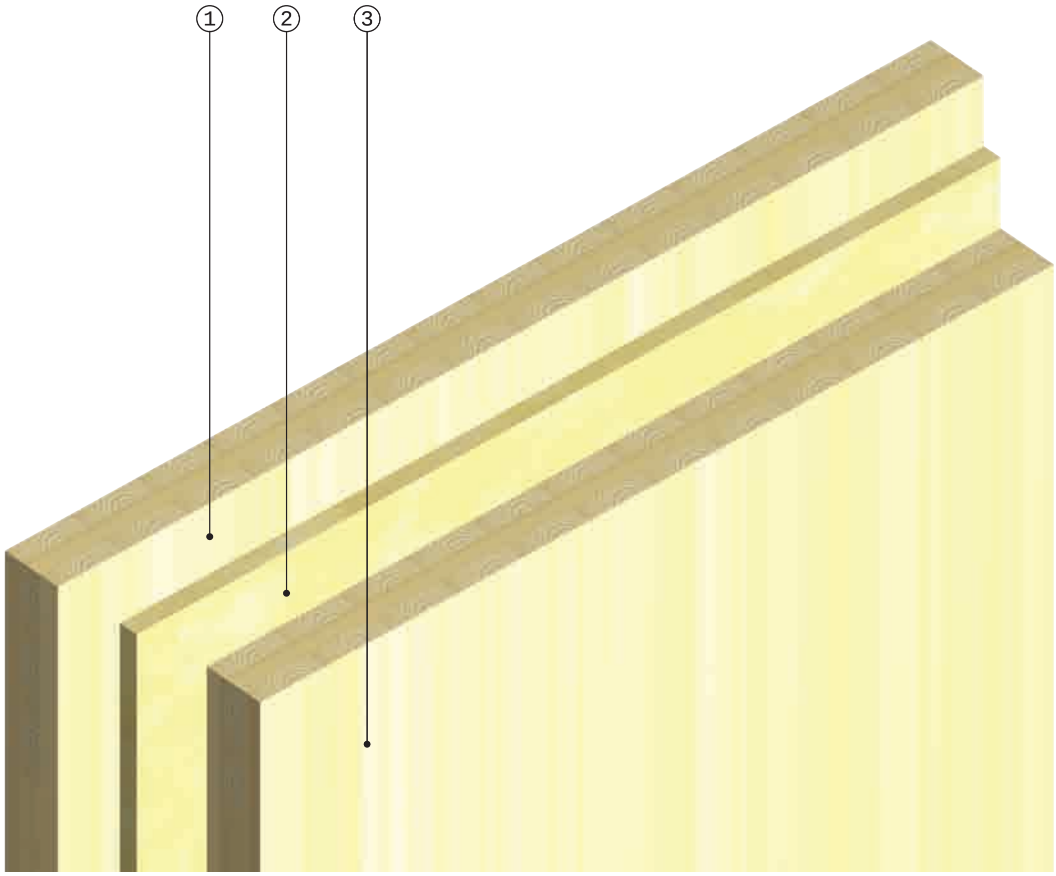
Wall Composition (W2)		Airborne (STC) dB
1	3-layer CLT panel (95 mm ~ 115 mm)	≤ 36 ~ 38
2	Gypsum board 15 mm	



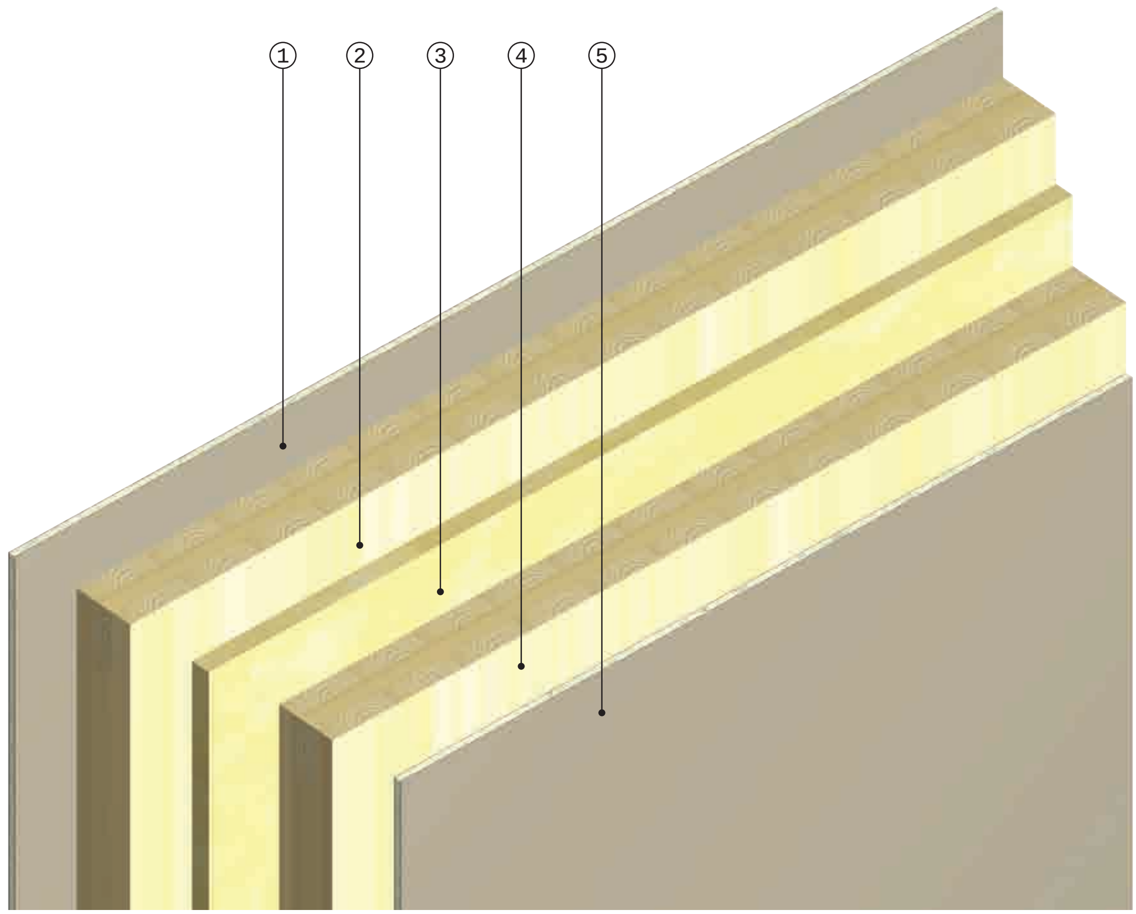
Wall Composition (W3)		Airborne (STC) dB
1	Gypsum board 15 mm	≤ 36 ~ 38
2	3-layer CLT panel (95 mm ~ 115 mm)	
3	Gypsum board 15 mm	



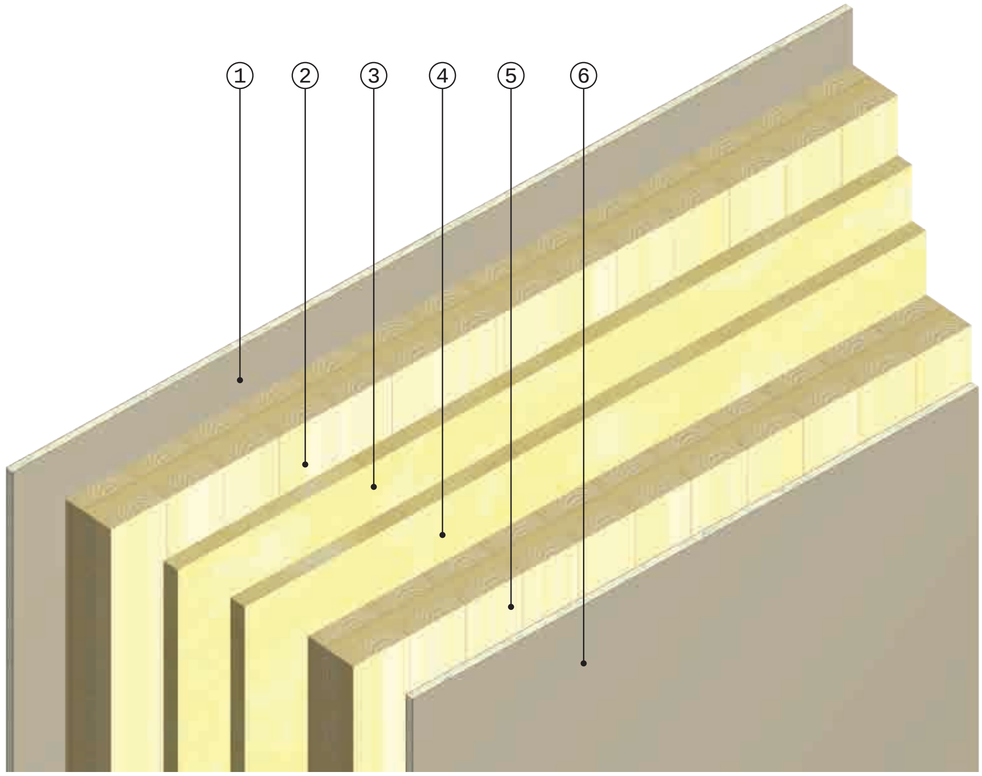
Wall Composition (W4)		Airborne (STC) dB
1	Gypsum board 15 mm	≤ 58
2	Mineral wool (~ 60 mm)	
3	Lumber studs (38 mm x 63 mm)	
4	3-layer CLT panel (95 mm ~ 115 mm)	
5	Mineral wool (~ 60 mm)	
6	Lumber studs (38 mm x 63 mm)	
7	Gypsum board 15 mm	



Wall Composition (W5)		Airborne (STC) dB
1	3-layer CLT panel (95 mm ~ 115 mm)	≤ 48 ~ 50
2	Sound insulation material (mineral or rock wool) (~ 30 mm)	
3	3-layer CLT panel (95 mm ~ 115 mm)	



Wall Composition (W6)		Airborne (STC) dB
1	Gypsum board 15 mm	≤ 55
2	3-layer CLT panel (95 mm ~ 115 mm)	
3	Sound insulation material (mineral or rock wool) (~ 30 mm)	
4	3-layer CLT panel (95 mm ~ 115 mm)	
5	Gypsum board 15 mm	



	Wall Composition (W7)	Airborne (STC) dB
1	Gypsum board 15 mm	≤ 60
2	3-layer CLT panel (95 mm ~ 115 mm)	
3	Sound insulation material (mineral or rock wool) (~ 30 mm)	
4	Sound insulation material (mineral or rock wool) (~ 30 mm)	
5	3-layer CLT panel (95 mm ~ 115 mm)	
6	Gypsum board 15 mm	

7 CONCLUSION

Adequate levels of noise/sound control in multi-family buildings are mandatory requirements of building codes in Canada, the United States, Europe and most developed Asian countries. In many jurisdictions, these requirements are as strictly enforced as those for structural sufficiency and fire safety. Much effort has been spent on evaluation of sound transmission class (STC) and impact sound insulation class (IIC) of floor and wall assemblies and on studies of flanking transmission in multi-family dwellings in Canada. However, very little work has been done in Canada on the acoustic performance of CLT systems in construction.

This chapter presented CLT floor and wall assemblies made of cross-laminated timber elements capable of good acoustic performance in residential, multi-residential and non-residential buildings in Canada and the USA. STC and IIC ratings of existing generic floor assemblies used in Europe have been provided as reference. Moreover, selected types of floor assemblies that may be easily replicated in Canada have been tested at the FCBA. STC and IIC ratings of these floors are also given in this document. Finally, some examples of CLT wall assemblies used in Europe were presented.

8 REFERENCES

Borello, G., and L. Gagliardini. 2007. Virtual SEA: Towards an industrial process. In *SAE Noise and Vibration Conference Proceedings*, May 15-17, 2007, St. Charles, Illinois, paper no. 2007-0123-02.

Canadian Standards Association (CSA). 2009. *Engineering design in wood (limit states design)*. CSA O86-09. Rexdale, ON: CSA. 222 p.

European Committee for Standardization. 2004. *Eurocode 5: Design of timber structures. Part 1-1: General – Common rules and rules for buildings*. EN 1995-1-1. Brussels: British Standards Institution (BSI). 124 p.

Hu, L. J. 2005. Acoustic performance of wood-frame buildings. Canadian Forest Service Report No. 3. Quebec: Forintek Canada Corp. 44 p.

Kouyoumji, J. L. 2004. Sound transmission loss prediction and vibro-acoustic SEA analysis of a wood framed floor. Paper presented at the 33rd International Congress and Exposition on Noise Control Engineering: Inter-noise 2004, August 22-25, 2004, Prague, Czech Republic.

______. 2007. Vibro-acoustic characterization of timber constructions: Measurements and modeling using statistical energy analysis (SEA). Paper presented at the 36th International Congress and Exhibition on Noise Control Engineering: Inter-noise 2007, August 28-31, 2007, Istanbul, Turkey.

Kouyoumji, J. L., and S. Gagnon. 2010. Experimental approach on sound transmission loss of cross laminated timber floors for building. Paper presented at the 39th International Congress and Exposition on Noise Control Engineering: Inter-noise 2010, June 13-16, 2010, Lisbon, Portugal.

Kouyoumji, J. L., S. Gagnon, and S. Boulet. 2009. Sound transmission loss of cross laminated timber 'CLT' floors, measurements and modelling using SEA. Paper presented at the 38th International Congress and Exposition on Noise Control Engineering: Inter-noise 2009, August 23-26, 2009, Ottawa, Canada.

Lyon, R. H., and R.G. DeJong. 1995. *Theory and application of statistical energy analysis*. 2nd ed. Boston: Butterworth-Heinemann. 277 p.

National Research Council (NRC). 2005. *National building code of Canada, 2005*. Ottawa: Canada. National Research Council. 2 v.

Richardson, L. R., and M. Batista. 2002. Fire-resistance and sound-transmission-class ratings for generic wood-frame assemblies. Quebec: Forintek Canada Corp. 27 p. + appendices.

Taguchi, G. 1986. Introduction to quality engineering: Designing quality into products and processes. Tokyo: Asian Productivity Association. 191 p.

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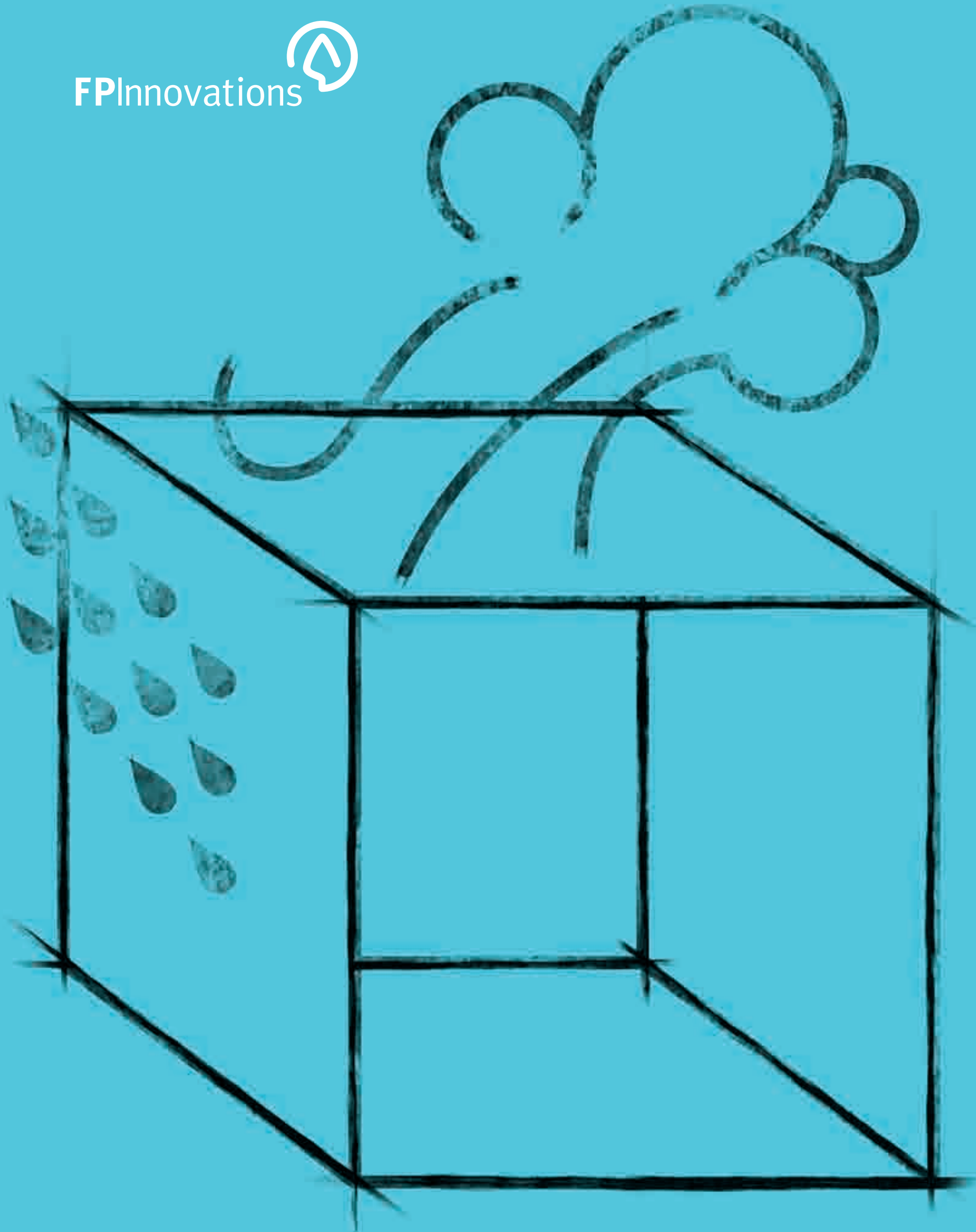

Enclosure

Building enclosure design of cross-laminated timber construction

CHAPTER 10

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ABSTRACT

Cross-laminated timber (CLT) has become popular in Europe for the prefabricated construction of wall, roof and flooring elements. The use of CLT in North America is gaining interest in both the construction and wood industries. Several North American manufacturers are in the process of product and manufacturing assessment or have already started pilot production.

For general principles of durability by design, the Best Practice Guide for Wood-Frame Envelopes (CMHC, 1999) and the Building Enclosure Design Guide – Wood-Frame Multi-Unit Residential Buildings (HPO, 2010) should be referred to for the design and construction of CLT buildings. The use of prefabricated CLT panels does not change the basic heat, air and moisture control design criteria for an exterior wall or roof assembly. However, different from conventional stick-built wood-frame buildings, the design of CLT building enclosures requires additional attention due to the unique characteristics of the product. CLT panels are massive solid wood elements and therefore have low vapour permeability and may provide a considerable level of insulation. They have a certain level of inherent air tightness but usually require an additional air barrier. The panels may absorb a large amount of moisture when exposed to excessive wetting and the consequent drying may be slow due to the mass of wood in such panels.

This chapter focuses on best practice heat, air and moisture control strategies for wall assemblies that utilize CLT panels in North American climate zones. The overlying strategies are to place insulation in such a way that the panels are kept warm and dry, to prevent moisture from being trapped or accumulating within the panel, and to control airflow through the panels, and at the joints and interfaces between them.

It is intended that these guidelines should assist practitioners in adapting CLT construction to North American conditions and ensuring a long life for their buildings. However, these guidelines are not intended to substitute for the input of a professional building scientist. This may be required in some jurisdictions, such as Vancouver BC, and is recommended in all areas at least until such time as CLT construction becomes common practice.

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1 INTRODUCTION

Cross-laminated timber (CLT) has become popular in Europe for the prefabricated construction of wall, roof, and flooring elements. The use of CLT in North America is gaining interest in both the construction and wood industries. Several North American manufacturers are in the process of product and manufacturing assessment or have already started pilot production.

CLT panels are typically constructed by laminating three or more layers of lumber together, with each layer rotated 90° relative to the neighbouring layers to create a solid wood panel. The lumber is most commonly adhered using a structural adhesive, with or without edge-gluing between lamina in the same layer. Manufacturing methods and lamina quality may have an impact on the final product properties but they do not affect the overall design strategy.

For general principles of durability by design, the Best Practice Guide for Wood-Frame Envelopes (CMHC, 1999) and the Building Enclosure Design Guide – Wood-Frame Multi-Unit Residential Buildings (HPO, 2010) should be referred to for the design and construction of CLT buildings. The use of prefabricated CLT panels does not change the basic heat, air and moisture control design criteria for an exterior wall or roof assembly. However, different from conventional stick-built wood-frame buildings, the design of CLT building enclosures requires additional attention due to the unique characteristics of the product. CLT panels are massive solid wood elements and therefore have low vapour permeability and may provide a considerable level of insulation. They have a certain level of inherent air tightness but usually require an additional air barrier. The panels may absorb a large amount of moisture when exposed to excessive wetting and the consequent drying may be slow due to the mass of wood in such panels.

Although occasionally used in this way experimentally, CLT panels are not a cladding material and are not designed to be exposed to the exterior environment. They are a moisture sensitive structural assembly, and therefore must be protected from rain and other moisture sources through the use of properly designed wall assemblies.

This chapter focuses on best practice heat, air and moisture control strategies for wall assemblies that utilize CLT panels in North American climate zones. The overlying strategies are to place insulation in such a way that the panels are kept warm and dry, to prevent moisture from being trapped or accumulating within the panel, and to control airflow through the panels, and at the joints and interfaces between them.

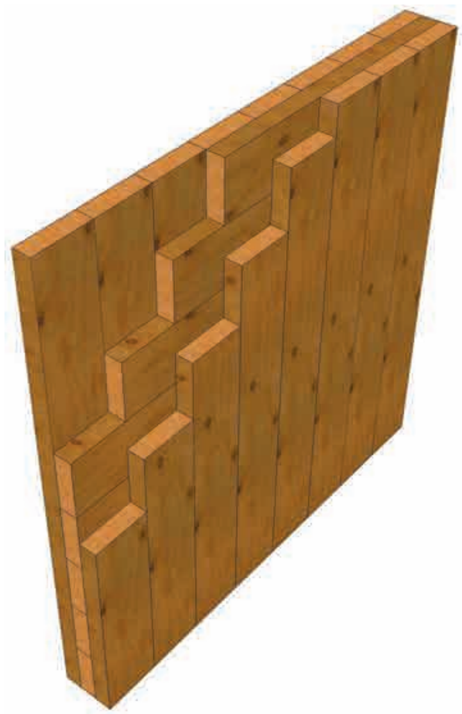

Figure 1
CLT panel constructed of three layers of cross-laminated board lumber to create a solid wood panel suitable for prefabrication of whole wall, floor and roof elements

2 HEAT, AIR AND MOISTURE CONTROL STRATEGIES

2.1 Heat, Air and Moisture Control Strategies

Heat flow control is achieved using insulation to minimize space heat loss or gain through the building enclosure. Air leakage control is also a key element of heat flow control.

Unlike stick-built wood-frame wall assemblies, where fibreglass batt insulation is traditionally placed between the studs, CLT panels are solid and therefore require insulation, placed appropriately, on one side of the panel.

Being laminated solid wood, CLT inherently offers a nominal amount of insulation. Softwood species which typically make up a CLT panel provide an R-value of approximately R-1.2 h·ft^2·°F/Btu per inch, (i.e. R-4.2 for a 3 ½ in. thick panel). While this inherent R-value is a good start, additional insulation must also be provided for the wall assembly to meet local energy code requirements. In most Canadian jurisdictions, a nominal insulation in the range of R-20 (RSI 3.52 K·m^2/W) may be required in walls. In the case of this 3 ½ in. panel, it would be necessary to add other insulating material with a minimum R-value of R-16 to the CLT wall assembly.

Table 1
Examples of insulation thicknesses for CLT building assemblies

Required Nominal Insulation R-value (RSI)	CLT thickness inch (mm)	CLT insulation R-value (RSI)	Required thickness of additional insulation with R-4 / inch inch (mm)
12 (2.11)	2.0 (50)	2.4 (0.42)	2.5 (64)
	3.5 (89)	4.2 (0.74)	2.0 (51)
	5.5 (140)	6.6 (1.16)	1.5 (38)
20 (3.52)	2.0 (50)	2.4 (0.42)	4.5 (114)
	3.5 (89)	4.2 (0.74)	4.0 (102)
	5.5 (140)	6.6 (1.16)	3.5 (89)
28 (4.93)	2.0 (50)	2.4 (0.42)	6.5 (165)
	3.5 (89)	4.2 (0.74)	6.0 (152)
	5.5 (140)	6.6 (1.16)	5.5 (140)

The placement of the insulation may significantly affect the moisture levels and durability of the wood panel in service. In all climate zones, most types of insulation should be placed on the exterior side of the CLT panels. This will keep the wood in a relatively constant warm and dry indoor environment and reduce the risk of moisture damage. Section 2.1.1 shows that the use of vapour permeable insulation materials such as mineral or wood fibreboards are recommended in lieu of less vapour permeable foam plastics. Where wood fibre insulation boards are used outside the weather resistant barrier, the wood should be treated to minimize water uptake and possible fungal growth, and wall penetrations properly detailed to prevent wetting of the insulation.

CLT panels themselves may offer aesthetic benefits and may be left exposed on the interior side to showcase the solid wood finish if the fire safety and acoustic requirements allow. This is another reason why thermal insulation should be placed on the exterior side of the panel. When used in certain building types, some jurisdictions may require that the interior exposed wood surface be covered with gypsum drywall or other non-combustible finish to meet fire safety requirements. In this scenario, it might be seen as desirable to place insulation on the interior side of the panel; however, this wall assembly is not recommended as the CLT panel will be more vulnerable to wetting caused by vapour diffusion from the interior, or wetting from rainwater or solar driven moisture from the exterior.

2.1.1 Choosing a Suitable Exterior Insulation Material

The common construction practice of building with CLT panels in Europe has been to insulate the panels with wood fibreboard or rigid mineral fibreboard insulation. These products currently have limited availability in some areas of North America, but have ideal properties for CLT construction.

Both rigid mineral fibre and wood fibre insulation boards are vapour permeable and have adequate R-values of R-3.5 to R-4.0 h·ft^2·°F/Btu per inch of thickness depending on density and other factors. In a jurisdiction requiring R-20 to R-30 (RSI 3.52 to 5.28 K·m^2/W) of nominal insulation, 4 to 7 in. (102 to 178 mm) of rigid wood or mineral fibre insulation board would be necessary.

Rigid mineral fibre or wood fibre boards are preferred products because they may be rigid enough to allow for furring or cladding supports to be structurally fastened directly through the insulation without the need for additional framing on the exterior. Long screws (> 6 to 8 in.) are available and can be used to attach furring directly through to the CLT panels to support the cladding, if this meets the structural requirements of the cladding attachment. Otherwise, additional support for the insulation should be provided at each floor level.

Extruded polystyrene (XPS) or expanded polystyrene (EPS) may also be sufficiently rigid to screw furring (strapping) through; however, the vapour permeability of these foam plastic insulations is relatively low, which reduces the drying capacity of the CLT panel and may trap moisture within the wood panels. Modeling has shown that drying through 3 to 5 in. of either EPS or XPS on the exterior side of a CLT assembly is slow, and can lead to damage to CLT panels that are initially wet, wetted during construction, or exposed to humid indoor conditions or a rainwater leak during service. The use of foam plastic insulation products is not ideal for insulating CLT wall assemblies, particularly for a heating-dominated climate, because of their low permeability and the consequently reduced drying capacity.

Less rigid, but vapour-permeable insulation materials including semi-rigid fibreglass or mineral wool boards commonly available in North America, are also suitable for exterior insulating CLT panels, but require additional framing on the exterior of the panels for cladding attachment. Because furring to support the cladding cannot be nailed through these less-dense insulation boards, 2x4 or 2x6 studs or intermittent wood blocks need to be attached or framed to the exterior of the CLT panels for cladding support. Semi-rigid insulation boards would then be placed tightly between the wood framing.

2.2 Vapour Flow Control

The purpose of vapour control within a wall assembly is to limit the flow of moisture by vapour diffusion, thereby preventing interstitial condensation. Vapour control is typically provided on the warm or high vapour pressure side of the insulating layer because the moisture drive will then be towards the more vapour open side of the assembly. Care must be taken to avoid a design where water vapour may become trapped within an assembly by incorrect vapour retarder or barrier placement.

In a traditional wood-frame wall assembly, vapour flow control is achieved on the inside surface of the batt insulation using a sheet of either asphalt impregnated kraft paper (a vapour retarder), polyethylene (a vapour barrier), or in some cases application of a vapour retarding paint on the gypsum board.

The vapour permeance of a 3½ in. thick softwood CLT panel is less than 30 ng/Pa·s·m^2 (~0.5 US perms) at normal indoor RH levels, based on the typical vapour permeance of solid softwood. Therefore, the CLT panel itself will control the flow of vapour through the assembly in most situations. This property must be considered in the design of a wall assembly and should be used as a design advantage instead of disadvantage. This further highlights the importance of placing insulation on the exterior side of the CLT panel to ensure the vapour retarding layer is on the warm side of the insulation.

In order to comply with the prescriptive requirements of some local building codes, the use of a vapour barrier material may be contemplated in design. However, the use of an additional interior vapour control layer (i.e. vapour retarder paint, impermeable finish or polyethylene sheet) can limit drying to the interior. Therefore, when gypsum sheathing is used on the interior surface of the CLT panel, the vapour permeance of the paint or wall finish should be carefully considered. This is particularly important in warmer humid climates where the predominant vapour drive is inwards. The installation of other materials within CLT panel assemblies which restrict panel drying, including foam plastic insulation and self-adhered bituminous membranes on the outside surface of the panels is not recommended as they will limit drying to the exterior. These recommendations apply to all climate zones on the principle that assemblies utilizing CLT panels should be designed to allow vapour to flow readily out of the assembly.

2.2.2 CLT Panels as Vapour Control Layer

The vapour permeance of wood varies with relative humidity and it has a lower permeance when exposed to a lower RH. The vapour permeance of various wood species and directions of grain can be found in the ASHRAE Handbook of Fundamentals, among other resources.

The vapour permeance of softwood lumber at normal indoor RH levels of 30 to 50% generally ranges from less than 10 $ng/Pa{\cdot}s{\cdot}m^2$ to as much as 110 $ng/Pa{\cdot}s{\cdot}m^2$ (0.17 to 1.9 US perms) for a 25 mm (1 in.) thick piece of lumber. For a CLT panel thickness of 89 mm (3½ in.), the total vapour permeance would range from less than 3 $ng/Pa{\cdot}s{\cdot}m^2$ to 30 $ng/Pa{\cdot}s{\cdot}m^2$ (0.05 to 0.5 US perms). Following the performance levels intended in building code requirements, the CLT panels themselves may meet the requirements for both a vapour retarder and a vapour barrier.

2.3 Rainwater and Exterior Moisture Control

Rainwater management is critical to the long-term performance of all wood-frame wall assemblies. Beyond basic design details such as using roof overhangs to shelter and reduce rainwater wetting of wall assemblies, CLT panel wall designs require further protection. The best practice strategy for rainwater penetration control is a drained and ventilated rainscreen cladding, which is common construction practice in the coastal climates of British Columbia and Washington State, and other wet climates across North America. While this rainwater control strategy may seem excessive in some climates, it may reduce problems associated with other exterior wetting mechanisms.

This strategy as it applies to CLT panel walls dictates that a weather resistant barrier (WRB) will be either mechanically fastened or adhered to the panels by the manufacturer in the factory or by the contractor immediately after the CLT panels are erected on site. This will keep the panels relatively dry through the early stages of delivery and construction. In line with the vapour open design philosophy and vapour control discussion, the WRB material should have a high vapour permeance, (i.e. a house wrap or comparable product).

