FPInnovations

Canadian CLT

Handbook

2019 EDITION

VOLUME

1

Edited by:

Erol Karacabeyli

Sylvain Gagnon

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Canadian CLT Handbook
2019 Edition

Volume 1

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Erol Karacabeyli, M.A.Sc., P.Eng., FPInnovations
Sylvain Gagnon, P.Eng., FPInnovations

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FOREWORD

Mass timber and hybrid systems started to play a notable role in sustainable construction of taller and larger buildings. CLT is one of several mass timber products considered to be the "game changer" in this endeavor and the catalyst to enable other wood products to be used in those applications.

A key to success is to provide the design and construction community with the most relevant up-to-date technical information related to CLT. FPInnovations, with its partners, delivered the Canadian and US versions of the CLT Handbook which have the following unique characteristics:

- In-depth multi-disciplinary and peer-reviewed information on all performance attributes in one publication;
- Insight from CLT researchers and expert users of CLT from the design and construction community;
- Information aligned with current codes and standards, and background on the future direction of codes and standards; and
- Critical information and guidance needed for the development of alternate solutions.

The Canadian edition of the CLT Handbook, published in 2011 under the Transformative Technologies Program of the Natural Resources Canada, played an imperative role in accelerating the use and acceptance of CLT in North America. Its introduction subsequently led to the publication of the US Edition. The Canadian Edition supported the early use of CLT products from Canadian manufacturers in many small to large projects across Canada and the US, and paved the way for CLT and other wood products to be used in new applications like tall and large buildings, and bridges.

Since then, additional research has taken place globally and substantial regulatory changes have occurred enabling more wood to be used in construction. Those developments highlighted a need for the CLT Handbook to be updated. The 2019 Edition of the CLT Handbook, for example, augments the recently developed CLT provisions in CSA Standard in Engineering Design in Wood and it includes a design example of an 8-storey CLT building. It helps expand the knowledge base of the designers about CLT enabling them to develop alternative solutions for taller and larger buildings that are beyond the boundaries of the acceptable solutions in building codes.

The CLT Handbook provides vital "How to" information on CLT for the design and construction community, and is a great source of information for regulatory authorities, fire services and others. The CLT Handbook is also a good textbook for university level timber engineering courses. In summary, the Canadian CLT Handbook will remain the most comprehensive reference for sharing the latest technical information on North American CLT.

ACKNOWLEDGEMENTS

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We would like to extend our thanks to The Canadian Wood Council, National Research Council Canada, and to APA – The Engineered Wood Association for their contribution to the making of this Handbook.

The contributions made by the authors and peer reviewers to the 2011 and 2019 Editions of the Canadian CLT Handbook are greatly appreciated.

Erol Karacabeyli
Sylvain Gagnon

INQUIRIES

Please submit inquiries about this document to:

FPInnovations
570 Saint-Jean Blvd
Pointe-Claire, QC
Canada H9R 3J9
www.fpinnovations.ca

Printable version	clt.fpinnovations.ca
For more information	info@fpinnovations.ca

Follow us on:

LIST OF CONTRIBUTORS

APA – The Engineered Wood Association	apawood.org	Borjen Yeh, *Ph.D., P.E.*
Aspect Structural Engineers	aspectengineers.com	Bernhard Gafner, *Struct.Eng., P.Eng.*
Canadian Wood Council	cwc.ca	Marc Alam, *B.Eng., Ph.D. Candidate*
		Robert Jonkman, *P.Eng.*
		Ineke Van Zeeland (now with CFM)
Colorado State University	colostate.edu	John van de Lindt, *Ph.D., P.E.*
Forestry Innovation Investment	bcfii.ca	Dave Patterson, *RPF*
		Antje Wahl, *M.Sc.*
FPInnovations	fpinnovations.ca	Zhiyong Chen, *Ph.D. P.Eng.*
		Samuel Cuerrier-Auclair, *P.Eng., M.Sc.*
		Christian Dagenais, *P.Eng., Ph.D. .*
		Sylvain Gagnon, *P.Eng.*
		Blane Grann, *M.Sc.*
		Lin Hu, *Ph.D.*
		Erol Karacabeyli, *M.A.Sc., P.Eng.*
		Conroy Lum, *Ph.D.*
		Lal Mahalle, *M.Phil., P.Ag.*
		Jan Michaelsen, *F.E.*
		Chun Ni, *Ph.D., P.Eng.*
		Anes Omeranovic
		Marjan Popovski, *Ph.D., P.Eng.*
		Lindsay Ranger, *P.Eng., M.A.Sc.*
		Diane Schorr, *Ph.D.*
		Jieying Wang, *Ph.D.*
		Alpha Barry (now retired), *Ph.D., MBA*
Graz University of Technology	tugraz.at	Dr. techn. Gerhard Schickhofer
GHL Consultants Ltd.	ghl.ca	Andrew Harmsworth, *M.Eng., P.Eng.*
Morrison Hershfield	morrisonhershfield.com	Mark Lawton, *Eng.*
Moses Structural Engineers Inc.	mosesstructures.com	David Moses, *Ph.D., P.Eng., P.E., LEED® AP*
		Mark Prescott, *M.Sc.*
Ministère des Forêts, de la Faune et des Parcs du Québec	mffp.gouv.qc.ca	Jean-Frédéric Grandmont, *P.Eng., Ph.D.*
National Research Council	nrc-cnrc.gc.ca	Noureddine Benichou, *Ph.D.*
		Michael Lacasse, *Ph.D.*
		Joseph Su, *Ph.D.*
Natural Resources Canada	nrcan.gc.ca	Mohammad Mohammad, *Ph.D., P.Eng.*
Nordic Engineered Wood.	nordicewp.com	Julie Frappier, *P.Eng.*
		Simon Gallagher, *P.Eng., M.A.Sc.*
		David Hanna, *P.Eng., M.Eng.*
		Williams Munoz Toro, *Ph.D.*
RDH Building Engineering Ltd.	rdhbe.com	Graham Finch, *Dipl.T, M.A.Sc., P.Eng.*
SCIUS Advisory Inc.	sciusadvisory.com	Helen Goodland, *MBA, RIBA*
Université Laval	ulaval.ca	Jean-Philippe Migneron
		Alexander Salenikovich, *P.Eng., Ph.D.*
University of Alberta	ualberta.ca	Y.H. Chui, *Ph.D.*
University of Auckland	auckland.ac.nz	Pierre Quenneville, *Ph.D.*
University of Ottawa	uottawa.ca	Ghasan Doudak, *Ph.D., P.Eng.*

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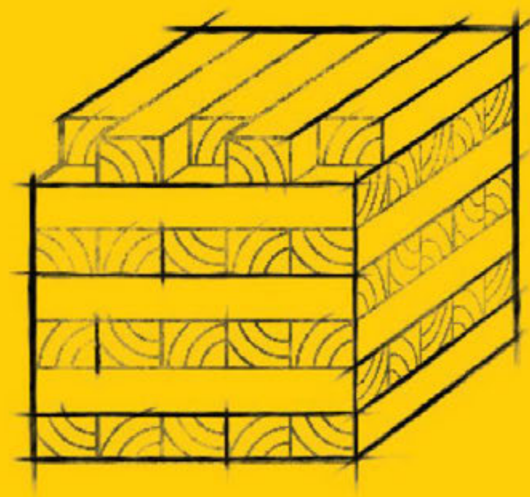

CHAPTER 1

Introduction to cross-laminated timber

Lead Authors
Erol Karacabeyli
Sylvain Gagnon

Reviewer
Conroy Lum

SP-532E (Ch.1)

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ABSTRACT

Cross-laminated timber (CLT), an engineered wood product that was originally developed in Europe in the 1990s, has been gaining worldwide popularity in helping to define a new class of timber products known as massive or “mass” timber.

In North America, significant progress has been achieved since the publication of the 2011 Edition of the Canadian CLT Handbook. This peer-reviewed Handbook was welcomed by the Canadian design and construction community, and it was instrumental in the design of early CLT projects. Subsequently, this Handbook was used as the base for the preparation of the 2013 Edition of the U.S. CLT Handbook. The technical information in these handbooks were instrumental in CLT’s inclusion in the Canadian and U.S. codes and standards. CLT has now a bi-national standard that is recognized by Canadian and U.S. regulatory systems.

The use of CLT in buildings has increased remarkably in the second decade of the 21st century. Hundreds of impressive buildings and other structures built around the world using CLT show the many advantages this product can offer to the construction sector. Construction of an eighteen-storey wood building in British Columbia and a thirteen-storey building in Québec, both started with assistance from Natural Resources Canada’s Tall Wood Building Demonstration Initiative, are recent examples of wood buildings in Canada that were made possible by CLT.

In this Chapter, we put forward an introduction to CLT as a product and also CLT construction in general, along with sections on compliance with building regulations, brief descriptions about Chapters related to manufacturing, performance of CLT, construction, and a design example of an 8-storey mass timber building assessment of markets, other examples of structures made with CLT panels.

1.1 BRIEF HISTORY

Cross-laminated timber (CLT) is a relatively recent building system of interest in North American construction and is helping to define a new class of timber products known as massive or “mass” timber. It is an engineered wood-based solution that complements the existing light frame and heavy timber options and is a suitable candidate for some applications that currently use concrete, masonry, and steel systems. CLT is an innovative wood product that was introduced in the early 1990s in Austria and Germany and has been gaining popularity in residential and non-residential applications in Europe.

In the mid-1990s, Austria undertook an industry-academia joint research effort that resulted in the development of modern CLT. After several slow years, construction with CLT increased in the early 2000s, partially driven by the green building movement, but also due to better efficiencies, product approvals, and improved marketing and distribution channels.

The use of CLT in buildings has increased remarkably in the second decade of the 21st century. Hundreds of impressive buildings and other structures built around the world using CLT show the many advantages this product can offer to the construction sector. The European experience shows that CLT construction can be competitive, particularly in mid-rise and high-rise buildings. Easy handling during construction and a high level of prefabrication facilitate rapid project completion. This is a key advantage. Lighter (relative to concrete and masonry) panels mean that foundations do not need to be as large and that smaller cranes can be used to lift the panels. Good thermal insulation, sound insulation, and performance under fire are added benefits that come as a result of a massive wood structure.

In this Chapter, we put forward an introduction to CLT as a product and CLT construction in general, along with different examples of buildings and other types of structures made with CLT panels. CLT is now available in North America and several projects already built in Canada and the United States, using CLT, are presented in this Chapter.

1.2 INTRODUCTION OF CLT IN NORTH AMERICA

The driving force behind the development of CLT in North America is the need to provide alternative wood-based products and systems to architects, engineers, and contractors. While this product is well established in Europe, work on the implementation of CLT products and systems has begun relatively recently in Canada and in the United States. Interest in the use of CLT in North America and other industrialized countries outside of Europe is increasing.

In North America, significant progress has been achieved with the publication of the 2011 Edition of the Canadian CLT Handbook (Gagnon and Pirvu, 2011). This peer-reviewed Handbook was welcomed by the Canadian design and construction community, and it was instrumental in the design of early CLT projects. Subsequently, this Handbook was used as the base for the preparation of the 2013 Edition of the U.S. CLT Handbook (Karacabeyli and Douglas, 2013). The technical information in these handbooks was instrumental in CLT's inclusion in the Canadian Standard for Engineering Design in Wood (CSA, 2016), and the National Design Specification (AWC, 2018) in the United States.

Using the draft product standards in the 2011 Edition of the Canadian CLT Handbook, a harmonized North American CLT product standard, *Standard for Performance-Rated Cross-Laminated Timber*, ANSI/APA PRG 320, has been developed by the ANSI (American National Standards Institute)/APA CLT Standard Committee (ANSI/APA, 2018). The ANSI/APA PRG 320 standard has been approved by the Structural Committee of the International Code Council (ICC) for the International Building Code (IBC, 2015).

The 2019 Edition of the Canadian CLT Handbook is based on a number of revisions that were made to the 2011 Edition. The revisions are guided by the following:

- Although the most current codes and standards are referenced (e.g. 2016 Update 1 to the 2014 Edition of the CSA O86 Standard (CSA, 2016)), CWC's 2017 Wood Design Manual (CWC, 2017), and the PRG 320 (APA, 2018)), the 2019 Edition of the Canadian CLT Handbook, particularly those sections pertaining to fire and lateral performance, includes practices we recommend based on the state-of-the-art research that has been undertaken worldwide to fill the information gaps. These practices, evaluated by the pertinent committees, are now integrated in the 2019 Edition of the CSA O86 (CSA, 2019) and are being considered for inclusion in the next editions of the National Building code (NBCC, targeted to be released in 2020).
- The primary audience for the Handbook is the Architecture, Engineering, and Construction (AEC) industry.
- References to published information are provided; where possible, summary data to support the development of alternative solutions are included.
- Metric (SI) units are used throughout the Handbook. Terminology consistent with that used in CSA O86, NBCC, NECC, PRG 320, the Technical Guide for the Design and Construction of Tall Wood Buildings in Canada (Karacabeyli and Lum, 2014), the RBQ Guide for twelve-storey mass timber buildings (RBQ, 2015), and Ontario's Tall Wood Building Reference (OMNRF, 2017) are adopted. When there are conflicts, Canadian documents took precedence over PRG 320 (APA, 2018).

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

- CLT manufacturing
- Structural design of CLT elements
- Lateral design (including wind and seismic performance) of CLT buildings
- Connections in CLT buildings
- Duration of load and creep factors for CLT
- Vibration-controlled designs for mass timber floors and tall wood buildings
- Fire performance of CLT assemblies
- Sound insulation of CLT assemblies
- Building enclosure design of CLT construction
- Environmental performance of CLT
- Lifting and handling (including transportation) of CLT elements
- A structural and fire design example of an 8-storey CLT building.

CLT has provided a significant lift to the wood sector's efforts to increase the use of wood in taller and larger buildings. There are now several CLT manufacturers in North America; worldwide, there are thousands of buildings that demonstrate the suitability and adaptability of CLT. Construction of an eighteen-storey wood building in British Columbia and a thirteen-storey building in Québec, both started with assistance from Natural Resources Canada's Tall Wood Building Demonstration Initiative, are recent examples of wood constructions in Canada that were made possible by CLT. Large demonstration wood building projects in the United States will most likely use CLT as well. Since CLT was first introduced by FPInnovations back in 2006, many projects incorporating CLT have been designed and are either under construction or have been completed, across Canada. A Summary Report (www.thinkwood.com) containing a Survey of International Tall Wood Buildings was prepared by Perkins+Will for the Forestry Innovation Investment and Binational Softwood Lumber Council. The Survey was focused on the experiences of four stakeholder groups involved in ten projects: Developers/Owners, Design Teams, Authorities having Jurisdiction, and Construction Teams. The Survey also included the topics of project insurance, project financing, and building operations and performance. Overall, the results in the report confirmed that a cost equivalent, high performing building with a timber structure is a viable option. Other case studies may also be found in recent books published by Mayo (2015), and Green and Taggart (2017).

Since its first publication in 2011, the Canadian CLT Handbook has been well-received by practitioners and educators. The 2019 Edition benefits from the immense state-of-the-art knowledge developed on CLT globally (Jeleč et al., 2018; Tannert et al., 2018), and also provides alignment with the new Canadian code provisions. Thus, the Handbook will continue to play an important role in increasing demand for CLT in construction, by making sure the most relevant design information is easily accessible by those familiar with and those recently introduced to CLT. The information in the CLT Handbook should be augmented by the Wood Design Manual (CWC, 2017), and information from manufacturers.

1.3 DEFINITION OF CROSS-LAMINATED TIMBER

Cross-laminated timber (CLT) panels consist of several layers of lumber boards stacked crosswise (typically at 90 degrees) and glued together on their wide faces and, sometimes, on the narrow faces as well. Besides gluing, nails, screws, or wooden dowels can be used to attach the layers. Innovative CLT products such as Interlocking Cross-Laminated Timber (ICLT) are in the process of development in some countries. However, non-glued CLT products and systems are out of the scope of this Handbook.

A cross-section of a CLT element has at least three glued layers of boards placed in orthogonally alternating orientation to the neighbouring 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/or additional double layers at the core of the panel) to obtain specific structural capacities. CLT products are usually fabricated with an odd number of layers; three to seven layers is common, even more in some cases.

The thickness of individual lumber pieces may vary from 16 mm to 51 mm (5/8 in to 2.0 in) and the width may vary from about 60 mm to 240 mm (2.4 in to 9.5 in). Boards are finger-jointed using adhesives meeting severe durability requirements. Lumber is visually graded or machine stress-rated and is kiln-dried.

Panel sizes vary by manufacturer; typical widths are 0.6 m (2.0 ft), 1.2 m (4.0 ft), 2.4 m (8.0 ft.), and 3 m (10 ft.), while length can be up to 18 m (60 ft.). In special cases, the thickness can be up to 508 mm (20 in), although typical thicknesses are 105 mm (4-1/8 in), 175 mm (6-7/8 in) and 245 mm (9-5/8 in), when used in buildings. Transportation regulations may impose limitations on CLT panel size.

Lumber in the outer layers of CLT panels used as walls are normally oriented up and down, parallel to gravity loads, to maximize the wall's vertical load capacity. Likewise, for floor and roof systems, the outer layers run parallel to the major span direction.