As shown in Figure 2, the WRB should then be covered with vapour permeable rigid insulation boards. Typically, a cladding, such as fibre-cement board and stucco, would then need to be structurally attached to the CLT panels. Cladding would not be attached directly to the CLT panels through insulation, because long screws (> 6 to 8 in.) would be necessary for every attachment point. For cladding attachment, continuous vertical furring (strapping) strips should be screwed through the rigid insulation to the CLT panel and the cladding should then be attached to the furring with short fasteners. Depending on loading conditions, a structural analysis of this cladding attachment scheme may be required. The gap between the furring strips creates an air space behind the cladding, which is beneficial for both drainage and ventilation. This air space should, at minimum, be vented and drained (opened at the bottom) or ideally ventilated and drained (i.e. by providing openings in the cladding at both the top and bottom).

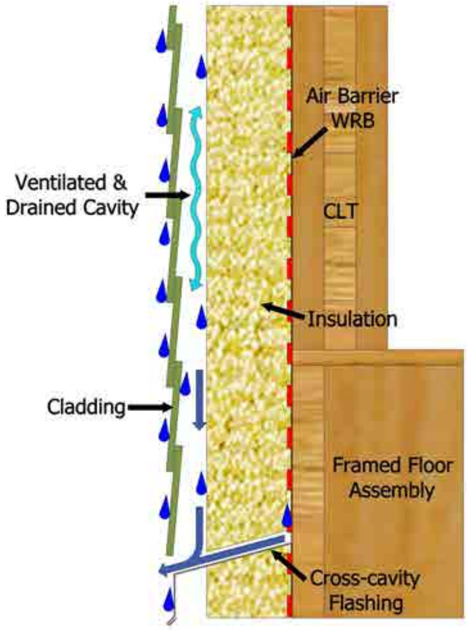

Figure 2
Best practice rainwater management strategy for CLT wall assembly. Detail shows a ventilated and drained cladding rainscreen system where primary cladding and secondary drainage planes are provided in addition to ventilation behind the cladding. This is typical of other exterior insulated rainscreen wall assemblies.

The practice of back-ventilating sidings such as wood, hardboard, and cement board is recommended by most manufacturers to better ensure the long-term performance of their products. It is also beneficial to provide an outlet for inward driven moisture from more absorptive claddings such as brick, stucco, stone and other porous materials.

The cladding surface will shed the majority of the rainwater load on the wall; however, it is not the only line of water penetration resistance. Moisture that does penetrate past the cladding will either run down the backside of the cladding, the strapping, the surface of the insulation, or the final line of protection, the sealed WRB. Any moisture which penetrates the cladding must then be drained back out of the assembly using flashings attached to the CLT panel behind the WRB at floor levels and around penetrations such as windows.

2.4 Air Flow Control

Air infiltration and exfiltration is controlled through enclosure assemblies that prevent interstitial condensation and minimize space heat energy loss or unwanted heat gain. Air flow through wall assemblies can be controlled using either a single material or, more commonly, a series of materials which together make up a continuous air barrier system.

In a traditional wood-frame wall assembly, air flow has to be controlled with an air barrier approach using either: sealed polyethylene, air-tight drywall, sealed sheathing or sealed sheathing membrane.

CLT is a massive wood component but the air-tightness of CLT panels is dependent on the joints between the individual boards and the individual layers. Gaps between individual boards or layers and checking in boards may occur due to shrinkage during storage, transportation and construction as a result of drying or cyclical wetting and drying. Manufacturing processes such as edge-gluing between boards can help improve the air-tightness of the panels. If the CLT panels are used as part of the air barrier assembly within a building, appropriate measures such as flexible sealant joints between CLT panels and other elements of the air barrier assembly would be required for air barrier continuity. However, in most cases, the CLT panel itself cannot be relied upon for air tightness, and it may be better practice to provide the primary air barrier system using other materials within the assembly.

The primary air barrier system could be the weather resistant barrier (WRB) adhered or mechanically fastened to the exterior of the panel (sealed WRB approach) or a carefully detailed drywall layer on the interior side of the panel (air-tight drywall approach). The effective implementation of the air barrier strategy would then rely on the details to achieve continuity at penetrations such as windows or doors, as well as at interfaces with floors, ceilings, balconies, decks, roofs and interior partitions. The details for such transitions would be similar to those used in traditional wood-frame construction. The use of the water resistive barrier is the preferred approach in most situations because there are fewer penetration and interface details to address.

3 RECOMMENDED CLT PANEL CONCEPTUAL DESIGN

3.1 Exterior Wall Assembly

Figures 3 and 4 demonstrate a CLT assembly where the exterior insulation is sufficiently rigid to allow for furring strips to be screwed directly through it to the CLT panel with minimal compression. In this assembly, a continuous WRB is applied before the rigid mineral fibreboard is placed on the exterior of the panel. Vertical furring such as strips of plywood or 1x4 lumber are fastened directly through one layer of insulation to the CLT panels with long screws (> 6 in.) to provide attachment points for the cladding, assuming this meets the structural requirements of cladding attachment. This assembly is thermally most efficient. The space between the furring strips is left open to provide drainage behind the cladding and openings are provided at the top and bottom of the wall for ventilation of the cavity. The assembly shown does not contain gypsum drywall on the interior. Where required for fire safety and acoustic control purposes, gypsum drywall would be fastened to the CLT panels or could be supported on vertical furring strips to allow for wiring and other services to be concealed.

Figures 5 and 6 illustrate two alternate cladding support framing strategies. Figure 5 shows two strapping members attached through the insulation to the CLT panels. The first strapping member would typically be a 2x2, and the second member a 2x2 or a size that suits the thickness of insulation. The first strapping member is attached to the CLT panels with shorter 5 in. screws and the second is then attached to the first strapping member. This method may be necessary where greater thicknesses of insulation are required (i.e. 6 in. insulation which would require 8 to 9 in. screws). It also offers benefits for detailing around penetrations, and allows the insulation to be installed with staggered joints. Where less rigid mineral fibre insulation boards or batts are used, or where structural analysis indicates a more rigid cladding support is needed, solid framing members (i.e. studs) would be required to support the cladding. Figure 6 shows 2x6 framing lumber directly attached to the CLT panels—thermally the least efficient of the systems discussed above.

Both the furring and framing members placed on the exterior of the WRB/air barrier should be protected with some level of wood preservative such as borate, CA, ACQ, or CCA (for plywood only), depending on the exposure and local building code requirements. Attention should be given to the selection of appropriate corrosion-resistant fasteners suitable for use with the preservative chosen for wood treatment.

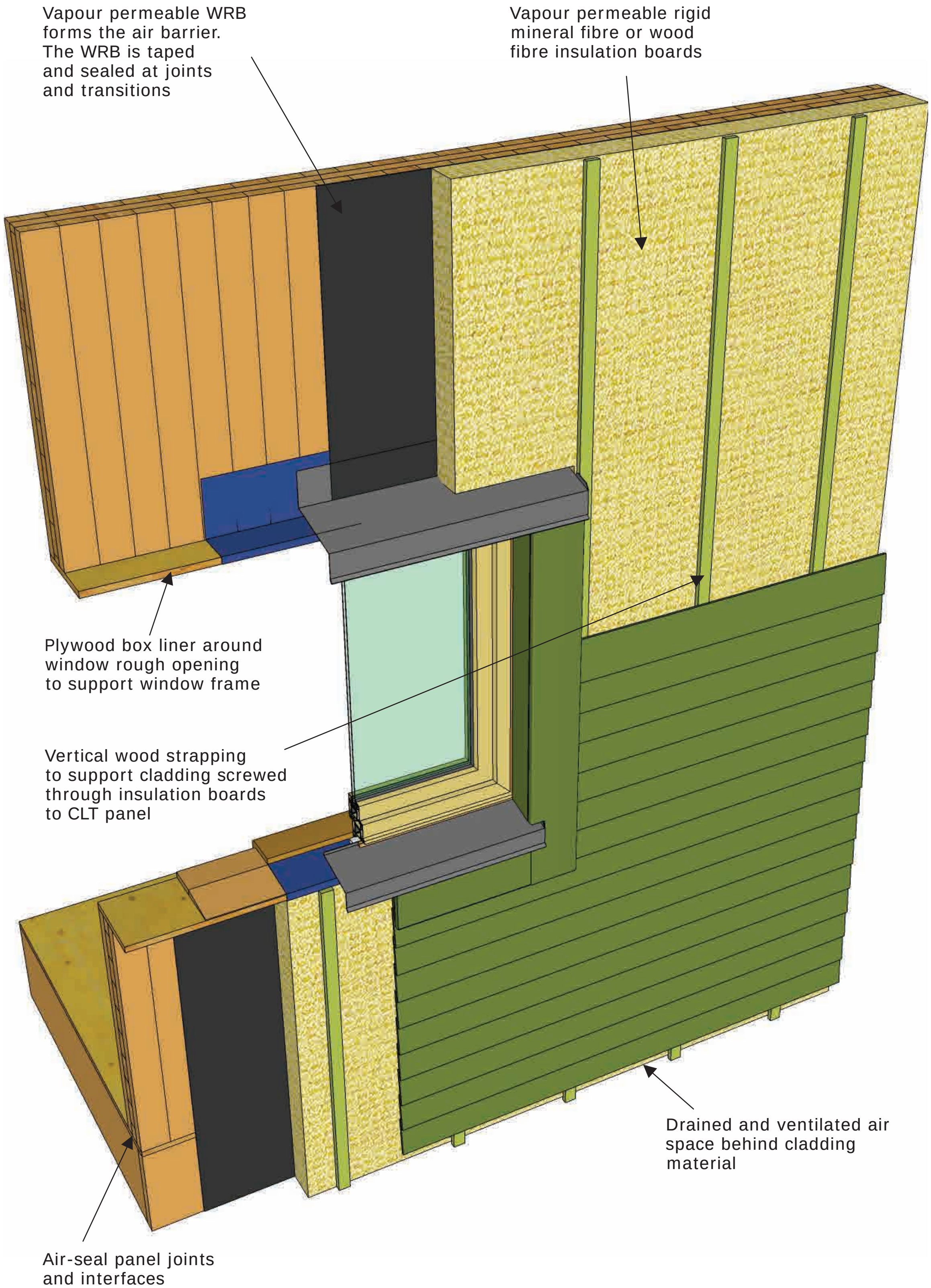

Figure 3

Exterior insulated rainscreen clad CLT exterior wall assembly showing material sequencing and schematic window penetration details

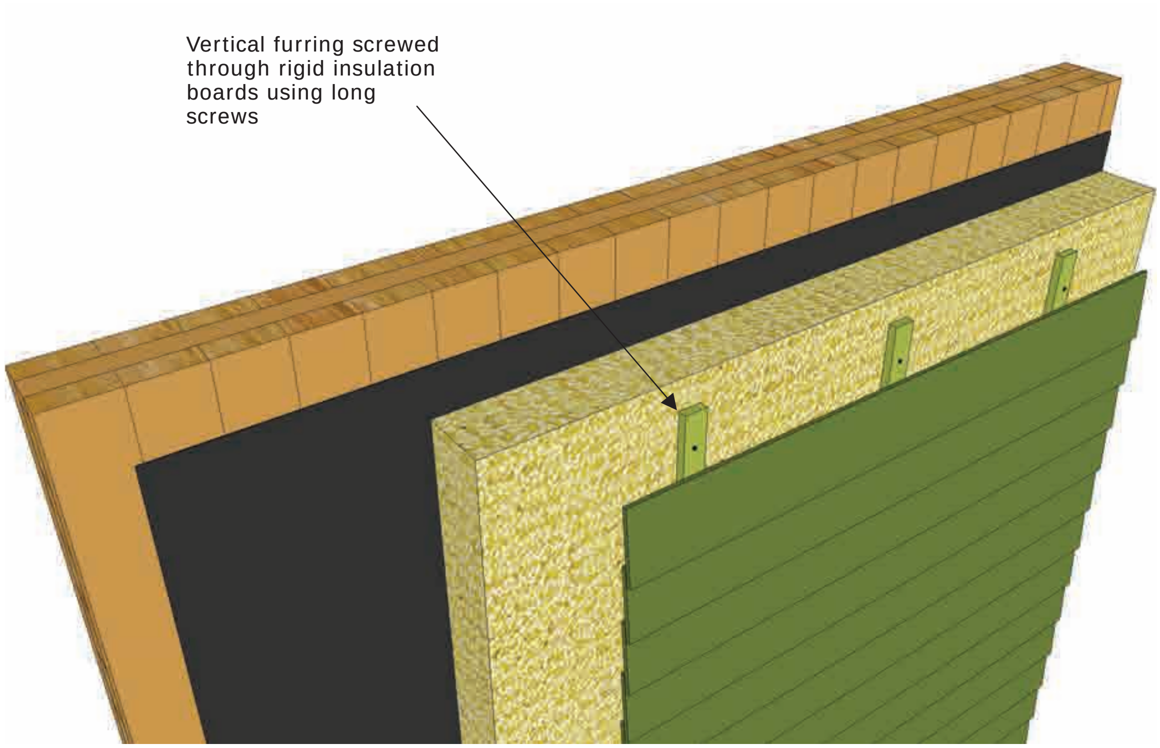

Figure 4
Cladding support strategy using vertical furring through rigid insulation board

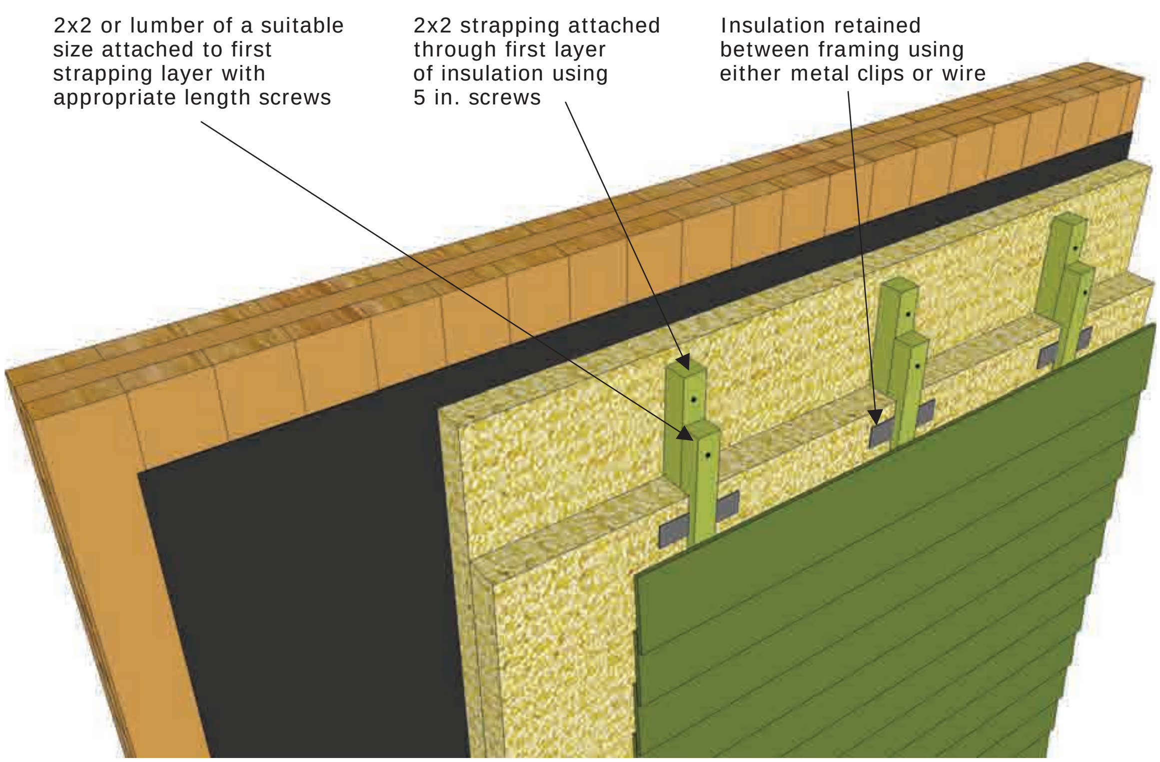

Figure 5

Cladding support strategy using two layers of rigid insulation and two strapping members: this configuration allows for the use of shorter screws and greater insulation thicknesses while minimizing thermal bridging

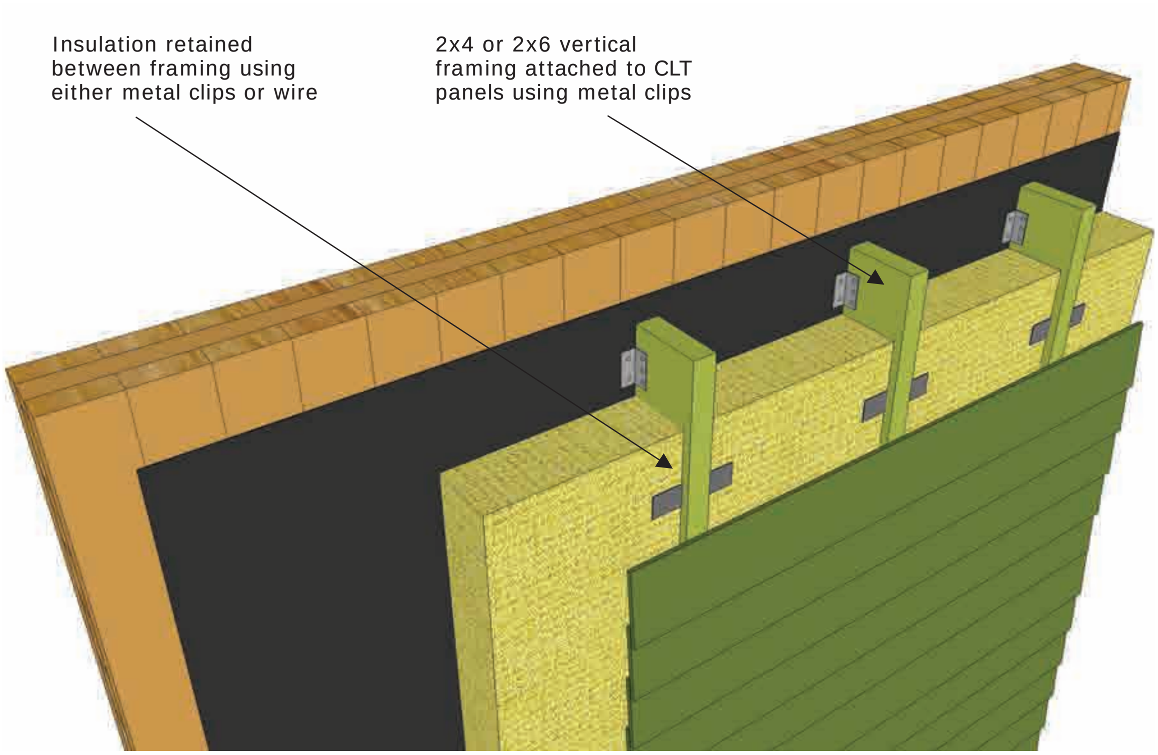

Figure 6
Cladding support strategy using stud framing attached directly to CLT panel with semi-rigid or batt insulation between framing

3.2 Roof Assemblies

The use of CLT panels within sloped roof assemblies will have similar design considerations as for walls. CLT roof panels are typically thinner than wall panels (30 to 50 mm thick) and are detailed to span between roof beams or trusses, and provide an interior finish similar to tongue-and-groove paneling. In this configuration, the insulation, moisture control layer and air barrier will be placed on the exterior side of the panel similar to a CLT wall assembly. Insulation requirements within building codes are typically higher for roofs than for walls dictating greater insulation thicknesses. Unlike with walls, an impermeable waterproofing membrane is used with roofs because drying is facilitated through the thinner CLT panel and exposed interior finish.

The type of roofing material will dictate the framing support structure on the exterior side of the CLT panels. Typically, this will involve the installation of purlins or intermittent structures and sheathing to support the roofing material. In this application, the CLT panel functions as the structural base and interior finish for the assembly. This type of exterior insulated assembly lends itself well to the use of metal or tile roofing materials. Figures 7 and 8 show material sequencing of a sloped CLT roof and tie-in details to a CLT wall assembly at the underside.

CLT panels may also be used in flat or low-slope roof assemblies—in which case using a conventional roofing assembly would be best practice to protect the CLT panel. Similar to a conventional above-deck insulated wood-frame roof with plywood, an air/vapour barrier, insulation and exposed roofing membrane is placed on top of the CLT panel. Figure 9 shows material sequencing of a low-slope CLT roof and tie-in details to a CLT wall assembly.

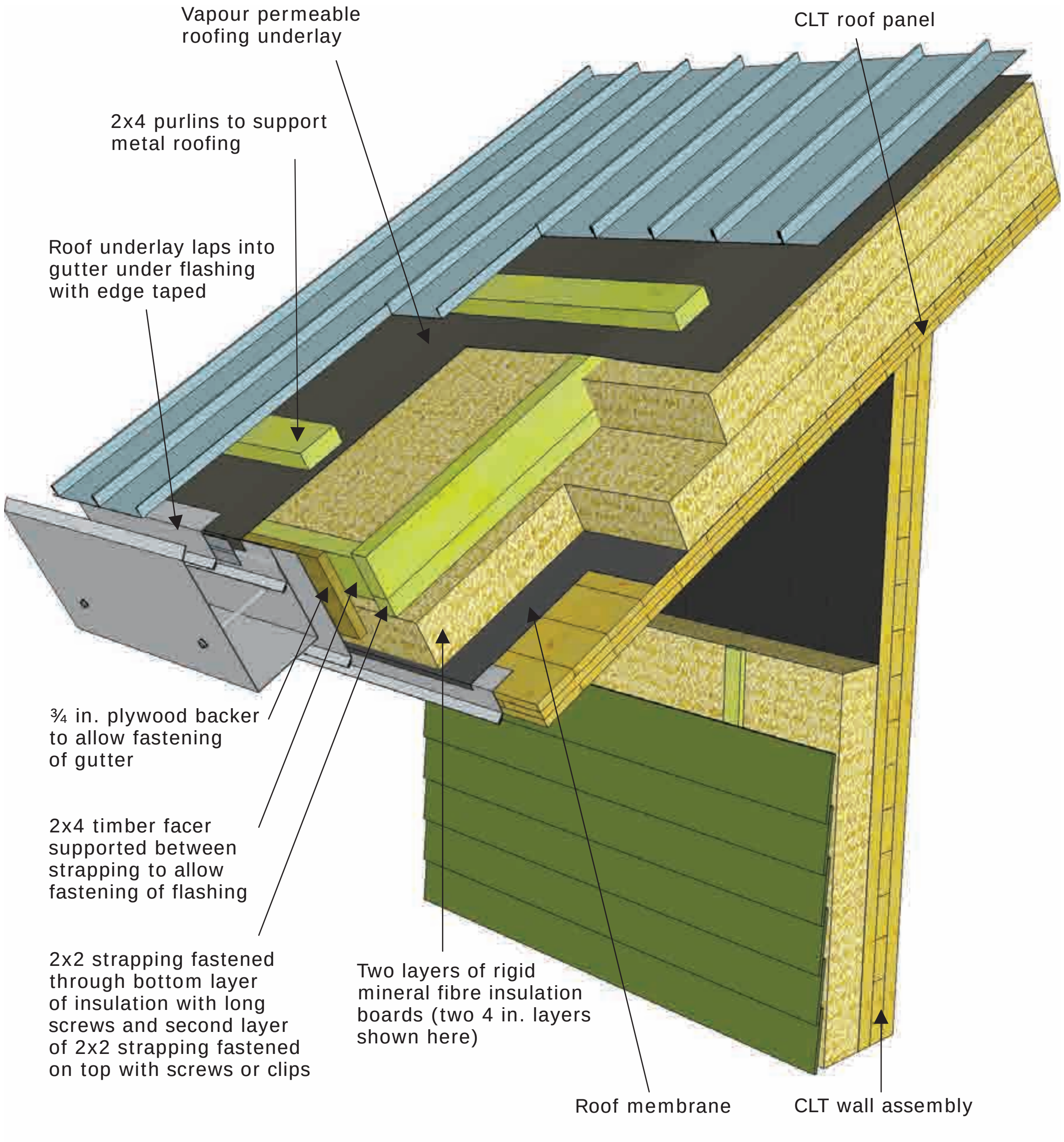

Figure 7

Top view of exterior insulated rainscreen clad CLT exterior wall transition to CLT sloped roof assembly showing material sequencing

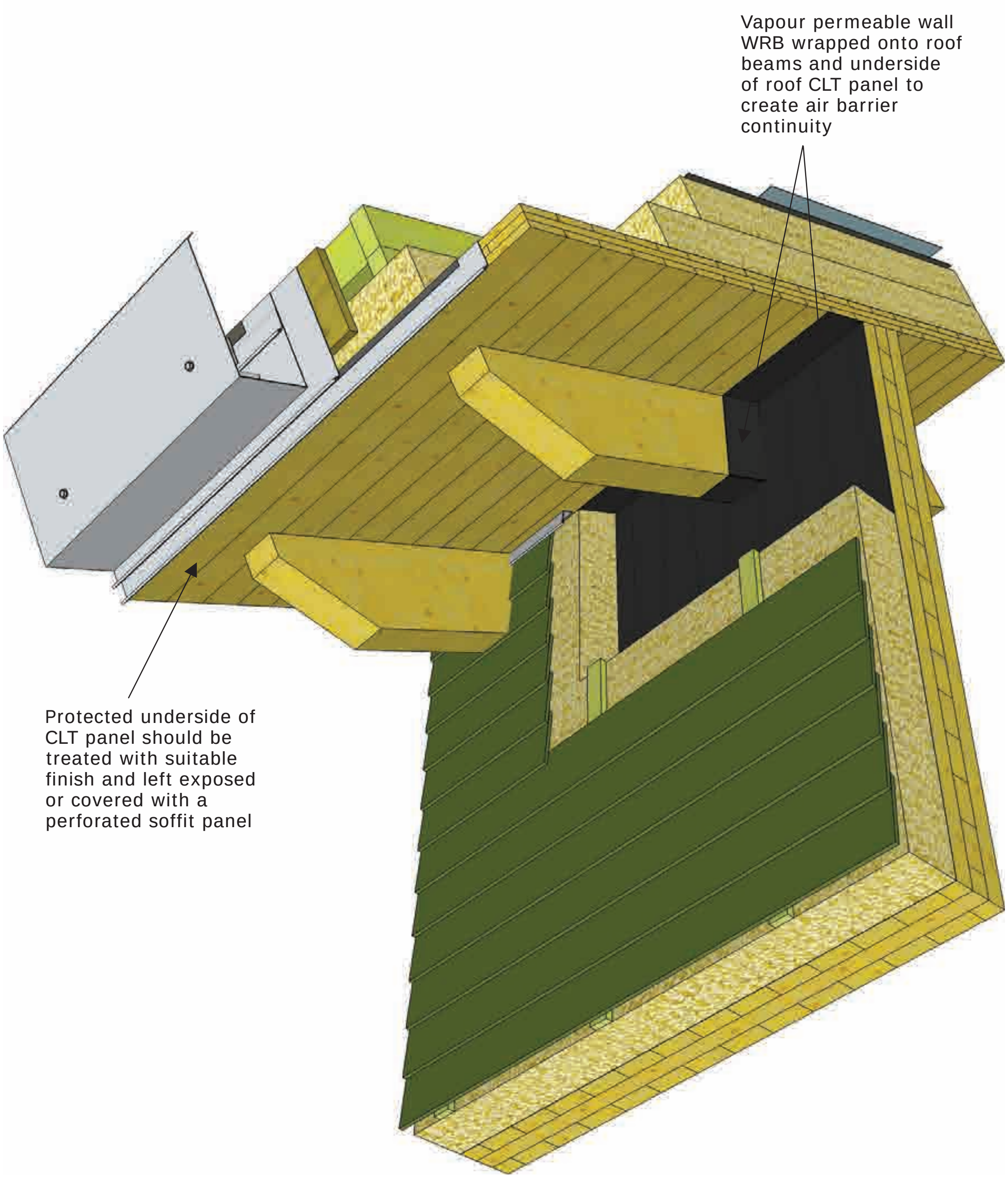

Figure 8
Bottom view of exterior insulated rainscreen clad CLT exterior wall transition to CLT sloped roof assembly showing material sequencing

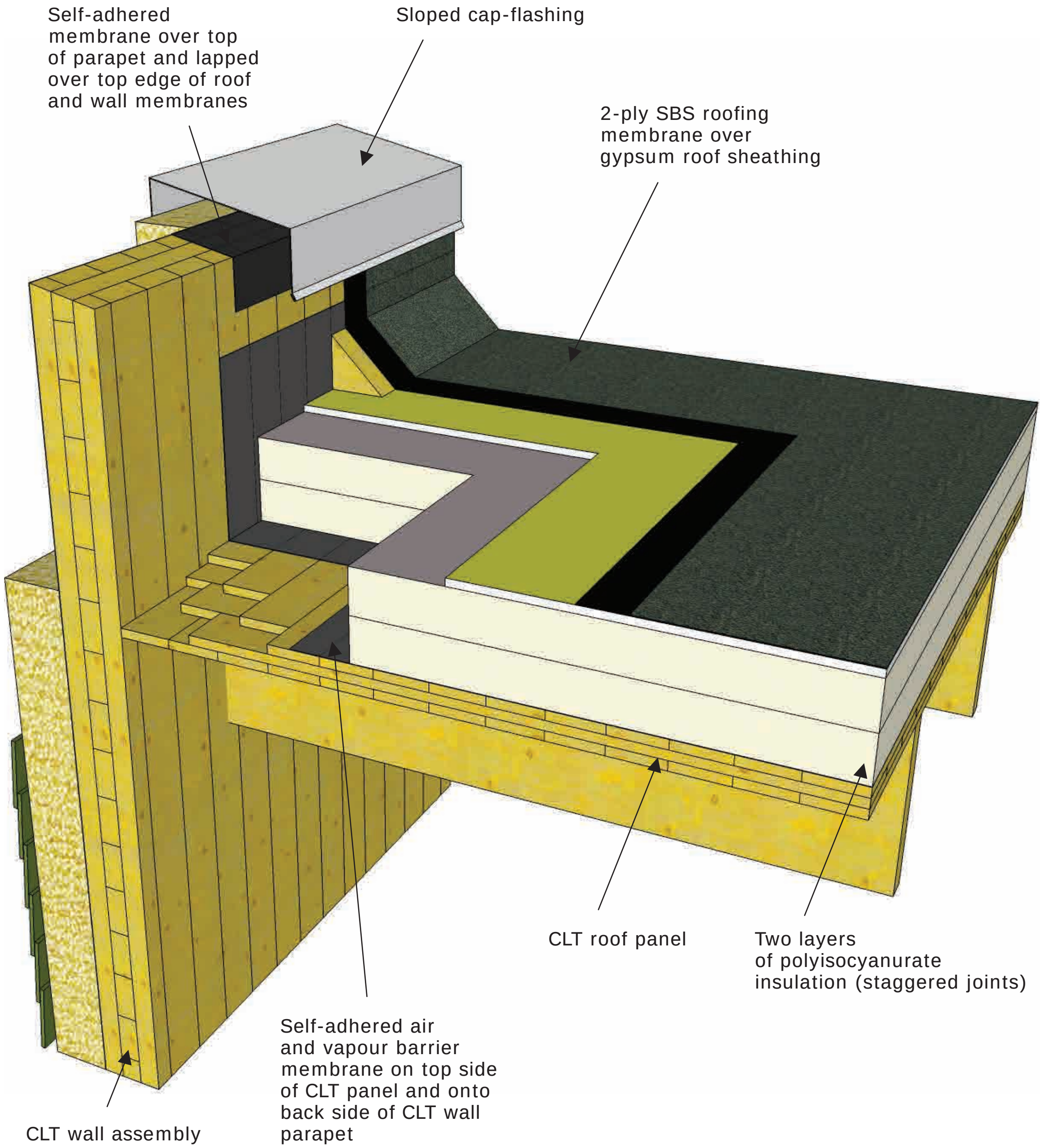

Figure 9

CLT flat roof detail showing material sequencing of a conventional roofing assembly with tie-in to parapet of CLT wall

3.3 Detailing Considerations: Installation of Windows

The installation of windows into an exterior insulated CLT panel wall assembly varies from traditional practice. When installing a window into an exterior insulated assembly, several window installation techniques are possible depending on the placement of the window frame. One such method which allows for the greatest flexibility is to support the window frame using a plywood box liner constructed within the window rough opening.