Figure 1 illustrates a CLT panel configuration, while Figure 2 shows examples of possible CLT panel cross-sections. Figure 3 illustrates a five-layer CLT panel with its two cross-sections.

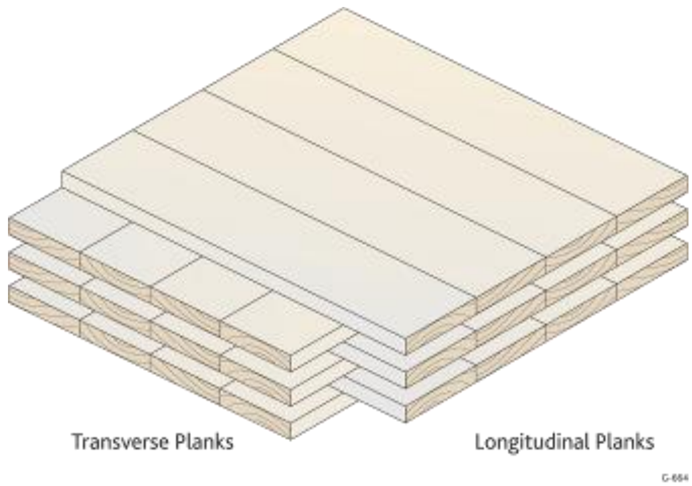

Figure 1 CLT panel configuration

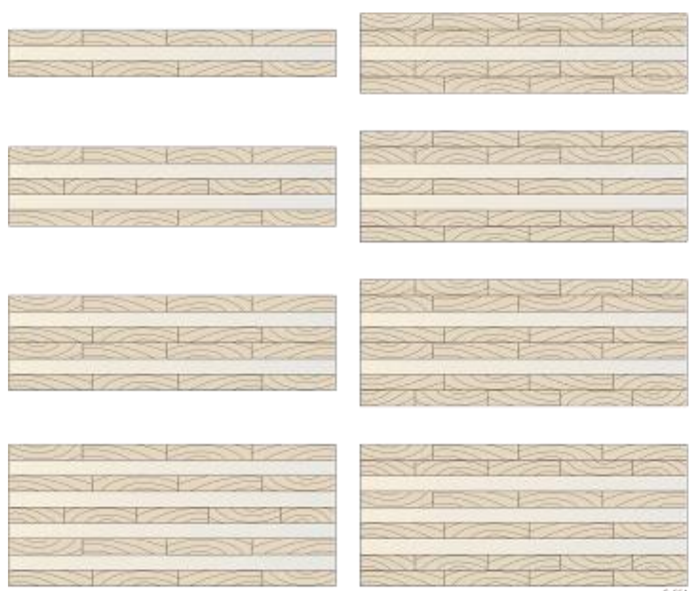

Figure 2 Examples of CLT panel cross-sections

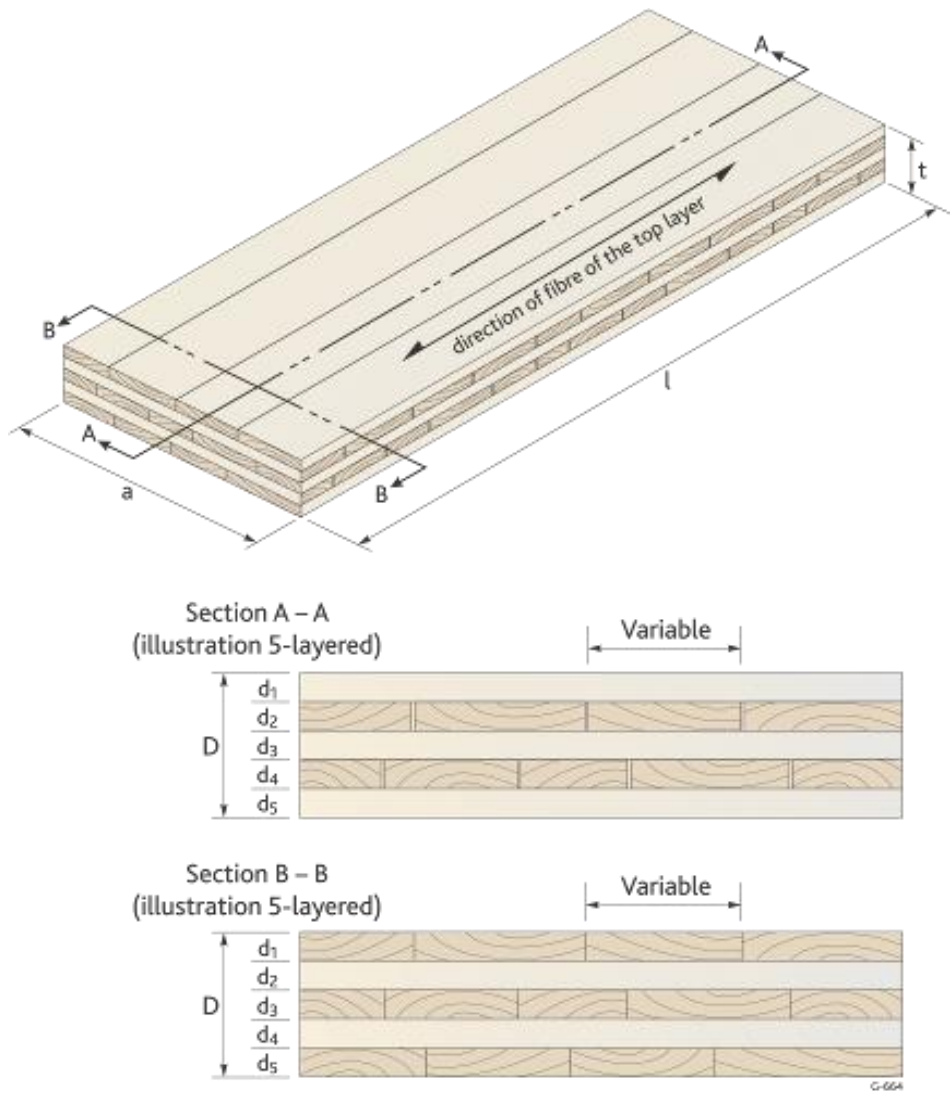

Figure 3 **Example of CLT panel cross-sections and direction of fiber of the top layers**

1.4 KEY ADVANTAGES OF CROSS-LAMINATING

CLT used for prefabricated wall and floor assemblies offers many advantages. The cross-laminating process provides improved dimensional stability to the product, which allows for prefabrication of long, wide floor slabs, long single-storey walls, and tall plate height conditions as in multi-storey balloon-framed configurations. Additionally, cross-laminating provides relatively high in-plane and out-of-plane strength and stiffness properties, giving the panel two-way action capabilities like those of 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.

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

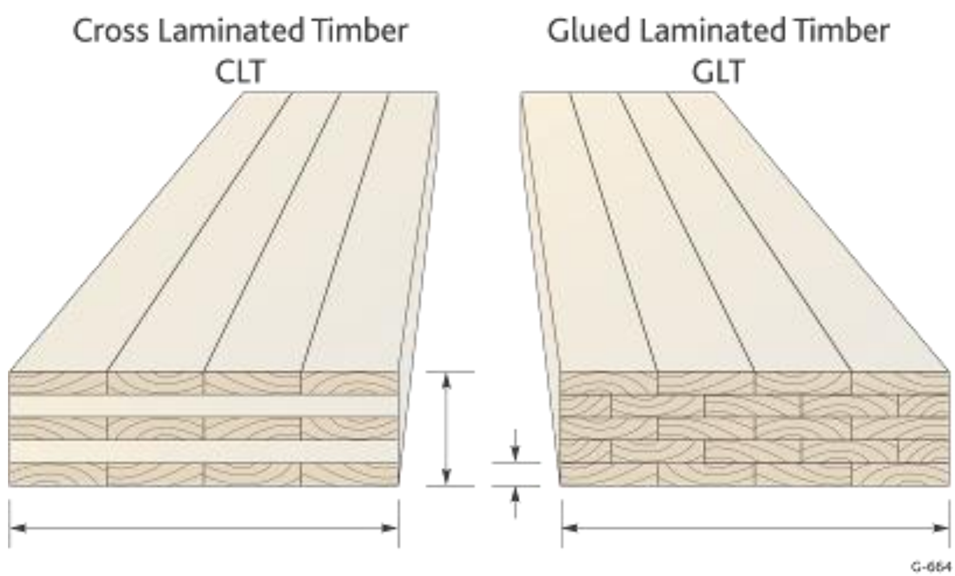

Figure 4 CLT vs. glulam panel

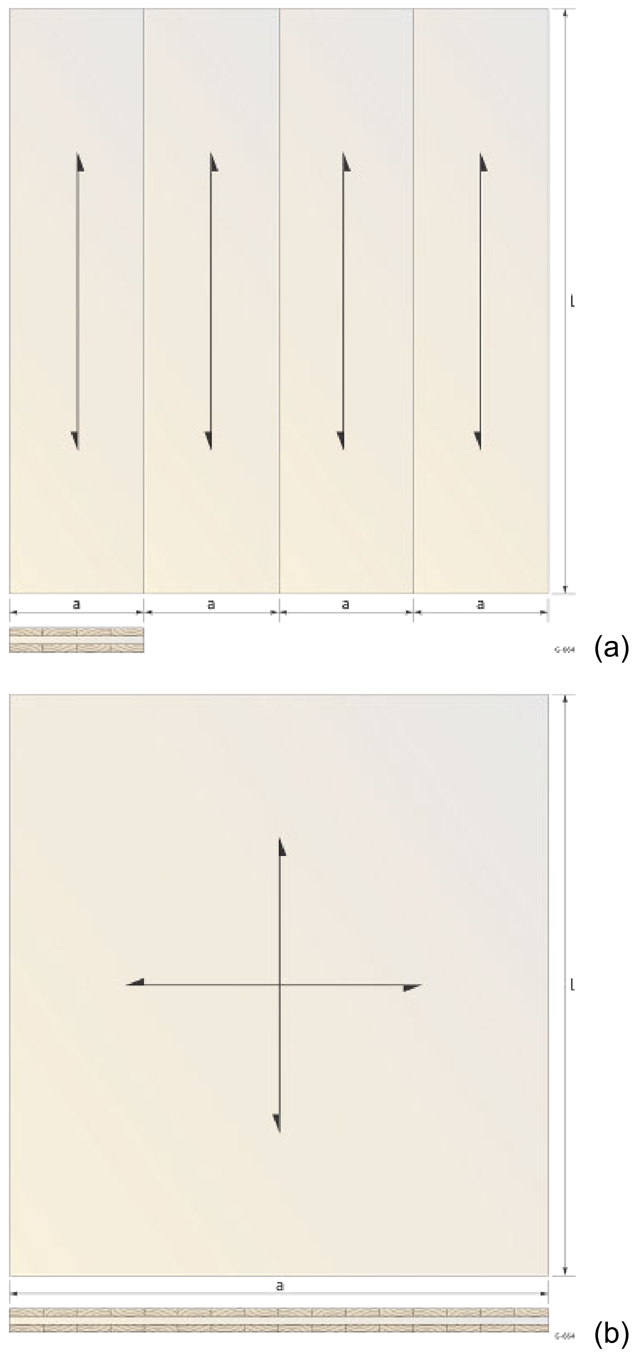

Figure 5 **(a) Floor assembly made of four 3-ply CLT panels acting in one direction and (b) Floor assembly made of one 3-ply CLT panel acting in both directions. Distance "a" may reach 3 metres**

1.5 MANUFACTURING PROCESS

Chapter 2, entitled *Cross-Laminated Timber Manufacturing,* provides general information about CLT manufacturing targeted mainly to engineers, designers, and specifiers. While this Chapter does not constitute a substitute to the manufacturing standard PRG 320 (ANSI/APA, 2018), it aims at providing background and additional information, as well as guidance related to the manufacturing of CLT products.

A typical manufacturing process for CLT includes the following steps: lumber selection, lumber grouping and planing, adhesive application, panel lay-up and pressing, product cutting, surface machining, marking, and packaging. Stringent in-plant quality control tests are required to ensure that the final CLT product will be fit for the intended application.

Panel dimensions vary by manufacturer. The assembled panels are usually planed and/or sanded for a smooth surface at the end of the process. Panels are cut to size and openings are made for windows, doors, service channels, connections, and ducts, using CNC (Computer Numerical Controlled) routers, which allow for high precision.

1.6 GOVERNING STANDARDS AND COMPLIANCE WITH BUILDING REGULATIONS

This section is intended to give users of this Handbook some basic understanding of the standards and approaches for compliance with building regulations that govern and/or influence the design of CLT in building structures.

There has been considerable progress made in the regulatory acceptance of CLT since the publication of the 2011 Edition of the Canadian CLT Handbook. The national standards have been updated and more changes are anticipated and embedded in various chapters of this 2019 Edition. Some provinces have been proactive in filling the gaps and facilitating the construction of CLT buildings; this is described in this section. In other jurisdictions, these approaches may be suggested as models or employed in the development of alternate solutions.

1.6.1 Governing Standards

1.6.1.1 ANSI / APA PRG 320: Standard for Performance-Rated Cross-Laminated Timber

ANSI/APA PRG 320 (ANSI/APA, 2018) is the standard for manufacturing CLT in North America. A special effort was dedicated to the development of this standard so that manufacturing, qualification, and quality assurance requirements for CLT would be the same across Canada and the United States. The Standard explicitly specifies performance requirements for several lay-ups made with E-rated or MSR (Machine Stress-Rated) lumber and for lay-ups made with visually graded lumber in longitudinal layers. It also includes provisions for custom lay-ups and appearance classifications and contains more options, such as the use of structural composite lumber (SCL) for laminations. Chapter 2 (*Manufacturing*) outlines the grades and thicknesses of CLT that are commonly produced.

Certification of CLT to attest that the product meets the requirements of the Standard for Performance-Rated CLT ANSI/APA PRG 320 is conducted by a Certification Body accredited under the International Standard ISO/IEC 17065, and all testing must be performed by a Testing Organization accredited under ISO/IEC 17025, as required by the Standards Council of Canada. Examples of two Products Reports may be found in APA PR L314 (2017) and APA PR L306C (2017). To further facilitate the acceptance of CLT in Canada, manufacturers also may obtain a CCMC (Canadian Construction Materials Centre) Listing. An example may be found in CCMC 13654-L (2016).

1.6.1.2 CSA O86 Standard – Engineering Design in Wood

CSA O86 Standard (CSA, 2014) makes general reference to the CLT product standard ANSI/APA PRG 320. Specific provisions for CLT related to structural, fire, and vibration performances, however, have been incorporated in the Update 1 of the CSA O86 Standard (CSA, 2016). The Update 1 of CSA O86 Standard is followed throughout this Handbook; however, where state-of-the art information is available (e.g. in the area of lateral design), guidance (which was recently implemented in the 2019 Edition of the CSA O86 Standard) beyond this standard is also given. Also, innovative fasteners such as self-tapping screws (STS) sourced from Europe are largely used in CLT construction but have not yet been implemented in CSA O86 Standard; however, relevant information on STS may be found in Chapter 5 of this Handbook.

For information, the Wood Design Manual (CWC, 2017) includes the Update 1 of the CSA O86 Standard, its Commentary, and design aids for CLT.

1.6.2 Compliance with Building Regulations

All buildings in Canada have to comply with building regulations or bylaws. The National Building Code of Canada (NBC) (NRC, 2015) is the model code that sets the standards for building construction in the country. The provinces adopt the NBC for their own provincial building codes in its entirety or with modifications.

NBC makes references to the latest edition of material standards. The NBC 2015 references CSA O86-14 Standard (CSA, 2014).

Compliance with provincial building codes (that is, with NBC) is by acceptable solutions or by alternative solutions, as defined in the code. The normal approach is to follow one of these two paths. Extensive information about "Building Code Compliance" along with information on acceptable and alternate solutions may be found in Karacabeyli and Lum (2014). In British Columbia, however, a third approach – Site-Specific Regulation (SSR) – has also been used (please see Section 1.6.2.3 for details). Regulatory Framework for CLT is shown in Figure 6.