A general schematic of a window installation is provided in Figure 10. Continuity of the air barrier and water shedding surface are critical and can be detailed in a number of different ways. Key points to consider when detailing include:

- Air barrier continuity must be retained from the WRB at the CLT surface, through the rough opening and to the window frame.
- The membranes used at the window sill should preferably be vapour permeable to prevent water from being trapped within the CLT panel or plywood.
- Water should not be drained behind the insulation/WRB interface below a window or other penetration. Water should be drained to the exterior of the insulation or directly to the exterior where possible.

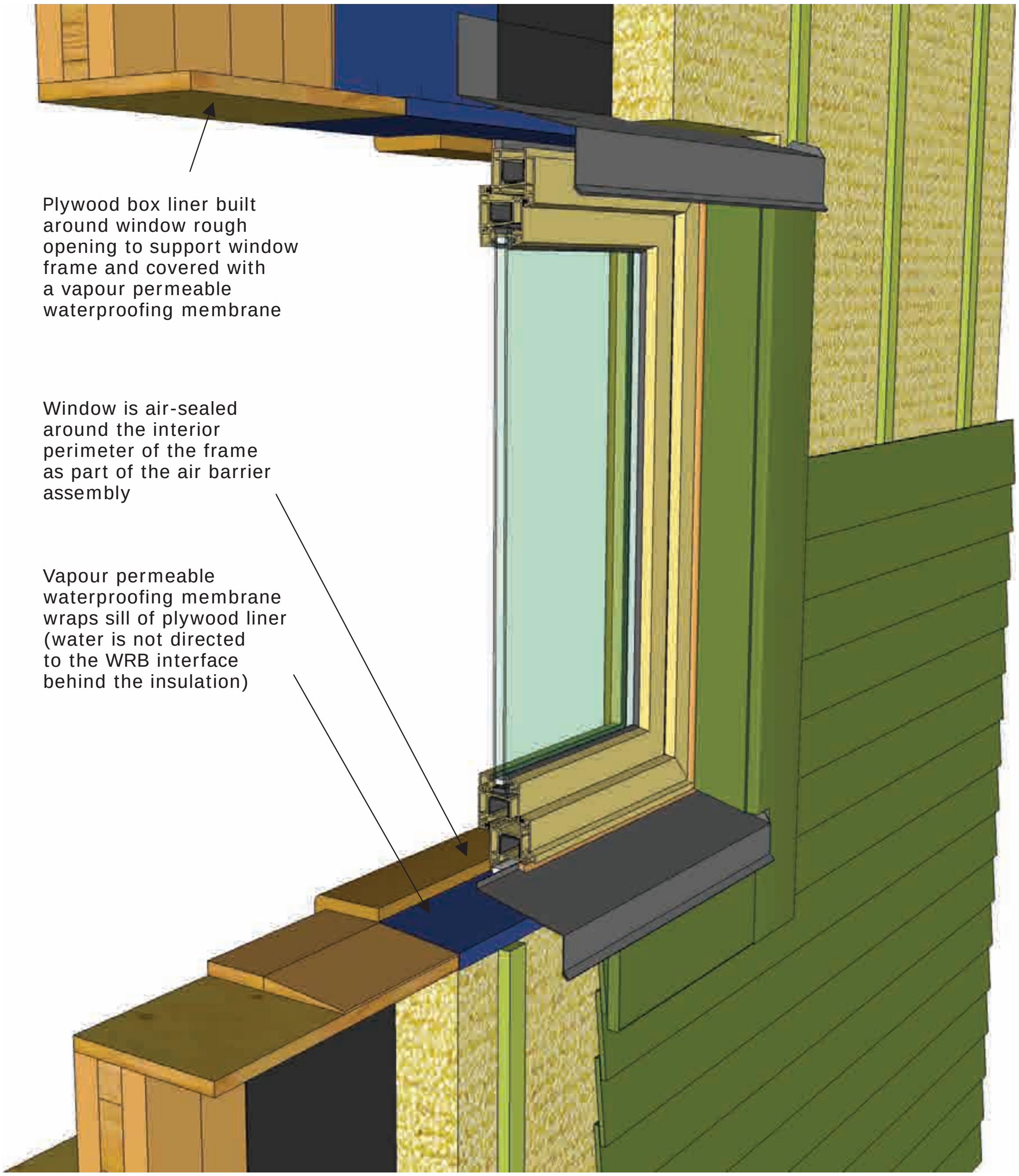

Figure 10

Window installation schematic using sloped wood sill and plywood box liner

3.4 Detailing Considerations: At Grade

CLT panels must be protected from moisture at grade. Typical wood-frame construction best practice regarding clearance between grade and wood should be followed and a minimum of 8 in. (200 mm) between the bottom of the CLT panel and grade should be maintained. The CLT panel should also be separated from the concrete using the foundation waterproofing and likely plastic shims to level the panels off at the top surface of the concrete wall. A sill gasket would be used if the CLT panels were in contact with the concrete.

The exterior insulated above grade CLT details easily into an exterior insulated below grade basement wall. Flashing is provided at the base of the above grade CLT wall which can be profiled to cover the below grade insulation. This insulation (typically extruded polystyrene, XPS) is placed on the exterior of the concrete and should be placed tight to the underside of the flashing. Since this can provide hidden access for termites, the XPS insulation should be borate treated or the flashing should be installed in such a manner to act as a termite shield where this hazard exists. Other termite management measures may be required by local building codes as discussed below.

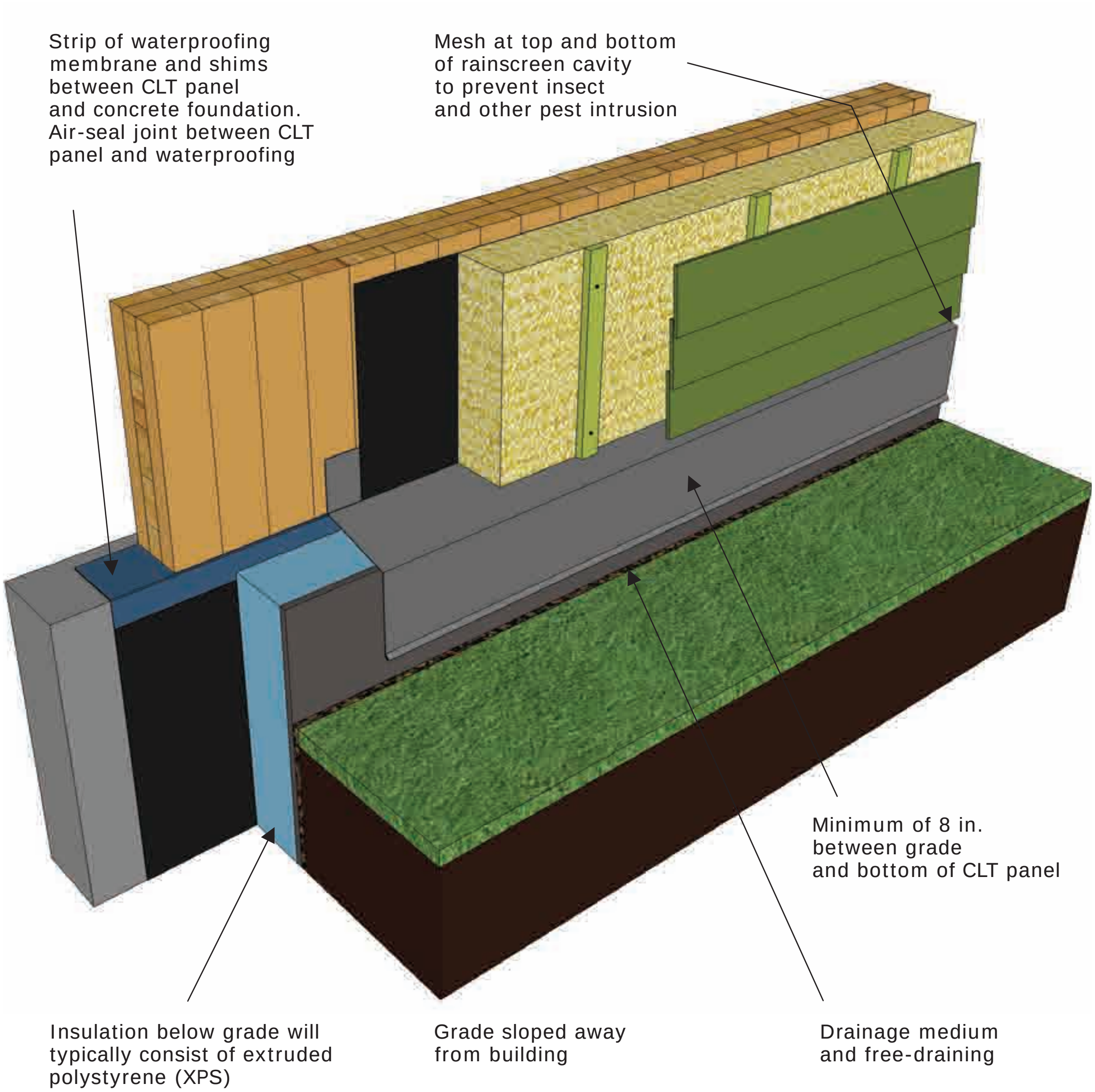

Figure 11
At-grade CLT wall assembly schematic

4

CONTROL OF MOISTURE DURING CONSTRUCTION

In Europe, a lot of attention has been paid to protecting CLT from getting wet during construction by delivering the product just in time, minimizing construction time, and providing temporary shelters during construction. CLT panels, similar to other wood products, should always be protected from exposure to rain, snow and the wet ground during the construction process. CLT panels are especially vulnerable to damage from wetting due to the nature of their laminated construction and because they are capable of absorbing large quantities of water through the faces, exposed ends and gaps between the panel laminations.

CLT panels are much more massive than plywood or standard dimension lumber, and will take a very long time to dry out if allowed to become wet. Therefore, prevention of wetting should be a priority in construction. Product standards may require a certain moisture content for finished CLT panels and building codes may require a moisture content of less than 19% at any location within a panel (surface, core or edge) before it is closed in. In addition, it is important to keep the panels at a stable operating moisture content, because moisture related expansion and contraction may damage the laminations and lead to distortion of the panels.

CLT panels should be temporarily protected by use of water-resistant sheet membranes or other effective methods to reduce environmental moisture uptake until they are protected by the building roof. Temporary protection can be attached in the manufacturing facility and should be maintained while stored on site. This protection should also be maintained as the panels are erected in place in order to protect the panels until the roof or other elements such as the sheathing membrane (WRB) provide adequate protection. If the protection is an impermeable material, it will need to be removed during construction.

Even with these precautions, it is likely that CLT panels will experience some wetting during transportation or construction, and be installed with built-in moisture in localized areas. Therefore, the most durable wall design strategies will keep the CLT panel warm (i.e. exterior insulated) and allow for excess moisture to escape fast enough (i.e. vapour open concept) from the assembly to prevent damage and deterioration.

5 WOOD PRESERVATIVE TREATMENT FOR DURABILITY

CLT panels (especially any exposed portions of the panels and parts in contact with foundations) would benefit from wood preservative treatment such as borate or copper-based preservatives, particularly in wetter or more humid climates or where termites are prevalent. While best practice construction and design strategies attempt to minimize exposure of the wood panels to wetting, inevitably some CLT panels will be exposed to moisture during their lifetime and the additional factor of safety provided by wood preservatives can be beneficial to the durability of the buildings.

In terms of treatment, preservatives used for treatment of lamina prior to manufacture of glulam posts and beams can generally be applied to CLT wall panels. Oil-based treatments used for industrial glulam may not be a preferred approach due to VOC emissions. Using pressure treated lumber for boards of the exterior lamination, applying post-lamination surface treatments to the exterior and end grains, or using boron rods for local protection may all help.

In areas with a high termite hazard, such as the Southeastern United States, multiple lines of defense should be used to prevent termite damage to CLT panels. Appropriate site termite prevention and the use of termite soil barriers such as termiticide soil treatment, and slab and foundation detailing to prevent termite intrusion should be taken into consideration during design. Preservative treated wood is also recommended for CLT panels and other wood furring and framing to prevent termite damage. In addition, termite control measures should also be provided to below grade insulation materials such as XPS.

The use of fire retardants may help meet fire safety requirements and warrant the use of exposed CLT panels for aesthetic purposes. Some fire retardants contain boron and will also provide decay and termite resistance.

6 CONCLUSIONS

It is intended that guidelines included in this chapter should assist practitioners in adapting CLT construction to North American conditions and ensuring a long life for their buildings. However, these guidelines are not intended to substitute for the input of a professional building scientist. This may be required in some jurisdictions, such as Vancouver, BC and is recommended in all areas at least until such time as CLT construction becomes common practice.

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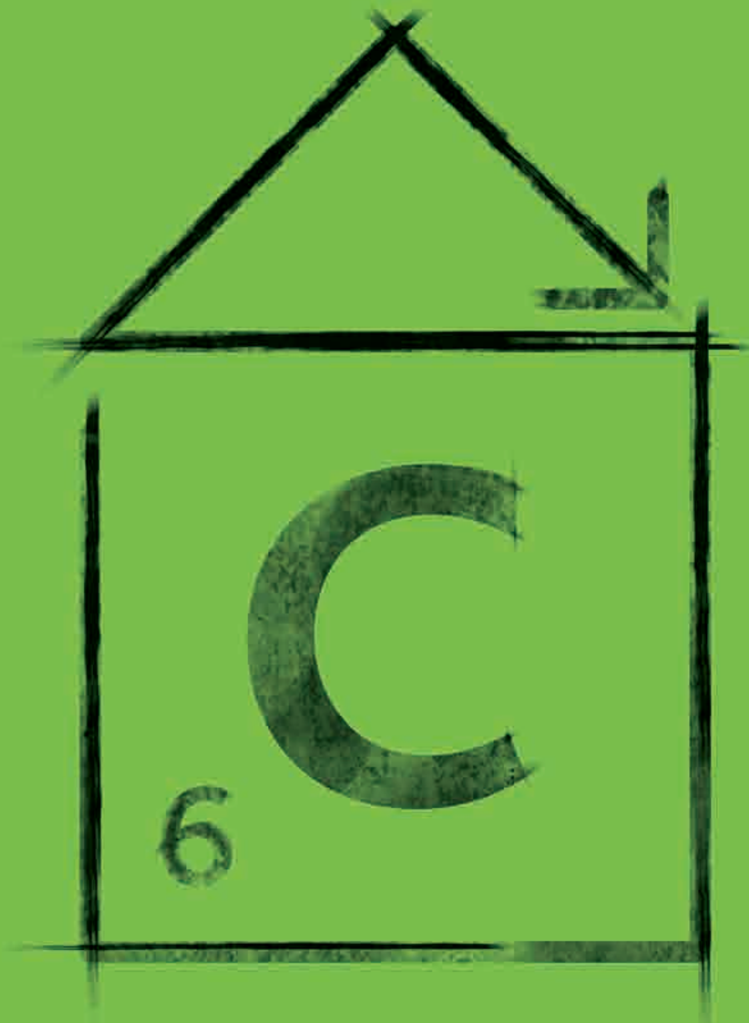
Environmental

Environmental performance of cross-laminated timber



CHAPTER 11

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ABSTRACT

Part 1 Environmental Footprint of CLT – Preliminary Findings

In this part, we approximately determine some quantified environmental characteristics of CLT as a construction material, without conducting a full life cycle assessment (LCA). Finding no existing comparative literature on CLT, we attempt several approaches to estimate the footprint of CLT and the comparison to concrete. Using existing LCA data on Canadian glulam as a proxy, we look at the footprint of the material itself compared to the materials in reinforced concrete, and at the material in a mid-rise building compared to concrete. We then modify glulam LCA data to approximate an LCA for a CLT floor section and compare it to a functionally equivalent concrete floor section. In all these cases, we estimate that the CLT version will substantially outperform concrete in every environmental metric addressed by LCA.

Part 2 Potential Indoor Air Quality Impact of Using CLT in Buildings – Preliminary Findings

Five cross-laminated timber products with different thicknesses and glue lines were tested for their volatile organic compounds (VOCs) including formaldehyde and acetaldehyde emissions in order to assist engineers and builders to better select their construction materials with less impact on indoor air quality. Emissions were evaluated according to ASTM D 5116 and were collected after 24 hours of samples exposure in the small chamber.

No correlation was observed between the cross-laminated timbers' thicknesses or glue lines and the amount of emitted individual VOCs (iVOCs), including formaldehyde and acetaldehyde or total VOCs (TVOCs). All five CLT products showed very low levels of iVOC and TVOC emissions; most of the detected VOCs consisted of terpene compounds originating from the softwood furnish used to manufacture the laminated timber products. Thus, their impact on indoor quality when CLT is used for construction will be very minor, if any.

In terms of evaluating the product's impact on indoor air quality, one can easily conclude that it would be negligible, if any. The five cross-laminated timber products' TVOCs and formaldehyde 24-hour results were generally lower than those set forth by some European emissions' labelling systems. Also, the European E1 grade for wood products' formaldehyde emissions set at 0.1 parts per million (ppm) or 100 parts per billion (ppb) is 6 to 20 times higher than those measured from the cross-laminated timber products.

By July 2012, the CARB (California Government standards) Phase 2 enforcement for all composite products will be completed and formaldehyde emission limits will vary from 0.13 ppm (130 ppb) for thin MDF (medium density fibreboard) to 0.05 ppm (50 ppb) for hardwood plywood with composite core (HWPW-CC). Comparing these limits to those from the cross-laminated timber products, one can conclude that these products easily meet the most stringent CARB limit of 50 ppb.

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1 ENVIRONMENTAL FOOTPRINT OF CLT – PRELIMINARY FINDINGS

1.1 Introduction

The environmental footprint of CLT is frequently discussed as potentially beneficial when compared to functionally equivalent concrete systems. However, to the best of our knowledge, there is no published information that credibly compares and quantifies the relative environmental performance of CLT versus other structural systems. In the work reported here, we try several approaches to approximate a quantification of CLT's environmental footprint without undertaking a full life cycle assessment (LCA) study, which is beyond the scope of the work.

There are many existing environmental comparisons between wood and other building materials, and the results are generally quite favourable to wood. However, these studies focus on light wood-framing using lumber, or post and beam using glulam, neither of which is at all similar to a CLT system. Differences include a mass wall or slab approach rather than a framed system, at least three times more wood material, and added processing and auxiliary materials such as adhesives. In other words, the footprint of a CLT building is not the same as a light-frame building, and we therefore cannot assume CLT will compare as favourably to concrete as previous LCA studies have shown for traditional wood systems.

1.2 Objective, Method and Limitations

The objective of this work was to approximately determine some quantified environmental characteristics of CLT as a construction material, without conducting a full LCA. Life cycle assessment is a rigorous, scientific analytical procedure, guided by international standards, to measure the flows to and from nature associated with a product or process over its full life cycle, and to interpret those flows in terms of environmental impact. LCA involves measuring flows at manufacturing facilities. At the time of writing this chapter, there were currently no commercial CLT manufacturing operations in North America. We could conduct such a study for European facilities and then modify the data to North American conditions, however, project resources would not allow for such an extensive piece of work. Our alternate method included a search for existing data and then hypothetical LCA-type exercises using data on Canadian glulam as a proxy for CLT. Note that glulam only partly satisfies as a proxy. While the manufacturing process may be similar, the construction method, design details and transportation issues are not; these factors are not included in this approach. Note as well that the glulam data used here are somewhat outdated. For these reasons, caution should be exercised in applying the theoretical information in this report.

1.3 Literature Review

We were unable to locate any existing LCA data or credible comparative studies addressing CLT. Several promotional pieces on the mid-rise Stadthaus building in England make comparative assertions about CLT, but these lacked support literature, clarity and methodological accuracy.

Gustavsson et al. (2010) performed a full life cycle assessment of energy use and greenhouse gas emissions for a CLT mid-rise building in Sweden (part of the Limnologen project). Energy use and carbon flows are tracked along the entire chain and include carbon stocks in building products and avoided fossil fuel combustion emissions where biofuel residues are used as a substitute energy source for fossil fuel. This study is not comparative; therefore, it does not tell us about environmental benefits over a concrete alternative. However, it is the only published study addressing whole-building, full life cycle environmental footprinting for a CLT construction application. The authors argue that a major carbon benefit for this wood-intensive building is the side effect of using wood residues as an energy substitute for fossil fuel. The biofuel can be collected in the form of harvesting residues, wood manufacturing residues, and—eventually—the CLT panels themselves at the end of their useful life.

Robertson (2010) conducted a comparative LCA study on a five-storey office building made of concrete versus a CLT and glulam hybrid building. This study will be published as a master's thesis in a near future, and will include details on the development of a life cycle inventory from primary data gathered at a CLT pilot plant in British Columbia. Results indicate a lower environmental impact for the glulam/CLT building over the concrete building in nine out of eleven environmental indicators.

A mid-rise LCA study by John et al. (2009) could theoretically provide a comparative basis for examining the CLT results in the Swedish study. This New Zealand study performed full LCA for four different structural approaches to a six-storey office building (concrete, steel, and two different wood versions). While results from the New Zealand study are not directly comparable to those of the Swedish study, we can potentially draw general conclusions about the likely comparative results for CLT. It is useful to look at the two versions of wood buildings in the New Zealand study. One used a fairly conventional quantity of structural wood while the other (called "timber plus" by the authors) increased the use of wood in that model by assuming wood substitution for additional products such as windows, ceilings and exterior cladding. The study found that total life cycle energy consumption and carbon footprint both decrease as the use of wood increases. A similar examination was performed by Meil et al. (2006) with similar results. In both studies, the reason for this benefit is the substitution of wood for non-wood materials that have a heavier energy/greenhouse gas footprint.

In the New Zealand study, various end-of-life scenarios were examined and operating energy was included; these are two important factors to consider when properly comparing wood to other materials in construction. In this study, thermal mass in the buildings was accounted for in the energy modeling, and the concrete building had the lowest operating energy consumption. However, this was overtaken by the embodied energy savings of the "timber plus" version over the concrete version due to product substitution. Taking the end-of-life study in landfill, the authors also contend that a significant portion of the carbon contained in the wood materials is stored permanently, giving both wood versions of the building lower total life cycle carbon footprints than the steel and concrete versions. The "timber plus" version has a substantially lower total carbon footprint than the other wood version due to embodied energy savings in product substitution, lower operating energy consumption due to thermal mass, and a greater mass of wood carbon in permanent landfill storage.

From this study, we can perhaps extrapolate some conclusions about the likely comparative performance of CLT. If we assume that CLT has a smaller manufacturing carbon footprint than concrete and that all other life cycle factors are similar to the "timber plus" model, it would follow that a CLT version would perform similarly or perhaps better than the "timber plus" model, given that it would have more wood mass available for permanent landfill storage at end of life.

1.4 Environmental Snapshot – CLT versus Concrete

We can broadly test our hypothesis that CLT has a smaller environmental footprint than concrete by accessing existing Canadian LCA data on building materials, and using glulam as a direct proxy for CLT, under the assumption that the manufacturing of CLT is very similar to glulam. The *Impact Estimator for Buildings* software and databases from the Athena Institute are tools for examining LCA information in a construction context and were used for this work. We created two different "snapshots": the basic materials on their own, and an approximation of a mid-rise building comparison using CLT and concrete.

1.4.1 Material Snapshot

Promotional literature on the UK building (Stadthaus) states that, according to architects' estimates, an equivalent building using reinforced concrete would consume 950 cubic meters of concrete and 120 metric tons of steel reinforcement, in place of approximately 910 cubic meters of CLT. Using the *Impact Estimator* and substituting glulam for CLT, we can roughly estimate the cradle-to-grave footprint for just the volume of the fundamental structural materials in these two versions of the building. This is a partial LCA; while it includes all life phases, it also omits many components that might alter comparative results (Figure 1). Note that CLT results are set as the benchmark (100%), with concrete shown in terms of performance relative to CLT. In all categories, the CLT materials have less environmental impact than a functionally equivalent amount of concrete (material amounts determined by the architects according to promotional literature).

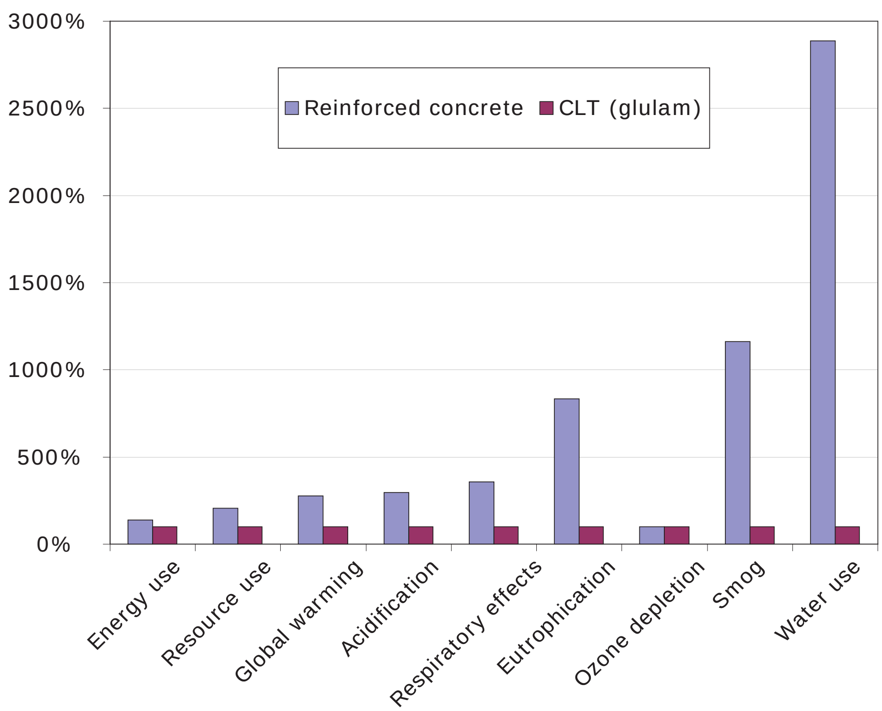

Figure 1
Functionally equivalent CLT materials versus reinforced concrete, LCA results, benchmarked to CLT

1.4.2 Mid-rise Snapshot

A more accurate approach to material comparison involves a whole-building study. Using the *Impact Estimator* and again using glulam as a proxy for CLT, we can roughly compare reinforced concrete to CLT (glulam) in the context of a mid-rise building.

In this calculation, the *Impact Estimator* (IE) built-in algorithms for building design are employed to model the two mid-rise CLT buildings for which we know wood material quantities: the UK Stadthaus and the Sweden Limnologen. A CLT version is developed based on a glulam column-beam-joist system with additional glulam added to bring the total quantity of wood up to the same amount in the real buildings; this is a proxy method, as the IE does not have a built-in "massive wood" structural system. Fasteners are included for the glulam quantities determined by the software but not for the additional glulam added to simulate the total mass of wood in the actual building. No other materials are included–they are deemed in this comparison to be equivalent for both the CLT and concrete versions. Construction, maintenance and disposal effects are included for the software-determined portion of wood, but not for the added portion. Because of those missing components for the CLT buildings, the environmental footprint may be somewhat underestimated compared to the concrete buildings. For the concrete versions, the same area, height and assumptions about bay sizes and structural spans were input to the IE, this time specifying a concrete slab-column-beam system; the IE determined material quantities including all fasteners. Additional materials were added to simulate exterior precast concrete wall panels, in order to properly compare to the CLT version (where the added glulam simulates the solid wood exterior walls among other components). Interior walls were not included in the concrete model, whereas they are largely included in the CLT model due to the added glulam materials; this omission may lead to an underestimate of the concrete building footprint.

For this rough whole-building snapshot, results from the two mid-rise buildings were combined in an area-weighted average. Results are shown in Figure 2 and are similar to the previous figure.

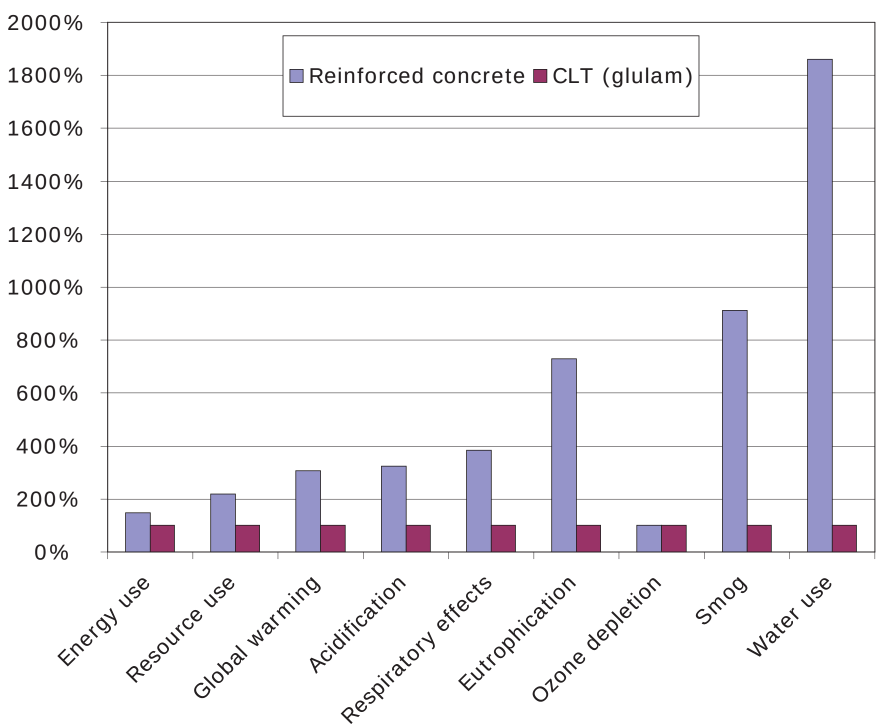

Figure 2
Rough LCA results, mid-rise comparison, CLT versus concrete, benchmarked to CLT

1.4.3 Greenhouse Gas Displacement

Displacement factors are a measure of the efficiency of wood materials in avoiding GHG emissions. Wood generally has lower embodied GHG emissions than functionally equivalent alternate materials; thus, when wood is used in place of other materials, GHG emissions are typically avoided. This is a permanent and cumulative benefit for climate change mitigation. A displacement factor specific to CLT could be generically applied to a given volume of CLT in a construction application, for an estimate of GHG benefit in using CLT, assuming the said application was typically built with a different material. Sathre and O'Connor (2010) discuss displacement factors at length and offer an average displacement factor for wood substitution, in units of mass of avoided CO_2 emissions per unit of mass of wood used. Their average number of 3.9 metric tons of avoided CO_2 emissions per metric ton of oven-dry wood is likely not applicable to CLT without modification, because it is based on traditional uses of wood. CLT systems use more wood while still displacing the same GHG emissions as a substitute material (compared to traditional wood systems such as light-frame), therefore it is less "efficient" at offsetting GHG emissions. Muter and O'Connor (2009) estimated an adjusted displacement factor of 0.66 metric tons of avoided CO_2 emissions per metric ton of oven-dry wood in CLT used in place of other materials. Using data from the above-mentioned snapshot LCA study on two 8-storey buildings, we can also calculate displacement factors, which establish at 0.81 and 0.65 metric tons of avoided CO_2 emissions per metric ton of oven-dry wood in CLT, for the Limnologen and Stadthaus buildings respectively. Until enough LCA data is available for a refinement of these estimates, we can perhaps use an average (0.71) of the above three numbers.

1.4.4 Carbon Storage

Entirely separate from the GHG avoided emissions due to wood substitution, the issue of carbon storage in wood can also be addressed when describing the environmental profile of wood construction elements. Wood is about half carbon, and wood in long-term service such as construction represents a significant pool for GHG. Over the long term, this carbon will return to the atmosphere and complete the natural carbon cycle. But the temporary GHG storage in wood products can be reasonably taken as a carbon credit, depending on time frame and end-of-life assumptions. If the time frame under consideration is short, perhaps 100 years or less, then the carbon in wood in long-term service is often deemed as a permanent removal of GHG from the atmosphere. This reasoning largely stems from the current urgency around climate change; a 100-year or so delay in carbon emission is helpful in current mitigation actions to reduce GHG emissions. Over a longer time frame, issues regarding landfill decomposition and potential release of methane become important. If the wood is burned at end of life for energy recovery to replace fossil fuel, the avoided GHG emissions from fossil fuel are included in the assessment.

For traditional wood structural systems, the carbon mass of wood is relatively small compared to the carbon emissions avoided by using wood instead of steel or concrete. Therefore, an important focus in the use of wood to combat climate change is to increase the rate of wood substitution for other materials, with less emphasis on carbon storage. With CLT, the relationship is the opposite: the carbon mass of wood is quite large compared to the avoided emissions of alternate materials. In this case, there would be an interest in putting a value on that stored carbon, with a motivation to keep the carbon in service for as long as possible, and to capture the energy value of that carbon to replace fossil fuel at the end of service life.

1.5 Preliminary LCA – CLT Floor versus Concrete Floor

We next developed a theoretical life cycle inventory for CLT manufactured in BC, using existing Canadian glulam life cycle inventory data with modifications, and we used LCA to compare CLT to concrete as a structural floor system. We examined various potential factory locations in BC and included transportation of the final product to a building site in Vancouver, in order to determine which manufacturing location would have the least environmental burden.

1.5.1 Study Method and CLT Results

Modifications to the existing glulam data included adjustments to wood and adhesive volumes based on in-house calculations. Floor sections were determined through in-house engineering. The approach followed international standards as set forth in the ISO 14040 series. Figure 3 illustrates the methodology for ISO 14040.
Impact assessment methods were a combination of the Tool for the Reduction and Assessment of Chemical and Other Environmental Impacts (TRACI) from the U.S. EPA and the Cumulative Energy Demand (CED) from EcoInvent.

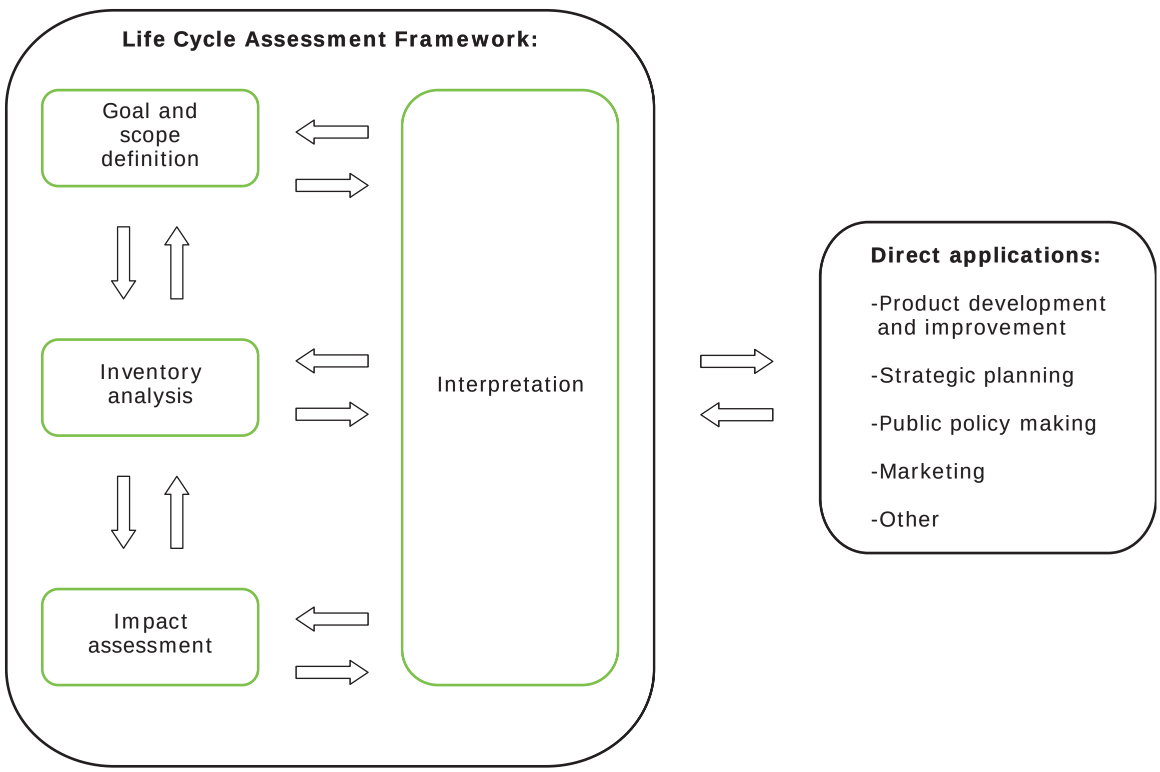

Figure 3
Life cycle assessment methodology: the ISO 14040 framework and applications

The life cycle inventory for CLT was based on a completely hypothetical production situation as CLT is currently not commercially manufactured in North America. We performed the analysis for three alternative locations of a potential CLT factory in BC: Vancouver, Kamloops, and Prince George. The study took into account timber extraction from forests, transport of logs and other raw material to mill gate, and energy and raw material consumption for the manufacturing of CLT. In addition, it included delivery of CLT to a building site for installation. The study boundary for concrete was the same.

Energy recovery from wood waste generated during CLT manufacturing, and substitution effects of this energy for natural gas were taken into account in the analysis as well as the forest carbon uptake that subsequently resides as stored carbon in the wood. These two effects give CLT a negative cradle-to-gate carbon footprint; as with most wood products, there is more carbon stored or offset by the wood than is emitted during its cradle-to-gate manufacturing.

CLT manufacturing is a multiple output process as it generates wood waste in addition to the primary product, CLT. This waste can be considered as a co-product as it is intended to be used for energy recovery. However, compared to CLT, the revenue generated from wood waste could be relatively minor and, therefore, environmental flows are not allocated to this co-product. To be conservative, the LCI is attributed entirely to the primary product.

There were no significant differences in CLT footprint as a function of factory location.

The next two figures show the system boundary of CLT and ready-mixed concrete, respectively.

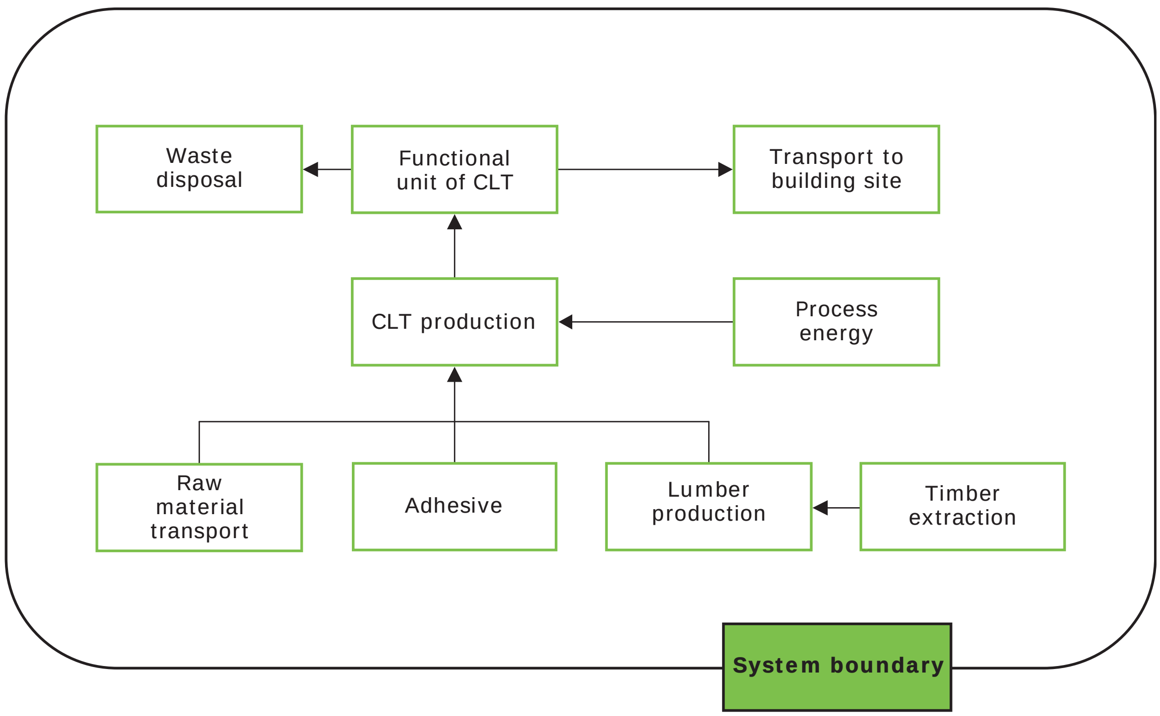

Figure 4
System boundary of CLT

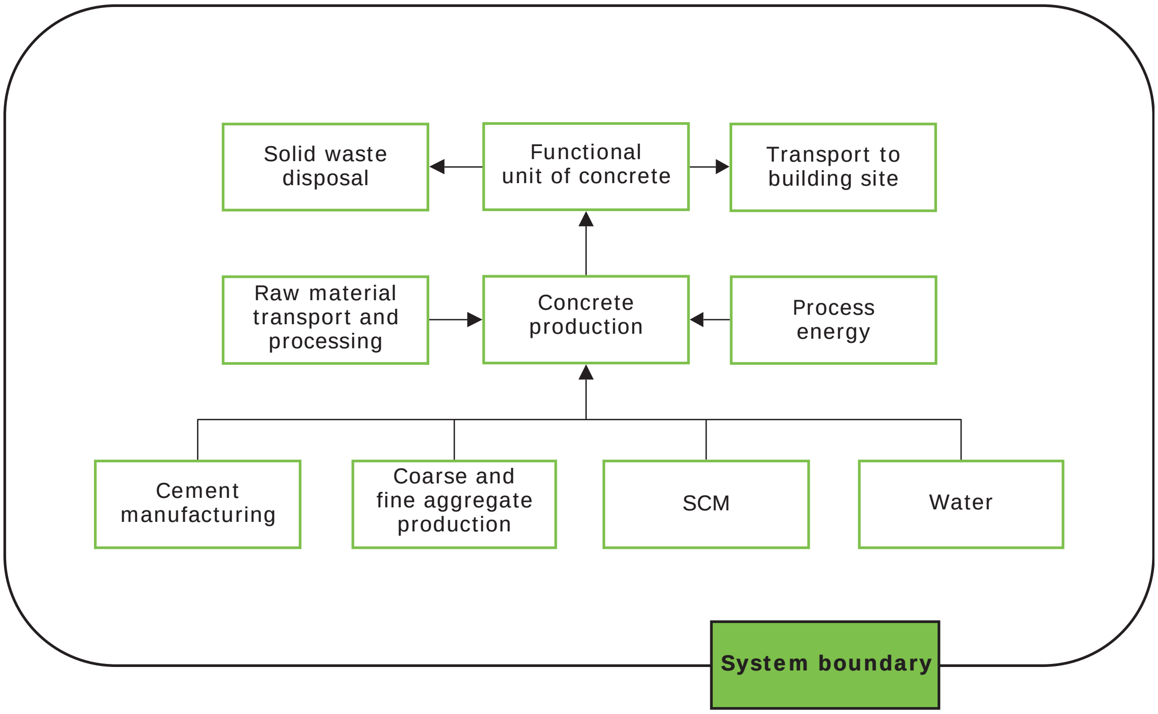

Figure 5
System boundary of ready-mixed concrete

1.5.2 Functional Unit for Comparison

All environmental inputs and outputs within the system boundary are normalized to a unit summarizing the function of the system that allows comparison of various product systems performing a similar function. For example, both CLT and concrete perform a similar function if they are used to construct a building floor. With the function of the system defined, a functional unit or reference flow has to be selected in order to provide a similar basis for the comparison of these two building materials. For this reason, the functional unit was considered to be one square metre of floor (slab) area for both CLT and concrete. However, the functionally equivalent thicknesses of these materials may not be similar due to engineering requirements. FPInnovations' in-house structural engineering staff advised that functionally equivalent thicknesses of floors can range from the same thickness for both CLT and concrete to a significantly thicker section for CLT. We chose to analyze a highly conservative situation of 200 mm and 120 mm (30 MPa) for CLT and concrete respectively.

1.5.3 Results of Floor Comparison – CLT versus Concrete

Summary results of the comparative assertion of CLT versus concrete are shown in Table 1 and Figure 6 on an absolute value and percent basis respectively. The CLT floor performs better.

Table 1
Comparative LCA results for CLT and concrete produced and used in Vancouver – absolute values

Impact Category	Unit	CLT 1 m^2 of Floor	Concrete 1 m^2 of Floor
Global warming	kg CO_2 eq.	-222.55*	90.12
Acidification	H+ moles eq.	8.77	23.00
Respiratory effects	kg PM2.5 eq.	0.010	0.058
Eutrophication	kg N eq.	0.014	0.115
Ozone depletion	kg CFC-11 eq.	7.15E-09	2.65E-07
Smog	kg NOx eq.	0.21	0.23
Non-renewable fossil fuel	MJ eq.	274.30	633.54

Note: *Net emissions, when taking into account forest carbon sequestration (248 kg CO_2 eq.) and reduction in carbon emissions from substituting wood residues for natural gas (21.8 kg CO_2 eq.).

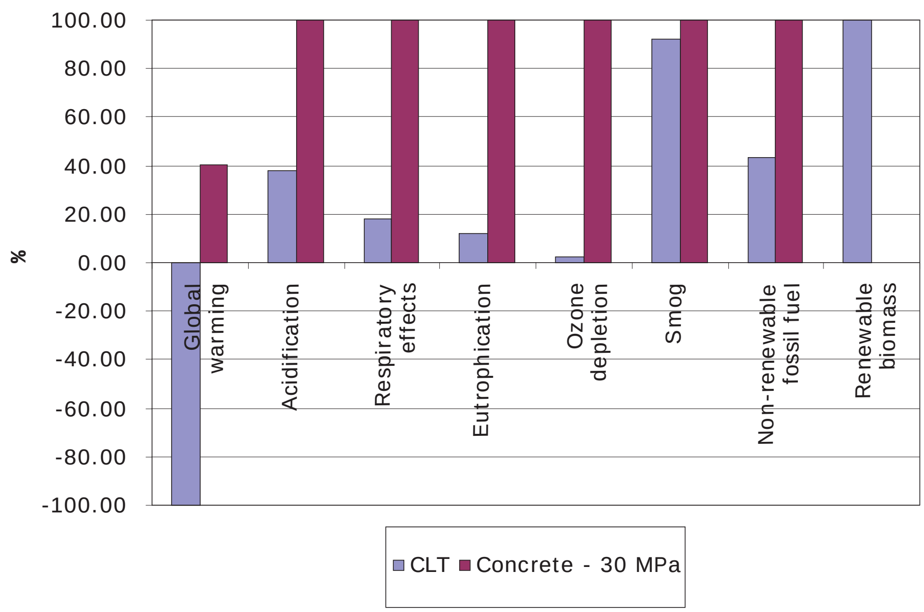

Figure 6
Comparative LCA between 1 square meter of CLT and concrete floor structure

Note: This Figure graphs the data shown in Table 1 on a percentage basis, with the baseline set at the highest number in each environmental performance category. For example, in fossil fuel consumption, the concrete system had the highest number and was set to 100%, with the CLT number shown as 43% of the fossil fuel use of concrete.

1.5.4 Sensitivity Analysis

A sensitivity analysis was performed to determine the effect of long-distance CLT transportation on the comparative LCA results against a concrete floor structure. We compared the preparation of a concrete floor structure using local materials to CLT manufactured in Vancouver and transported 5000 km, to simulate a construction site in the Eastern USA. Both road and rail transport modes were considered as separate scenarios. The same concrete manufacturing profile developed for Vancouver was used for the comparison, but with modifications to reflect the Eastern USA energy grid.

A summary of the sensitivity analysis results are shown in Table 2. Results indicate that long-distance transportation significantly increases the environmental impacts of CLT leaving the mill gate. However, CLT still compares fairly well with locally manufactured concrete provided that rail is the primary transport mode.

Table 2

LCA, CLT versus concrete floor, various transportation scenarios

Impact Category	Unit	Concrete Produced in the USA	CLT Produced in Vancouver	CLT Transpoted to the USA via Road	CLT Transported to the USA via Rail
Global warming	kg CO_2 eq.	96.33	-222.55	-182.11	-217.81
Acidification	H+ moles eq.	25.72	8.77	24.28	19.95
Respiratory effects	kg PM2.5 eq.	0.068	0.010	0.028	0.022
Eutrophication	kg N eq.	0.114	0.014	0.043	0.040
Ozone depletion	kg CFC-11 eq.	2.63041E-07	7.15E-09	4.97E-06	4.97E-06
Smog	kg NOx eq.	0.26	0.21	0.53	0.47
Non-renewable fossil fuel	MJ eq.	719.98	274.30	891.31	406.71

Note: Impact indicators were calculated per functional unit of both products

1.6 Green Market Potential for CLT

In green programs such as LEED®, there are currently no incentives that would particularly encourage the use of CLT on environmental merits. In fact, CLT would likely first experience an environmental backlash on the basis of material volume. A strong mandate in the sustainable construction world is the minimization of resources. At first glance, and particularly with a North American light-frame perspective, "massive wood" systems appear highly wasteful of materials. There are increased market sensitivities in particular with regard to wasteful use of forest resources, which many feel are in need of preservation.

A marketing push around the environmental footprint of CLT would first need to direct attention away from a natural North American comparison to light-frame and towards a comparison to concrete slab systems exclusively. At that point, possible opportunities for CLT in the green market may revolve around health, comfort and well-being benefits, if the wood is allowed to be exposed. If CLT incorporates post-consumer wood waste materials, it would certainly find a market. CLT also has excellent potential in terms of disassembly and re-use, which would have major value from a sustainable construction perspective as it addresses conservation of resources and would help overcome possible perceptions that CLT may be material wasteful. The effect of CLT on operating energy performance may be attractive and should be looked at in future work.

To realize green market potential based on the real environmental benefits of CLT as discussed in this report will require a market education process. Results discussed here show a likely strong performance against concrete in most environmental indicators, however bringing that data to construction decision makers is currently a challenge. This is due to the complexity of the data, challenges in weighting the importance of various environmental indicators, uncertainty in the results, lack of real LCA data on CLT systems, and general lack of public knowledge about LCA. Once designers are routinely using LCA to assess the environmental footprint of structural options, they may find CLT is a good choice over concrete. Note, however, that CLT may not perform well against other wood-frame options on an LCA basis. For a low-rise building that could be constructed with a light-frame or post-and-beam wood system, CLT would likely be a hard sell on environmental benefits. A better bet for capturing the green value of CLT would be the mid- and high-rise market, or low-rise buildings rarely constructed in wood such as industrial facilities, to avoid a comparison with other wood systems. In either case, the emergence of carbon trading options for wood systems could seriously tilt the market towards CLT. If a value is attached to the carbon stock represented by wood products in long-lived construction service, then CLT could become quite attractive compared to other materials, including lumber.

1.7 Conclusions

Establishing the environmental footprint of CLT as a substitute material for concrete requires significant further work. This would include development of a real life cycle inventory for Canadian-produced CLT and its use in structural systems, and then cradle-to-grave life cycle assessments of functionally equivalent CLT and concrete buildings. In the absence of this information, we can only hypothesize about the comparative environmental footprint of CLT as a structural substitute for concrete.

Our hypothesis, based on preliminary data, is that CLT likely has a lighter footprint than equivalent concrete systems in many cases and has potential value as a carbon storage mechanism and eventual biofuel at end of life. These two aspects of footprint are discussed separately.

As a concrete replacement, CLT would likely displace (avoid) a number of environmental emissions and other impacts such as consumption of natural resources. The lower consumption of fossil fuel and subsequent GHG emission reduction in the manufacturing of CLT versus reinforced concrete are particularly attractive benefits. However, CLT is a massive material with a corresponding high transportation environmental footprint; if transported long distances, CLT may have trouble competing on fossil fuel and GHG emissions with concrete, which is nearly always locally manufactured. This may not be an issue if regional CLT manufacturing develops.

We note as well that CLT may not be the best wood-frame option for some cases of concrete substitution, if GHG offset “efficiency” is important. For example, light-frame and post and beam wood structural systems may equally well be able to substitute for a concrete building as CLT. In that case, the same amount of GHG savings will result, but in a CLT version, a much greater mass of wood will be used. Instead of one CLT building replacing one concrete building, a more efficient strategy for climate change mitigation would be several light-frame buildings replacing several concrete buildings, using the same amount of wood as in the one CLT building. For building applications where light-frame is not an option, this caveat does not apply. Examples include mid- and high-rise buildings.

A separate discussion addresses the carbon storage aspect of CLT which, like most wood products, has more carbon contained in the product than is emitted during its harvesting, manufacturing and transporting. CLT can thus be considered carbon-negative in a cradle-to-gate context. Environmentally, this is a delayed GHG emission; the carbon storage is temporary (the carbon stored in wood will eventually return to atmosphere) and, therefore, over a long time frame, has no effect on global carbon balances. However, in the short term (100 years or so), any delayed emission is helpful for the immediate societal need to reduce emissions in order to slow down climate change. The significance of this role in the global carbon balance requires discussion. Generally, displaced emissions are more meaningful as they are larger than typically stored carbon, and they are permanent and cumulative. Stored carbon is only affecting climate change if the pool of carbon in wood products is increasing;

otherwise, this pool is in equilibrium with removals of wood carbon from service and with carbon absorption by forests. A rapid market uptake of CLT could theoretically help increase the pool of carbon in wood products. However, the consideration of carbon flows in forest ecosystems is essential to accurately understand the climate impacts of wood product use. Similarly, the boundary needs to include consideration of carbon dynamics in landfills, as well as predictions for likelihood that the CLT will be used as biofuel at end of life.

While it is likely that CLT has an attractive environmental profile compared to concrete, realizing market value from that benefit may prove difficult. Displaced emissions due to the use of CLT will only be visible to a decision maker if life cycle assessment methods are employed in the process of selecting materials. This is currently not standard practice in the construction sector, although this may change in the near future with the inclusion of LCA in two popular green building rating systems as well as in the emerging US green construction code. Stored carbon in construction materials is currently not valued at all; however, this too may change due to rapid policy activity worldwide. The green building world in North America may initially react with scepticism regarding the environmental credentials of CLT due to concerns about resource conservation and forest protection. These potential reactions should be anticipated in marketing campaigns for CLT.

1.8 References

Fell, D. 2010. *Wood in the human environment: Restorative properties of wood in the built indoor environment.* PhD diss., University of British Columbia. 134 p.

Gustavsson, L., A. Joelsson, and R. Sathre. 2010. Life cycle primary energy use and carbon emission of an eight-story wood-framed apartment building. *Energy and Buildings* 42 (2):230-242.

John, S., B. Nebel, N. Perez, and A. Buchanan. 2009. *Environmental impacts of multi-storey buildings using different construction materials*. Research Report 2008-02. Christchurch, New Zealand: University of Canterbury, Department of Civil and Natural Resources Engineering. 135 p.

McKeever, D. B., C. Adair, and J. O'Connor. 2006. *Wood products used in the construction of low-rise nonresidential buildings in the United States*, 2003. N.p.: Wood Products Council. 63 p.

Meil, J., M. Lucuik, J. O'Connor, and J. Dangerfield. 2006. A life cycle environmental and economic assessment of optimum value engineering in houses. *Forest Products Journal* 56 (9):19-25.

Muter, D., and J. O'Connor. 2009. Market and GHG assessment of new wood applications. Vancouver, BC: FPInnovations. 32 p.

Olutimayin, S. O., C. Simonson, J. O'Connor, M. Salonvaara, and T. Ojanen. 2005. Effect of moisture storage on indoor humidity and vapor boundary layer thickness. In *Proceedings of the 10th Canadian Conference on Building Science and Technology, May 12-13, Ottawa*, vol. 1, 194-203.

Robertson, A. 2010. Reinforced concrete or cross-laminated timber – a life cycle comparison of office building construction. Poster presented at CYCLE 2010: 4th Canadian Forum on the Life Cycle Management of Products and Services, May 4-5, Montréal.

Sathre, R., and J. O'Connor. 2010. Meta-analysis of greenhouse gas displacement factors of wood product substitution. *Environmental Science and Policy* 13 (2):104-114.

2

POTENTIAL INDOOR AIR QUALITY IMPACT OF USING CLT IN BUILDINGS – PRELIMINARY FINDINGS

2.1 Objectives

Preliminary potential indoor air quality impact of using CLT products for flooring or wall structures in residential or non-residential buildings was evaluated by FPInnovations.

2.2 Background

As regulatory and non-governmental organizations (NGOs) address indoor air quality issues, they tend to focus on volatile organic compounds (VOCs), including formaldehyde, as key factors relating to the discomfort reported by people working or living inside "air tight" buildings. This effect is known as the "Sick Building Syndrome". The World Health Organization (WHO) has defined VOCs as organic compounds with boiling points between 50°C and 260°C. Wood composite products are suspected of emitting some of these organic chemicals, namely formaldehyde, alpha- and beta-pinene, carene, camphene, limonene, aldehydes, ketones and acetic acid. Although VOC and formaldehyde emissions from unfinished and finished wood composite panels are very well documented, very little data exist on thick multiply products, if any.

2.3 Procedures and Results

All measurements were done in general agreement with specified standards and protocols. The precision levels were in accordance with the technical requirements.

2.3.1 Materials Sampling, Packaging, Transportation and Conditioning

Duplicate test samples of 280 mm x 760 mm (Figure 7) were cut 300 mm from each end of a 5.5 metre long original CLT panel. In order to avoid any potential contamination of samples, latex gloves were worn during the whole sampling and packaging processes; also, before cutting the samples, a towel was used to clean the saw blade. Samples were wrapped with plastic foil with no writing on the sample or on the packaging and stacked in a conditioned room (23±1°C and 50±5% RH) until ready for testing.

VOC and formaldehyde tests were performed from the same sample and at similar conditions, at a loading ratio of 0.44 m^2/m^3 with all edges sealed with a non-emitting aluminum tape material leaving two flat surfaces exposed.

Figure 7
Prepared sample with edges sealed ready to be put in the chamber

2.3.2 Method

A constant and adjustable airflow, conditioned for relative humidity, was fed through the small environmental chamber at a rate which corresponds to an air change rate of one per hour. The VOC sampling procedures excluding formaldehyde were similar to those described in the ASTM D 5116-97 and ANSI/BIFMA M 7.1-2007 standards. The chamber was in stainless steel and the interior surfaces were electropolished to minimize chemical adsorption. The chamber was equipped with suitable accessories such as inlet and outlet ports for airflow and an inlet port for temperature/humidity measurements. The air sampling was accomplished from the airflow outlet port. The small chamber was placed inside a controlled temperature room. The humidity of the air flowing through the chamber was controlled by adding deionized water to the air stream, as shown in Figure 8.

Figure 8
General view of the 1 m^3 environmental chamber used for emissions testing

The collection of VOCs on an appropriate adsorbent medium is required to avoid overloading of the analytical equipment. In order to maintain integrity of the airflow in the small chamber, the sampling flow rate was 100 mL/min for a sampling period of 120 minutes for VOC sampling, while the formaldehyde sampling rate was set at 1.5 L/min for 120 minutes for a total of 180 L.

Tenax cartridges were used to sample VOCs including high molecular weight aldehydes and derivatized DNPH cartridges were used to sample formaldehyde and acetaldehyde. VOC sample tubes were analyzed by desorbing the VOCs through a thermal desorption system and then injected into a gas chromatograph equipped with a mass detector (GC/MS). Aldehyde tubes were desorbed with acetonitrile solvent and injected into a high performance liquid chromatograph (HPLC). Table 3 describes the small chamber operating conditions while Table 4 summarizes the GC/MS and the HPLC operating conditions.

Table 3
Small chamber operating conditions

Parameter	Symbol	Unit	Value
Chamber volume	V	m^3	1.0
Loading ratio	Lr	m^2/m^{-3}	0.44
Temperature	T	°C	23±1
Relative humidity	RH	%	50±5
Air exchange rate	ACH	h^{-1}	1.0
Sampling time		Hours	24

Table 4

TDU/GC/MS and HPLC operating conditions

Thermal Desorption Unit (Type ACM 900)	
Desorption temperature	250ºC
Desorption time	6 min
Cryofocus Unit Model 951	
Cooling temperature	-50ºC
Time	4 min
Desorption temperature	150ºC
Desorption time	15 min
GC/MS: Agilent 5890 Series II Plus	
Carrier gas	He, 43.2 cm/sec
Column J&W Scientific DB-1	30 m x 0.25 mm ID, 1.0 µm
Injection type	Split: 22:1 at 230ºC
Oven heating program	10 min at 70ºC
	8ºC/min at 200ºC
	3 min at 200ºC
Detector	MSD, transfer line temp. 280ºC
HPLC Type: Agilent Series 1100	
Column Zorbax Eclipse XDB-C18	Analytical 4.6 mm x 150 mm, 5 microns
Phase mobile	70% ACN:30% water
Flow rate	1.0 mL/min
Total injected volume	25 µL
Column temperature	20ºC
Detector	DAD 360 mm

2.3.3 Quantification of Formaldehyde

Formaldehyde emissions were quantified according to the modified National Institute of Occupational Safety and Health (NIOSH) Test Method 3500. The method can be summarized as follows: 4 mL of the scrubber's content and 0.1 mL of 1.0% chromotropic acid are poured in a 50 mL Pyrex® test tube with a screw top cap. Six mL of concentrated sulphuric acid (96%) are slowly added and agitated for 2 minutes, then heated for 30 minutes at 100°C and cooled and tested in triplicate. Solution absorbencies were read through a UV-visible spectrophotometer set at 580 nm. Distilled water was run as a blank, and with a formaldehyde solution calibration curve, each absorbency reading being converted into µg/mL of formaldehyde. When the condensate samples were too concentrated to yield absorbencies in the linear range of the calibration curve, aliquots of these samples were diluted with distilled water to a level within the linear range of the calibration curve. The concentration obtained from this dilution was back-calculated to the original concentration and presented as micrograms of formaldehyde per litre, which is then converted into parts per million (ppm) and in emission factors as $mg/m^2.h$.

2.3.4 Quantification of the TVOC

VOC measurements from panel samples were conducted in accordance with the ASTM D 5116-97 guide and described in great detail in Barry et al. (1999). A Thermal Desorber/Gas Chromatograph/Mass Spectrometer (TDU/GC/MS) system was utilized to desorb and quantify the total volatile organic compounds (TVOC). A "cryo-trap" device was connected to the GC column in order to "cryofocus" the thermally desorbed chemicals prior to their injection into the GC. The GC oven was programmed for 10 min at 70°C, followed by ramping up the heat to 200°C at a rate of 8°C/min, and held for 10 minutes. The mass scan ranged from 29 to 550 atomic mass units (amu). Quantitative evaluation was achieved by comparing the chromatogram peak area of each compound to the corresponding peak area of a standard.