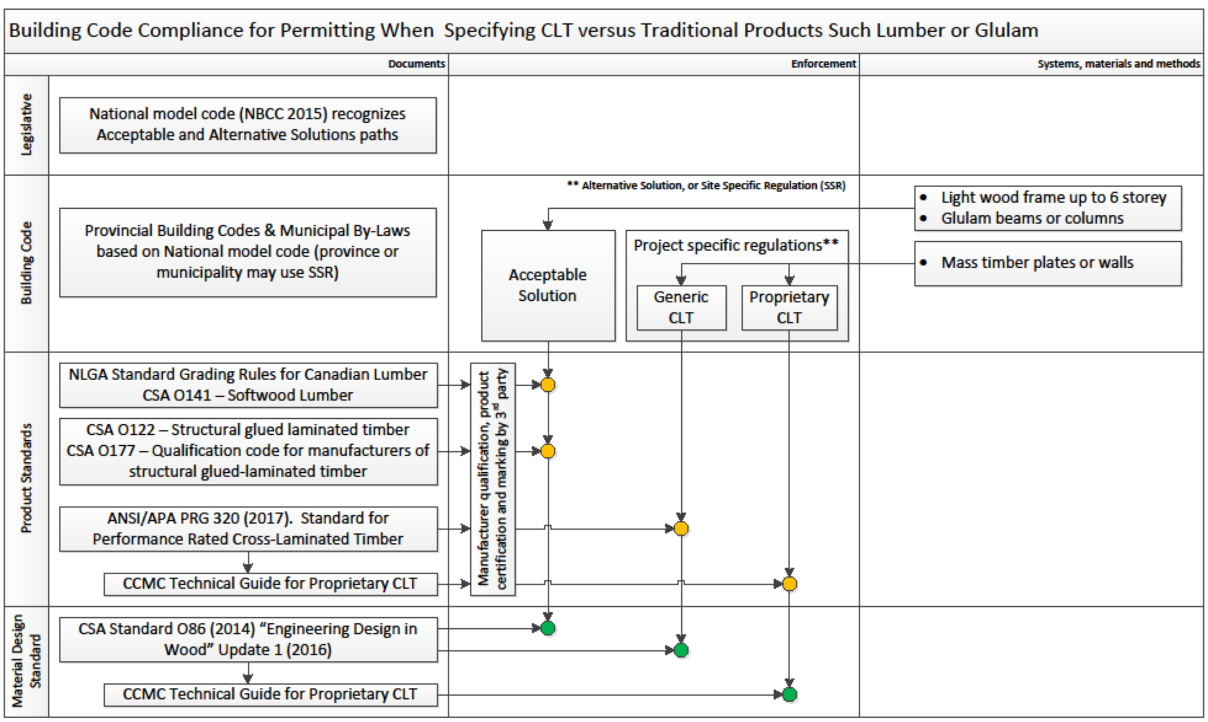

Figure 6 Regulatory framework for CLT

1.6.2.1 Acceptable Solution Path

NBC 2015 (NRC, 2015) and provincial building codes make reference to the CSA O86 Standard (CSA, 2014). As long as the wood building is within the height (e.g. up to six storeys) and area limitations of the acceptable solutions in the building codes, one can use these standards and PRG 320 (ANSI/APA 2018) to design CLT components in that building.

1.6.2.2 Alternative Solution Path

When the wood building is outside the height and area limitations of the building codes (e.g. CLT building in Vancouver with more than six storeys), the alternative solution path will have to be followed so that the building, with respect to acceptable solutions, will provide the same level of performance and safety relative to the objectives and functional statements provided in the NBC. Two different ways by which the Provinces of Québec and Ontario facilitated the design and construction of tall wood buildings are presented below. It is anticipated that these developments will assist the introduction of future provisions in building codes in Canada.

1.6.2.2.1 Acceptable Solutions with RBQ Guide in Québec

Pursuant to Article 127 of the Building Act, the Régie du bâtiment du Québec (RBQ – the Authority Having Jurisdiction in Québec) released a Guide (RBQ, 2015) for Mass Timber Buildings of up to twelve storeys (or thirteen storeys when a concrete podium is used). To allow the equitable use of wood in construction in Québec, the RBQ permits construction of mass timber buildings of up to twelve storeys, without requiring an application for equivalent (or alternative) measures, provided all guidelines set out in Part 1 of this Guide are respected and the points set out in the Explanatory Guide are taken into account. This historic moment makes Québec the first Province or State in North America that facilitated tall wood buildings with its Guide.

The RBQ Guide is based on the guidelines in the 2010 Edition of the Québec Building Code, FPInnovations' Technical Guide for the Design and Construction of Tall Wood Buildings in Canada (Karacabeyli and Lum, 2014), experience gained from the tall wood building demonstration projects by Natural Resources Canada and the Canadian Wood Council, and the results of tests conducted at the National Research Council Canada. In March 2013, a working group consisting of government departments, agencies, and fire departments, operating under the responsibility of the RBQ was formed. The RBQ consulted this working group in the development of its guidelines for mass timber construction exceeding six storeys.

1.6.2.2.2 Alternative Solutions with the OMNRF Reference in Ontario

The Ministry of Natural Resources and Forestry in the Province of Ontario published a technical reference document (MNRF, 2017) to assist architects, engineers, builders, and developers in the development of Alternative Solutions for tall wood projects with mass timber, and to facilitate the approval by a Chief Building Official (CBO) under Ontario's Building Code.

Like the RBQ Guide, Ontario's Tall Wood Building Reference also made extensive use of FPInnovations Technical Guide for the Design and Construction of Tall Wood Buildings in Canada (Karacabeyli and Lum, 2014) that was developed under Natural Resources Canada's Transformative Technologies Program.

1.6.2.3 Site-Specific Regulation Path in BC

The Province of British Columbia, through its Building and Safety Standards Branch, used the Site-Specific Regulation approach for the design and construction of two wood buildings (BSSB, 2015):

- The Wood Design and Innovations Center in Prince George, B.C.
- The Brock Commons building (18 storeys) at the University of British Columbia campus.

Compared to the regular approach to compliance with the building code, the Site-Specific Regulation approach could be considered to be more elaborate, from an administrative point of view.

More recently, the Province of British Columbia adopted the proposed provisions for the 2020 Edition of the NBC for 12-storey mass timber buildings.

1.7 STRUCTURAL, FIRE, SERVICEABILITY, BUILDING ENCLOSURE AND ENVIRONMENTAL PERFORMANCE OF CLT

CLT structures are well suited for use in a wide variety of structural applications, from low-rise commercial and institutional buildings, to mid- and high-rise residential and non-residential buildings. A number of buildings, as high as twenty-four storeys, have already been constructed around the world, which use CLT in their structural system.

CLT panels are typically used as load-carrying plate elements in structural systems such as walls, floors, and roofs, and sometimes as beams and lintels. For floor and roof CLT elements, the key critical characteristics that must be taken into account are the following:

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

For wall elements, the following are key characteristics that must be taken into account at the design stage:

- Load-bearing capacity (critical criterion)
- In-plane shear and out-of-plane bending strength
- Fire performance
- Sound insulation
- Durability
- Energy efficiency

The following sections provide brief summaries of the key design and performance attributes of CLT panels and assemblies.

1.7.1 Lateral Design of CLT Buildings

Chapter 4 of this Handbook entitled *Lateral Design of Cross-Laminated Timber Buildings* provides design guidelines and recommendations about the design of CLT structures for lateral loads, such as loads due to earthquakes and strong winds. A brief literature review on the research work conducted around the world related to the seismic performance of CLT wall panels and structures is also included. The design recommendations presented in Chapter 4 of this Handbook are based on the research information available world-wide, the CSA O86-14 Standard Update 1 (CSA, 2016), the information provided in the CWC Wood Design Manual (CWC, 2017), and the general requirements of the National Building Code of Canada (NRC, 2015). Since most of the research conducted on this topic around the world is related to platform-type CLT buildings, the design recommendations are mostly related to this type of structural system.

While most low- to mid-rise CLT buildings are platform-framed, they are far less susceptible to the development of soft storey failure mechanisms than other platform-framed structural systems. Since the nonlinear behaviour (and the potential damage) is localized in the hold-down and L-bracket connection areas, the panels - that are also the vertical load carrying elements - are virtually left intact, in place, and uncompromised, even after failure of the connections. In addition, all CLT walls (including the ones that are not part of the lateral load-resisting system) on a single level contribute to the lateral and gravity resistance, providing a degree of redundancy and a system sharing effect. Vertical and lateral load sharing can also take place between levels, creating a honeycomb effect.

1.7.2 Connections and Construction of CLT Structures

Connections in timber construction, including those built with CLT, play a crucial role in maintaining the integrity of the timber structure and in providing strength, stiffness, stability, and ductility. Consequently, they require thorough attention from the designers. The structural efficiency of a 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 together.

Chapter 5*, Connections in Cross-Laminated Timber Buildings*, of this Handbook focuses on connector systems that reflect present-day practices, some being conventional, others being proprietary. Examples and a flow chart for analysis and design of connections that is in line with the recommendations in Chapter 4*, Lateral Design of Cross-Laminated Timber Buildings,* are also included.

1.7.3 Duration of Load and Creep Behaviour

CLT products are used as load-carrying slabs, wall elements, and beams in structural systems; thus, load duration and creep behaviour are critical characteristics that should be taken into account in the design. Given the nature of CLT, with its orthogonal arrangement of layers that are 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.

Chapter 6 of this Handbook entitled *Duration of Load and Creep Factors for Cross-Laminated Timber* aims to describe how the duration of load and creep effects are taken into account in the design of CLT structures. This Chapter also contains a discussion on different parameters that may affect the duration of load and creep effects, including the effect of adhesive, edge-gluing, and release grooves.

Mechanically fastened CLT products are outside the scope of the CSA standard and of this CLT Handbook, but research has found that they may deflect and creep to a greater extent than adhesively bonded CLT.

1.7.4 Vibration Performance of Floors and Tall Wood Buildings

Chapter 7 of this Handbook entitled *Vibration-Controlled Designs for Mass Timber Floors and Tall Wood Buildings* first addresses the vibration serviceability of CLT floors related to normal human activities, as well as the vibration serviceability of tall wood buildings under wind-induced excitation.

Studies at FPInnovations found that bare CLT floor systems differ from traditional lightweight wood-joisted floors. Hence, the existing standard vibration-controlled design methods for lightweight and heavy floors may not be applicable to CLT floors.

Chapter 7 is an update of the same chapter published in the 2011 Edition of the Canadian CLT Handbook. In 2016, a vibration design method for CLT floors was accepted by the CSA O86 Technical Committee and was subsequently published in the Update 1 of the CSA O86-14 Standard (CSA, 2016). This method was largely based on the method presented in the 2011 Edition of the Canadian CLT Handbook, with a few modifications. This updated Chapter is in line with the provisions in the Update 1 of the CSA O86-14 (CSA, 2016). The revised vibration design method for CLT floors in the CSA O86 Standard has two key features: 1) the vibration-controlled span is directly calculated using the CLT floor effective bending stiffness in the major strength direction and its mass, without iteration; and 2) an empirical approach to account for the effects of multiple-spans, toppings, and non-structural elements such as partition walls and finishes.

This updated Chapter was then extended to include a preliminary design method for Timber-Concrete Composite (TCC) floors, based on recent research conducted by FPInnovations, and a more sophisticated stiffness requirement for floor supporting beams than in the previous Edition.

This updated Chapter provides preliminary guidelines for the control of tall mass timber building vibrations. These guidelines are based on the recent technical information and data collected by FPInnovations and others, and propose simple equations to calculate the first two transverse natural frequencies of wood buildings as well as recommendations for damping ratios of wood buildings.

1.7.5 Fire Performance of Cross-Laminated Timber Assemblies

Mass timber products are generally known to perform well under fire conditions due to their slow rate of charring, which generates a thick layer of low-density insulating char and thereby protects the timber below from elevated heat effects. Charring is a material-specific property attributed to timber; understanding this behaviour is fundamental in estimating the reduced thickness of full-strength timber when exposed to fire, which designers can then use to calculate a member's residual strength for a given fire exposure.

Chapter 8 of this Handbook entitled *Fire Performance of Cross-Laminated Timber Assemblies* provides up-to-date information related to the fire performance attributes of CLT elements conforming to the CLT manufacturing standard ANSI/APA PRG 320 (APA, 2018). This bi-national standard has been revised in 2018, when new mandatory performance requirements for adhesives at elevated temperatures have been implemented. These changes positively impact the charring behaviour of CLT elements.

Acceptance of CLT construction into the Canadian regulatory environment necessitates compliance with the fire-related provisions of the NBC (NRC, 2015), among other regulations. Part 3 of Division B of the NBC provides prescriptive fire safety provisions in order to meet these objectives, based on a building's major occupancy group, its height and area, as well as the presence of automatic fire sprinklers. Chapter 8 addresses some of the common code-mandated fire performance requirements.

CLT elements are used in building systems in a manner similar to concrete slabs and solid wall elements, as well as heavy timber construction; limiting concealed spaces with the use of mass timber elements reduces the risk of concealed space fires. Moreover, CLT construction typically uses CLT panels for floor and load-bearing walls, which allow inherent fire-rated compartmentalization, therefore further reducing the risk of fire spread beyond its point of origin (compartment of origin).

In an attempt to provide the scientific and technical information related to CLT fire performance attributes to allow building code implementation, extensive fire testing has been conducted in North America on CLT elements. The results have shown that CLT elements, with or without gypsum board protection, can achieve significant fire resistance, beyond three hours in some cases. Surface flame spread tests confirm that the risk of ignition of mass timber elements is greatly reduced compared to traditional interior finish wood products. Tests have also shown that fire stops approved for concrete construction are suitable for CLT elements, so long as adequate detailing is provided. The informative calculation method from Annex B of the Update 1 of CSA O86-14 (CSA, 2016) is detailed in this revised Chapter 8. A refined stepped charring model is also presented.

In addition, Chapter 8 includes a discussion on the use of CLT in vertical exit stair shafts as an alternative to traditional non-combustible construction, as well as an overview on how to incorporate CLT in a performance-based fire design. Safety during construction is also discussed.

1.7.6 Sound Insulation of Cross-Laminated Timber Buildings

Noise control (mitigation of unwanted sound) is an important serviceability consideration for the design of multi-family occupancies. There is a need for a noise control procedure that would guide designers, architects, contractors, and any practitioners to fulfill their design goals. This was the motivation behind Chapter 9*, Acoustics Performance of CLT,* in the 2011 Edition of the Canadian CLT Handbook.

When the 2011 Edition of Chapter 9 was published, NBC (NRC, 2010) was in the process of replacing the Airborne Sound Transmission Class (STC) by an Apparent Sound Transmission Class (ASTC). Similarly to STC, ASTC is a single number rating of the apparent airborne sound insulation performance of the combined wall and floor/ceiling assemblies in buildings, as perceived by the occupants. The apparent airborne sound insulation accounts for direct transmission through the demising element, as well as flanking transmission. In 2015, ASTC was implemented in the NBC (NRC, 2015), as a measure for airborne sound insulation performance.

Moreover, CLT production was at its infancy in Canada in 2011; therefore, Chapter 9 was based on European experience, designs, and materials. Canadian CLT panels are now more readily available in the Canadian market, and significant research efforts have been undertaken to study sound insulation performance of CLT wall and floor/ceiling assemblies, and to develop solutions for the assemblies to meet code requirements and consumer expectations. A number of CLT buildings have been constructed using Canadian products and solutions, and apparent sound insulation performance testing has been conducted on some of these buildings. Feedback on their sound insulation performance has also been monitored. These studies have resulted in significant advancements in knowledge and have provided solutions for CLT building sound insulation in Canada; this has led us to update Chapter 9.

The updated Chapter 9 first outlines a procedure for CLT building noise control based on current knowledge, which involves the following essential components: 1) basic knowledge of the noise source and measurement of noise transmission; 2) 2015 NBC requirements and occupant perception; 3) noise management principles; 4) effects of CLT mass (thickness) and construction details on sound insulation performance of CLT wall and floor/ceiling assemblies; 5) systems approach to manage noise, which includes meeting other performance requirements such as fire resistance, structural integrity, vapour barrier, ease of installation, and cost effectiveness; 6) prescriptive design examples of CLT wall and floor/ceiling assemblies with sound ratings; and 7) on-site installation quality control – flanking management. The sound insulation of wood elevator shafts, wood stairwells, and stepped storey wood buildings is briefly covered.

The goal of Chapter 9 is not only to provide solutions for noise control, but also to provide a road map for controlling noise, by showing how to use a systematic and logical approach to control noise transmission in buildings. The examples of acoustic design solutions presented in this Chapter were carefully selected from various sources, to ensure they met or exceeded the code requirements for sound insulation. The design solutions are ready to be applied to new CLT building projects, and also illustrate the effects of various details on sound insulation. By following the road map and examples provided in this Chapter, new and innovative design solutions may be derived.

1.7.7 Building Enclosure Design of Cross-Laminated Timber Construction

As mentioned previously, when the 2011 Edition of the Canadian CLT Handbook (Gagnon and Pirvu, 2011) was being written (between 2009 and 2011), there were very few built examples of CLT construction in Canada or in the United States. Research had started and those seriously interested in CLT were looking at the early examples in Europe. The 2013 Edition of the U.S. CLT Handbook (Karacabeyli and Douglas, 2013) provided additional information for the climates in the United States. Fast forward to 2019 and CLT has now been used to build hundreds of small to large buildings in Canada and the United States. This includes the eighteen-storey UBC Tallwood House (Brock Commons) in Vancouver, British Columbia, where CLT was used within the floor system (Figure 14). CLT will be an integral component of tall wood buildings currently being proposed across North America. As its title implies, Chapter 10, *"Building Enclosure Design and Construction Moisture Management of Cross-Laminated Timber"* focusses on the building enclosure (also known as the building envelope) system, i.e. the component of mass timber buildings that protects the structure from moisture and environmental elements, separates the indoors from the outdoors, and is a key passive design element within energy-efficient and sustainable buildings. The building enclosure may incorporate CLT structural elements or be placed in position outside of a structure. The proper design and long-term performance of the building enclosure is therefore critical to the sustainability of mass timber buildings.

Chapter 10 provides building science guidance on best practices for the design of building enclosures incorporating CLT panels. This guidance is based on a combination of research, testing, and acquired experience with the construction of buildings with CLT building enclosure systems. A brief primer on relevant building code requirements and the building science of heat, vapour, air, and moisture control for CLT walls and roofs is followed by sections on CLT wall and roof designs and detailing. The final section covers strategies and solutions for addressing construction moisture, service moisture, and preservative treatment to ensure long-term durability.

1.7.8 Environmental Performance of CLT

In Chapter 11 of this Handbook, several important dimensions concerning the environmental performance of CLT are presented. In the first section of this Chapter, results from a life cycle assessment study comparing a four-storey CLT apartment building to a functionally equivalent building with a concrete slab and column structure, and light gauge steel stud walls are presented. Results from this comparison show that the CLT building provides a reduction in life cycle greenhouse gas (GHG) emissions, a finding that is consistent with results obtained in other case studies. The second section explores the topic of fibre availability in Canada, showing the availability of a sustainable wood supply from Canadian forests in the form of a regulated Annual Allowable Cut (AAC). Finally, in the third section, results from a study on indoor air emissions from several CLT samples showing that CLT panels easily achieve the most stringent emission standards are summarized.

1.8 CLT IN CONSTRUCTION

CLT as a building system is quite adaptable, can be suitable for long spans in floors, walls and roofs, and has the potential for a high degree of off-site preinstallation of exterior and interior finishes. Its ability to be used as either a panelized or a modular system makes it ideally suited for additions to existing buildings. It can be used jointly with any other building material, such as light wood frame, heavy timber, steel or concrete, and accepts a range of finishes. CLT panels can also be built compositely with reinforced concrete, to enable longer spans (i.e., longer than 9 meters). Good thermal insulation and sound insulation, as well as an impressive performance under various fire conditions are added benefits resulting from the massiveness of the wood structure.

The prefabricated nature of CLT permits high precision and a construction process characterized by faster completion, increased safety, less demand for skilled workers on-site, less disruption to the surrounding community, and less noise and waste. Openings for windows, doors, staircases, and utilities are pre-cut using Computer Numerical Controlled (CNC) machines at the factory. Buildings are generally assembled on-site, from panels prefabricated and brought to the site, where they are connected by means of mechanical fastening systems such as bolts, lag bolts, self-tapping screws, or other connection systems.

Throughout the design, the project team will need to consider the routing of services between floors, within ceiling spaces, and within walls. This topic is discussed in the Technical Guide for the Design and Construction of Tall Wood Buildings in Canada (Karacabeyli and Lum, 2014) under the heading “Integrating Systems”, where guidance and some examples are given for integration of mechanical/plumbing systems, electrical systems, and fire suppression systems in mass timber and hybrid buildings.

In Chapter 12 of this Handbook, a wide range of lifting systems and devices that can be used in the construction of structures made of CLT panels is presented. Some are in-use currently, while others are suggested. We also discuss the basic theory required to understand and implement 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 contractors build safe and efficient CLT structures. Issues related to the transportation of CLT assemblies from factory to building site are also discussed.

1.9 8-STOREY MASS TIMBER BUILDING DESIGN EXAMPLE

In the 2019 Edition of the Canadian CLT Handbook, a new Chapter (13) was added to include a design example of an 8-storey residential mass timber building with emphasis on the structural design (gravity and lateral), and design for fire resistance of its key components. The building consists of a first storey in concrete with the upper seven storeys in mass timber (CLT roof panels, CLT floor panels, CLT elevator and stair cores, and a glulam post-and-beam frame). The elevator and stair cores and additional shear walls are platform-type CLT to resist the seismic and wind loads.

1.10 ASSESSMENT OF MARKET OPPORTUNITY

1.10.1 CLT Canadian Market Prospects

As Canadian construction increasingly shifts to taller buildings, CLT is one of the primary building products that will allow the Canadian wood sector to address this market. Over the past two decades, residential construction has shifted from primarily detached homes to multifamily homes. In 2017, 61% of all housing starts were multifamily, compared to only 37% in 1997. As multifamily units are smaller and have shared walls and other systems, a multifamily start consumes about 1/3 of the volume of wood as a single-family start. This makes the shift to multifamily construction a demand issue for the wood products industry (Figure 7).

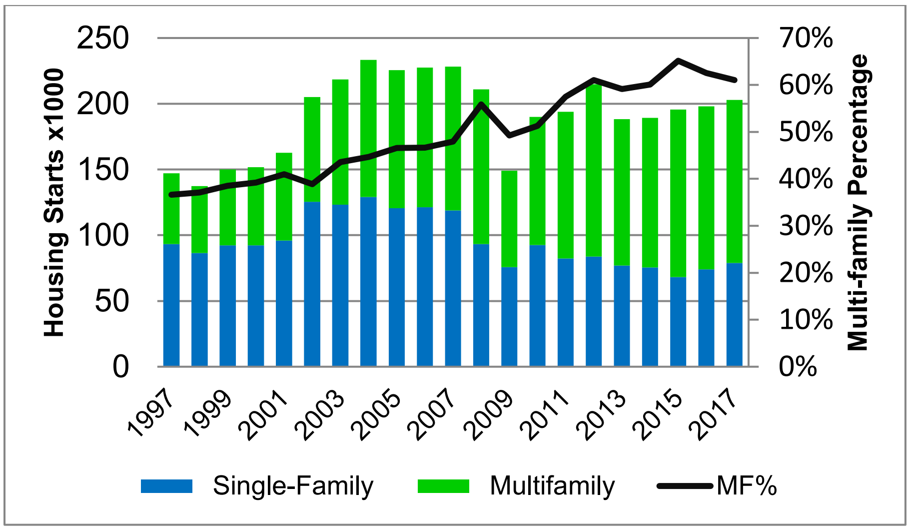

Figure 7 Canadian housing starts shift to multifamily (source: Statistics Canada)

One of the ways that CLT can help bridge the demand gap is to introduce it in existing multifamily construction. As of NBC 2015, wood multifamily construction as an “Acceptable Solution” is allowed up to six storeys, and even higher in Québec, up to twelve storeys. In multifamily construction, this usually equates to a light-frame wood condominium or apartment building, sometimes with a concrete podium. There are three existing and two potential applications for CLT in light-frame multifamily buildings. These are:

1. Elevator shafts/cores
2. Floor plates-wood hybrid buildings
3. Podiums
4. Buildings higher than six storeys
5. Non-residential

1.10.2 Elevator Shafts/Cores

CLT can be used to construct elevator shafts in light-frame buildings. The strength of CLT in these elevator shafts can also be designed to be part of the lateral resistance system of the building. These systems are already being used in Canada in both the CLT and NLT (Nail-Laminated Timber) form. However, if CLT were to be used in all four- to six-storey elevator shafts in Canada, the approximate market size would be between 70 - 135 MMBF.

Figure 8 CLT elevator shaft constructed in the Arbora development

1.10.3 Floor Plates – Wood Hybrid Buildings

CLT is making in-roads as floor plates in light-frame buildings, the result of a new wood hybrid building method. CLT floor slabs with cut-outs provide builders with large prefabricated systems that are fast to erect, and easy to transport. This also leads to a safer building site when solid floors are installed and can be used right away, rather than be constructed on-site by workers at heights.

1.10.4 Podiums

In British Columbia, 45% of five- to six-storey wood multifamily buildings were constructed with concrete podium ground floors, in 2016. As mid-rise construction is adopted across Canada, it is expected that podium floors will continue to be used, as many of these buildings will be for mixed use. For wood buildings having a total height of five to six storeys, the 2015 NBC allows podiums made of combustible material, which clears a path for CLT. While this has not yet been put into practice, CLT is the prime wood material that can provide a robust horizontal separation between the commercial and residential areas in a building. The use of wood instead of concrete podiums has the potential to boost the use of wood in a six-storey building by up to 23%.

1.10.5 Buildings Higher than Six Storeys

Not only has Canada made a large shift towards multifamily construction, this construction class is also increasingly being built to more than six storeys, as allowed as "Acceptable Solution" for wood by the 2015 NBC. It must be noted that the Province of Québec already developed provisions for 12-storey mass timber buildings; the pertinent committees proposed provisions for the 2020 NBC for 12-storey mass timber buildings that were adopted by the Province of British Columbia. In 2016, 55% of multifamily construction in Canada was built to seven storeys or more. To address this market, wood construction must shift from light-frame to mass timber.There are multiple examples of tall CLT construction, the tallest in Canada being at 18 storeys (2017-UBC Brock Commons). Current code work in Canada and the United States is targeting tall mass timber construction for the 2020 (Canada) and 2021 (United States) editions of building codes.

The potential size of the tall construction market for wood can be broken-up by storey classes. If a CLT system were used for all seven- to twelve- storey residential construction, this would equate to 535 MMBF of CLT consumption. A target market share scenario assuming partial adoption of wood in this height class was calculated at 219 MMBF. In addition, there are 94 MMBF in potential wood use in non-residential buildings at these heights (24 MMBF market share scenario).

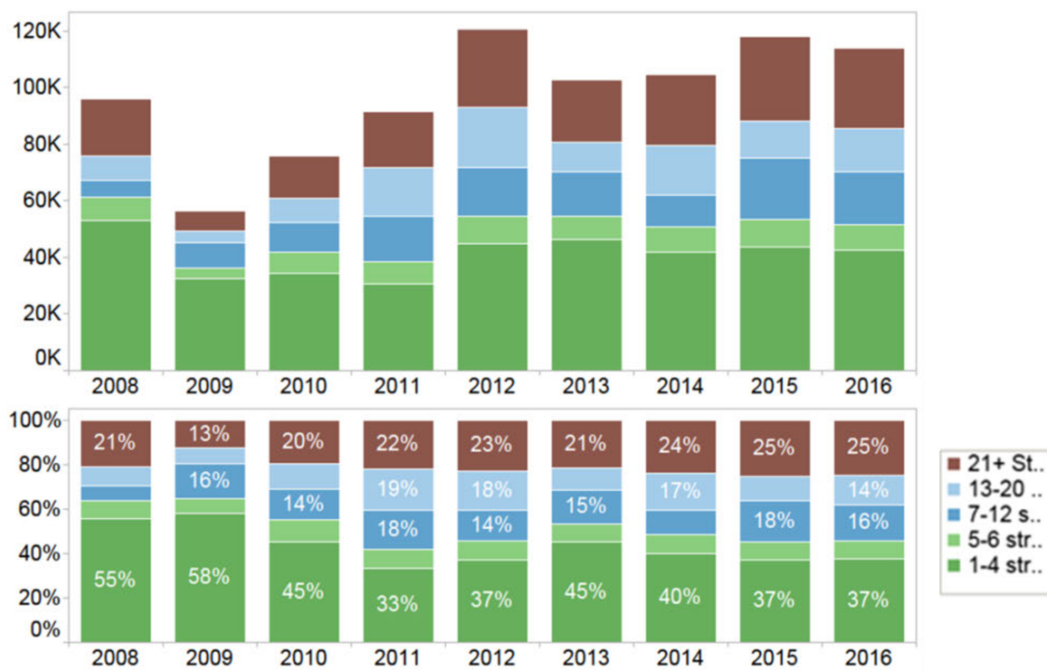

Figure 9 Canadian multi-family construction shifts to taller buildings (source: FPInnovations, CMD)

1.10.6 Non-Residential

CLT can also play an important role in the non-residential construction sector. FPInnovations breaks the non-residential sector into three main groups, two of which are suitable for CLT applications. The conventional construction sector encompasses buildings with multiple floors and partition walls, similar to multifamily buildings. This sector includes offices, hotels, dormitories, and health buildings such as long-term care facilities and treatment offices (non-hospital). The second potential category is large buildings, such as warehouses and stores, which are usually single-storey constructions. The final category is labelled as “restricted”, because it often does not lend itself to the use of wood.

Within the conventional category, wood has already achieved a moderate market share with light-frame construction. However, CLT has been playing an increasing role in this sector, with multiple light-frame / CLT hybrid projects completed or planned in Canada. The main non-residential application for this system has been the hotels, but it could also include the long-term care and dormitory sectors as well; there are already examples of mass timber construction in these sectors. Mass timber offices have been built in Vancouver (MEC) and in Minnesota (T3). Information on these buildings may be found in www.thinkwood.com. While both of these projects involved glulam (Glued Laminated Timber) and NLT (Nail Laminated Timber), CLT could also have been used. Specific to these buildings is a wood-forward appearance intended to promote occupant health.

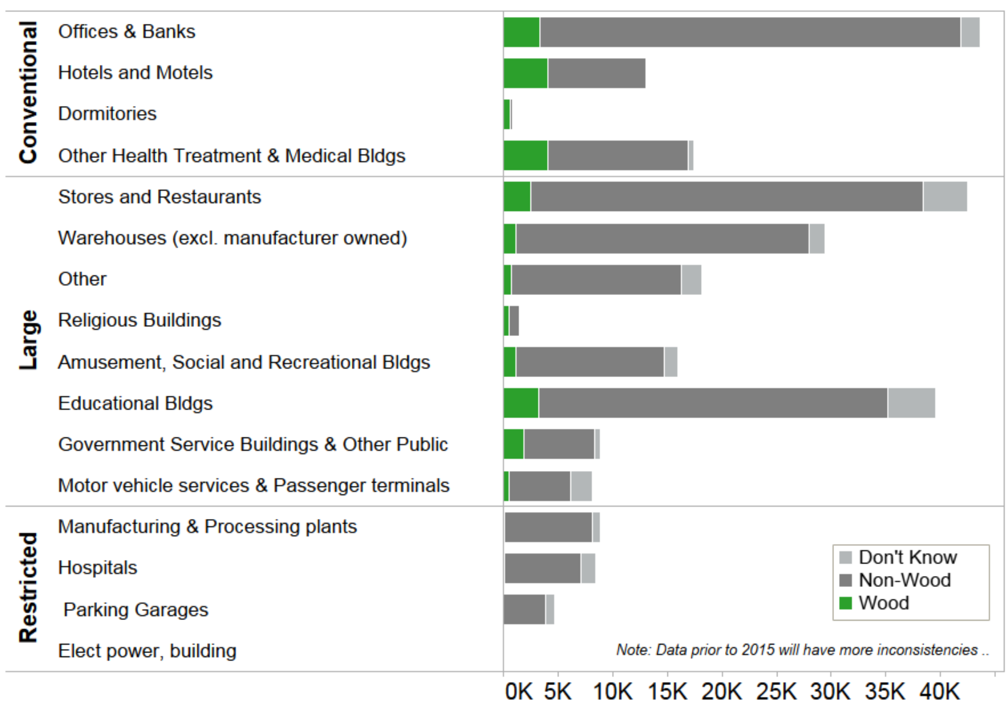

Figure 10 Canadian one- to four-storey non-residential floor area constructed - 2012-2016 (sq.ft x 1000) (source: FPInnovations, CMD)

The greatest portion of non-residential construction is built in the large construction category, in the form of retail buildings and warehouses. These are often single-storey buildings built on concrete slab floors, but CLT can be used for wall and roof applications in this sector. The key factors in this sector are tall walls, large open spans and, most importantly, price. From 2012 to 2016 the average building cost for warehouses in Canada was $104/sf, while stores and restaurants were $119/sf. This compares to $203/sf for schools and $153/sf for offices. Thus, CLT tall walls and long-span roofs must be produced at low costs, or market niches that will pay for exposed wood will have to be identified. On the other hand, the education market is quite large and typically has larger project budgets.

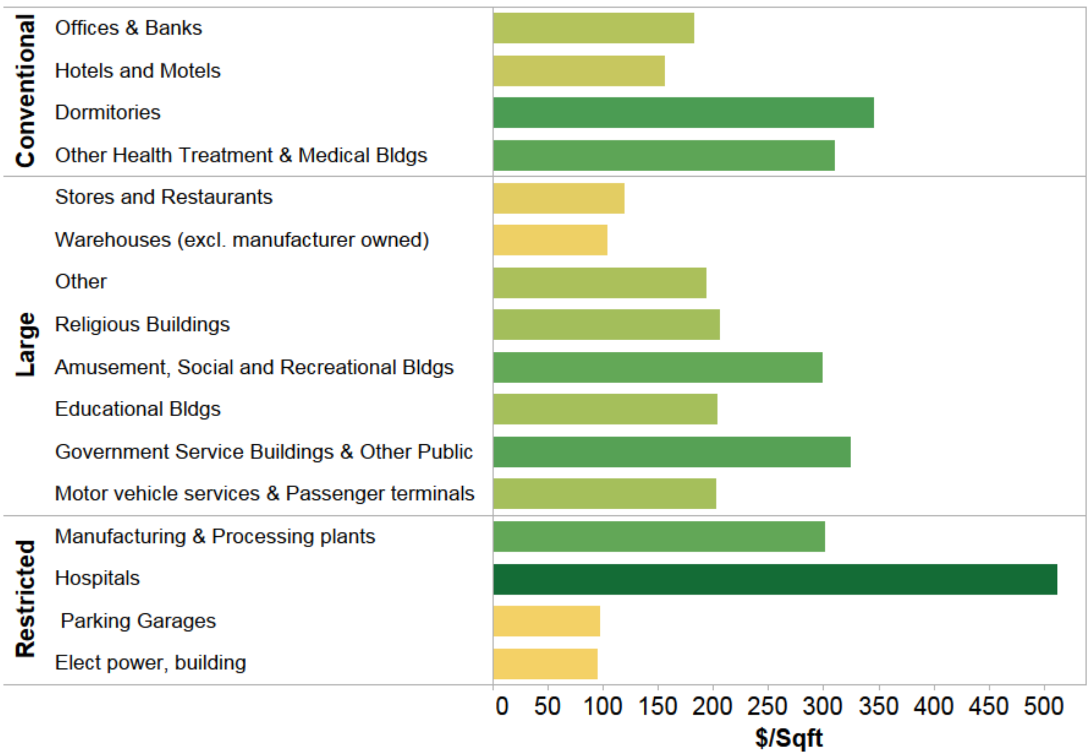

Figure 11 Canadian non-residential construction costs - 2012-2016 (source: FPInnovations, CMD)

1.11 BUILDING EXAMPLES

Since publication of the 2011 Edition of this Handbook, numerous buildings have been constructed and documented worldwide. The purpose of this section is to briefly introduce some examples of buildings built in Canada and the United States, using CLT elements. Additional information may be found in [www.thinkwood.com,](http://www.thinkwood.com) where the readers have access to research papers as well as case studies.