2.4 Results and Discussions

Tables 5 and 6 summarize the emitted VOCs including formaldehyde, acetaldehyde and acetone expressed in micrograms per cubic meter (µg/m^3). To better illustrate the variation of emissions as a function of the product types, the results are graphically shown in Figures 9 and 10; the same scale was applied to both Figures for easy comparison. As can be seen from these Figures, no correlation exists between emission results and the number of glue lines involved in each product category or product thickness. Also, most of the emitted VOCs, if we except formaldehyde and acetaldehyde, are those usually emitted from softwood species, indicating that only formaldehyde and acetaldehyde could really be associated with the products manufacturing processes. Figure 11 compares the total volatile organic compounds (TVOC), excluding formaldehyde, acetaldehyde and acetone, emitted from the five different products tested; as for individual VOCs, no correlation can be established between TVOCs, the thickness or the number of plies in cross-laminated lumber products.

Table 5

Samples 24-hour individual VOCs (iVOCs), TVOCtoluene, between n-C6 and n-C16 including formaldehyde (µg/m^3) (114-3S and 95-3S products)

VOCs	CAS #	114-3S			95-3S		
		A	B	Mean	A	B	Mean
Acetic acid	64-19-7	N/A*	6.7	6.7	2.4	<2.0	2.4
Hexanal	66-25-1	5.0	9.4	7.2	2.9	4.3	3.6
Alpha-pinene	7785-70-8	134.7	218.1	176.4	44.7	26.2	35.4
Beta-pinene	18172-67-3	14.6	32.7	23.6	9.9	7.8	8.8
Alpha-phellandrene	99-83-2	4.7	N/A*	4.7	2.7	3.1	2.9
3-carene	13466-78-9	19.1	51.0	35.0	3.6	8.3	6.0
Para-cymene	99--87--6	78.6	5.9	42.3	43.0	45.4	44.2
Limonene	95327-98-3	7.6	11.7	9.6	3.3	2.8	3.0
Unknown	- - -	- - -	- - -	- - -	4.9	5.3	- - -
$TVOC_{alpha\text{-}pinene}$	- - -	264.3	335.5	299.9	117.3	103.2	110.2
Formaldehyde	50-00-0	16.6	21.5	19.1	9.6	8.7	9.1
Acetaldehyde	75-07-0	70.1	149.7	109.9	107.3	51.0	79.1
Acetone	67-64-1	33.2	65.3	49.2	45.7	24.4	35.0

* Compound for which the concentration is below the quantification limit allowed by ANSI/BIFMA.

Table 6

Samples 24-hour individual VOCs (iVOCs), TVOCtoluene, between n-C6 and n-C16 including formaldehyde (µg/m^3) (190-5S, 152-5S and 210-7S products)

VOCs	CAS #	190-5S			152-5S			210-7S		
		A	B	Mean	A	B	Mean	A	B	Mean
Acetic acid	64-19-7	3.8	3.9	3.9	2.8	<2.0	2.8	2.8	2.1	2.4
Hexanal	66-25-1	4.4	3.8	4.1	3.1	2.6	2.8	4.4	2.1	3.2
Alpha-pinene	7785-70-8	67.9	143.5	105.7	98.6	20.5	59.6	64.9	35.2	50.0
Beta-pinene	18172-67-3	14.0	8.5	11.3	7.3	4.5	5.9	7.9	6.7	7.3
Alpha-phellandrene	99-83-2	2.7	<2.0	2.7	<2.0	2.3	2.3	<2.0	<2.0	<2.0
3-carene	13466-78-9	9.3	9.6	9.5	36.2	5.9	21.1	8.3	5.5	6.9
Para-cymene	99--87--6	36.4	<2.0	36.4	2.8	32.5	17.7	3.0	13.6	8.3
Limonene	95327-98-3	10.7	4.6	7.7	3.4	2.3	2.8	4.2	2.7	3.5
Unknown	- - -	- - -	- - -	- - -	- - -	2.4	- - -	- - -	- - -	- - -
$TVOC_{alpha\text{-}pinene}$	- - -	149.2	174.1	161.7	154.3	72.9	113.6	95.4	68.0	81.7
Formaldehyde	50-00-0	9.4	8.8	9.1	5.7	6.5	6.1	6.0	5.4	5.7
Acetaldehyde	75-07-0	71.5	68.5	70.0	72.6	74.4	73.5	73.6	59.4	66.5
Acetone	67-64-1	22.3	27.6	24.9	31.2	29.8	30.5	21.5	16.1	18.8

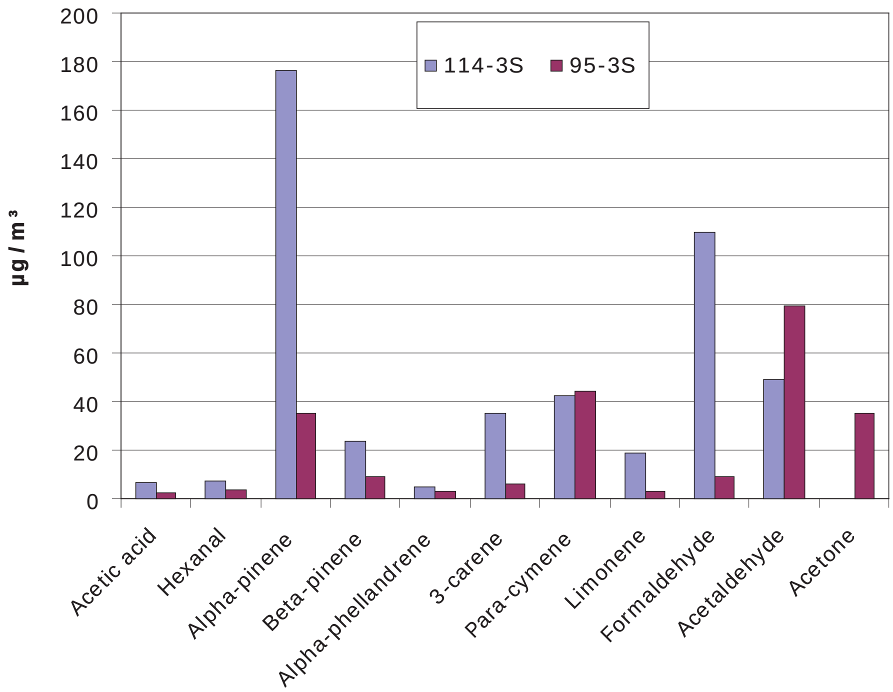

Figure 9

24-hour VOCs including formaldehyde and acetaldehyde off gassing as a function of samples types (114-3S and 95-3S products)

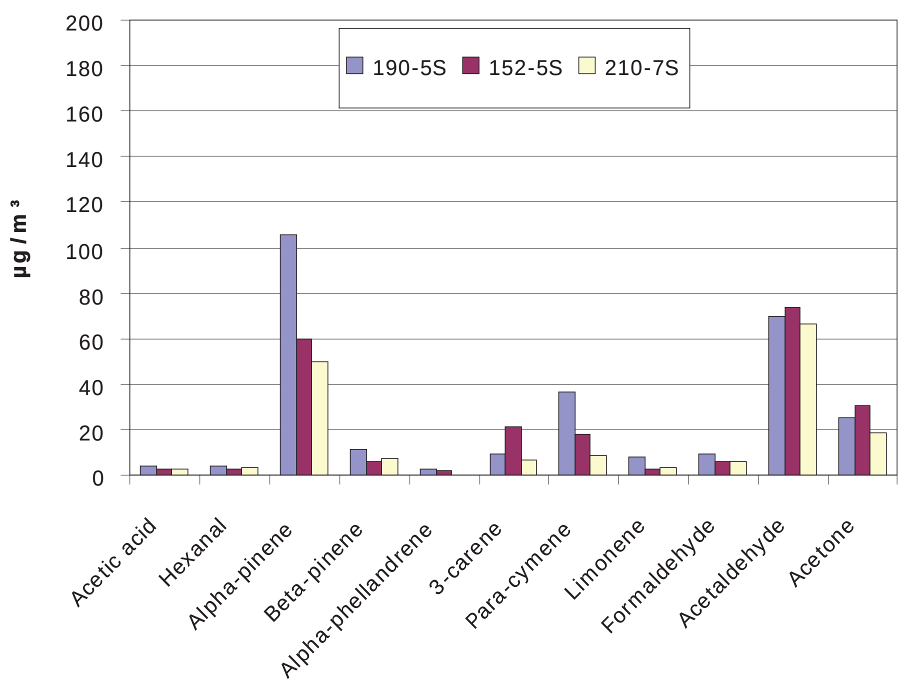

Figure 10

24-hour VOCs including formaldehyde and acetaldehyde off gassing as a function of samples types (190-5S, 152-5S and 210-7S products)

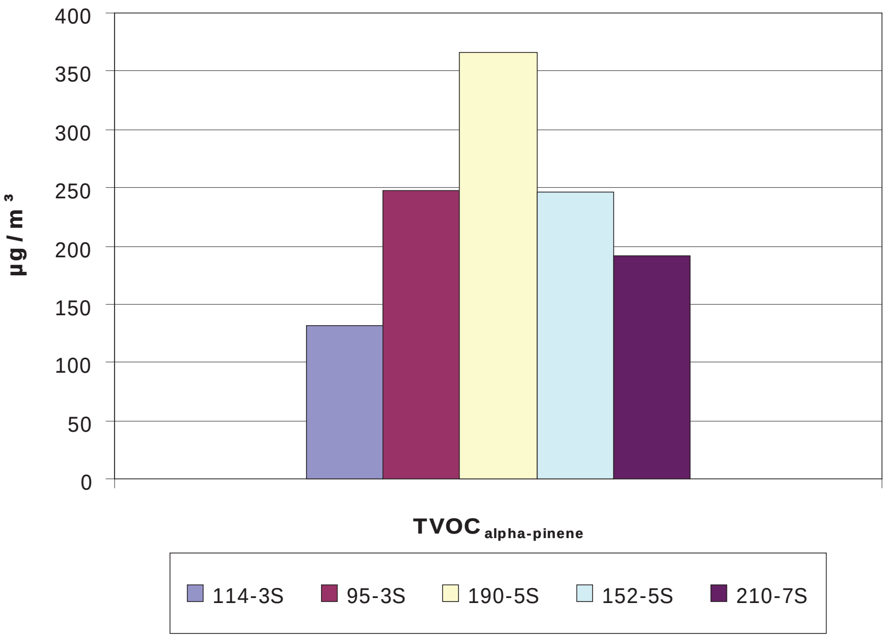

Figure 11
24-hour TVOC emissions as a function of cross-laminated products

Examples of emission labelling systems in Europe in terms of VOCs, including formaldehyde and acetaldehyde, are summarized in Table 7 in order to put the tested cross-laminated timber products emissions in context, and to inform manufacturers interested in labelling their products for overseas markets. Because few individual VOC emission limits are expressed in emission factors, i.e. mass of the emitted VOC per square metre of the product tested per hour ($\mu g/m^2.h$), the cross-laminated timber products emission results have been converted into emission factors and summarized in Tables 8 and 9. Results of emission factors reported in Tables 8 and 9 were calculated from the 24-hour sampling time compared to the voluntary limits listed in Table 7 calculated after 3, 10 or 28 days of sample exposure in the environmental chamber. One should expect that the cross-laminated timber emission factors would be much lower if their exposure is prolonged for an additional 3, 10 or 28 days and meet the most stringent Blue Angel or GUT (Germany) TVOC emission limits not met after 24 hours of exposure.

Table 7

Example of some European emission labelling systems

Label	Origin	TVOC	Aldehydes Additional Requirements
AgBB	Germany	10 mg/m^3 (3 days) 1 mg/m^3 (28 days)	DIBt: 120 µg/m^3 (28 days)
CESAT	France	5000 µg/m^3 (3 days) 200 µg/m^3 (28 days)	Formaldehyde: 10 µg/m^3 (28 days)
M1	Finland	200 µg/m^3 (28 days)	Formaldehyde: 50 µg/m^3 (28 days)
LAQI Scheme	Portugal	5000 µg/m^{2}h (3 days) 200 µg/m^{2}h (28 days)	Formaldehyde: 10 µg/m^3 (28 days)
Natureplus	Germany	5000 µg/m^{2}h (3 days) 200 µg/m^{2}h (28 days)	Formaldehyde: 36 µg/m^3 after 3 days or 28 days
Blue Angel	Germany	200 or 300 µg/m^3 (28 days)	Formaldehyde: 60 µg/m^3 (28 days)
Austrian Ecolabel	Austria	1.2 mg/m^3 (3 days) 0.36 mg/m^3 (28 days)	Hexanal: 70 µg/m^2 h (28 days), nanonal: 20 µg/m^2 h after 3 days
GUT	Germany	300 µg/m^3 (3 days)	Formaldehyde: 10 µg/m^3 after 3 days
EMICODE EC1 such as adhesives	Germany	500 µg/m^3 (10 days)	Formaldehyde and acetaldehyde: 50 µg/m^3 each after 24 hours
Scandinavian Trade Standards	Sweden	Declaration of TVOC at 28 days and 26 weeks no limits specified	Formaldehyde and acetaldehyde according to WHO

Table 8

Samples 24-hour iVOCs, TVOCtoluene, between n-C6 and n-C16 emission factors including formaldehyde (μg/m².h) (114-3S and 95-3S products)

VOCs	CAS #	114-3S			95-3S		
		A	B	Mean	A	B	Mean
Acetic acid	64-19-7	<2.0	2.9	2.9	5.1	N/A*	5.1
Hexanal	66-25-1	2.2	4.1	3.2	6.3	10.1	8.2
Alpha-pinene	7785-70-8	59.0	95.5	77.2	96.6	61.5	79.1
Beta-pinene	18172-67-3	6.4	14.3	10.3	21.3	18.3	19.8
Alpha-phellandrene	99-83-2	2.1	<2.0	2.1	5.9	7.3	6.6
3-carene	13466-78-9	8.3	22.3	15.3	7.7	19.6	13.7
Para-cymene	99--87--6	34.4	2.6	18.5	92.9	106.6	99.8
Limonene	95327-98-3	3.3	5.1	4.2	7.1	6.5	6.8
Unknown	- - -	- - -	- - -	- - -	10.5	12.4	- - -
$TVOC_{alpha\text{-}pinene}$	- - -	115.7	146.9	131.3	253.5	242.3	247.9
Formaldehyde	50-00-0	8.4	10.9	9.7	16.2	18.7	17.5
Acetaldehyde	75-07-0	35.9	76.5	56.2	182.0	109.6	145.8
Acetone	67-64-1	16.7	32.9	24.8	77.5	52.6	65.1

* Compound for which the concentration is below the quantification limit allowed by ANSI/BIFMA.

Table 9

Samples 24-hour iVOCs, TVOCtoluene, between n-C6 and n-C16 emission factors including formaldehyde (μg/m².h) (190-5S, 152-5S and 210-7S products)

VOCs	CAS #	190-5S			152-5S			210-7S		
		A	B	Mean	A	B	Mean	A	B	Mean
Acetic acid	64-19-7	8.7	9.0	8.8	5.9	N/A*	5.9	6.4	5.1	5.8
Hexanal	66-25-1	9.9	8.7	9.3	6.6	5.9	6.2	10.3	4.9	7.6
Alpha-pinene	7785-70-8	153.1	326.6	239.9	210.1	46.2	128.1	151.9	83.1	117.5
Beta-pinene	18172-67-3	31.6	19.5	25.5	15.6	10.1	12.9	18.6	15.8	17.2
Alpha-phellandrene	99-83-2	6.2	N/A*	6,2	N/A*	5.2	5.2	N/A*	N/A*	N/A*
3-carene	13466-78-9	21.0	21.9	21.4	77.1	13.4	45.2	19.4	13.1	16.2
Para-cymene	99--87--6	82.1	N/A*	82.1	6.0	73.5	39.8	7.1	32.2	19.6
Limonene	95327-98-3	24.1	10.5	17.3	7.2	5.3	6.2	9.9	6.4	8.1
Unknown	- - -	- - -	- - -		- - -	5.3	2.7	- - -	- - -	
$TVOC_{alpha\text{-}pinene}$	- - -	336.7	396.2	366.4	328.5	164.7	246.6	223.5	160.6	192.0
Formaldehyde	50-00-0	18.9	20.1	19.5	10.6	12.3	11.5	14.1	12.8	13.5
Acetaldehyde	75-07-0	144.2	155.9	150.0	132.9	139.5	136.2	172.3	140.2	156.3
Acetone	67-64-1	45.0	62.9	54.0	58.0	56.6	57.3	50.3	38.1	44.2

* Compound for which the concentration is below the quantification limit allowed by ANSI/BIFMA.

On the other hand, the levels of the emitted formaldehyde converted into parts per billion (ppb) are summarized in Table 10 and, as one can see, emissions are just in the order of a few ppb. Compared to the European E1 wood products formaldehyde emission limit of 0.1 ppm (100 ppb), all five cross-laminated timber tested products had emissions 6 to 20 times lower than the E1 required emission limits, indicating that these products could be installed in any European country embracing the E1 grade. When compared to the voluntary formaldehyde emission limits for labelling (Table 7), three of the five samples meet the formaldehyde emission limits and two samples encoded as 114-3S and 190-5S would need to be tested for longer period of time ranging from 2 to 3 days in order to qualify for the most stringent GUT (Germany) labelling for which the formaldehyde emission limit is set at 10 μg/m³ after three days.

Table 10
24-hour formaldehyde emissions as a function of product types

	CAS #	114-3S		95-3S		190-5S		152-5S		210-7S	
		µg/m³	ppb	µg/m³	ppb	µg/m³	ppb	µg/m³	ppb	µg/m³	ppb
Formal-dehyde	50-00-0	19.1	15	9.1	7	19.5	16	6.1	5	5.7	5

The new formaldehyde emission limits set forth by the California government are known under the acronym of CARB Phase I and Phase II for wood composite products particleboard, MDF, thin MDF and hardwood plywood (HWPW) with composite core (HWPW-CC) or veneer core (HWPW-VC). By July 2012, phase II will be enforced and formaldehyde emission limits will vary from 0.13 ppm (130 ppb) for thin MDF to 0.05 ppm (50 ppb) for HWPW-CC. Comparing these limits to those from the cross-laminated timber products in Table 10, one can conclude that the cross-laminated timber products easily meet the most stringent CARB limits of 50 ppb.

2.5 Conclusions and Recommendations

Five CLT products were tested for their volatile organic compounds (VOCs), including formaldehyde and acetaldehyde emissions, in order to assist engineers and builders to better select construction materials with low-emitting characteristics having less impact on indoor air quality. The tested laminated products had different thicknesses and different numbers of glue lines. Emissions were collected after 24 hours of sample exposure in the environmental chamber.

Results did not show any correlation between individual VOCs, including formaldehyde and acetaldehyde or TVOC and the cross-laminated timber thickness or numbers of glue lines. All five products showed very low levels of iVOC and TVOC emissions; most of the detected VOCs consisted of terpene compounds originating from the softwood furnish used to manufacture the cross-laminated timber products.

In terms of evaluating the products' impact on indoor air quality, one can easily conclude that it would be negligible, if any. The five cross-laminated timber products TVOCs and formaldehyde 24-hour results were generally lower than those set forth by some European emission labelling systems even if those limits were emissions measured after 3, 10 or 28 days of sample exposure. Also, the European E1 grade for wood products formaldehyde emissions set at 0.1 parts per million (ppm) or 100 parts per billion (ppb) is 6 to 20 times higher than those measured from the cross-laminated timber products.

Comparing the limits that are planned to be enforced by CARB by 2012, one can conclude that the CLT products tested in this study would easily meet the most stringent CARB limit of 50 ppb.

2.6 References

American Society for Testing and Materials (ASTM). 1996. *Standard test method for determining formaldehyde concentration in air from wood products using a small-scale chamber*. ASTM D 6007-96. West Conshohocken, PA: ASTM. 8 p.

______. 2002. *Standard test method for determining formaldehyde concentrations in air and emission rates from wood products using a large chamber*. ASTM E 1333-96(2002). West Conshohocken, PA: ASTM. 12 p.

______. 2003. *Standard test method for determination of formaldehyde and other carbonyl compounds in air (active sampler methodology)*. ASTM D 5197-03. West Conshohocken, PA: ASTM. 15 p.

______. 2006. *Standard guide for small-scale environmental chamber determination of organic emissions from indoor materials/products*. ASTM D 5116-06. West Conshohocken, PA: ASTM. 15 p.

Barry, A. 1995. Measurement of VOCs emitted from particleboard and MDF panel products supplied by CPA mills: A report by Forintek Canada Corp. for the Canadian Particleboard Association. Quebec, QC: Forintek. 20 p.

Barry, A., and D. Corneau. 1999. Volatile organic chemicals emissions from OSB as a function of processing parameters. *Holzforschung* 53:441-446.

Barry, A., D. Corneau, and R. Lovell. 2000. Press volatile organic compounds emissions as a function of wood processing parameters. *Forest Products Journal* 50 (10):35-42.

BIFMA International. 2007. *Standard test method for determining VOC emissions from office furniture systems, components and seating*. ANSI/BIFMA M7.1-2007. Grand Rapids, MI: BIFMA International. 53 p.

Commission of the European Communities. 1989. *Formaldehyde emission from wood based materials: Guideline for the determination of steady state concentrations in test chambers*. Report no. 2 of EUR 12196 - European concerted action: Indoor air quality and its impact on man. COST Project 613. Brussels, Luxembourg: Office for Official Publications of the European Communities. 24 p.

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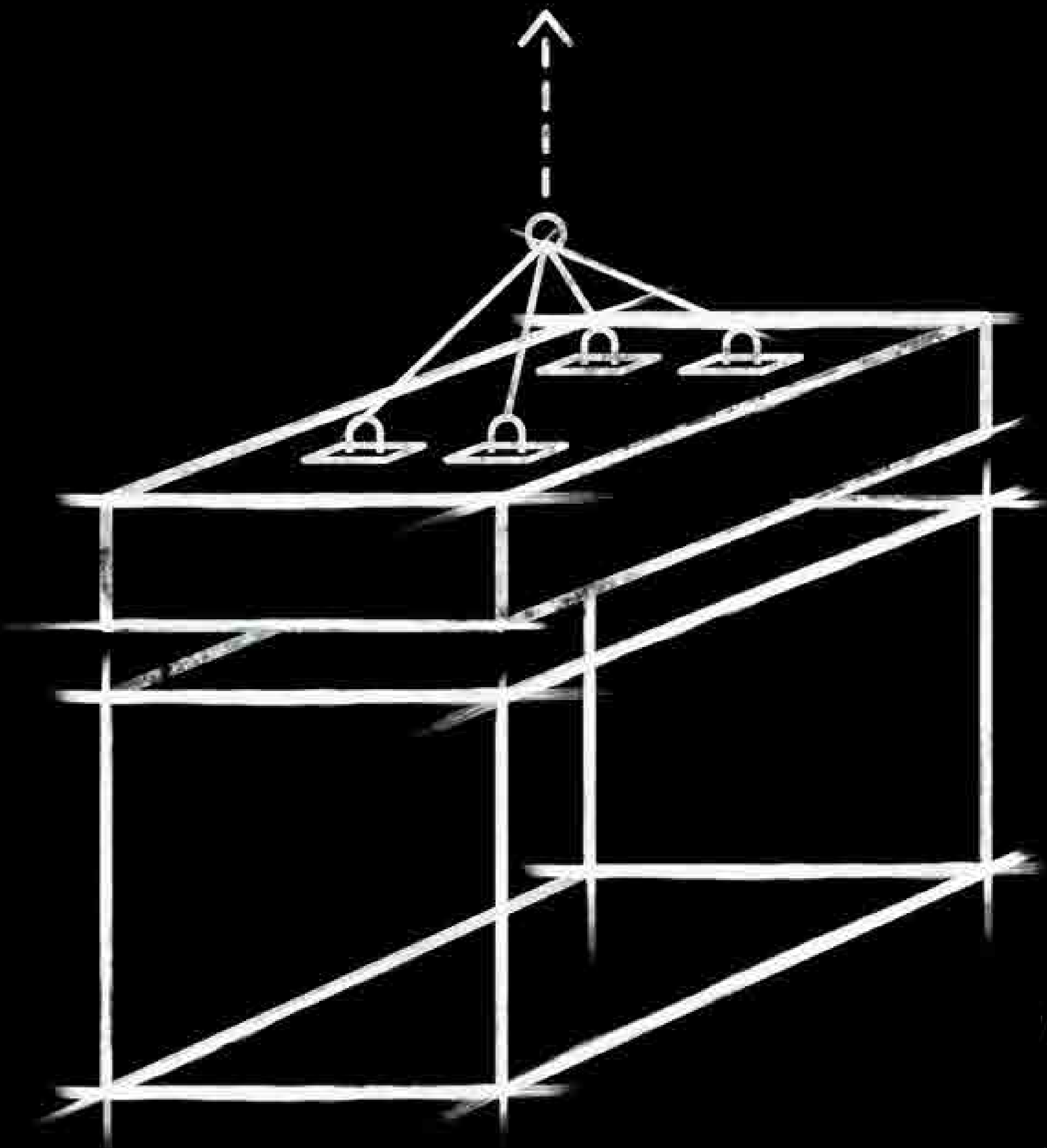

Lifting

Lifting and handling of CLT elements

CHAPTER 12

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ABSTRACT

Cross-laminated timber (CLT) construction is a relatively new process. There is therefore very little specific technical documentation for the erection of heavy structures designed and built with CLT panels. Current CLT manufacturers propose lifting systems to set up prefabricated wood assemblies. However, technical documents currently available mostly come from Europe and may appear incomplete to some engineers and builders in North America.

In this chapter, we present a variety of lifting systems that can be used in the construction of structures using CLT panels. We discuss the basic theory required for proper lifting techniques. In addition, we introduce various tools and accessories that are frequently required for CLT construction, as well as good building practices to help manufacturers build safe and efficient CLT panel structures. Finally, we discuss issues related to the transportation of CLT assemblies from factory to building site. Regulatory aspects of transportation are also discussed.

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1 INTRODUCTION

In this chapter, we present a variety of lifting systems that can be used in the construction of structures made of cross-laminated timber (CLT) panels. We discuss the basic theory required for proper lifting techniques. In addition, we introduce various tools and accessories that are frequently required during CLT construction, as well as good building practices to help manufacturers build safe and efficient CLT panel structures. Finally, we discuss issues related to the transportation of CLT assemblies from factory to building site. Regulatory aspects of transportation are also discussed.

1.1 Parallel with Prefabricated Concrete Industry

CLT construction is a relatively new process. There is therefore very little specific technical documentation for the erection of heavy structures designed and built with CLT panels. Current CLT manufacturers propose lifting systems to set up prefabricated wood assemblies. However, technical documents currently available mostly come from Europe and, to some engineers and builders in North America, these documents may seem incomplete or insufficiently adapted to the standard construction techniques that they use.

A close look at Figure 1 reveals that prefabricated concrete construction using large concrete slabs is, in many ways, similar to the recent techniques used in CLT construction. As the prefabricated concrete construction industry is more developed and experienced, it is easier to obtain or use systems and lifting accessories adapted to this industry or to build on this experience.

For example, certain systems discussed in this chapter, which are sometimes used in CLT construction, are directly inspired by the systems used in prefabricated concrete construction. In addition, a large amount of technical data contained in the following sections is taken from documentation developed and provided by major producers of prefabricated concrete or by manufacturers of lifting devices specialized in factory-made concrete.

Figure 1
Lifting and handling of prefabricated concrete elements

1.2 Lifting and Handling of CLT Elements

The emerging CLT construction industry offers various techniques for lifting and handling CLT panels so that they can be used in the erection of buildings and other structures. The complexity of the building or its location often dictates the techniques and systems to be used. Of course, erecting an 8-storey building in a downtown area typically requires more preparation and precaution than a single-family residence built in the country. But if that country house is to be perched high in the mountains, the techniques used may often be surprising (Figures 2 and 3).

Figure 2
Lifting and handling of CLT elements by cableway (courtesy of KLH)

Figure 3
Lifting and handling of CLT elements by helicopter (courtesy of KLH)

Figures 4 to 10 show examples of CLT panels during the lifting and handling process on construction sites. The techniques and lifting systems used are discussed in detail further in this chapter.

Figure 4
Lifting and handling of relatively light CLT elements, in Norway (courtesy of Brendeland and Kristoffersen, Architects)

Figure 5
Lifting and handling of CLT wall elements, in Belgium (courtesy of HMS)

Figure 6
Lifting and handling of CLT elements, in Longueuil, Québec (courtesy of KLH Élément)

Figure 7
Lifting and handling of CLT elements in an hybrid structure, USA (courtesy of Binderholz)

Figure 8
Lifting and handling of CLT elements, in Saint-Lambert, Québec (courtesy of Nordic Structures Bois)

Figure 9
Lifting and handling of CLT elements, in Sweden

Figure 10
Lifting and handling of CLT elements, in Norway
(courtesy of Brendeland and Kristoffersen Architects)

2 SLINGING AND FASTENING SYSTEMS FOR THE LIFTING AND HANDLING OF CLT PANELS

A variety of systems available for lifting and handling CLT panels are presented in this section. Some systems are commonly used in CLT construction. Others are for illustrative purposes, some of which are inspired by systems used in the prefabricated concrete industry.

Many of the systems proposed use slings. A sling is a cable that connects the fastening system to the lifting device. It usually consists of textile rope, synthetic fiber woven strips, steel cables, or chains. Slings must always be calibrated (working load permitted) and validated (wear and tear) before use.

2.1 Contact Lifting Systems

Lifting systems that use steel plates that provide compressive resistance on the lower face of the panels during lifting are popularly considered the safest CLT panel handling methods. However, to avoid accidents on the lower levels of the building once the panels are in place, great care must be taken when removing the system as the steel plates are usually not secured once the system is unbolted.

This lifting technique requires in-plant drilling to allow the insertion of dowels or threaded sleeves with nuts. This technique uses the wood's efficient strength in compression. However, when CLT elements are intended to be visible inside the building, local repairs will be required using wooden dowels.

It is important to note that in all cases outlined hereafter, the holes must be sealed to ensure proper air tightness and to limit the spread of sound, smoke, and fire.

The following examples describe some contact lifting systems.

2.1.1 Single Lifting Loop with Threaded Sleeve Used with Socket Steel Tube Welded onto Flat Steel Plate

The system comprised of a single lifting loop with threaded sleeve is widely used in the construction of prefabricated concrete. The system shown in Figure 11 is a modification of the system commonly used to lift prefabricated concrete. Instead of enclosing the welded plate socket in concrete at the plant, the socket is welded onto a steel plate and inserted into a previously machined hole. The lifting loop is then screwed from above using the threaded sleeve. This system is considered simple, safe, economical, and quick to use on the construction site.

The single lifting loop used in the concrete industry can be reused but it requires enhanced inspection and quality control to ensure safety. This system is not recommended for frequent panel rising, which may imply a high lifting angle (>30°) and therefore bending of the steel cable. The recommended maximum angle (β) is 30°. The use of a spreader beam can help reduce the lifting angle. It is also recommended that the radius of the hook be at least equal to the diameter of the lifting loop steel cable. When handling is completed, the two components must be removed carefully.

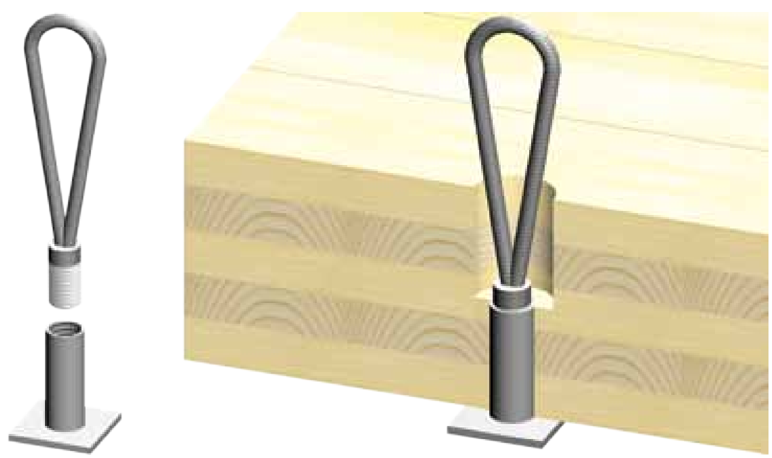


Figure 11
Single lifting loop with threaded sleeve used with socket steel tube welded onto flat steel plate

2.1.2 Articulated Lifting Loop with Threaded Sleeve Used with Socket Steel Tube Welded onto Flat Steel Plate

The system made of an articulated lifting loop with threaded sleeve also comes from the prefabricated concrete industry and is installed in the same manner as the previous system. One advantage of this system is the ability of the steel cable to rotate in all directions around the threaded sleeve. This system can thus be used more easily for rising panels. However, the lifting angle should still be limited to 30°. When handling is completed, the two components must be removed carefully.