1.11.1 Multi-family Residential Buildings

Figure 12 Arbora Complex, Montréal, 8-storey multi-family residential building (courtesy of Nordic, top; FPInnovations, bottom)

Figure 13 Origine, Québec City, 13-storey multi-family residential building (courtesy of Stéphane Groleau, top; FPInnovations, bottom)

Figure 14 Brock Commons, Vancouver, 18-storey residential building (courtesy of Structurlam)

1.11.2 Office, Commercial and Institutional Buildings

Figure 15 Shoreline Medical Center (courtesy of Structurlam)

Figure 16 Garibaldi Fire Department, Oregon, United States (courtesy of Structurlam)

Figure 17 Soccer Stadium St-Michel, Montréal (courtesy of Stéphane Groleau)

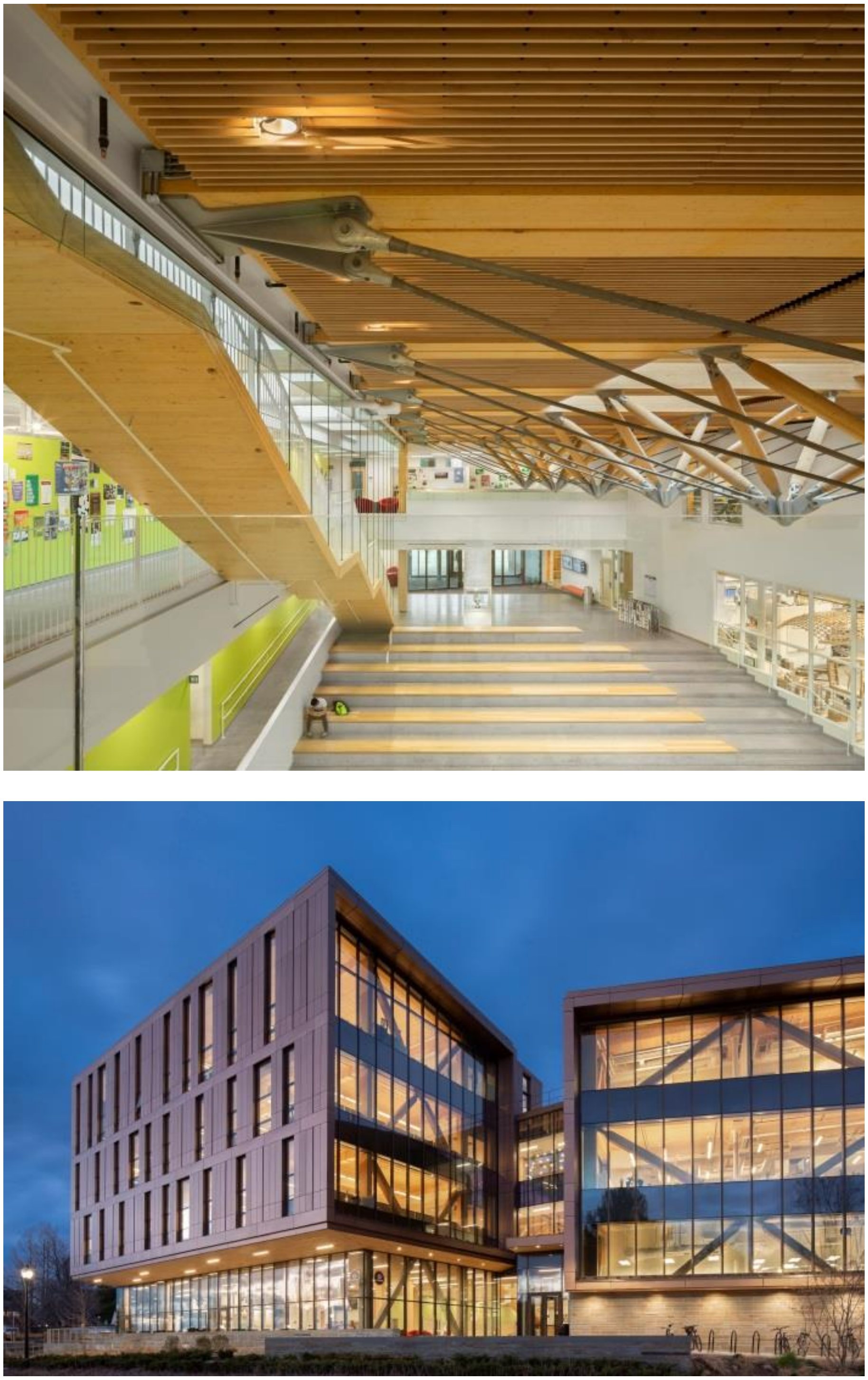

Figure 18 John W. Olver Design Building, Amherst, Massachusetts (courtesy of Stéphane Groleau)

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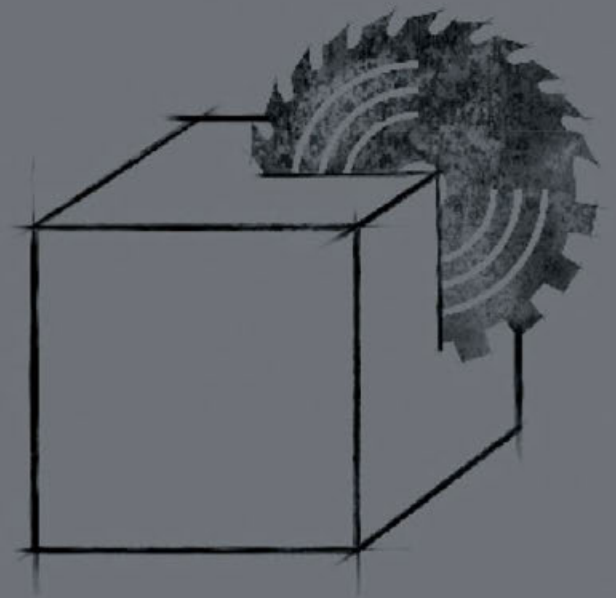

CHAPTER 2

Cross-laminated timber manufacturing

Lead Authors
Jean-Frédéric Grandmont
Borjen Yeh
Christian Dagenais

Reviewer
Sylvain Gagnon

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ABSTRACT

Based on the seed documents on Cross-Laminated Timber Plant Qualification and Product Standards that were published in the 2011 Edition of the Canadian CLT Handbook, a bi-national standard on CLT (ANSI/APA PRG 320) that is recognized by Canadian and U.S. regulatory systems was first released in 2011, and then updated in 2012, 2017, and 2018. In the meantime, a Canadian Construction Material Center Technical Guide (CCMC TG) on CLT was published in 2016 to contribute to CLT adoption across Canada, namely for CLT products that are non-conforming to ANSI/APA PRG 320. With these efforts, CLT made its way in both the Canadian and American wood design standards and building codes.

CLT panel manufacturing requires a good understanding of the properties and behaviour of the various components entering in its manufacturing process and an equally good understanding of the end-product characteristics and performance. This Chapter outlines multiple key elements of the manufacturing process that need to be considered when manufacturing CLT panels as per ANSI/APA PRG 320 Standard as well as CCMC TG.

Future development of CLT panels regarding their manufacturing may concentrate on the following elements:

- Determination of adequate manufacturing parameters for CLT panels made with SCL (e.g., surface preparation and tolerances, pressure) that would lead to the publication of guidelines and recommendations
- Harmonization of glue bond durability test requirements between Canada and the United States.

2.1 INTRODUCTION

Cross-laminated timber, or CLT, is hereby defined [1] as "*a prefabricated engineered wood product made of at least three orthogonal layers of graded sawn lumber or structural composite lumber (SCL) that are laminated by gluing with structural adhesives*" (see Figure 1). CLT is manufactured under controlled factory conditions by gluing laminations in layers, which are stacked crosswise, i.e., at 90 degrees, in a generally alternating manner. In special cases, double outer laminations may be parallel and not alternating crosswise. In typical CLT products, laminations from the outer layers correspond to the panel's major strength direction, while those arranged perpendicular to the outer layers correspond to the panel's minor strength direction.

CLT is a relatively new engineered wood product when considering alternative structural wood construction materials. Its development only dates back to the early 1990s, with commercial production in Europe starting in the early 2000s. It must therefore be stated that its very definition may evolve with time. For instance, ongoing work on curved CLT panels might eventually lead to a broadening of the aforementioned definition.

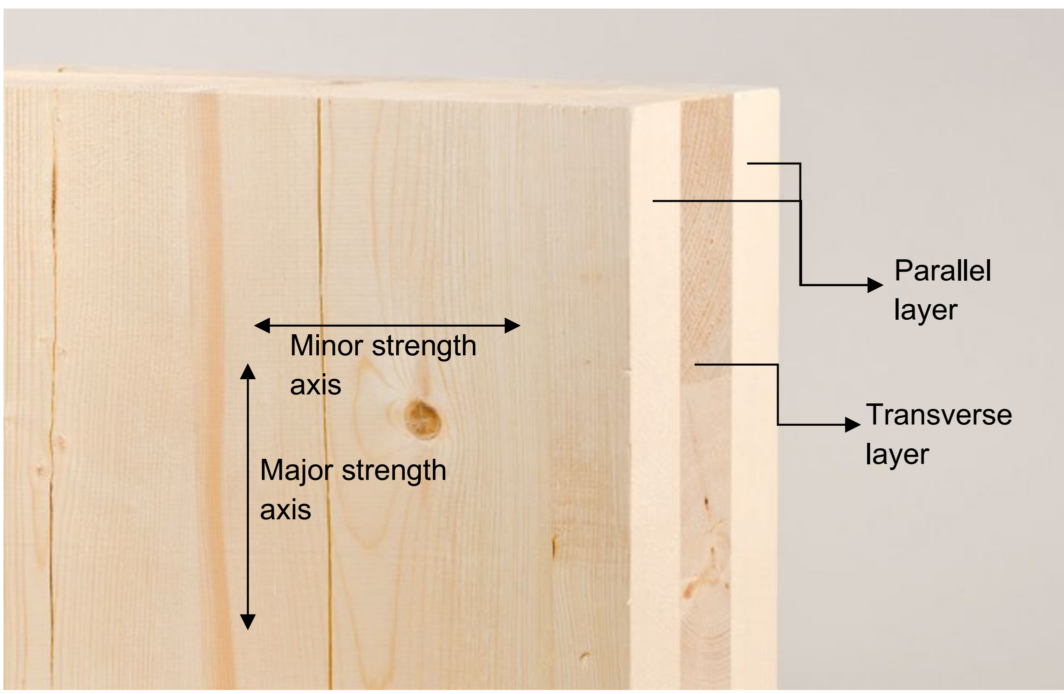


Figure 1 Cross-section of a three-layer CLT panel made with orthogonal layers of sawn lumber

CLT made its way in both the Canadian and American wood design standards shortly after a joint Canadian-American consensus-based product standard was first released in 2011: ANSI/APA PRG 320 Standard for Performance-Rated Cross-Laminated Timber [1]. Inclusion in Canada was part of the 2016 Supplement of *CSA O86-14 - Engineering design in wood* [2], while inclusion in the United States was in the 2015 International Building Code (IBC), the 2015 International Residential Code (IRC), and the 2015 edition of the *National Design Specification for Wood Construction* [3]. Furthermore, the 2015 National Building Code of Canada (NBC) implicitly recognizes ANSI/APA PRG 320 through its inclusion in CSA-O86-14 [4].

The bi-national product standard, ANSI/APA PRG 320, was first published in 2011, then updated in 2012, 2017, and 2018. In the meantime, a Canadian Construction Material Center Technical Guide (CCMC TG) [5] was prepared and published in 2016 to contribute to CLT adoption across Canada, namely for CLT products that are non-conforming to ANSI/APA PRG 320. Products conforming to ANSI/APA PRG 320 are not required to be evaluated according to the CCMC TG. This Technical Guide is available only to the clients of the CCMC.

Caution is warranted, as only CLT products complying with ANSI/APA PRG 320 were included in the aforementioned North American wood design standards. Designers, engineers, and other stakeholders should be aware of the fact that the differences between CLT products complying with ANSI/APA PRG 320 and those do not, go beyond a conversion of the design properties. Important attributes such as heat durability, wood density, moisture durability, and fire performance may vary greatly depending on the manufacturing process of the CLT panels, the laminating stock, and the adhesives that are used [6]. These fundamental characteristics are as important as the design values for the inclusion and design of CLT panels in North American wood design standards.

It is noted that while the definition given above and the information provided herein address CLT made with sawn lumber and SCL, the structural design provisions implemented in the 2016 Supplement of CSA O86-14 - *Engineering design in wood* [2] are not applicable to CLT made entirely or in part with SCL. The same restriction does not apply to the NDS in the United States.

2.2 COMPONENT REQUIREMENTS

Selection of the various components used in the manufacturing of CLT panels is of paramount importance. Not only is it important to have an adequate understanding of the components such as the lamination materials and adhesives, but of equal importance is a good understanding of their combined behaviour and of the effect the manufacturing processes have on the end-product performance. Evaluated and approved products by a certification body as meeting either ANSI/APA PRG 320 [1] or the CCMC Technical Guide [5] provide assurance of product quality and performance.

CLT can be manufactured from sawn lumber (see Figure 1), structural composite lumber (SCL, see Figures 2 and 3), or a combination of both (see Figures 4 and 5). SCL includes products such as laminated veneer lumber (LVL), laminated strand lumber (LSL), oriented strand lumber (OSL), and parallel strand lumber (PSL). Common to both types of lamination is that face bonding is accomplished using a structural adhesive. It should be noted that none of the illustrated product featuring SCL laminations are either currently available or certified through ANSI/APA PRG 320 or the CCMC TG. As such, only CLT panels made of sawn lumber are currently manufactured and certified according to ANSI/APA PRG 320, while none are certified according to the CCMC TG.

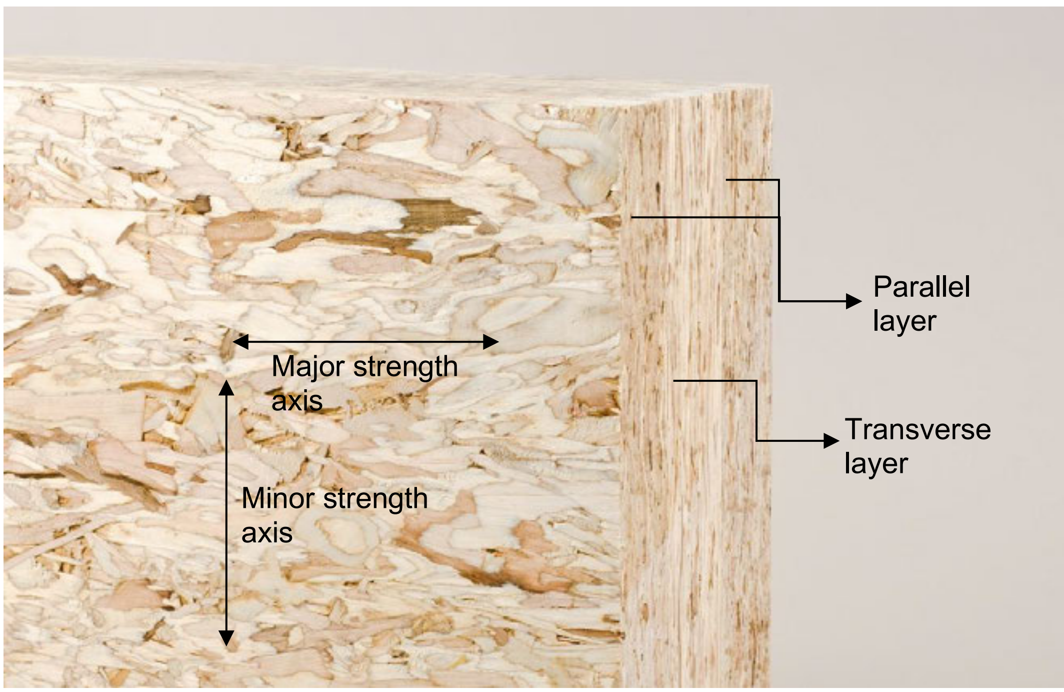


Figure 2 Cross-section of a three-layer CLT panel made with orthogonal layers of LSL