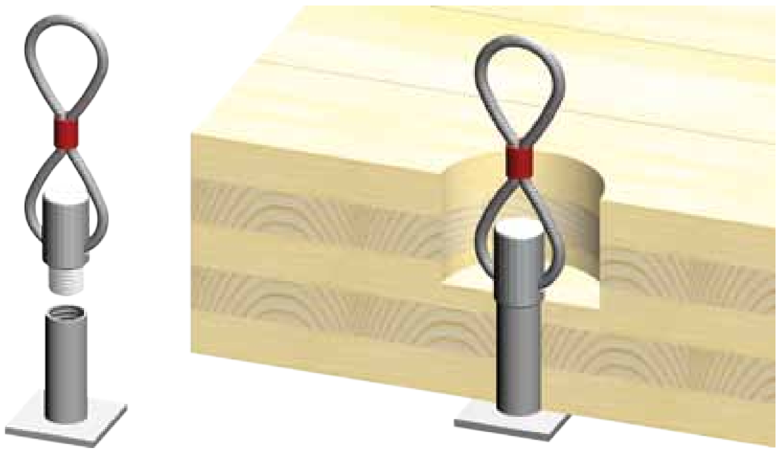


Figure 12
Articulated lifting loop with threaded sleeve used with socket steel tube welded onto flat steel plate

2.1.3 Articulated Lifting Hook with Threaded Sleeve Used with Socket Steel Tube Welded onto Flat Steel Plate

The system comprised of articulated lifting hook and threaded sleeve used with a socket steel tube welded onto a flat steel plate also comes from the prefabricated concrete industry. The hook allows for quick installation on the lifting system. This system can be used for rising panels because of the ability of the hook to rotate around the steel ring. When handling is completed, the two components must be removed carefully.

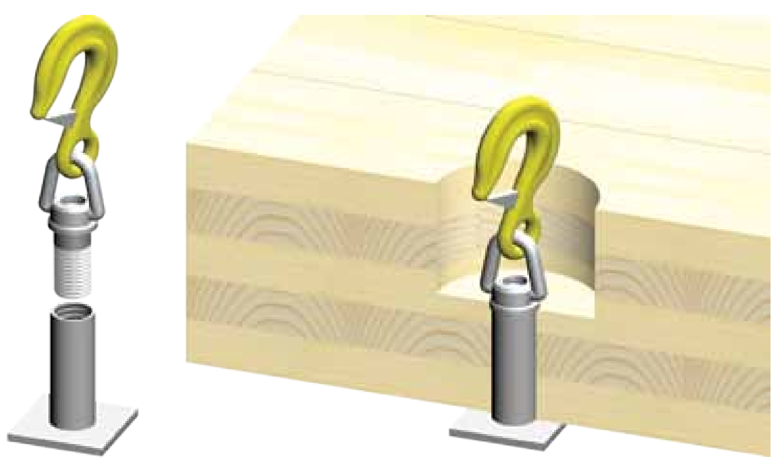


Figure 13
Articulated lifting hook with threaded sleeve used with socket steel tube welded onto flat steel plate

2.1.4 Threaded Eyelet Bolt Used with Socket Steel Tube Welded onto Flat Steel Plate

The threaded eyelet bolt used in conjunction with a socket steel tube welded onto a flat steel plate is also a good option for quick and safe lifting. However, it is important to choose the right eyelet bolt and install it correctly (Figures 14 and 15). It is recommended to use an eyelet base bolt when lifting heavy loads at an angle. Ensure there is proper contact between the base and the wood panel. Plain or regular eyelet bolts (without base) are normally used in straight tension when lifting heavy loads; that is, when used with a spreader beam or with only one attachment point. Also, according to good practice, the eyelet bolts must be oriented in the same direction as the tensioned slings since the eyelet could bend under heavy oblique loads (Canadian Centre for Occupational Health and Safety, CCOHS). When handling is completed, the two components must be removed carefully.

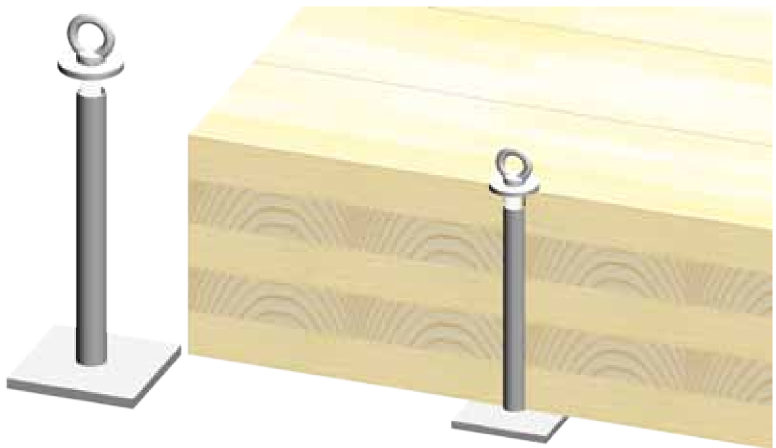


Figure 14
Threaded eyelet bolt (with base) used with socket steel tube welded onto flat steel plate

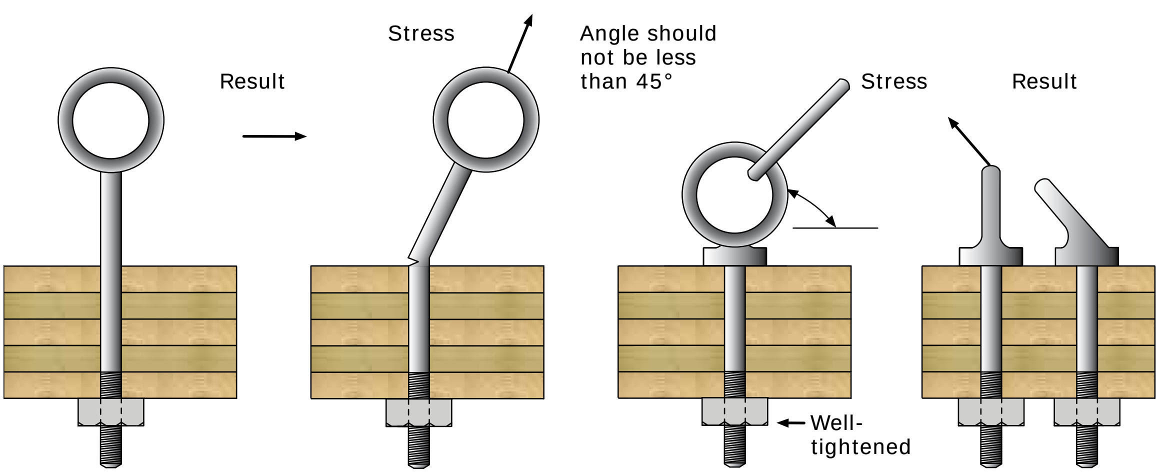


Figure 15
Correct use of threaded eyelet bolt (with and without base)

2.1.5 Threaded Eyelet Bolt Used with Plate and Nut

The system using threaded eyelet bolt in conjunction with a steel plate and nut is widely used in CLT construction. However, it is important to choose the proper eyelet bolt and install it correctly. The use of an eyelet base bolt when lifting at an angle is also recommended. Also, according to good practice, the eyelet bolts must be oriented in the same direction as the tensioned slings since the eyelet could bend under heavy oblique loads. When handling is completed, the system must be completely removed.

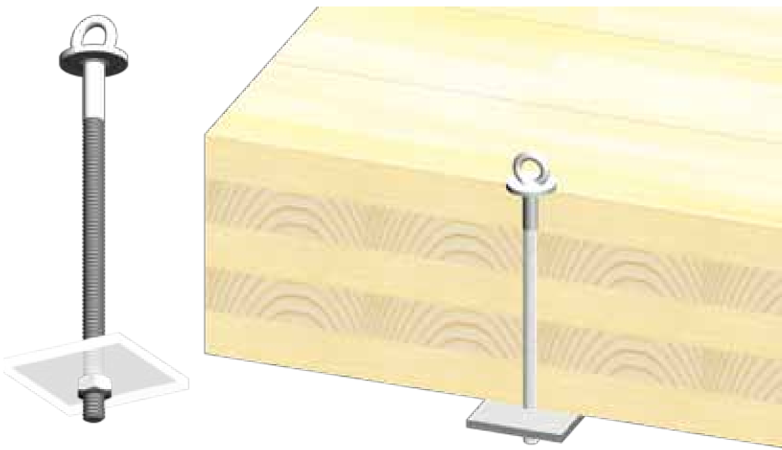


Figure 16
Threaded eyelet bolt used with plate and nut

2.1.6 Eyelet Used with Bolt or Threaded Sleeve and Steel Plate

The following system is similar to the system presented above, and the same recommendations apply. In this case, the eyelet is independent from the sleeve or bolt. It is important to use an eyelet with base when lifting at an angle. Baseless eyelets should only be used when lifting in straight tension. When handling is completed, the system must be completely and carefully removed.

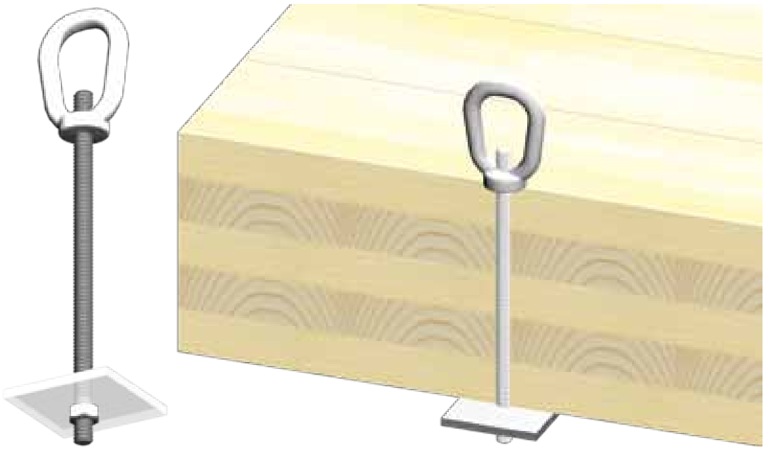


Figure 17
Eyelet used with threaded bolt or sleeve and steel plate (courtesy of Nordic Structures Bois)

2.1.7 Threaded Eyelet Bolt, Threaded Socket, Threaded Bolt and Steel Plate

The threaded eyelet bolt can be used with a threaded socket, a threaded bolt, or a threaded rod and steel plate. The threaded socket can be installed in plant for future use. On the construction site, the eyelet bolt and the single bolt or the threaded rod are screwed to the plate. Again, it is important to choose the right eyelet bolt and install it correctly. When handling is completed, the two bolts and the steel plate are removed. The threaded socket remains in place for future use.

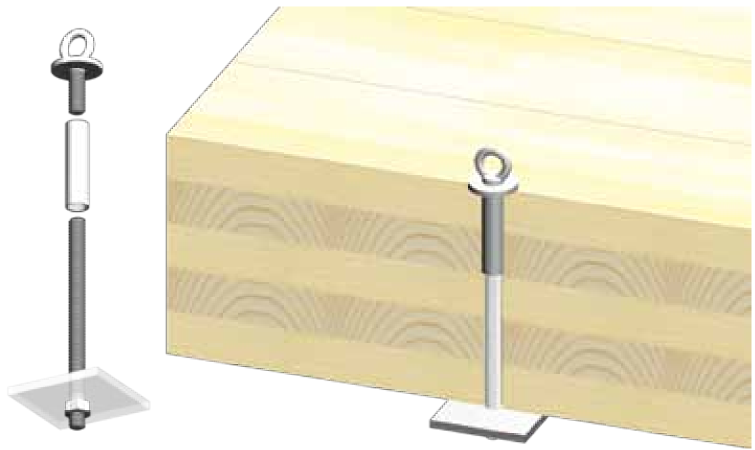


Figure 18
Threaded eyelet bolt, threaded socket, threaded bolt or sleeve and steel plate

2.1.8 Threaded Eyelet Bolt, Threaded Socket and Steel Round Rod

Another system that comes from the prefabricated concrete construction industry can inspire a new, fast, and safe system for lifting CLT panels. A threaded socket with holes at the tip is inserted into the CLT slab. This socket is normally embedded in the concrete. An eyelet bolt is screwed into the socket. The lifting system is then locked with a steel round rod that is in contact with the wood. When handling is complete, the three elements are removed. However, this system can leave marks on the timber and may not be suitable if the panel must remain visible on the underside.

Figure 19
Threaded eyelet bolt, threaded socket and steel round rod

2.1.9 Soft Lifting Sling Used With Support

Another system widely used in CLT construction is shown in Figure 20. A hole is drilled into the panel at the plant (50 ~ 75 mm). On the construction site, a soft sling is inserted into the hole and a locking piece is used on the underside. The next figure shows a piece of dimensional lumber being used. However, it is important to ensure that the locking parts are properly fixed and will not slip during handling.

Figure 20
Single lifting sling used with support

2.1.10 Soft Lifting Sling Without Support for Vertical Elements

The lifting systems presented in the previous examples are intended mainly for floor and roof slabs. For wall assemblies, a simple system requiring only one hole and a flexible sling is often used. Since walls are often lighter than thick floor slabs, this system is often appropriate for lifting and rising. The holes must be plugged once handling is completed, especially those in the exterior walls.

Figure 21
Lifting sling without support (with hole)

2.1.11 Soft Lifting Sling Without Support for Horizontal Elements

This simple lifting system requires no hole. However, this technique comes with a risk of instability due to the possibility of slings slipping during lifting. Also, in order to leave enough space to release the slings once the element is in place, the panels cannot be completely juxtaposed. Therefore, they must be drawn together with the appropriate tools (refer to Sections 4.3 to 4.5).

Figure 22
Lifting sling without support (without hole)

The next technique requires two holes drilled in plant for each anchor point. These holes have a diameter of approximately 50 mm and are relatively close together. A soft sling is inserted as shown in Figure 23.

Figure 23
Lifting sling without support (with holes)

2.2 Screw Hoist Systems

There are several lifting techniques that rely only on the withdrawal resistance of fastenings. Although these techniques are simple and effective, they require strict control during installation and use. One advantage of this system is that it does not affect the wood appearance when sections must remain visible on one side. This section describes some examples.

2.2.1 Screwed Anchor

The most widely used screw hoist system in Europe is shown in Figure 24. This system is based on an anchor used in prefabricated concrete construction. The original system uses an anchor embedded in the concrete with a protruding head to allow connection to a lifting ring.

Figure 24 shows the two components required for lifting. A self-tapping screw makes the connection between the CLT panel and the lifting ring. It is strongly recommended to use the self-tapping screw only once. The lifting ring must be inspected frequently to ensure safety. This system can be installed on both the top and side of the panels. It is important to refer to the manufacturer's technical data to determine the allowable loads and for usage and installation specifications.

Figure 24
Lifting system with self-tapping screw

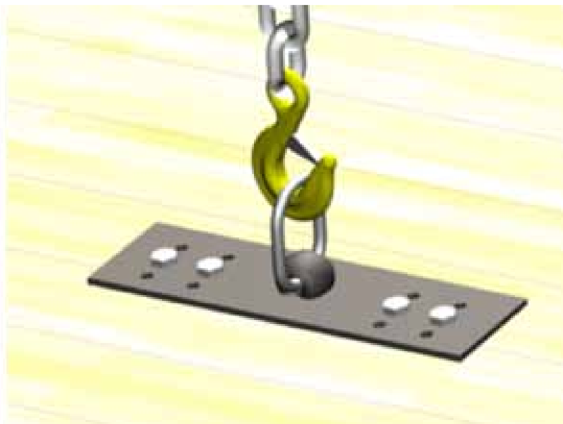
Figure 25
Screwed anchor

2.2.2 Screwed Plate and Lifting Ring

There are various lifting systems using screws or lag screws in combination with steel plates with holes. Figure 26a shows a system that uses only two self-tapping screws. This system is recommended for light-to-medium loads. Therefore, this system offers very little flexibility in terms of allowable capacity.

However, it is possible to increase the number of screws in order to increase its lifting capacity. Figure 26b shows a much more flexible system. The plate has sufficient pre-drilled holes to accommodate several lag screws or wood screws. Thus, the plate provides the engineer in charge of designing the lifting systems with much more flexibility since the same plate can be used repeatedly. The steel plates, lifting ring, and lag screws should be checked regularly. Figure 26c shows a light panel being lifted.

a)



b)

c)

Figure 26
Screwed plate and lifting ring (courtesy of Tergos)

2.2.3 RAMPA-Type Double-Threaded Socket with Eyelet Bolt or Lifting Loop with Threaded Sleeve

Another lifting method consists of using a double-threaded socket (i.e., threaded inside and outside) together with an eyelet bolt or lifting loop. This RAMPA-type socket is screwed in plant into the panel. Similar to a lag screw, a hole with a diameter equal to 75 ~ 90% of the socket diameter must first be drilled in the wood.

On the construction site, the eyelet bolt (or lifting loop with threaded sleeve) is installed for lifting. Once handling is completed, the bolt is removed. The double-threaded sleeve remains in place for future use. This system can be installed on both the top and side of the panels. It is important to refer to the manufacturer's technical data to determine the allowable loads and for usage specifications.

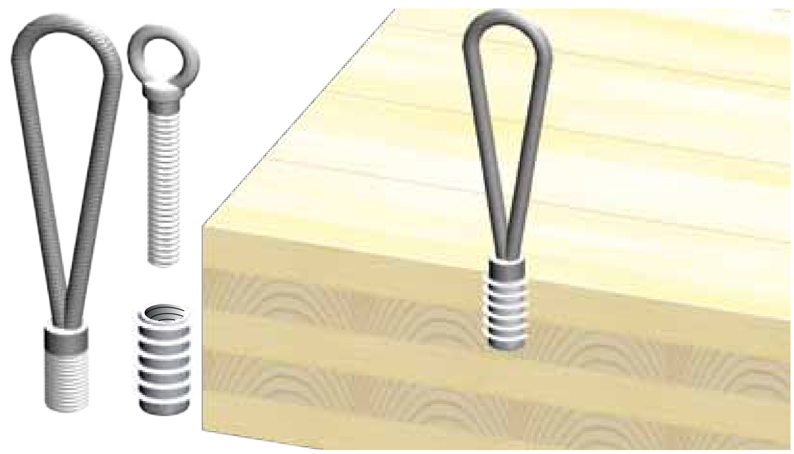


Figure 27
RAMPA-type double-threaded socket with eyelet bolt or lifting loop with threaded sleeve

2.2.4 SIHGA-Type Lifting System with Wood Screws and Eyelet Bolt

SIHGA, an Austrian company, offers an anchoring system that is screwed to the wood panels and uses an eyelet bolt. The Idefix IFS system is shown in Figure 28.

Wood screws are used to screw the cylindrical steel component to the panel. This piece is usually attached on the top of the panel. However, a recess can be performed into the panel at the plant in order to embed the fixed piece, thus allowing stacking of the panels during transportation. It is important to refer to the manufacturer's technical data to determine the allowable loads and for usage specifications.

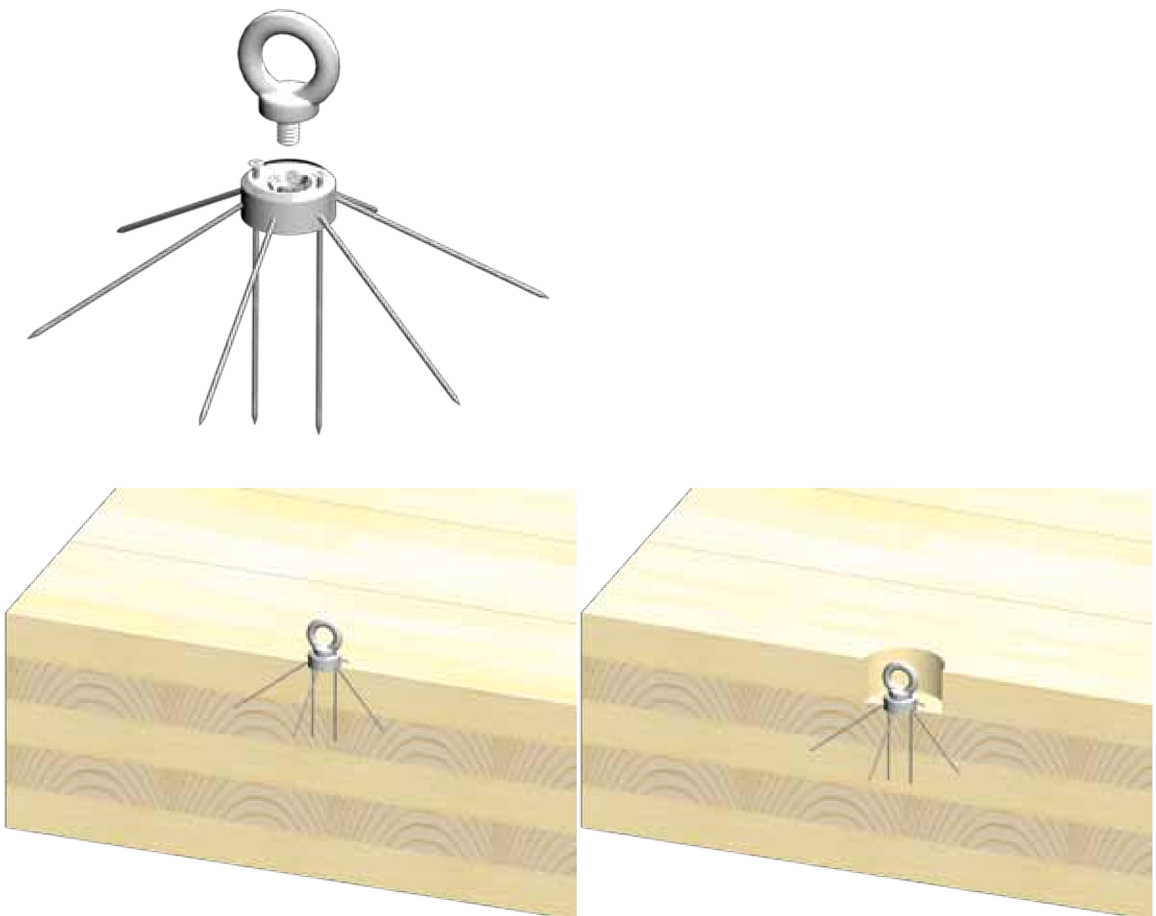


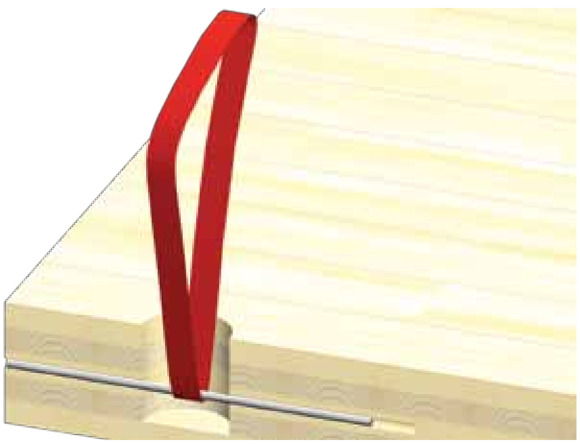
Figure 28
SIHGA-type lifting system with wood screw (with or without recess)

2.3 Integrated Lifting Systems

The principle of using plant-integrated support parts lends speed to job execution on construction sites. These systems are simple and safe. In addition, if the ceilings of buildings should remain visible, no major repair is required. However, it is better to seal the holes to ensure air tightness and to limit the spread of sound, smoke, and fire. Some examples are given in this section.

2.3.1 Inserted Rod with Soft Sling

This technique is frequently used in Europe. It consists of first drilling one hole on the top of the panel, a few centimetres from the edge. This hole, which has a diameter of about 50 ~ 75 mm, is placed at a depth equivalent to about one half to two thirds of the thickness of the panel. Then, using a long drill, a hole is drilled on the side facing the axis of the hole made on the top of the panel. A steel rod with a diameter equal to that of the hole is then forced into the hole. It is possible to use smooth rods or reinforcement steel bars. Upon insertion of the rod, a soft sling is installed and held by the rod. The sling should be able to get into the hole for easy stacking during transportation. Once the lifting and handling steps have been completed, the sling is either cut or inserted into the hole for future use. Figure 29a shows the first system.



a)

Figure 29b shows a similar system. However, instead of drilling a hole, a groove is made on the top of the panel a few centimetres from the edge. The alteration is performed at a depth equivalent to about one half to two thirds of the thickness of the panel. Then, using a long drill, a hole is drilled on the side. A steel rod with a diameter equal to that of the hole is then forced into the hole. Once the panel is on the construction site, a soft sling is simply slid under the rod; this sling can be removed once the panel is positioned. The steel bar remains in place and the hole should be sealed.

b)

Figure 29
Inserted rod with soft sling

2.3.2 Inserted Rod with PFEIFER-Type Lifting Hook

The next system is once again inspired by the prefabricated concrete construction industry. This method consists of drilling one hole on the top of the panel (re-entrant), a few centimetres from the edge. The diameter of this hole must be large and deep enough to allow insertion of a lifting hook, as shown in Figure 30. Then, with a long drill, a second hole is made on the side facing the axis of the hole made on top of the panel. A steel rod with a diameter equal to that of the hole is then forced into the hole. It is possible to use smooth rods or steel rebars. Once the panel is on the construction site, the hook is attached to the rod for the lifting and handling phase. The steel bar remains in place and the hole should be sealed.

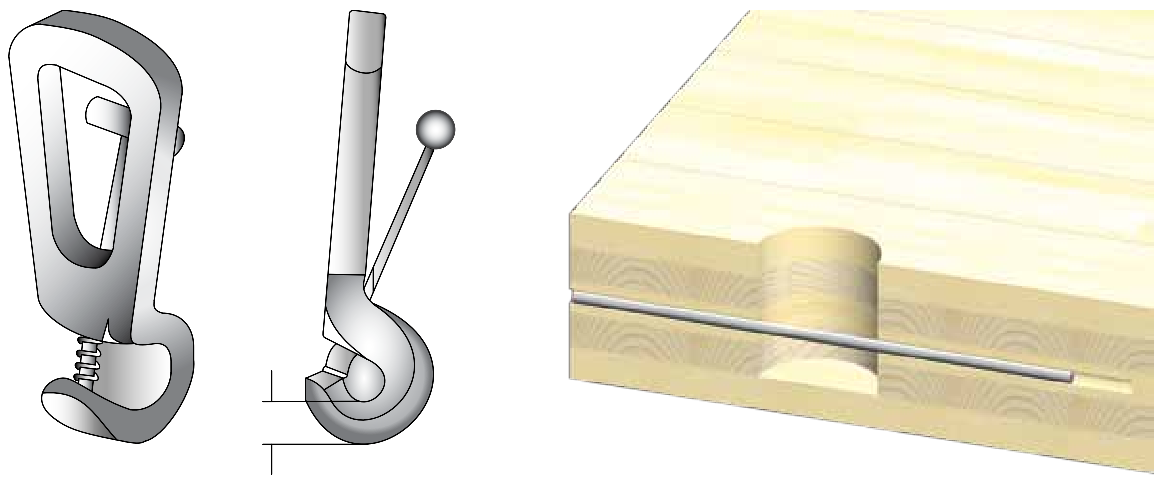


Figure 30
Inserted rod with lifting hook

3 GENERAL PRINCIPLES FOR LIFTING AND HANDLING CLT ELEMENTS

There are several types of lifting equipment that can be used on construction sites. Each has its own characteristics for lifting and handling heavy loads such as CLT panels. It is therefore essential to choose the right lifting and handling system for each type of component.

It is also of the outmost importance that lifting equipment be installed and operated properly. Several criteria must be verified and validated prior to and during work on site. Engineers and builders in charge of a construction project involving CLT panels need to consider certain important points. Some of these considerations are presented in the following sections.

3.1 Lifting Station and Devices

The lifting station is undoubtedly a key element of the construction site. The lifting device must be selected and positioned according to several criteria. Certain construction sites may require more than one lifting stations and some sites may need to change the type of device being used during the construction phase.

Here are some of the elements to be considered when choosing a lifting device. The device must, without limitation:

- Be able to lift all required loads for the duration of construction:
  - Types of loads may vary on the same construction site;
  - The lifting device should not be moved;
- Reach appropriate height with required maximum load:
  - Appropriate range must be attained for all required distances, from point A to point B;
- Be fast, keep a good pace and be flexible, while keeping safety first.

In addition, consideration must be given to the type of land upon which the construction will be done, as well as the immediate surroundings. To avoid nasty surprises, it is strongly recommended to inspect the site before choosing the type of device.

The grounds (slopes, streams, etc.) and the soil's bearing capacity (sand, clay, etc.) are important points to consider. As well, the stability of the operating devices must be maintained at all times. For example:

- A crane can collapse under the weight of excessive load;
- The ground can erode under the device's bearing points;
- A device that is too close to a slope can cause a landslide and tip over;
- The device's range may be too large. The device can come into contact with obstacles and fall (e.g., buildings, trees, a second crane, power lines, etc.).

Despite all precautions that can be taken, accidents may occur. Thus, it is strictly forbidden to handle loads directly above workers or the public. Also, to avoid serious accidents, the worker in charge of positioning the slings should never stand between the load to be lifted and a fixed object, in case of load instability or improper operation during lifting. Other safety-related recommendations are available from regulatory authorities.

3.2 Determining the Weight and Center of Gravity of CLT Elements

Before choosing the proper lifting system, it is important to know the total weight of the element to be lifted, as well as the position of its center of gravity.

Although density of wood greatly varies depending on wood species and moisture content, i.e. between 320 and 720 kg/m^3 (Wood Handbook, 1999), it is generally recommended to use an average density varying between 400 and 600 kg/m^3 for the calculation of the total weight of CLT elements. As a result, the density used for evaluating lifting system loads will vary between 4 and 6 kN/m^3 for CLT elements. Note that this density is about five times lower than the density used for prefabricated reinforced concrete elements, which are usually between 23 and 25 kN/m^3. Nevertheless, the total weight of CLT elements can be considerable. As an example, a 2.4 m x 16 m x 300 mm thick CLT slab weighs about 6 tonnes (≈ 60 kN). We suggest using values provided by manufacturers.