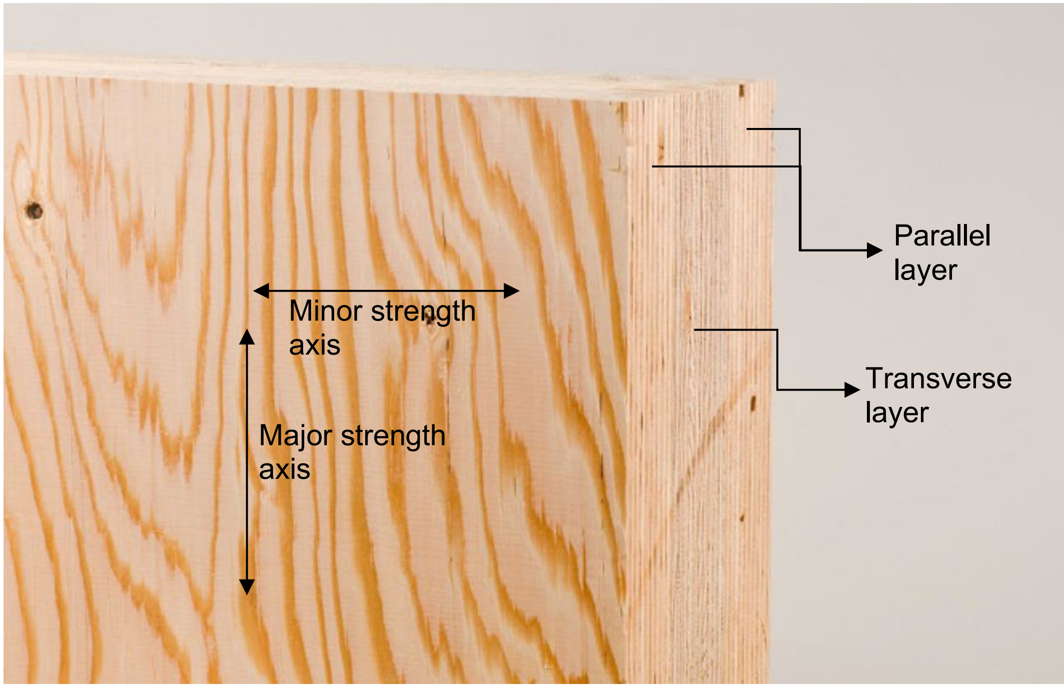


Figure 3 **Cross-section of a three-layer CLT panel made with orthogonal layers of LVL**

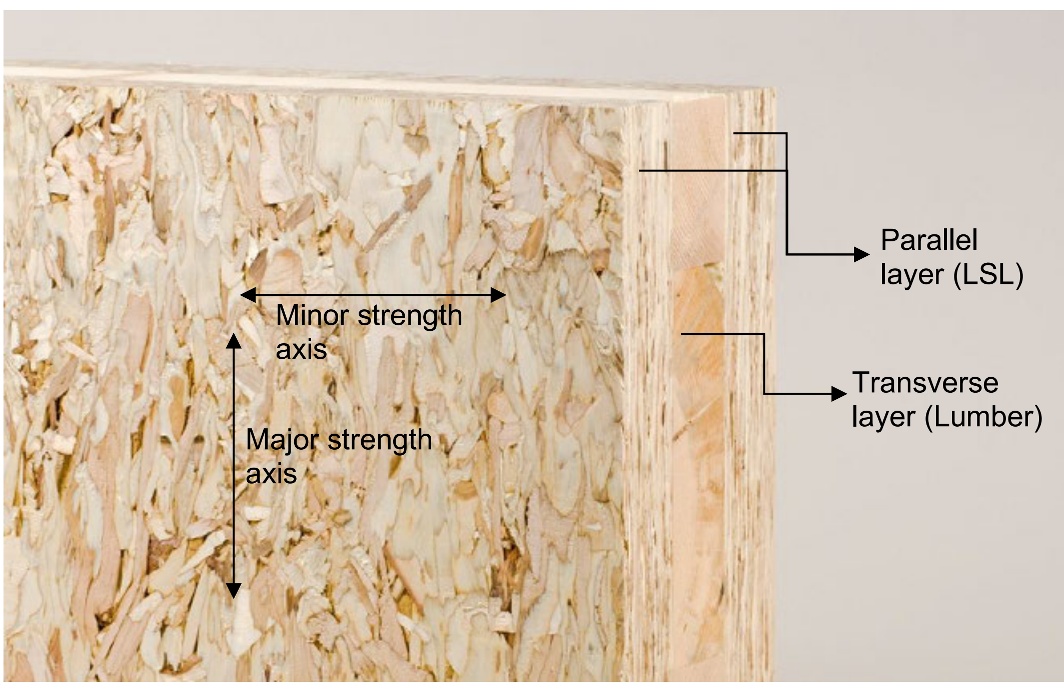


Figure 4 **Cross-section of a three-layer CLT panel made with a combination of LSL (outer layers) and sawn lumber (transverse inner layer)**

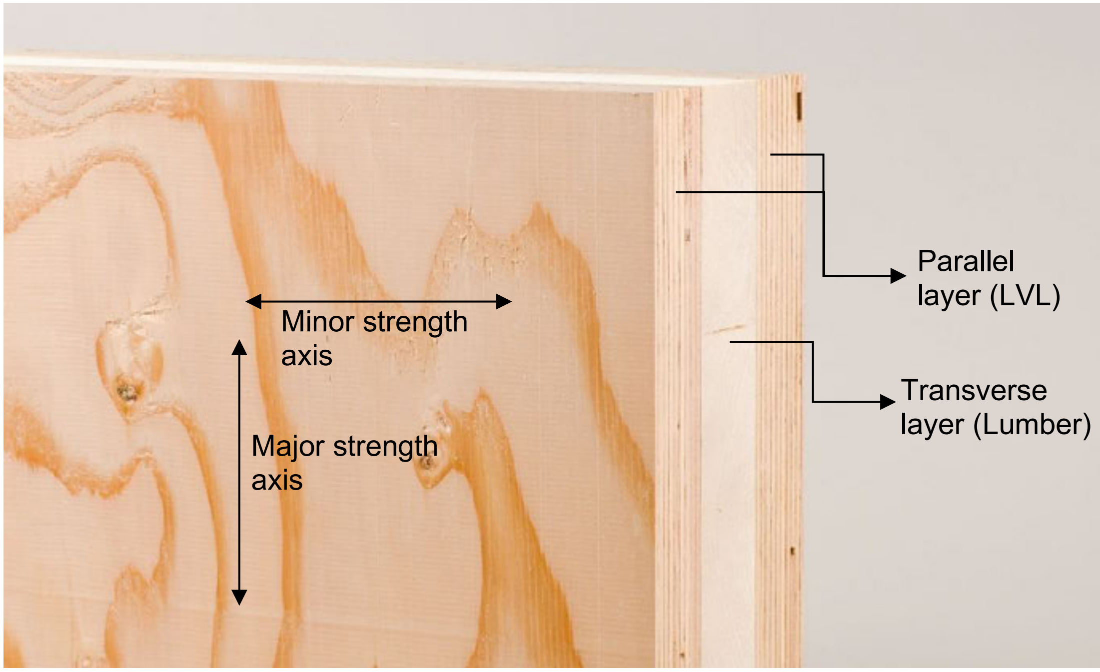


Figure 5 Cross-section of a three-layer CLT panel made with a combination of LVL (outer layers) and sawn lumber (transverse inner layer)

2.2.1 Laminations

2.2.1.1 Theoretical Considerations for Selecting Sawn Lumber Laminations

This Section aims at outlining a number of wood properties that have an important effect on the end-product properties that are mostly due to the orthogonally layered nature of CLT panels. It does not constitute an exhaustive review of the theoretical considerations on the matter.

CLT panels made of sawn lumber are the only currently available products that are certified according to ANSI/APA PRG 320, despite other options being available. The advantages of using graded sawn lumber are numerous and include:

- Lowest raw material cost for manufacturing CLT panels
- Layered nature of the CLT panels allows for use of specific and optimized lumber grades for the various layers, which in turn allows for better raw material recovery; thus, layers where the stress is lower can advantageously be made of lower grade lumber
- Using heat-treated lumber graded according to recognized standards allows, amongst other things, for compliance with exportation phytosanitary requirements
- Analytical calculations of the CLT panel properties can be performed based on the design values of the laminations
- Layered nature of the CLT panels allows for improved dimensional stability of the end-product compared to solid wood
- Possibility of finger-jointing sawn lumber according to recognized standard to manufacture large size panels and improve wood fiber recovery.

Lamination properties will greatly dictate the performance and properties of the end-product. Laminations must therefore be carefully considered to ensure that the in-service performance of the end-product matches expectations.

Wood is a heterogeneous and hygroscopic material. Its heterogeneity is due to its anatomy and its properties are therefore anisotropic. Its properties are considered different in three main directions – longitudinal, radial and tangential (see Figure 6) – but the anisotropy is not random. Wood is consequently a cylindrical orthotropic material.

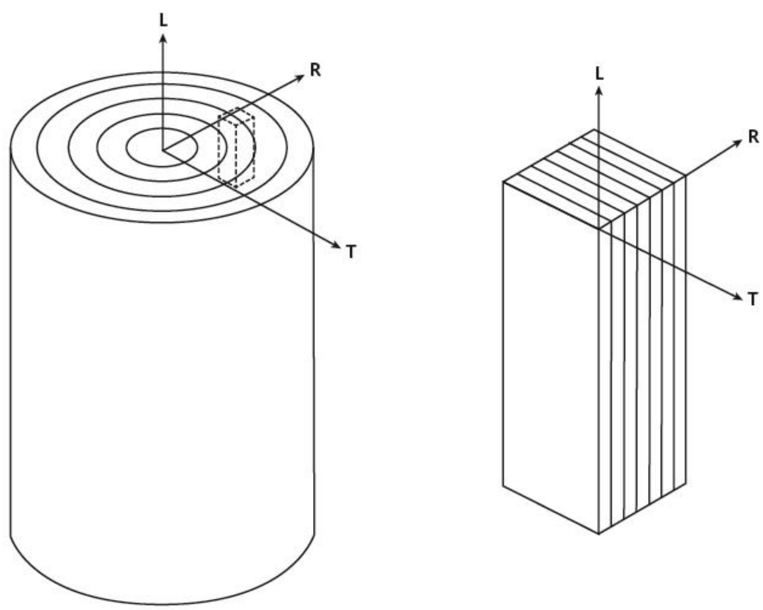


Figure 6 Wood's main directions: longitudinal (L), radial (R), and tangential (T)

For practical purpose, radial and tangential directions are typically assumed to be similar and therefore considered as one direction, which is also referred to as the transverse direction, since wood logs are not cut following these specific directions. The cross-section of the resulting products has a somewhat random orientation of their radial and tangential directions.

The three main directions display specific sets of properties, i.e. the properties differ for all three directions. Stiffness is one of the main characteristics upon which the material may be selected. The ratios between the modulus of elasticity (E) in the three main directions generally follow the relationship below [7]:

Equation 1: $\boldsymbol{E_L}:\boldsymbol{E_R}:\boldsymbol{E_T} \approx \mathbf{20:1.6:1}$

where E_L, E_R and E_T are the modulus of elasticity along the longitudinal, radial and tangential direction, respectively. For the purpose of CLT modeling, however, it is customarily assumed that the modulus of elasticity in the transverse direction is about 1/30 of that in the longitudinal direction.

Although wood is often primarily used in its longitudinal direction (which has the highest strength and stiffness), CLT properties, however, will be greatly influenced by the properties in the other two directions as well, because of the orthogonally layered structure of the panels.

Strength and other properties must also be considered for all three directions to ensure proper material selection. Properties must be considered for the various applications and layups. Orthogonally layered engineered wood products are far from new, as plywood is structured in this way. CLT panels, however, differ from plywood because the thickness of the layers is far greater. This characteristic makes it more sensitive to planar shear (or rolling shear) when subjected to out-of-plane loading (e.g., flexure).

Wood is sensitive to splitting in the tangential direction. In other words, wood's lowest tensile strength is in the tangential direction, followed by the radial direction. When a CLT panel is subjected to out-of-plane loading, planar stress occurs and causes rolling shear stress, as illustrated in Figure 7. Rolling shear is the result of the lower resistance of the wood lamination in the radial and tangential directions, inducing failure across the annual rings. Sensitivity to planar shear may become a limiting factor when selecting a CLT panel, due to local tensile stress in the transverse direction of the lamination induced by the rolling shear forces.

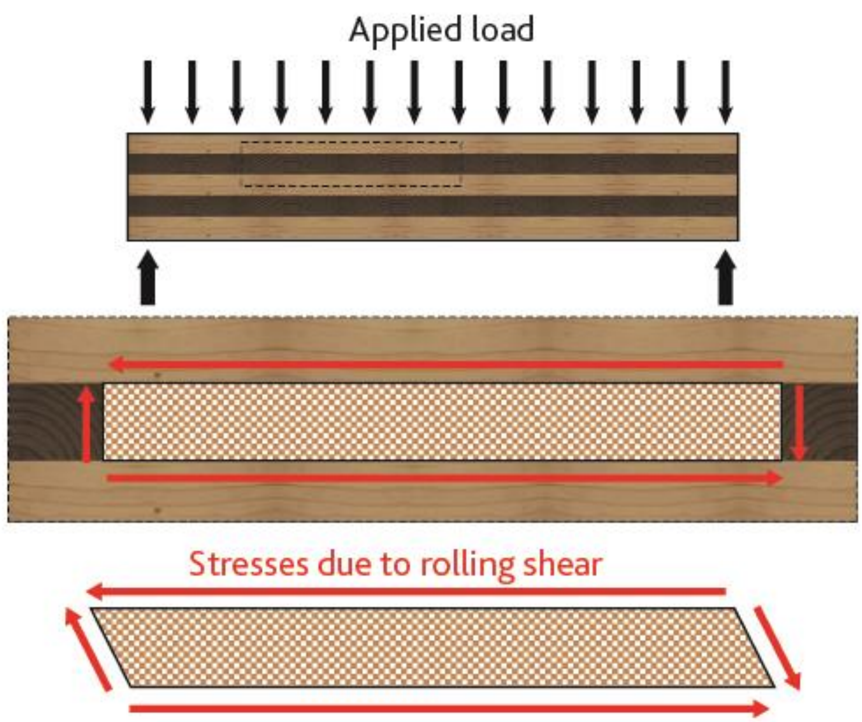


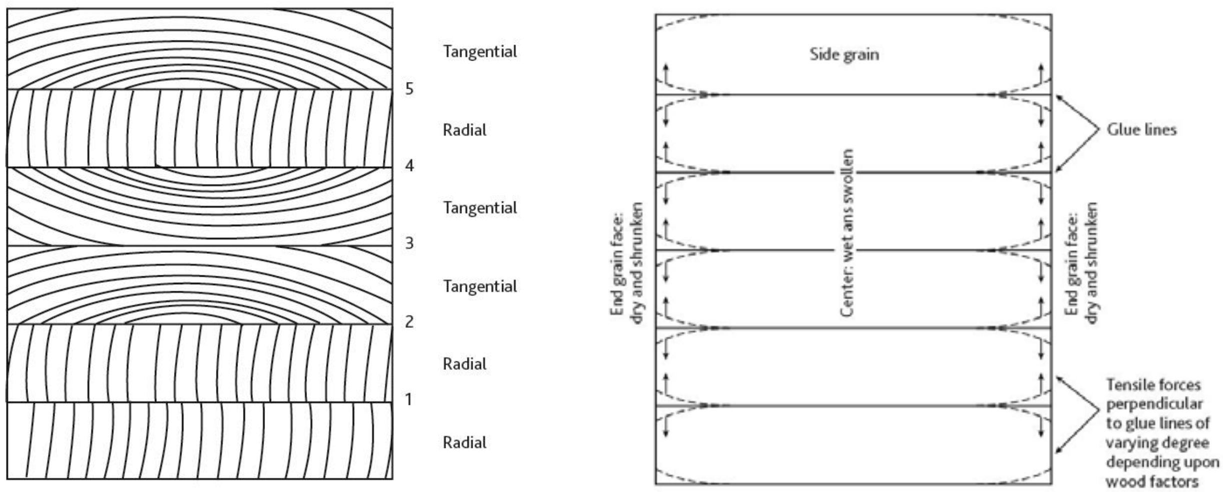
Figure 7 Rolling shear in CLT

As such, lamination material selection may be influenced by the wood species and the grade's inherent ability to resist rolling shear in a CLT panel. CLT layup design has been shown to have an influence over the sensitivity of the panel to rolling shear, with increased mechanical properties associated with thinner laminations and reduced gaps between laminations [8]. A combination of carefully designed CLT layup and material selection appears to be a key parameter in achieving the manufacturing of optimised products. These parameters may provide possible solutions for special cases when planar shear governs the CLT panel design.

Moreover, the thickness of the layers constituting a CLT panel makes it sensitive to moisture content (MC) changes, when compared to other mass timber products. Moisture content changes lead to dimensional changes of the wood component. Swelling occurs when the substrate adsorbs water and shrinkage occurs when it desorbs water. The dimensional changes follow the three main directions (longitudinal, radial, and tangential) of the lumber and follow a trend opposite to most mechanical properties (i.e., the shrinkage and swelling is higher for the tangential direction, followed by the radial direction and the longitudinal direction). Longitudinal dimensional changes are often considered negligible, which is obviously not the case for both tangential and radial directions. These phenomena occur between moisture contents ranging from 0% to the fiber saturation point (which varies from one species to another but generally ranges between 25-30% MC). Black spruce, for instance, has a total shrinkage from green to oven dry state of 4.1% in the radial direction and 6.8% in the tangential direction ([9], [10]). In the longitudinal direction, total shrinkage for the same MC conditions is considered to vary between 0.1% and 0.2% for most wood species [10]. In service, a change in MC will therefore have an impact on the CLT panels.