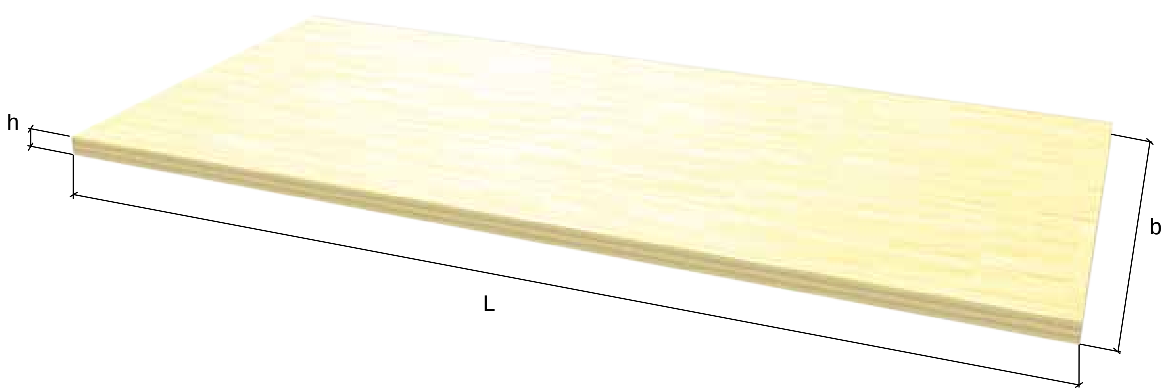


Figure 31
Calculating the weight of a CLT element

The total weight of a CLT element is simply calculated as follows:

$$P = V \times \rho_{CLT} \quad [1]$$

$$V = b \times L \times h \quad [2]$$

where:

P = CLT element weight (kN)
V = Volume of element to be lifted and handled (m^3)
b = Element width (m)
L = Element length (m)
h = Element thickness (m)
ρ_{CLT} = CLT element average density (4~ 6 kN/m^3)

3.3 Dynamic Acceleration Factors

3.3.1 Lifting System Used

During lifting and handling manoeuvres, elements are subject to dynamic forces that must be taken into account. These forces mainly depend on the chosen system, the lifting speed, and the type of ground on which the elements are being handled.

Table 1 provides an overview of recommended lifting and handling dynamic acceleration factors for specific devices used in construction. These factors should be taken into account for the calculation of forces. The values are provided for informational purposes only. It is important to refer to normalized values as provided by the relevant authorities (e.g., provincial, federal, municipal, etc.).

Table 1
Dynamic acceleration factors (f)

Lifting Device	Dynamic Coefficient of Acceleration f
Fixed crane	1.1 ~ 1.3
Mobile crane	1.3 ~ 1.4
Bridge crane	1.2 ~ 1.6
Lifting and moving on flat terrain	2.0 ~ 2.5
Lifting and moving on rough terrain	3.0 ~ 4.0 and +

Sources: PFEIFER, SNAAM, HALFEN, PEIKKO, ARTEON

3.3.2 Other Effects to Consider

Wind can significantly increase forces in lifting systems. Engineers in charge of a project must consider such loads in their calculations based on the surface in contact with the wind as well as the location and height of assemblies requiring lifting.

However, it is normally forbidden to lift loads when weather conditions are deemed dangerous. Prefabricated CLT elements are subject to wind movement and this phenomenon should not be underestimated.

Guide ropes may sometimes be required to prevent rotation of assemblies during lifting.

Finally, it is recommended that each lifting job be performed in a single operation or in compliance with (sequence of) operations intended by the engineer.

3.4 Dissymmetrical Distribution of Load According to Center of Gravity

It is always better to fix anchors in a way to limit the eccentricity due to the center of gravity of the element to be lifted. If anchors are dissymmetric with regard to the center of gravity, forces will not be equally distributed during lifting and must be calculated accordingly. Tensile and shear forces must be calculated for each component to be lifted, or the most critical elements must be taken into consideration.

Furthermore, to limit the tilt and sway of parts during lifting and handling, it is possible to use a spreader system. Simply align the center of gravity of the element as calculated exactly facing the hook installed on the spreader beam to prevent rotation. Figure 32 shows the appropriate method. However, if the lifting of an element is done without a spreader beam, which is often the case in CLT construction, it is important to check the balance of the load when lifting. Wind can also swing and spin the load.

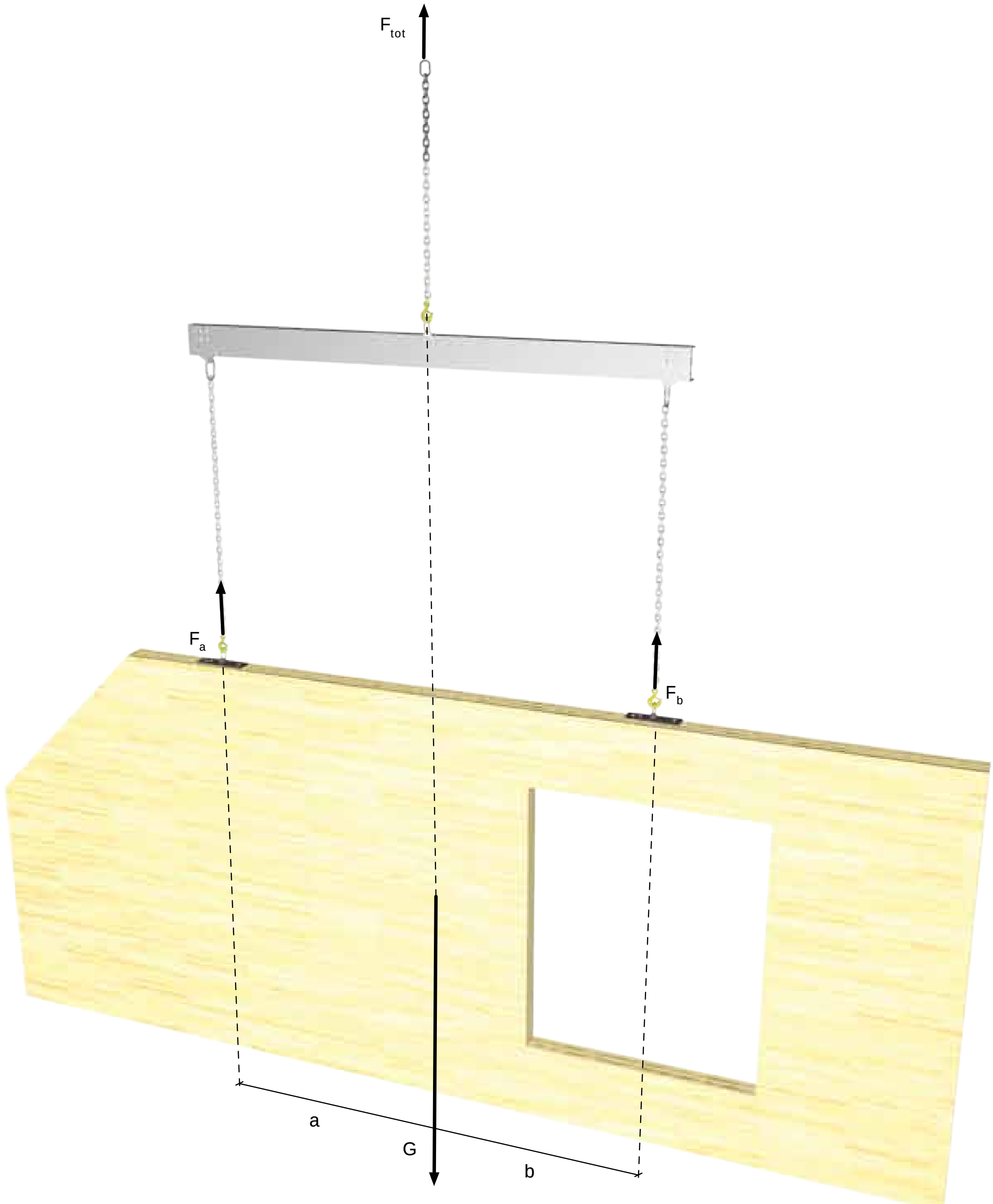


Figure 32
Element lifted with a spreader system

For example, the next equations are used to calculate forces in two anchors placed dissymmetrically to the center of gravity of an element that is being lifted with a spreader system (note here that $F_{tot} > G$).

$$F_a = \frac{P \times b}{(a + b)} \quad [3]$$

$$F_b = \frac{P \times a}{(a + b)} = P - F_a \quad [4]$$

3.5 Determining Forces According to Lifting Angles

When a spreader system, similar to that shown in Figure 32, is not used when handling assemblies, it is necessary to adjust forces in the anchors by taking into account the lifting angles. In this case, the inclination angle of the cables or slings will vary depending on their length.

The adjustment is done by evaluating the coefficient of angle z. A range of coefficients z is presented in Table 2 according to the inclination angle β. Refer to Figure 34 for more details about angles.

Table 2
Coefficient of lifting angle (z)

Cable Angle ß (1)	Angle α (2)	Angle Coefficient z (3)
0°	0°	1.000
7.5°	15°	1.009
15°	30°	1.035
22.5°	45°	1.082
30°	60°	1.155
37.5°	75°	1.260
45°	90°	1.414
52.5°	105°	1.643
60°	120°	2.000

(1) It is strongly recommended to limit β to 30°
(2) α = 2 x β
(3) z = 1/cos β

3.6 Determining Load Distribution According to the Number of Effective Anchors (Suspension in Several « N » Effective Points)

It is common practice to use only two anchor points when CLT wall or beam elements are handled on the construction site. In such cases, it is normally sufficient to determine forces in the two anchors (N=2) according to the position of the center of gravity, the lifting system, and the lifting angle.

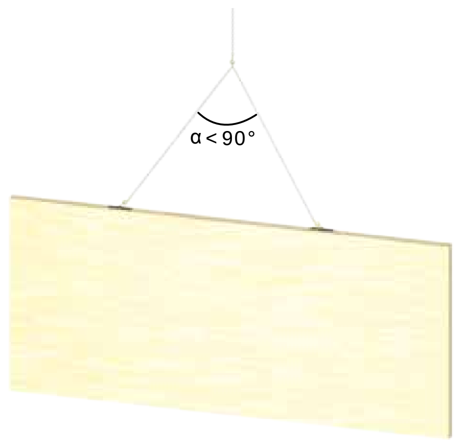
However, for floor and roof slabs, or for long wall assemblies, the use of three or four anchors is generally required. Thus, if more than two anchors are used, it may be impossible to accurately determine the load applied to each anchor, even when anchors are positioned symmetrically to the center of gravity. Indeed, there is no guarantee that the load will be perfectly symmetrical to the center of gravity or that the slings will be exactly the same length. It is therefore necessary to correctly establish the maximum force by using only two effective anchors (N=2).

In special cases, for example, when the loads are not precisely known, or the element is irregular in shape, each anchor shall be calculated so as to be capable of supporting the total load of the assembly (N=1).

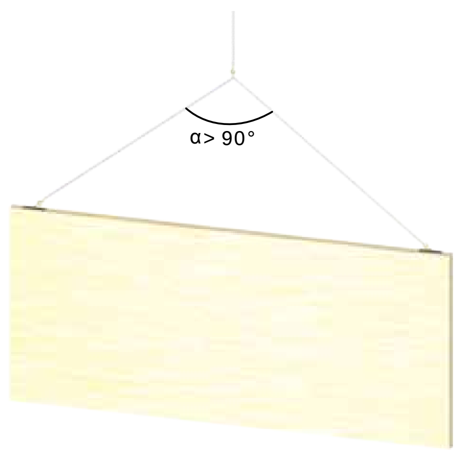
Furthermore, to ensure proper distribution of forces in each anchor considered effective, it is important to use systems with no friction. The use of free spreaders, pulleys, or shackles helps reduce unwanted friction.

Note that in all cases, it is recommended not to use excessively long slings so as to avoid instability or to create high angles when lifting. Also, if assemblies that require lifting and handling are too long, the use of a spreader system might be a better option, as it will limit the length of the slings.

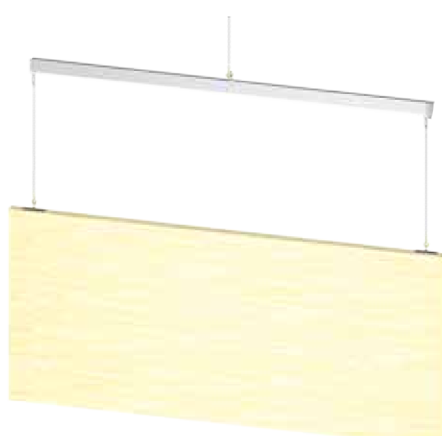
Figures 33 to 40 present typical cases of CLT element lifting and the number of effective anchors used in the calculations.



Proper angle

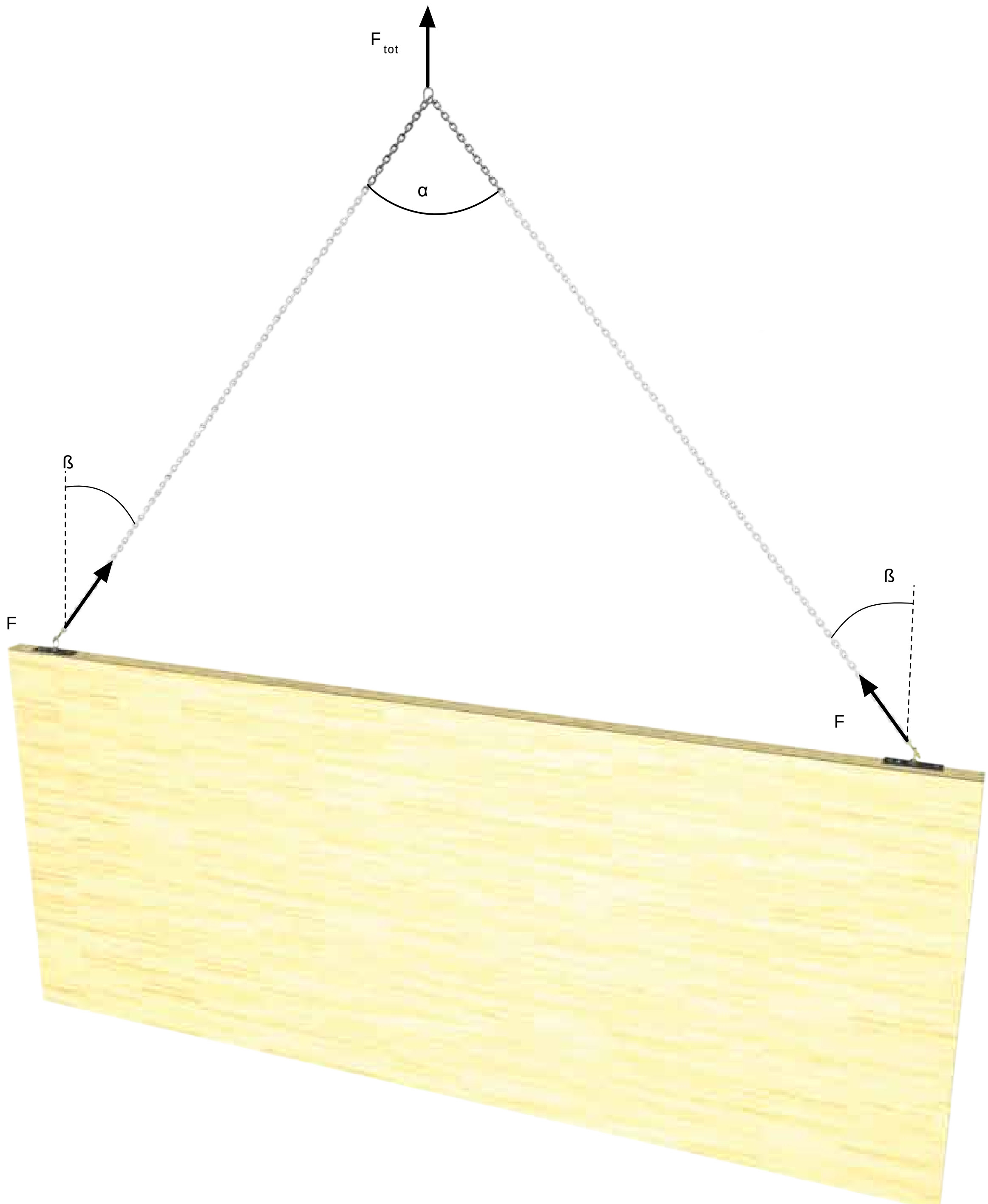


Angle too sharp



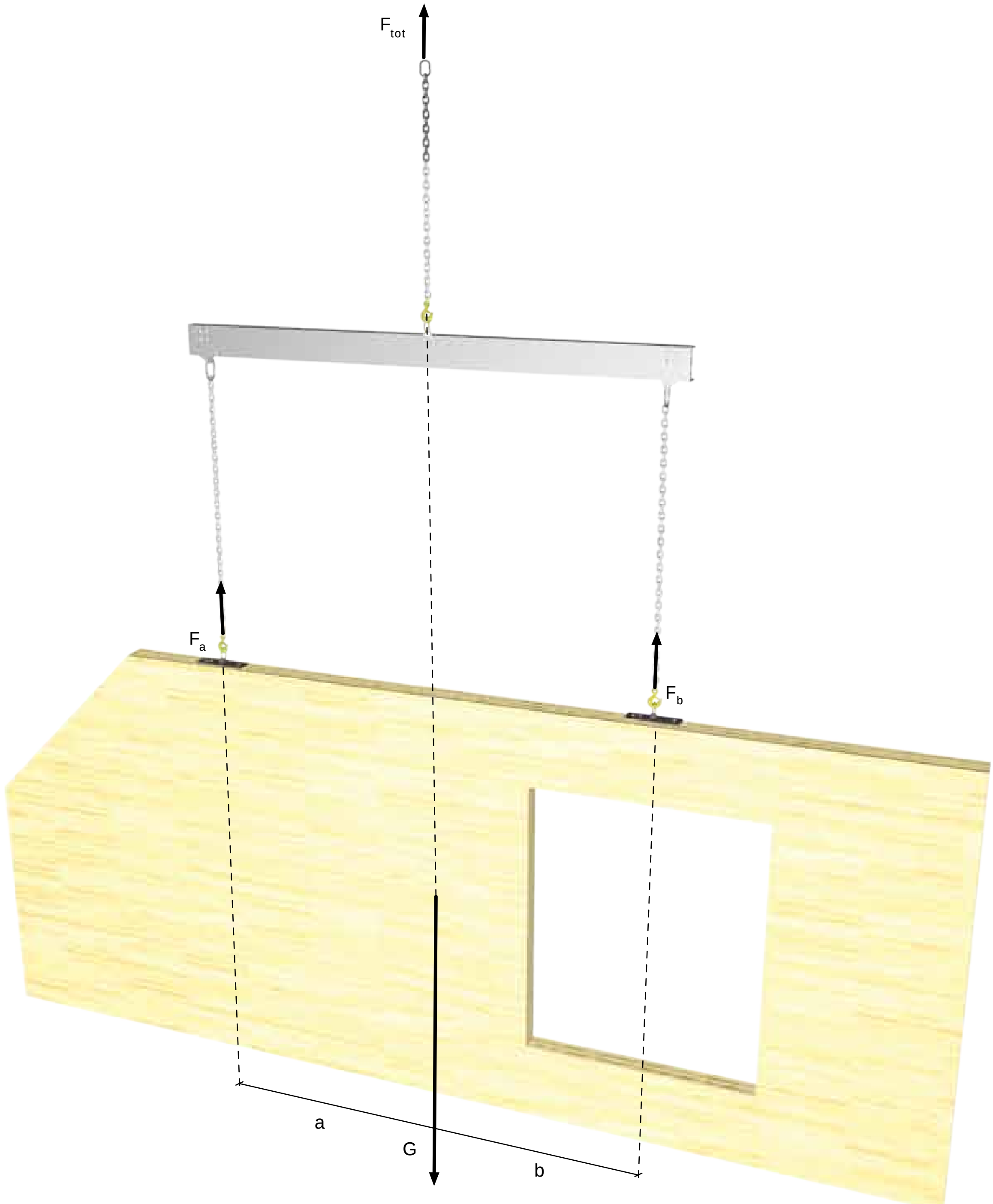
Better performance using spreader

Figure 33
CLT wall lifted with two slings symmetrically positioned – Good and bad practices



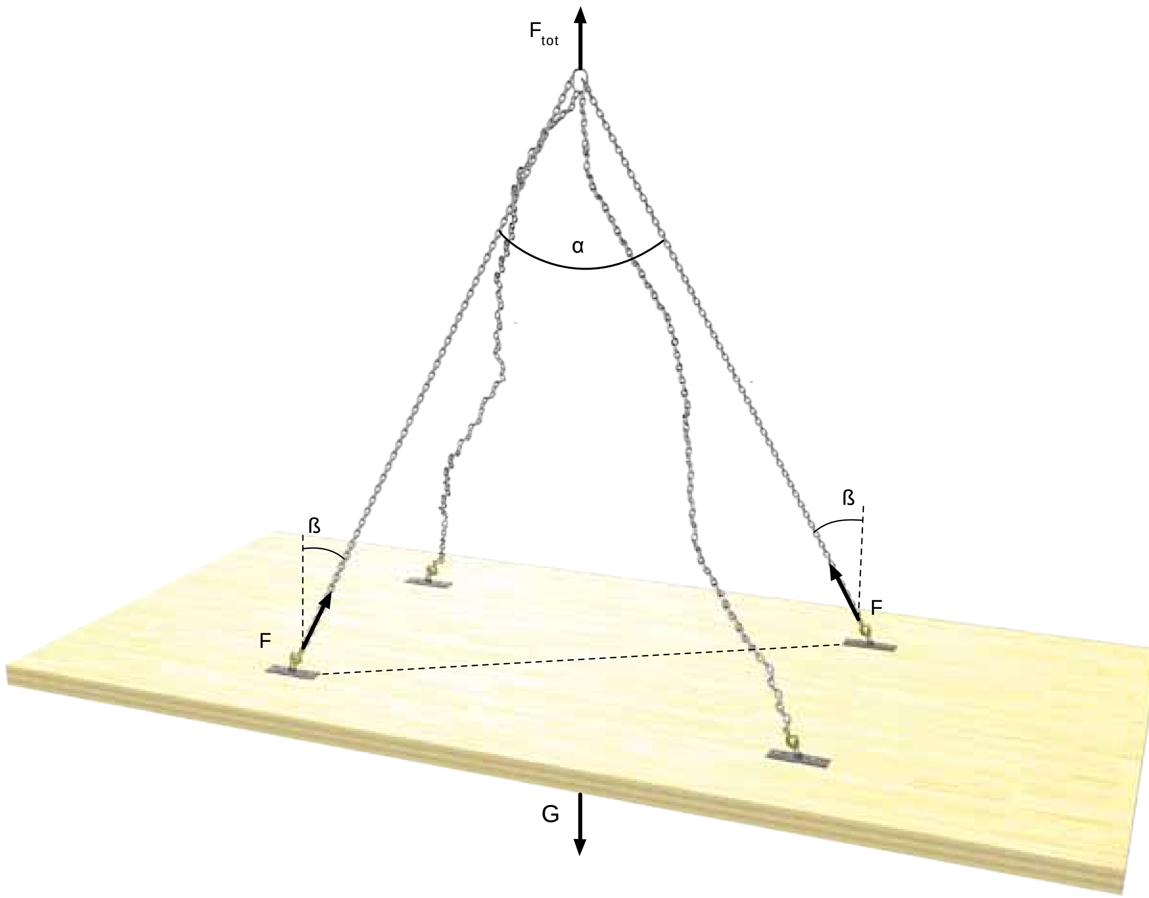
Number of effective anchors = 2

Figure 34
CLT wall lifted with two slings symmetrically positioned to the center of gravity (N=2)



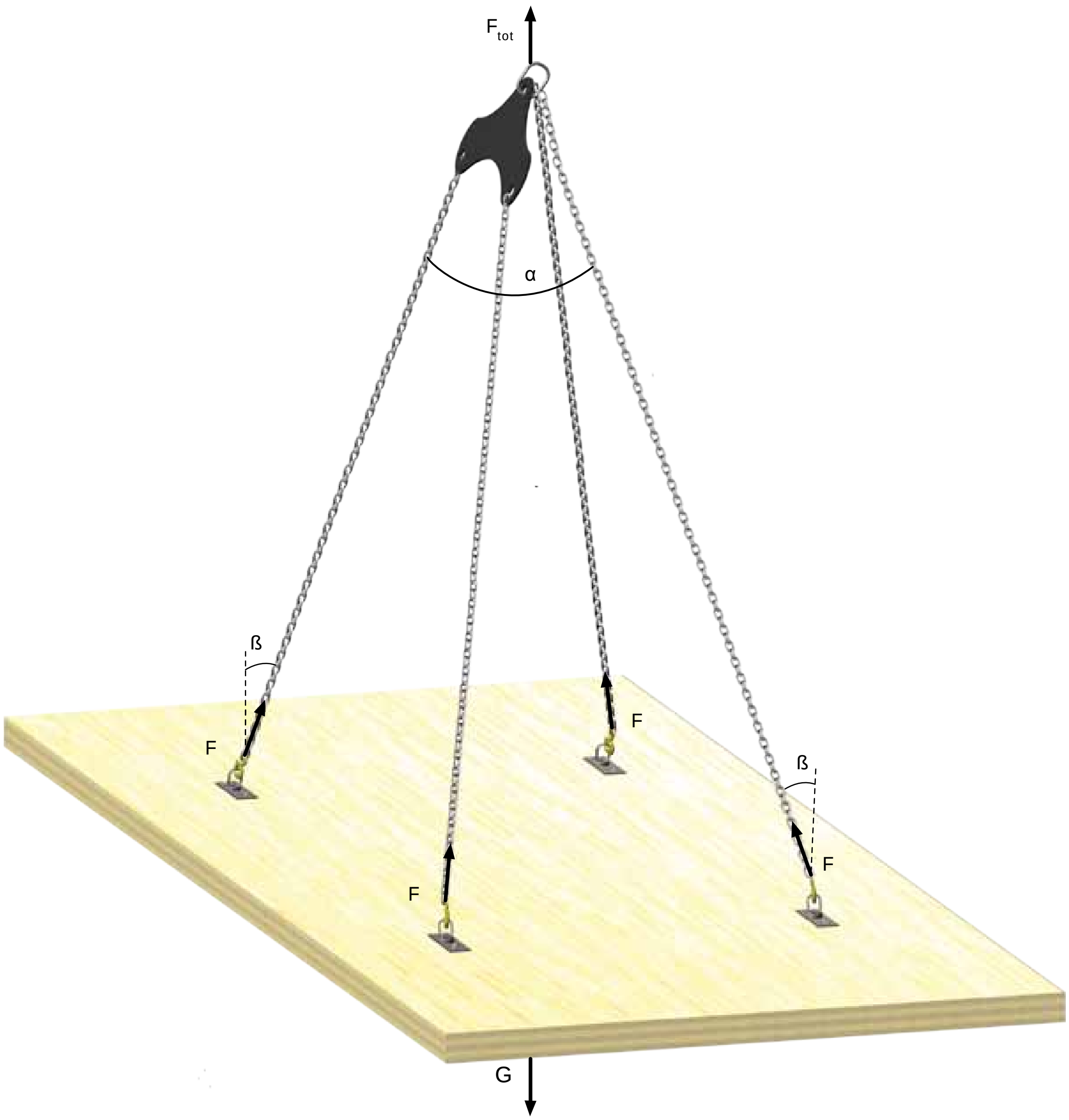
Number of effective anchors = 2

Figure 35
CLT wall lifted with two slings asymmetrically positioned to the center of gravity, with single spreader (N=2)



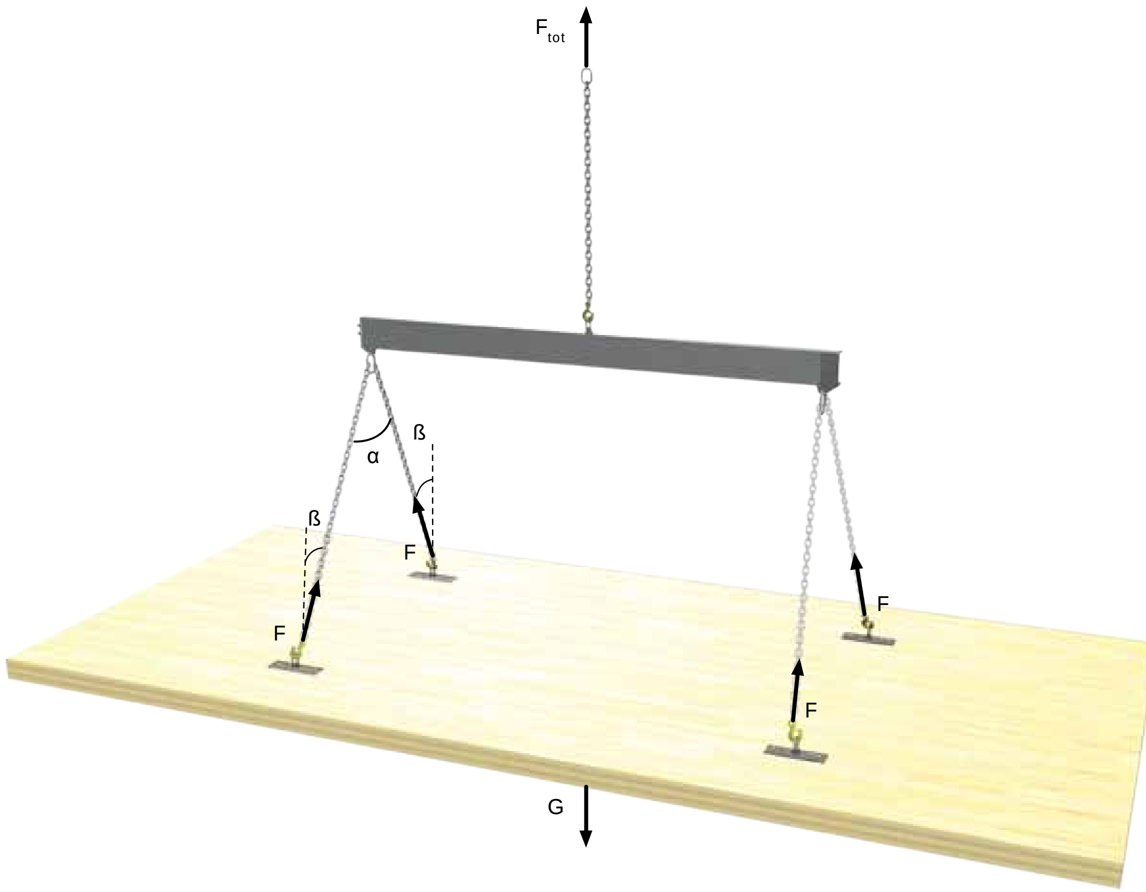
Number of effective anchors = 2

Figure 36
CLT slab lifted with four slings symmetrically positioned to the center of gravity, without spreader and without compensation system (N=2)



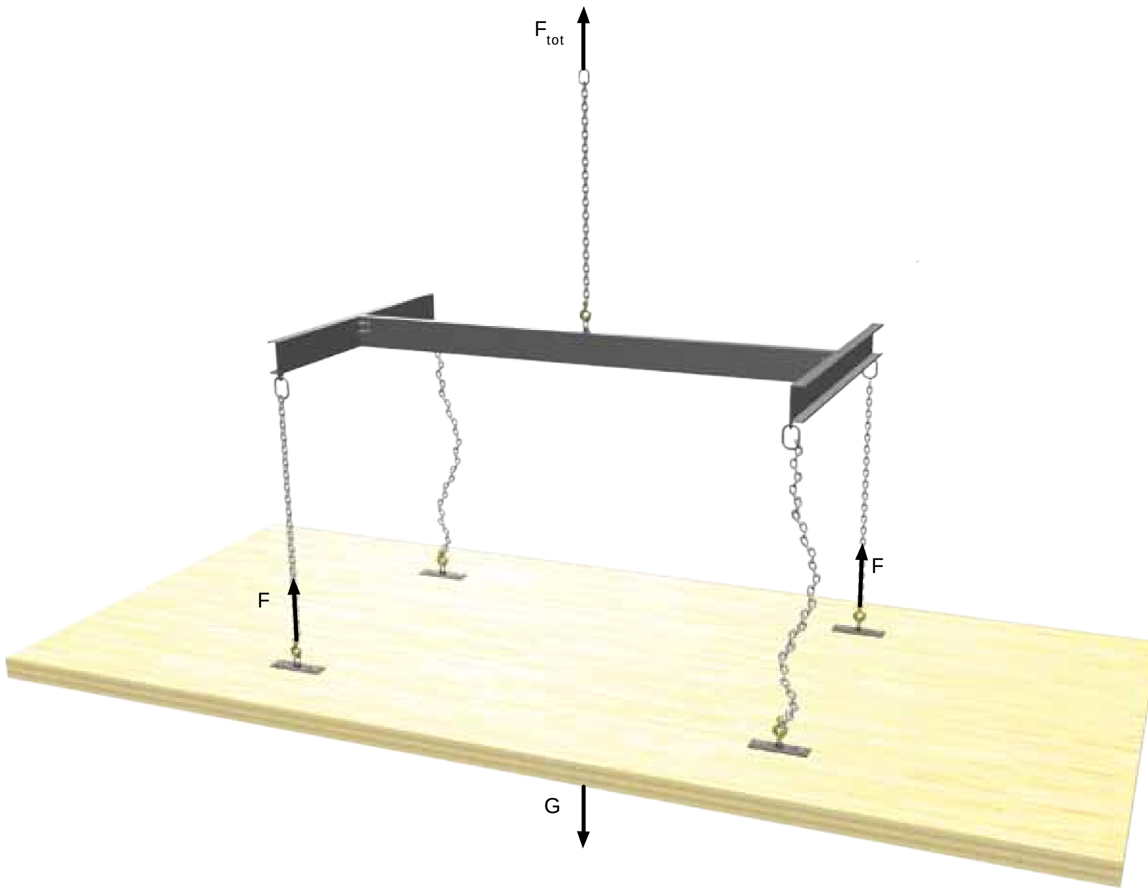
Number of effective anchors = 4

Figure 37
CLT slab lifted with four slings symmetrically positioned to the center of gravity, with compensation system (N=4)