Similarly to CLT panels, glue-laminated timber (glulam) is made of thick laminations of solid wood. The impact of the lamination dimensional changes due to variations in MC in a glulam is well documented. In a glulam, all the laminations are oriented in the same direction. As a result, the dimensional changes have an almost negligible effect in the longitudinal direction of the glulam. However, stress occurs when there is an MC change and when the radial and tangential directions are mixed in the cross-section. Furthermore, whenever there is an MC gradient within the lamination or the glulam itself, stress occurs, as shown in Figure 8a). Glulam delamination tests are based on this principle: specimens are soaked with water before being subjected to a harsh drying environment for a given period of time. Specimens are evaluated for delamination at the bond line, while the core of the specimen is still wet and swollen, and the surfaces are dry, which generates stress at the bond lines, as seen in Figure 8b).

The aforementioned dimensional change effects on the internal stress of glulam also occur in CLT panels. However, the cross layering of the laminations in a CLT panel causes an additional stress when wood laminations are subjected to dimensional changes due to varying MC conditions. Given that longitudinal shrinkage or swelling is almost negligible when compared to the transverse directions, any change in the lamination cross-section will be constrained by the adjacent orthogonal CLT layers (in a typical layup), which will result in differential dimensional changes across adjacent layers and induce greater stress at the glue lines. The magnitude of these constraints depends on multiple manufacturing parameters, including the wood species. This is typically not observed with glulam, as all laminations are oriented along the same direction.



a) due to moisture gradient during cycling

b) due to differential dimensional change between radial and tangential cut boards

Figure 8 Forces acting in the lumber delamination specimen (adapted from [11])

Different wood species are characterized by different total shrinkage and swelling values. Figure 9 shows two laboratory-made CLT specimens using either a Maple-SPF-Maple layup (sugar maple and spruce-pine-fir wood species group) or a SPF-SPF-SPF layup. There were no gaps between laminations in either panel at the time of manufacturing. Black spruce, which is one of the wood species included in the SPF wood species group, is presumed to have a total shrinkage from green to oven dry state of 4.1% in the radial direction and of 6.8% in the tangential direction; sugar maple comes in at 4.8% total shrinkage in the radial direction and 9.9% in the tangential direction [10]. Those differences are the root cause of the larger gaps that formed between laminations for the sugar maple laminations, when compared to those of the SPF laminations, as shown in Figure 9. Given that sugar maple has a higher density and a higher stiffness in the transverse directions than black spruce, greater dimensional changes in those directions as well as higher stress levels at the bond line are to be expected for a similar change in the lamination MC. Stress levels can be estimated by simple calculations based on expected dimensional changes and transverse stiffness of the laminations, to evaluate the potential suitability of using a given wood species for manufacturing CLT panels. Transverse direction stiffness can be estimated based on Equation 1, for that specific purpose.

Figure 9 Cross-sections of laboratory-made CLT specimens made with (a) Maple-SPF-Maple and (b) SPF-SPF-SPF panels after conditioning at 20°C and 65% RH (initially with no measurable gaps between laminations), followed by conditioning at 35°C and 22% RH [12]

2.2.1.2 Theoretical Considerations for Selecting SCL Laminations

This Section aims at outlining a number of SCL properties that may have an important impact on the end-product properties that are mostly due to the orthogonally layered nature of the CLT. It does not constitute an exhaustive review of the theoretical considerations on the matter.

While SCL complying with ASTM D5456 [13] is deemed suitable for manufacturing CLT ([1], [5]), no CLT manufacturing plant is currently using SCL. The use of SCL could potentially bring some benefits to the manufacturer and the product characteristics [12], provided the considerations detailed herein are properly examined, including cost.

As mentioned in Section 2.2.1.1, the cross-layered nature of CLT panels constrains the shrinkage and swelling movements of the laminations. As most SCL products are characterized by advantageous dimensional stability when compared to sawn lumber, their use in CLT layers may minimize internal stress, due to smaller differential swelling or shrinkage.

Additionally, while most SCL products are sold at specified dimensions, they generally are manufactured in much larger dimensions than traditional sawn lumber. SCL can be manufactured in either a continuous or a discontinuous process, before being cut to final dimensions. The uncut billets of SCL may be advantageously used to manufacture CLT panels considering that:

- A smaller number of pieces of laminations would need to be manipulated at the manufacturing facility
- Less adhesive would be wasted in gaps between laminations
- A smaller number of gaps between laminations may minimize sensitivity to rolling shear forces [8].

Other potential advantages include:

- Potentially higher planar (rolling) shear resistance
- Potentially less variability of their physical and mechanical properties
- Free of natural defects such as wane, shake, and knots.

However, the use of SCL in manufacturing CLT panels may be challenging in some respects. Gluing together several layers of SCL laminations may require surface preparation of the glued surface before the assembly, to activate the surface and clean it from contaminants. Surface preparation of SCL billets may be a difficult task to achieve despite the availability of some equipment to perform that operation.

Density is also a property that is worth considering, as it may influence the selection of the laminations. OSL and LSL are characterized by a higher density than most softwood lumber species currently used in CLT manufacturing. Since one of the advantages of CLT—when compared to other building materials—is its lower weight, the use of denser laminations should therefore be carefully considered.

2.2.1.3 Laminations for CLT Manufacturing

ANSI/APA PRG 320 [1] was initially developed to take into account the construction applications prevalent in North America. Provisions were therefore included in the CLT product standard from the beginning, to take advantage of readily available dimensional lumber. Nonetheless, the standard also permits the use of SCL when qualified in accordance with ASTM D5456 [13]. The CCMC TG also has similar provisions.

2.2.1.3.1 Lumber Laminations

Sawn lumber is not only widely available in North America, but is also standardized in both Canada and the United States. ANSI/APA PRG 320 permits the use of any softwood lumber species or species combinations recognized by the Canadian Lumber Standards Accreditation Board (CLSAB) under CSA O141 [15], or the American Lumber Standards Committee (ALSC) under PS 20 [14], with a minimum specific gravity (SG) of 0.35, as published in the Engineering Design in Wood (CSA O86) [2] in Canada or the National Design Specification for Wood Construction (NDS) [3] in the United States. The use of sawn lumber complying with either the Canadian or American standard (or both) facilitates CLT exports and exchanges from both sides of the Canada-USA border, since these products are typically heat-treated as part of the drying process. Complying product can then be stamped as heat-treated (HT), as they fulfill phytosanitary regulations while ensuring traceability.

The minimum SG of 0.35 is intended as the lower boundary for CLT connection design since it is near the minimum value of commercially available wood species in North America, e.g., Western Woods in the United States and Northern Species in Canada. To avoid differential mechanical and physical properties of the lumber, the standard requires that the same lumber species or species combination be used within the same layer of the CLT, while permitting adjacent layers of CLT to be made of different species or species combinations, whenever practical.

Lumber grades in the major strength axis of CLT panels are required to be at least 1200f-1.2E MSR or visually graded No. 2. Visually graded No. 3 is the minimum lumber grade required in the minor strength direction. Remanufactured lumber is permitted as equivalent to sawn lumber when qualified in accordance with ANSI A190.1 [17] in the United States or SPS 1, 2, 4, or 6 ([18], [19], [20], [21]) in Canada. Proprietary lumber grades meeting or exceeding the mechanical properties of the lumber grades specified above are permitted, provided they are qualified in accordance with the requirements of an approved agency, which is defined in the standard as an independent inspection agency accredited under ISO/IEC 17020 [22] or an independent testing agency accredited under ISO/IEC 17025 [23] in the United States, or a certification agency accredited under ISO/IEC 17065 [24] in Canada. This allows for a great flexibility in the utilization of forest resources in North America.

Provisions are similar for the CCMC TG regarding lumber grading but specify that if more than 3% of the original thickness on either face is removed, the lumber shall be re-graded before being used as lamination.

The ANSI/APA PRG 320 standard requires the net lamination thickness for all CLT layers at the time of gluing to be at least 16 mm (5/8 inch), but no thicker than 51 mm (2 inches), to facilitate face bonding. In addition, the lamination thickness is not allowed to vary within the same CLT layer, except when it is within the lamination thickness tolerances; at the time of face bonding, variations in thickness across the width of a lamination is limited to ±0.2 mm (0.008 inch) or less, and the variation in thickness along the length of a lamination is limited to ±0.3 mm (0.012 inch).

These maximum tolerances may need to be adjusted during qualification, in order to produce acceptable face bond performance.

The provisions in the CCMC TG [5] are slightly different, as the minimum lamination thickness is set to 17 mm. Also, the TG allows for the addition of an aesthetic (non-structural) layer glued to one of the CLT surfaces. These laminations shall be thinner than 17 mm but thicker than 5 mm. The addition of such a layer is not considered to cause the resulting CLT to be qualified as unbalanced.

The net lamination width is required to be at least 1.75 times the lamination thickness for the longitudinal layers in the major strength direction of the CLT. This means that if nominal 2x lumber (35 mm or 1-3/8 inches in net thickness after surfacing prior to gluing) is used in the longitudinal layers, the minimum net lamination width must be at least 61 mm (2.4 inches), i.e., nominal 2x3 lumber. On the other hand, the net lamination width is required to be at least 3.5 times the lamination thickness for the transverse layers if the laminations in this direction are not edge-bonded, unless the planar shear strength and creep of the CLT are evaluated by testing. This means that if nominal 2x lumber is used in the transverse layers, the net lamination width must be at least 122 mm (4.8 inches), i.e., nominal 2x6 lumber.

This minimum lamination width in the transverse layers (minor strength direction) could become a problem for CLT manufacturers who prefer to use nominal 2x3 (net 38 mm x 63 mm or 1-1/2 inches x 2-1/2 inches) or nominal 2x4 lumber (net 38 mm x 89 mm or 1-1/2 inches x 3-1/2 inches). However, the ANSI/APA PRG 320 Technical Committee was concerned about the unbonded edge joints, which could leave gaps and act as potential stress risers. These, in turn, may reduce the effective planar shear strength and stiffness, and may result in excessive creep. Therefore, in this case, the manufacturers will have to either edge-glue the laminations or demonstrate the conformity to the standard by conducting planar shear tests and ASTM D6815 [25] creep tests. It should be noted that this is an interim measure due to the lack of data available at this point in time to address these concerns. Further research is being conducted in this area to better understand the potential for improvement.

2.2.1.3.2 SCL Laminations

Both the CCMC TG and the ANSI/APA PRG 320 standard require SCL to be compliant with ASTM D5456 [13]. The CCMC TG however, further requires SCL to be approved under its own set of performance requirements.

Some SCL mechanical properties may need to be further evaluated for use as laminations in CLT manufacturing. Rolling shear properties are amongst those and the CCMC TG includes an appendix dedicated to the determination of the aforementioned property. The methodology is based on *Standard Test Method A - Planar Shear Loaded by Plates, in ASTM D2718* [26]. The same procedure is deemed suitable for lumber laminations as well.

2.2.2 Adhesives

2.2.2.1 Considerations Concerning Adhesives and CLT Panels

This Section aims at outlining some adhesive characteristics that have an important effect on the end-product properties, namely the orthogonally layered nature of CLT. It does not constitute an exhaustive review of the considerations on the matter.

Adhesives are essential and critical to the manufacturing of CLT panels. Being a structural component, CLT panels are required to be manufactured in a way that will ensure public safety throughout its service life. Therefore, there is a strong emphasis in both ANSI/APA PRG 320 and the CCMC TG on the evaluation of the adhesives' performance.

As pointed out in Section 2.2.1.1 of this Chapter, CLT's orthogonal layered nature causes additional stress at the bond lines due to either differential shrinkage or swelling, when compared to glulam. The Canadian [27] and American [28] glulam standards were nonetheless used as reference for part of the development of both the ANSI/APA PRG 320 standard and the CCMC TG, as the products they respectively cover are mostly similar, in principle.

Ideally, the structural wood adhesive should not be a limiting factor in the performance of an engineered wood product that has been made with said adhesive, under any circumstances. The spectrum in which that philosophy is pursued can vary, depending on the application in which the product is used: exterior use calls for performance under broader types of potentially adverse conditions than interior use.

It should be pointed out that there are currently no adhesive standard dedicated to CLT products. Therefore, both ANSI/APA PRG 320 and the CCMC TG rely on adhesive standards that were developed for specific products such as glulam (ANSI 405 [28]), or for generic structural applications (CSA O112.9 [29] and CSA O112.10 [30]). Consequently, even if a given adhesive complies with these standards, it may fail in the manufacturing of CLT panels that comply with the CLT manufacturing standards. As adequately noted in CSA O112.10: *"This Standard is applicable primarily to the evaluation of structural and semi-structural wood adhesives intended for bonding solid wood to solid wood, such as in laminated lumber, edge-glued lumber, and finger-joined lumber. It serves as a pre-screening test for adhesives to be used for products intended for dry service conditions. Additional requirements might be specified by the applicable product Standard based on the product configuration and the product's end use."* The aforementioned quote outlines that due diligence is warranted even though the standard covers structural uses of the adhesives. It is worth noting that the glulam and the structural adhesives standards referred to in this Section are essentially based on parallel-to-grain glued specimens and therefore may not adequately address the performance of joints between laminations that have wood fiber components orthogonally oriented to one another. Both ANSI/APA PRG 320 and the CCMC TG therefore have additional provisions regarding glue-bonded surface performance.

The bond performance in CLT panels is not solely based on the adhesive performance, but rather on the combination of multiple manufacturing elements. ANSI/APA PRG 320 and the CCMC TG have provisions that allow for pre-qualification of CLT panels. It may therefore be advisable to perform tests on adhesives theoretically complying with the CLT manufacturing standards, prior to testing them on production scale panels. Section 2.2.2.2 provides additional information relative to CLT panels adhesive requirements that are included in ANSI/APA PRG 320 and the CCMC TG.

2.2.2.2 Adhesives for CLT Panels

In Canada, CLT adhesives must meet the requirements of the CSA O112.10 standard [30] and Sections 2.1.3 and 3.3 (ASTM D7247 heat durability [31]) of ANSI 405. In addition, in both Canada and the United States, CLT adhesives have to be evaluated for elevated temperature performance in accordance with the small-scale flame tests of CSA O177 [27] (using CLT specimens rather than glulam specimens) and a full-scale compartment fire test, as specified in the mandatory Annex B of the ANSI/APA PRG 320 – 2018 edition. The intent of the elevated temperature performance evaluation is to identify and exclude adhesives that allow CLT char layer fall-off resulting in fire regrowth during the cooling phase of a fully developed fire. It is recognized that CLT manufactured in accordance with previous editions of ANSI/APA PRG 320 may exhibit char layer fall-off under an extensive and unattended fire, resulting in an increased effective charring rate and a potential for fire re-growth in a compartment fire ([33], [34], [35]). It should be noted that the fire design in the current NDS and CSA O86 is based on the assumption that such a phenomenon can occur, and that fire re-growth is a concern for wood buildings if the CLT is left unprotected.

The CCMC TG has similar requirements. Variables such as time and temperatures for ASTM D7247 heat durability [31] are stated directly in the TG (i.e., 220°C for both the bonded and solid wood specimens), while ANSI/APA PRG 320 references ANSI 405, which in turn references ASTM D7247 for the same test and qualification requirements. While the CCMC TG does not mandate a full-scale compartment fire test, it requires a full-scale fire-resistance test on a CLT floor specimen to assess its charring behavior. The specimen tested must be representative of the commercial production and manufactured using laminations of the lowest thickness and density used by the manufacturer. A minimum of three bond lines are to be entirely charred when exposed to the standard fire curve in CAN/ULC S 101 [36]. The charring rate is measured with thermocouples positioned at various locations across and within the CLT specimen.

Several types of structural adhesives have been evaluated for CLT commercial production and pilot/experimental production, as listed below:

- Phenolic types such as phenol-resorcinol formaldehyde (PRF)
- Emulsion polymer isocyanate (EPI)
- Melamine formaldehyde (MF)
- One-component polyurethane (PUR).

As the full-scale compartment fire test and the small-scale CSA O177 tests are relatively new (i.e., approved in ANSI/APA PRG 320-2018), not all the adhesives mentioned above have been evaluated for compliance.

PRF is a well-known adhesive for structural use, namely for glulam manufacturing in North America. MF adhesives also are commonly used for glulam manufacturing and have the advantage of being characterized by a very light color once cured, one of the reasons why some producers chose this adhesive. EPI adhesives are typically used for end jointing sawn lumber for wood I-joists and for face gluing laminations. PUR adhesives have been and are still the most commonly used type of adhesive in Europe, Canada and the United States to produce CLT. They are also used to manufacture other engineered wood products.

It should be noted that there is a large variety within a given type of adhesives and that not all formulations within a given type will meet the requirements of the structural adhesive standard. There may be considerable variations in working properties within each adhesive type. Documentation showing that a given adhesive has met the appropriate standards is required for CLT product certification. In addition, the working properties of the adhesive required by the manufacturing process should be discussed with the adhesive supplier.