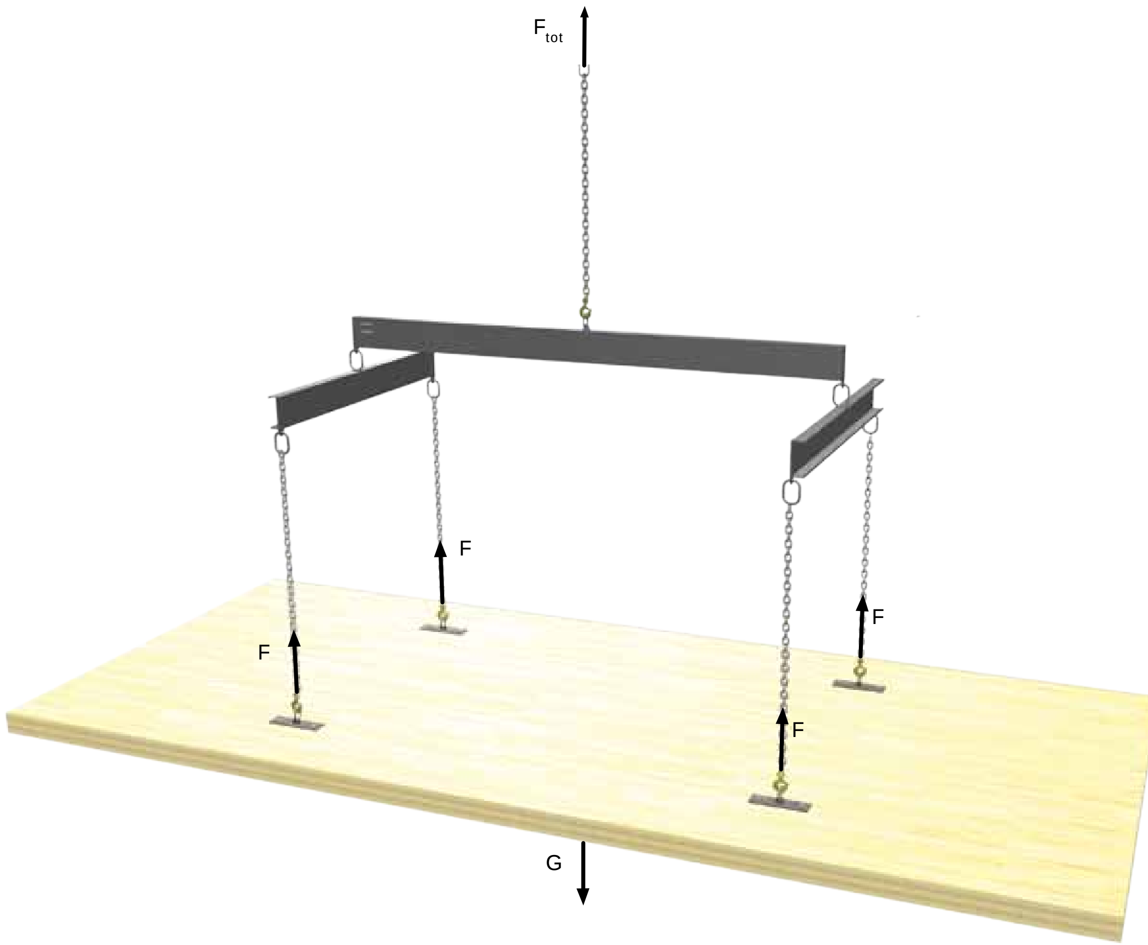
Number of effective anchors = 4

Figure 38
CLT slab lifted with four slings symmetrically positioned to the center of gravity, with single spreader (N=4)



Number of effective anchors = 2

Figure 39
CLT slab lifted with four slings symmetrically positioned to the center of gravity, with three fixed spreaders (N=2)



Number of effective anchors = 4

Figure 40
CLT slab lifted with four slings symmetrically positioned to the center of gravity, with three free spreaders (N=4)

3.7 Calculation of Forces Resulting from Lifting per Anchor

The maximum forces resulting from lifting using anchors must be evaluated at each stage of lifting and handling. The maximum unfavorable value will determine the design of the lifting systems.

For example, different lifting systems can be used in the plant and on site (e.g., travelling crane in the plant vs. regular crane on construction site). Furthermore, a component can be raised and handled in several stages and with slings of different lengths. Also, if the same lifting systems are used more than once during handling between the plant and its final destination, it may be required to use an oversize anchor to accommodate the effects of repetition.

For loads that require lifting with slings placed symmetrically to the center of gravity, force per anchor is calculated as follows:

$$F_i = \frac{F_{tot} \times f \times z \times \mu}{N} \qquad [5]$$

where:

F_i = Resultant anchor force i (kN)
F_{tot} = P = Total weight of assembly to be lifted (kN)
f = Dynamic acceleration factor (Table 1)
z = Coefficient of lifting angle (Table 2)
μ = Other majoration coefficient if required
N = Number of effective fasteners (see figures)

Finally, tensile and shear stress in fasteners can be established based on the lifting angle. The anchoring system can therefore be correctly designed by the engineer by taking into account the CLT element to be lifted (i.e., wood density, number of CLT layers, direction of the grain, etc.) and the lifting system chosen.

Important notes:

- If anchors are not symmetrical to the center of gravity, they must be increased by using the appropriate static equations (see [1] and [2]).
- Other effects such as wind may significantly influence load movement on lifting systems.
- If the same lifting system is used for rising, it may be necessary to reduce the allowable anchor capacity in the calculations.
- In Canada, calculations must be done using limit states. It is important to ensure that the calculated and provided capacities of anchor systems are compatible.
- Laboratory tests may be required.

4 OTHER ACCESSORIES AND MATERIALS

Numerous construction accessories and materials are required on a construction site. In this section, in addition to the items and tools normally required in conventional wood construction, we suggest products, tools, and accessories that may be useful or essential on a construction project using CLT panels.

4.1 Fire-resistant Rope and Joint Sealing Tapes

To ensure proper sealing of CLT panel joints (i.e., floor-to-floor or floor-to-wall junctions), it is recommended to use products that are specifically intended for this purpose. There are plenty of products on the market.

Typically, the proposed products should:

- Help reduce sound transmission through floors and walls;
- Ensure effective protection against fire and gas;
- Improve energy efficiency by reducing heat loss and by limiting air flow (for CLT elements that are part of the enclosure).

Fire

Fire-resistant ropes are normally cylindrical and flexible. Some products are made from incombustible mineral fiber inserted into a fiberglass wire netting. These ropes must provide effective protection against fire and gas for a sufficient time.

Acoustics

Acoustic membranes or tapes are specifically designed and formulated to effectively stop sound transmission between walls and partitions. Some suppliers also indicate that the tapes are used to control the vibrations of floor slabs (damping).

Air

To ensure air tightness, polyethylene foam-type products are often used on concrete foundation joints and on the roof. Other types of membranes (e.g., rubber-based) can be used.

Figures 41 to 43 show some examples of tight joints between CLT elements.

Figure 41
Sealing joint between floor, wall, and connectors

Figure 42
Joint between floor and wall with semi-rigid membrane

Figure 43
Joint between two floor slabs with flexible membrane

4.2 Adjustable Steel Shores

During frame assembly, it is crucial to have the right tools at hand. Figure 44 shows adjustable steel shores for adjusting wall plumbs. Shores can be adjusted with screws or with steel dowels that can be placed at frequent intervals. This instrument is essential to ensure a precise angle of installation. The fastening at both ends is done with screws. If the CLT panels are to remain visible, repairs may be required when the operation is complete.

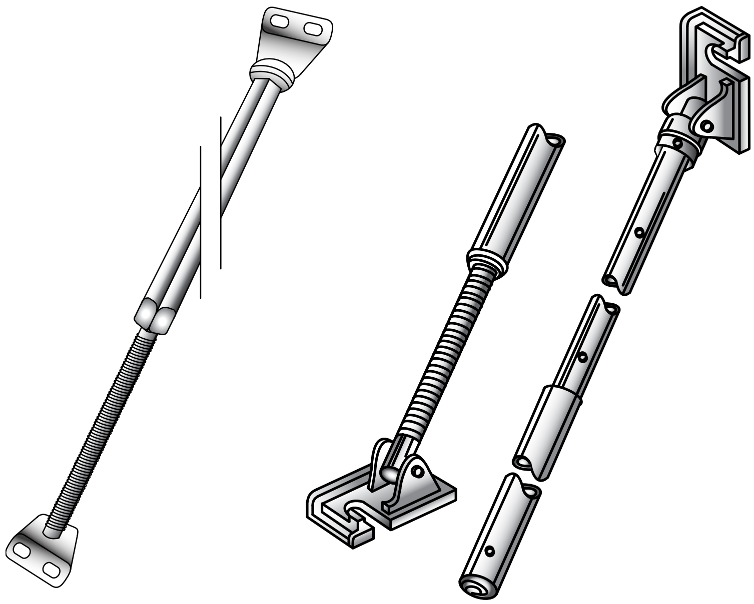


Figure 44
Adjustable shores for walls

4.3 Beam Grip with Ratchet and Hooks

Figure 45 shows a beam grip with ratchet and hooks. This instrument is primarily used to bring the CLT panels together once they are supported and juxtaposed. It is necessary to use this type of instrument to ensure that there is proper contact between wall, floor, or roof panels. Figure 45 shows a beam grip being used to bring two floor panels together. It can be noticed that the forged hooks have been driven in line with the exterior walls that will be subsequently installed. If the floor must remain visible, it is essential to position the beam grip strategically so as not to mark the wood.

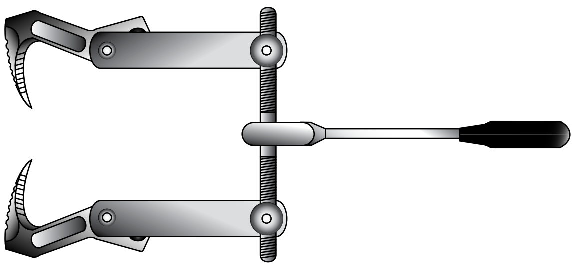


Figure 45
Beam grip with ratchet and hooks

4.4 Beam Grip with Ratchet and Screw Plate

The beam grip can also be used to ensure proper contact between two panels that are installed perpendicularly. Instead of hooks, the beam grip is used with two perforated plates. The beam grip is screwed onto the CLT wall and roof elements. The clamping is then performed and the panels are screwed to one another using self-tapping screws or other systems (refer to Chapter 5 for more information). Tightening will ensure proper contact between the elements to limit air infiltration and sound transmission. Note in Figure 46 the weatherproofing membrane used at the junction of the panels.

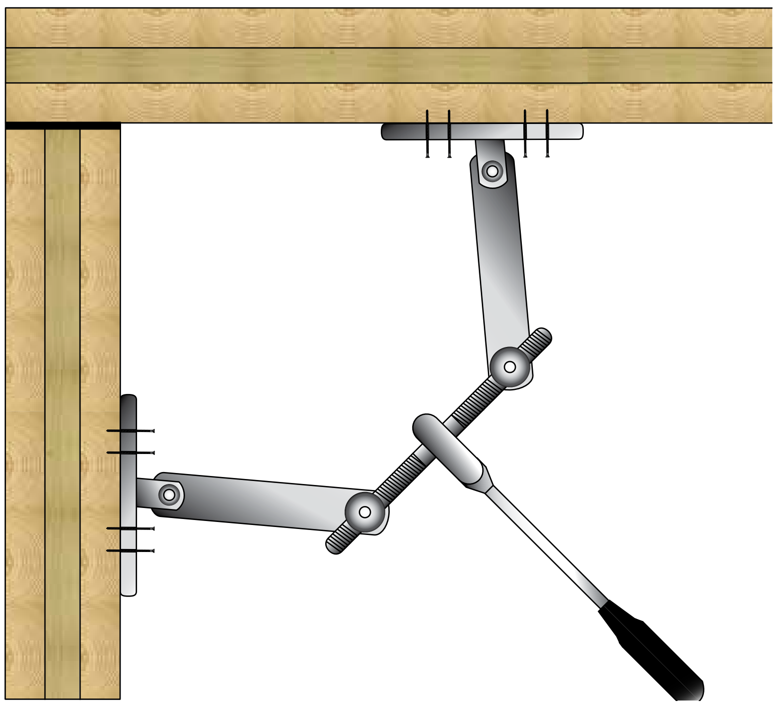


Figure 46
Beam grip with ratchet and screw plate

4.5 Manual Winch with Cables or Slings

Instead of a beam grip, a manual winch attached to cables or slings can be used to bring the CLT panels together. Figure 47 shows the system in use. Steel plates are installed on the panels with screws or lag screws. A flexible sling is used as the link between the winch and the plate. Once proper contact has been made between the panels, they are assembled using self-tapping screws or wood screws (refer to Chapter 5 for more information).

Figure 47
Manual winch used with soft slings

4.6 Steel Shims and Cement-based Grout with no Shrinkage

It is sometimes necessary to use steel shims of different thicknesses under CLT walls, at the junction of concrete foundations, for them to be perfectly square. Once the wall has been properly installed and is at a right angle, the gap is usually filled with a cement-based grout. It is imperative to use a waterproof membrane at the base between the concrete and the wood to limit the migration of water into the wood.

Figure 48
Junction between concrete foundation and CLT walls with steel winch and cement-based grout without shrinkage

5 TRANSPORTATION OF CLT ELEMENTS

Before undertaking the design of a CLT building, consideration must be taken with regards to the transportation of the prefabricated CLT elements. Transporting CLT panels can be costly and, depending on the size of the element, may require specialized transportation services.

As shown in Chapter 1, CLT panels can be quite large. Typical panel widths are 1.2 m, 2.4 m, and 3 m, while maximum lengths are dependent on the press type and may reach 18 m. As well, panels can be quite heavy. Because of the potential size and weight of the elements, there are two main factors regarding transportation that must be considered when planning CLT elements: highway regulations and construction site limitations.

5.1 Standard Weights and Dimension Regulations

In Canada, vehicle weights and dimensions (W&D) fall within provincial jurisdictions and limits vary from province to province. However, the provinces and territories have agreed on National Standards for the weight and dimension limits of heavy vehicles used in interprovincial transportation. These are contained in a Federal/Provincial/Territorial Memorandum of Understanding (MOU). Under the terms of the MOU, each of the provinces and territories will permit vehicles which comply with the appropriate weights and dimensions described in the agreement to travel on a designated system of highways within their jurisdiction. Keep in mind, however, that the provinces are allowed (and many do) to set more liberal W&D within their jurisdictions. More information on the MOU may be obtained by visiting the Council of Ministers Responsible for Transportation and Highway Safety website.

5.1.1 Dimension Limits

In terms of dimension limits, here are the main points with regard to road vehicles (according to dimensional limits applicable to the U.S., which are slightly more restrictive than Canadian limitations):

- Vehicle height, including load, is limited to 4.11 meters (13’6”);
- Vehicle width, including load but excluding load covering or securing devices, cannot exceed 2.6 meters (102”);
- Semi-trailer length, including load, cannot exceed 16.15 m (53’).

Figure 49 presents these limits in a graphical format.

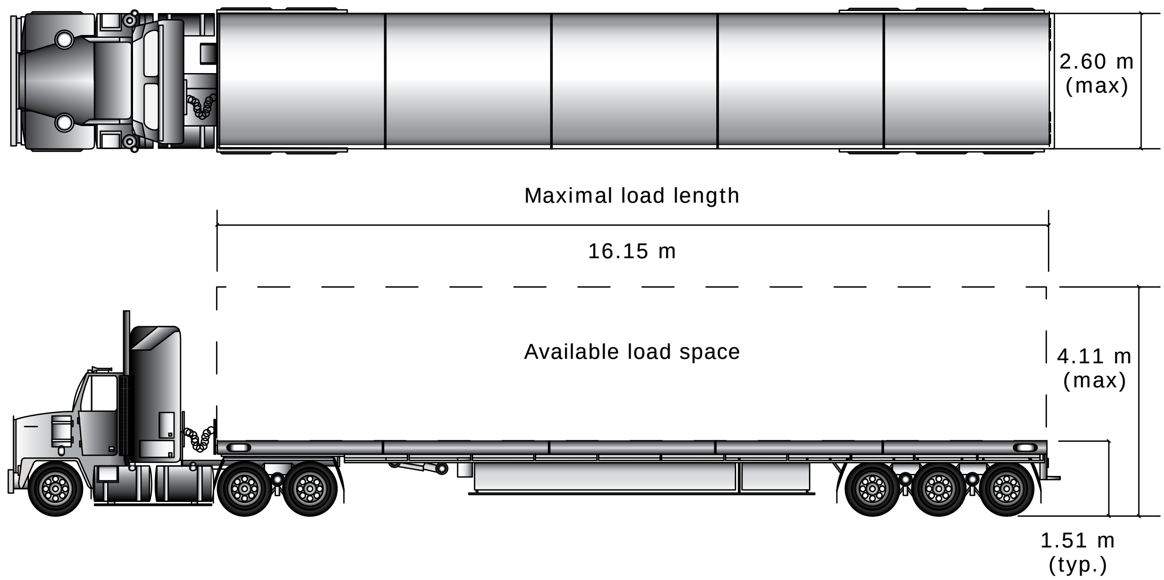


Figure 49
Available load space on a flatbed semi-trailer

The majority of CLT panels will be transported by the use of a flatbed semi-trailer (Figure 50). These trailers have the advantage of being open on all sides, which facilitates loading, and having a continuous deck space from front to back. Given that the normal height off the ground of the deck of a flatbed semi-trailer is about 1.51 m (at the front of the trailer, which is the highest point), this permits load heights of 2.60 meters. Overall, this means that a CLT load, comprised of one or more elements, must fit into a box with a height of 2.6 m, a width of 2.6 m, and a length of 16.15 m if it is to be transported by a flatbed semi-trailer.

For taller structures, dropdeck (also called stepdeck) semi-trailers can also be used. However, as can be seen in Figure 51, unlike flatbed semi-trailers, the deck of a dropdeck is not continuous. A dropdeck flatbed with smaller 255/70R22.5 type tires (but still using normal axle hubs and brakes) can be used to allow a 3 m tall load on the rear 12.8 m section and a 2.6 m tall load on the front 3.35 m section.

Other semi-trailers with even more load height are available, such as doubledrop decks (Figure 52), but they can be difficult to load, and the deck is divided into three sections with the lowest section having a length of about 9 m and a deck height of 0.55 m, allowing products of up to 3.56 m in height.

Although all of these semi-trailer types can be as long as 16.15 m, many are 14.63 m (48') in length.
The dimensions given here are presented as guidelines.

It is important to check with transportation providers to verify the dimensions of their vehicles before going forward with any transportation plan.

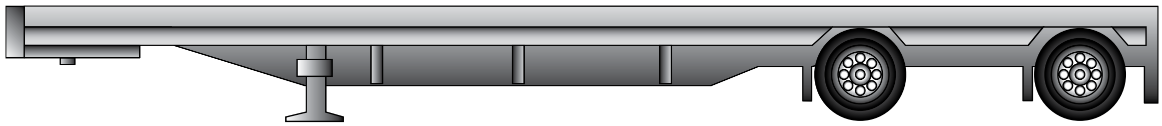


Figure 50
Flatbed semi-trailer

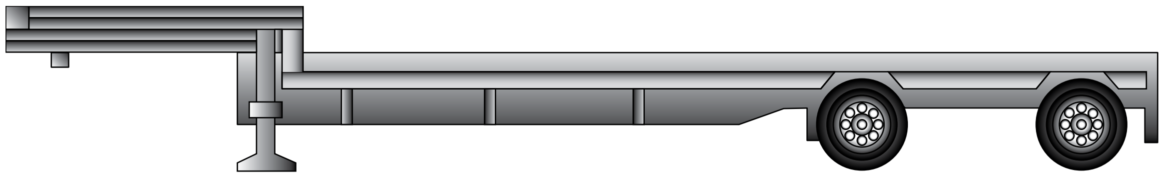


Figure 51
Dropdeck semi-trailer

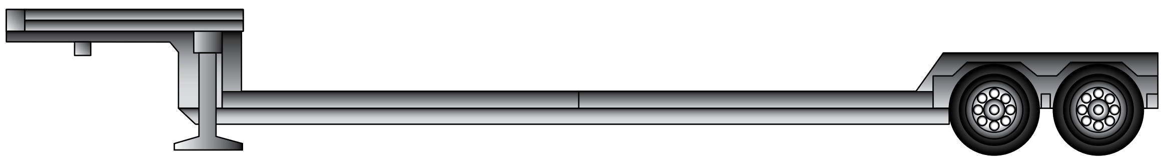


Figure 52
Double dropdeck semi-trailer

5.1.2 Weight Limits

When it comes to weight limits, the situation is more complex. Legal Gross Vehicle Weight (GVW) is the weight of the vehicle and its load. Legal GVW varies not only by province, as previously mentioned, but also by the type of vehicle, the number of axles on the vehicle, and the distance between the axles. Nonetheless, a simplified picture can be drawn. When delivering within Canada, 6-axle semi-trailer combinations (e.g., a tandem drive tractor with a 3-axle semi-trailer) can be used in every jurisdiction although at different allowable GVWs. In the USA, tractor/semi-trailer combinations are limited to 5 axles.

Table 3 presents the maximum payloads authorized with 5- and 6-axle flatbed combinations by jurisdiction, taking into account the typical tare weights for these units (14.5 t for a 5-axle unit and 16 t for a 6-axle unit) and the legal GVW in each jurisdiction. It should be kept in mind that these are only guidelines. It may be possible to have higher payloads with some of the superlight trailers available on the market. Also, trucks are limited in the amount of weight that different individual axles or axle groups can carry. With odd-shaped loads, it is often difficult to distribute the load properly between axles and thus the legal GVW cannot be obtained while maintaining legal axle or axle group weights.

Table 3
Maximum payloads by jurisdiction for 5- and 6-axle tractor/semi-trailer combinations (t)

Territory	5-axle Combinations	6-axle Combinations
MOU*	23.0	28.5
Atlantic Provinces and Québec	25.0	31.5
Ontario†	25.0	33.1
USA	20.0	-

*Manitoba, Saskatchewan, Alberta and B.C. limits all follow the MOU
† Although higher GVW may be allowed in the regulation, we have included the highest practical GVW

5.1.3 Other Canadian Legal Configurations

Québec also allows the use of 4-axle semi-trailers while Ontario allows 4- and even 5-axle semi-trailers with much higher payloads. Given that these vehicles cannot travel outside their jurisdictions, we have not presented payload maximums for these types of units. As well, the Canadian MOU allows the use of 8-axle B-train units (a tractor pulling two semi-trailers; see Figure 53) at a GVW of 62.5 tonnes. However, the length of both trailers combined is 20 m, with a lead trailer typically having a deck length of 9.75 m and a rear trailer with a deck of 8.5 m. Because each trailer unit articulates separately (steering and suspension systems), a load cannot span from the deck of the lead unit to the rear unit. As such, the longest panels that super B-trains can accommodate are 9.75 m. Typical tares are in the range of 18 t, so loads of up to 44.5 t are possible.

Different possible configurations are also available in the USA, the most common being spread tandem axle semi-trailers. In these configurations, the space between the two axles of a tandem group is increased from the standard 48 inches to a space reaching up to 121 inches.

Figure 53
Super B-train flat deck combination

5.2 Oversize and Overweight Permits

In every Canadian and U.S. jurisdiction, oversize and overweight permits are required when the dimensions or weight of a vehicle exceeds the normal limits permitted by legislation. Larger CLT panels exceed these dimension limits and a truckload of panels may also cause the vehicle to exceed the legally allowable Gross Vehicle Weight. Keep in mind that these permits are only available for indivisible loads.

The regulations, permitting, and logistics of oversize and overweight transportation are quite complex. The planning and organization of such hauls is best left to transport companies that specialize in this type of work. If it is determined that CLT elements do not fit in the standard legal dimensions or weights described in Section 5.1, it is important to contact one of the specialists. For more information on oversize and overweight permitting, refer to local provincial or state authorities. A complete list may be obtained on the U.S. Federal Highway Administration website.

5.3 Construction Site Limitations and Considerations

Transporting CLT elements to the construction site is only part of the challenge. The construction site itself may have restrictions that are more limiting than weights and dimension regulations. First off, make sure that the route from the plant to the construction site will allow movement on the truck, including its load, without any obstacles. This is especially critical for oversize loads.

A common problem at construction sites occurs when a long trailer arrives and the width of the driving space (which was fine for a short dump truck) does not allow enough clearance for the off-tracking of the rear trailer wheels when a short radius turn is needed. Moving a fence, a shed, piles of materials, for example, to make driveway changes can disrupt and delay deliveries and increase costs.

This can be a challenge when working in tight urban areas where the space for piling building materials and the allowance for turns is very limited. The off-tracking is a function of the sum of the squares of the vehicle combination wheelbases so an extra-long trailer will intrude inward on a tight turn much more than shorter wheelbase trailers. A data chart and other methods to estimate off-tracking (SAE J 695) are available to the Society of Automotive Engineers.

Awareness of local city regulations and pre-planning to match construction site challenges are advisable to ensure a smooth efficient delivery without delays and cost overruns.

5.4 Other Transport Considerations

It is a large advantage to design the loads to fit on normal equipment which allows the option to use for-hire carriers to deal with long distance one-way hauls where many loads must arrive and be staged at a jobsite within a close period of time. It also reduces the vulnerability by having access to replacement vehicles when a specialized vehicle has downtime and to deal with swings in demand.

When normal flatbeds are used, it is generally best to lay the load horizontally for easiest tarping and to have the load centre as low and stable as possible for safety and load security. Tarping and load tie-down requirements must take into account the fact that federal safety regulations limit the height at which workers can work without a fall restraint system to 3 m off the ground and that many drivers are not willing to climb up high to manually tarp a difficult load because of the safety risk.

Having each lift of CLT wrapped in a waterproof package can be helpful as long as it has a way to drain trapped water and breathe out condensation at the bottom in case the wrapping gets damaged during handling or in case there is an air void that allows condensation to accumulate. It is best to also have a physical tarp over the load as the primary protection against rain, ice, debris, and wind forces. For more information on good tie-down practices, a driver's guide may be downloaded from the Publications page of the Canadian Council of Motor Transport Administrators website.

Closed top trailers with a lower rear floor similar to dropdeck flatbed trailers can also be used if loading and unloading are done from the rear door. Rollers should be recessed into the floor and be raised with air pressure to allow the CLT sections to roll when loading and unloading.

Ideally, when placing the load, the center of the payload mass (at maximum payload) should be about 1.2 meters ahead of the center of the trailer for best weight distribution and traction.

6 POSITIONING OF MATERIALS ON CONSTRUCTION SITE AND PROTECTION AGAINST WEATHER

6.1 Positioning of Materials on Construction Site

Once the materials have been delivered to the construction site, wood-based building materials must be stored properly if they are not used immediately. Good planning is essential to ensure that materials have the necessary space and proper logistics control during construction. There are costs associated with handling each shipment.

If panels must be placed temporarily on the ground prior to use, great care must be taken to protect them against weather elements and vandalism. The panels must be installed on skids in sufficient numbers to protect them from water runoffs and appropriate tarpaulin should be used to protect them.

Figure 54 shows CLT panel packs in the process of being unloaded from a truck for storage on site. The packs are completely wrapped (six faces) and are deposited on wood skids to protect them from water runoffs. Although this packaging practice may be adequate, it is crucial to use high-quality tarpaulin and to ensure that the packs remain sealed. If there are openings, water could infiltrate and remain trapped.

Also, CLT bundles should be stacked properly to avoid overloading the lower assemblies. Skids must be properly aligned to ensure load transfer from one bundle to another.

It should be noted that the stacking of the panels on the construction site must match the planned installation sequence. Unnecessary handling leads to additional costs and risks of accidents or breaking.

Figure 54
Storage on construction site – individually wrapped bundles

Figure 55
Truck platform left on construction site – it will be recovered on the next trip

6.2 Construction Load on Frame

Stacking and storage of CLT elements or other heavy materials must be made while taking into account the maximum anticipated loads for the building. If assemblies need to be placed on the construction frame, ensure that the provisional loads do not exceed the engineer's expected loads during construction.

It is recommended that CLT slabs be placed flat on the frame so they are not exposed to winds. Skids in sufficient numbers and at regular intervals should be placed between panels.

Figure 56
CLT slabs temporarily stored on a floor

6.3 Temporary Protection During Construction

When necessary, the wood components should be protected as much as possible against the elements during frame set-up operations. The CLT components are primarily intended for use in dry conditions with limited exposure to water, so they should be protected from direct rain, snow and ice, especially from long exposure to these elements. Otherwise, the wood may tarnish or become dirty during construction.

In addition, due to the hygroscopic nature of wood, CLT panels may vary slightly in size during construction and problems can occur at joints. For example, connections can be difficult to perform on the construction site, especially if accuracy is important.

There are some effective techniques used to provide adequate protection against weather elements during frame set-up operations. Figures 57 to 59 show techniques used mainly in Europe to protect components from the weather during construction.

Figure 57
Use of a temporary tarpaulin (courtesy of Fristad Bygg, Sweden)

Figure 58
Use of a permeable tarpaulin outside scaffoldings – Germany

Figure 59
Use of an adjustable tent – Sweden

7 REFERENCES

Canadian Centre for Occupational Health and Safety (CCOHS). 2010. *Materials handling: Lifting with eye bolts.* http://www.ccohs.ca/oshanswers/safety_haz/materials_handling/eye_bolts.html.

Canadian Council of Motor Transport Administrators (CCMTA). 2005. *Driver's handbook on cargo securement.* http://www.ccmta.ca/english/pdf/cargo_driver_handbook.pdf.

Council of Ministers Responsible for Transportation and Highway Safety. 2011. *National standards for heavy vehicle weights and dimensions.* http://www.comt.ca/english/programs/trucking/standards.html.

Society of Automotive Engineers (SAE). 2011. *Turning ability and off tracking: Motor vehicles.* SAE J695. Warrendale, PA: SAE. 13 p.

U.S. Department of Agriculture. Forest Products Laboratory (FPL). 1999. *Wood handbook: Wood as an engineering material.* Madison, WI: FPL. 1 v.

U.S. Department of Tranportation. Federal Highway Administration. 2012. *Oversize/overweight load permits.* http://www.ops.fhwa.dot.gov/freight/sw/permit_report/index.htm

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