In addition to cost and working properties, each adhesive type possesses other attributes that may be important. For example:

- Open assembly time
- Curing time
- Colour (e.g., PRF is dark brown whereas EPI and PUR are light-coloured)
- One-component vs two-components adhesives
- Presence of solvent or formaldehyde (e.g., PUR is free of both)
- Moisture reactivity and sensitivity
- Gap filling capability (structural and non-structural gap filling).

It is therefore paramount for the CLT panel manufacturer to select an adhesive that can both yield panels with the desired end-product attributes and meet the needs of the manufacturing process.

2.2.3 Lamination Joints

There are three common types of lamination joints within CLT panels:

1. End-joints
2. Face-joints (joints on the wide face of the lamination)
3. Edge joints (joints on the narrow face of the lamination) (optional).

According to ANSI/APA PRG 320, end-joints must be qualified in the same way as they are for glulam, in both the United States and Canada. Consequently, it is required that they be qualified in accordance with the relevant sections of the glulam standard, ANSI A190.1 [17] in the United States and CSA O177 [27] in Canada. The CCMC TG does not include specific provisions for end-joints, so they must therefore be qualified based on the accepted types of products, as listed in Section 2.1. It must be pointed out, however, that the TG limits the amount of material that can be removed from the laminations when preparing the surface for CLT assembly to 3%. Removing more material than prescribed would require the CLT manufacturer to re-grade the laminations according to the corresponding standard.

Furthermore, ANSI/APA PRG 320 requires that face joints between adjacent laminations be qualified in accordance with the relevant sections of the glulam standard, CSA O177 [27] in Canada and ANSI A190.1 [17] in the United States, except that the planar shear strength criteria is not applied, due to the lower planar shear strength when adjacent laminations are in the transverse direction. The CCMC TG requirements for face-joint qualification are inspired by the CSA O177 requirements, but include some minor differences, such as the specific requirement to perform a three-cycle in twelve days delamination test, instead of offering the possibility of relying on intensive quality control testing. The delamination test is also slightly different, with a pressure-vacuum cycle that calls for a two-hour positive pressure step before a two-hour vacuum step, instead of a thirty-minute positive pressure step.

ANSI/APA PRG 320 does not require edge-gluing between laminations in the same layer of a CLT, unless CLT structural and/or fire performance is qualified based on the use of adhesive-bonded edge joints. The CCMC TG does not provide specific provisions for edge-glued laminations, with the exception of those meeting accepted types of laminations, such as NLGA SPS-6 [21]. As the CCMC TG provisions are included for proprietary grades, it may be conceivable to consider edge-glued laminations exceeding the maximum sizes in which SPS-6 lumber can be produced (which are limited to a maximum nominal size of 2x12 by 7.3 m, or 24 feet).

2.3 CLT REQUIREMENTS

2.3.1 Dimensions and Dimensional Tolerances

CLT thickness is currently limited to 508 mm (20 inches) or less, in ANSI/APA PRG 320. This is considered an upper limit for handling of the CLT panels in production and transportation. In addition, the dimension tolerances permitted at the time of manufacturing are as follows:

- Thickness: ± 1.6 mm (1/16 inch) or 2% of the CLT thickness, whichever is greater
- Width: ± 3.2 mm (1/8 inch)
- Length: ± 6.4 mm (1/4 inch)

The reference moisture content of the CLT panel for those dimensional tolerances is the lamination moisture content at the time of manufacturing, which is limited to 12 ± 3% for lumber and 8 ± 3% for SCL.

The CCMC TG requirements are slightly different, in that the reference moisture content is set to 15% for lumber and 7-8% for the dimensions of the CLT panels. Dimensional tolerances are slightly different (most likely due to rounding) and are as follows:

- Thickness: ± 2 mm or 2% of the CLT thickness, whichever is greater
- Width: ± 3 mm
- Length: ± 6 mm

In either case, textured or other face or edge finishes are permitted to alter the tolerances. However, designers need to compensate for any loss in cross-section and/or specified strength due to such alterations.

The ANSI/APA PRG 320 standard and the CCMC TG also specify the CLT panel squareness, defined as the length of the two panel face diagonals measured between panel corners, to be 3.2 mm (1/8 inch) or less and 3 mm, respectively. In addition, the CLT panel straightness, defined as the deviation of the edges from a straight line between adjacent panel corners, is required not to exceed 1.6 mm (1/16 inch) in the ANSI/APA PRG 320 standard and 2 mm in the CCMC TG.

2.3.2 Panel Classification

CLT panels are classified structurally based on the CLT layup. The lumber species and stress grades used in the major strength layer and those used in the transverse layers dictate the CLT panel classification.

2.3.2.1 Layups and Stress Classes

The CCMC TG contains CLT stress classes that must be determined by acceptable engineering analysis or by the applicable design standard, based on the panel composition.

As part of a standardization effort, seven CLT layups are stipulated in ANSI/APA PRG 320; custom CLT products are also recognized, provided the products are qualified by an approved agency in accordance with the qualification and mechanical test requirements specified in the standard. The different CLT layups are described in terms of structural properties, such as bending strength (F_bS), bending stiffness (EI), interlaminar shear strength (V_s), and shear rigidity (GA), along the major and minor strength directions (as shown in Figure 1, for example). This provides CLT manufacturers with the needed flexibility to produce panels in conformance with the product standard, based on the available material resources and required design capacities. Designers should consult with CLT manufacturers concerning their actual layups and stress classes, as their offerings may include additional options to the generic layups included in the ANSI/APA PRG 320 standard.

The generic CLT layups in ANSI/APA PRG 320 were developed based on the following prescriptive lumber species and grades available in North America:

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

The required characteristic strengths and moduli of elasticity for CLT laminations are listed in Table 1. As seen from the list above, both mechanically graded lumber (for “E” classes) and visually graded lumber (for “V” classes) are included in the product standard. Also included are three major species groups in North America, i.e., Douglas Fir-Larch, Spruce-Pine-Fir, and Southern Pine. Using published values for the lumber properties, the design capacities of the CLT were derived based on the “shear analogy” method developed in Europe ([37], [38]) and the following assumptions:

- The modulus of elasticity of lumber in the direction perpendicular to the grain, E_{90}, is 1/30 of the modulus of elasticity of lumber in the direction parallel to the grain, E_0
- The modulus of shear rigidity of lumber in the direction parallel to the grain, G_0, is 1/16 of the modulus of elasticity of lumber in the direction parallel to the grain, E_0
- The modulus of shear rigidity of lumber in the direction perpendicular to the grain, G_{90}, is 1/10 of the modulus of shear rigidity of lumber in the direction parallel to the grain, G_0.

Table 1 **Required characteristic strengths and moduli of elasticity for CLT laminations**

CLT Layup	Laminations used in major strength direction						Laminations used in minor strength direction					
	f_b (psi)	E (10^6 psi)	f_t (psi)	f_c (psi)	f_v (psi)	f_s (psi)	f_b (psi)	E (10^6 psi)	f_t (psi)	f_c (psi)	f_v (psi)	f_s (psi)
E1	4,095	1.7	2,885	3,420	425	140	1,050	1.2	525	1,235	425	140
E2	3,465	1.5	2,140	3,230	565	185	1,100	1.4	680	1,470	565	185
E3	2,520	1.2	1,260	2,660	345	115	735	0.9	315	900	345	115
E4	4,095	1.7	2,885	3,420	550	180	945	1.3	525	1,375	550	180
V1	1,890	1.6	1,205	2,565	565	185	1,100	1.4	680	1,470	565	185
V2	1,835	1.4	945	2,185	425	140	1,050	1.2	525	1,235	425	140
V3	1,575	1.4	945	2,375	550	180	945	1.3	525	1,375	550	180

For SI: 1 psi = 0.006895 MPa

a. See Section 4 for symbols

b. Tabulated values are test values and shall not be used for design. See Annex A for design properties

c. Custom CLT layups that are not listed in this table shall be permitted in accordance with 7.2.1.

d. The characteristic values shall be determined as follows from the published reference design value unless otherwise justified by the approved agency.

Fb = 2.1 x published ASD reference bending stress (Fb)
Ft = 2.1 x published ASD reference tensile stress (Ft)
Fc – 1.9 x published ASD reference compressive stress parallel to grain (Fp)
Fv = 3.15 x published ASD reference shear stress (Fv)

Note 8. The “E” designation indicates a CLT layup based on the use of E-rated or MSR laminations in the longitudinal layers and the “V” designation indicates a CLT layup based on the visually graded laminations in the longitudinal layers. Visually graded laminations are used in the transverse layers for both “E” and “V” layups. The specific species and grade of the longitudinal layers and the corresponding transverse layers for each “E” and “V” designation are based on the layups shown in Annex A.

For use in Canada, limit state design (LSD) specified strengths are given in Table 2 and LSD resistances are shown in Table 3. The LSD resistances are not compatible with the allowable stress design (ASD) reference design capacities in the United States. There is currently no published LSD specified strength and modulus of elasticity for Southern Pine lumber in Canada. Thus, the CLT layups E4 and V3 are not listed in Tables 2 and 3.

Table 2 **LSD specified strength and modulus of elasticity for CLT laminations**

Stress grade	Longitudinal layers						Transverse layers					
	f_b (MPa)	E (MPa)	f_t (MPa)	f_c (MPa)	f_s (MPa)	f_{cp} (MPa)	f_b (MPa)	E (MPa)	f_t (MPa)	f_c (MPa)	f_s (MPa)	f_{cp} (MPa)
E1	28.2	11700	15.4	19.3	0.50	5.3	7.0	9000	3.2	9.0	0.50	5.3
E2	23.9	10300	11.4	18.1	0.63	7.0	4.6	10000	2.1	7.3	0.63	7.0
E3	17.4	8300	6.7	15.1	0.43	3.5	4.5	6500	2.0	5.2	0.43	3.5
V1	10.0	11000	5.8	14.0	0.63	7.0	4.6	10000	2.1	7.3	0.63	7.0
V2	11.8	9500	5.5	11.5	0.50	5.3	7.0	9000	3.2	9.0	0.50	5.3

Notes:

(1) Tabulated values are based on the following standard conditions:
   a. Dry service; and
   b. Standard-term duration of load

(2) The specified values are taken from Table 6.3.1 for MSR lumber and Table 6.3.1A for visually stress-graded lumber. The specified strength in rolling shear, f_s, is taken as approximately 1/3 of the specified strength in shear, f_v, for the corresponding species combination. See Figure 8.2.4 for clarification of rolling shear.

(3) The transverse modulus of elasticity, $E_\perp$, may be estimated as E/30.

(4) The shear modulus, G, may be estimated as E/16.

(5) The rolling shear modulus, $G_\perp$, may be estimated as G/10. See Figure 8.2.4 for clarification of rolling shear.

(6) The modulus of elasticity for design of compression members, E05, shall be taken from Table 6.3.1A for visually stress-graded lumber and 0.82E for MSR lumber.

Table 3 **LSD stiffness and unfactored resistances values for CLT (for use in Canada)**

CLT layup	Lamination thickness (mm) in CLT layup								Major strength direction				Minor strength direction			
	CLT t_p (mm)	=	⊥	=	⊥	=	⊥	=	$(f_bS)_{eff,f,0}$ (10^6 N-mm/m of width)	$(EI)_{eff,f,0}$ (10^9 N-mm^2/m of width)	$(GA)_{eff,f,0}$ (10^6 N/m of width)	$V_{z,0}$ (kN/m of width)	$(f_bS)_{eff,f,90}$ (10^6 N-mm/m of width)	$(EI)_{eff,f,90}$ (10^9 N-mm^2/m of width)	$(GA)_{eff,f,90}$ (10^6 N/m of width)	$V_{z,90}$ (kN/m of width)
E1	105	35	35	35					42	1,088	7.3	34	1.4	32	9.1	12
	175	35	35	35	35	35			98	4,166	15	47	12	836	18	34
	245	35	35	35	35	35	35	35	172	10,306	22	59	28	3,183	27	46
E2	105	35	35	35					36	958	8.0	43	0.94	36	8.2	15
	175	35	35	35	35	35			83	3,674	16	59	8.1	929	16	42
	245	35	35	35	35	35	35	35	146	9,097	24	74	19	3,537	25	58
E3	105	35	35	35					26	772	5.3	29	0.92	23	6.4	10
	175	35	35	35	35	35			60	2,956	11	40	8.0	604	13	29
	245	35	35	35	35	35	35	35	106	7,313	16	50	18	2,299	19	40
V1	105	35	35	35					15	1,023	8.0	43	0.94	36	8.7	15
	175	35	35	35	35	35			35	3.922	16	59	8.1	929	17	42
	245	35	35	35	35	35	35	35	61	9,708	24	74	19	3,537	26	58
V2	105	35	35	35					18	884	7.2	34	1.4	32	7.5	12
	175	35	35	35	35	35			41	3,388	14	47	12	836	15	34
	245	35	35	35	35	35	35	35	72	8,388	22	59	29	3,183	23	46

For SI: 1 mm = 0.03937 in.; 1 m = 3.28 ft; 1 N = 0.2248 lbf

a. See Section 4 for symbols
b. This table represents one of many possibilities that the CLT could be manufactured by varying lamination grades, thicknesses, orientations, and layer arrangements in the layup.
c. Custom CLT layups that are not listed in this table shall be permitted in accordance with 7.2.1.

Custom CLT layups are permitted in ANSI/APA PRG 320 when accepted by an approved agency in accordance with the qualification and mechanical test requirements specified in the standard. This may include double outer layers or unbalanced layups when clearly identified for installation, as required by the manufacturer and the approved agency. However, the product standard requires that a unique CLT layup be assigned by the approved agency if the custom product represents a significant product volume of the manufacturer, to avoid duplication with an existing CLT layup that has been assigned to other manufacturers.

2.3.3 Appearance Classification

There are currently no mandatory appearance classifications for CLT in ANSI/APA PRG 320 nor in the CCMC TG. CLT appearance classifications are therefore to be agreed upon between the buyer and the seller. The appearance classification does not affect the CLT structural capacities/resistances.

However, examples of non-mandatory classifications, based on selected glulam appearance classifications in ANSI A190.1, are provided in ANSI/APA PRG 320 and are as follows:

1. Architectural Appearance Classification
   An appearance classification normally suitable for applications where appearance is an important, but not overriding consideration. Specific characteristics of this classification are as follows:
   - In exposed surfaces, all knot holes and voids measuring over 19 mm (3/4 inch) are filled with a wood-tone filler or clear wood inserts selected for similarity with the grain and color of the adjacent wood.
   - The face layers exposed to view are free of loose knots and open knot holes are filled.
   - Knot holes do not exceed 19 mm (3/4 inch) when measured in the direction of the lamination length, with the exception that a void may be longer than 19 mm (3/4 inch) if its area is not greater than 323 mm^2 (1/2 in^2).
   - Voids having a width greater than 1.6 mm (1/16 inch) created by edge joints appearing on the face layers exposed to view are filled.
   - Exposed surfaces are finished smooth with no misses permitted.
2. Industrial Appearance Classification
   An appearance classification normally suitable for use in concealed applications where appearance is not of primary concern. Specific characteristics of this grade are as follows:
   - Voids appearing on the edges of laminations need not be filled.
   - Loose knots and knot holes appearing on the face layers exposed to view are not filled.
   - Members are surfaced on face layers only and the appearance requirements apply only to these layers.
   - Occasional misses, low laminations or wane (limited to the lumber grade) are permitted on the surface layers and are not limited in length.

A series of guidelines for the development of a protocol for classifying CLT panels into different appearance classifications based on gaps and checks have been drafted by FPInnovations, based on research findings [38]. Depending on market demand, the appearance classifications may be standardized in the future, as more CLT is produced and used in North America.

The CCMC TG includes the aforementioned methodology. It relies on small specimens of full thickness which are no less than 610 mm (24 in.) in either the major or minor direction. These appearance tests should only be performed after pre-qualification or qualification, and are based on comparison of measured check depth and width at different points in time:

- After an initial conditioning until the equilibrium moisture content (EMC) is reached under 20°C and 65% RH conditions
- After exposure to 30°C and ambient RH conditions (about 30% RH) for 10 days
- After exposure to 50°C and ambient RH conditions (about 30% RH) for 10 days

No acceptance criteria or classification are suggested in the CCMC TG, only that the ratios of check length to lamina length and check width should be used as criteria for values that would be agreed upon between the seller and the buyer.

2.4 CLT MANUFACTURING PROCESS

CLT panels are manufactured in three or more layers of sawn lumber or boards; the layers may have the same or a different thickness and are arranged in an orthogonally layered pattern. This layer arrangement in CLT panels adds dimensional stability and two-way action capability to the product. In certain cases, multiple adjacent layers can be aligned in the same direction to meet certain specifications. Fundamentally, it is possible to produce any CLT thickness by combining layers of thicknesses up to a maximum of 51 mm (2 inches). As previously mentioned, the maximum CLT thickness is limited to 508 mm (20 inches) in ANSI/APA PRG 320, mainly for practical reasons, but is not bound to a specific maximum value in the CCMC TG.

Figure 10 shows a schematic representation of a typical CLT manufacturing process, which involves the following nine basic steps:

1) Primary lumber selection
2) Lumber grouping
3) Lumber planing
4) Lumber or layer cutting to desired length
5) Adhesive application
6) CLT panel layup
7) Assembly pressing
8) CLT in-line quality control, surface sanding and cutting
9) Product marking, packaging and shipping.

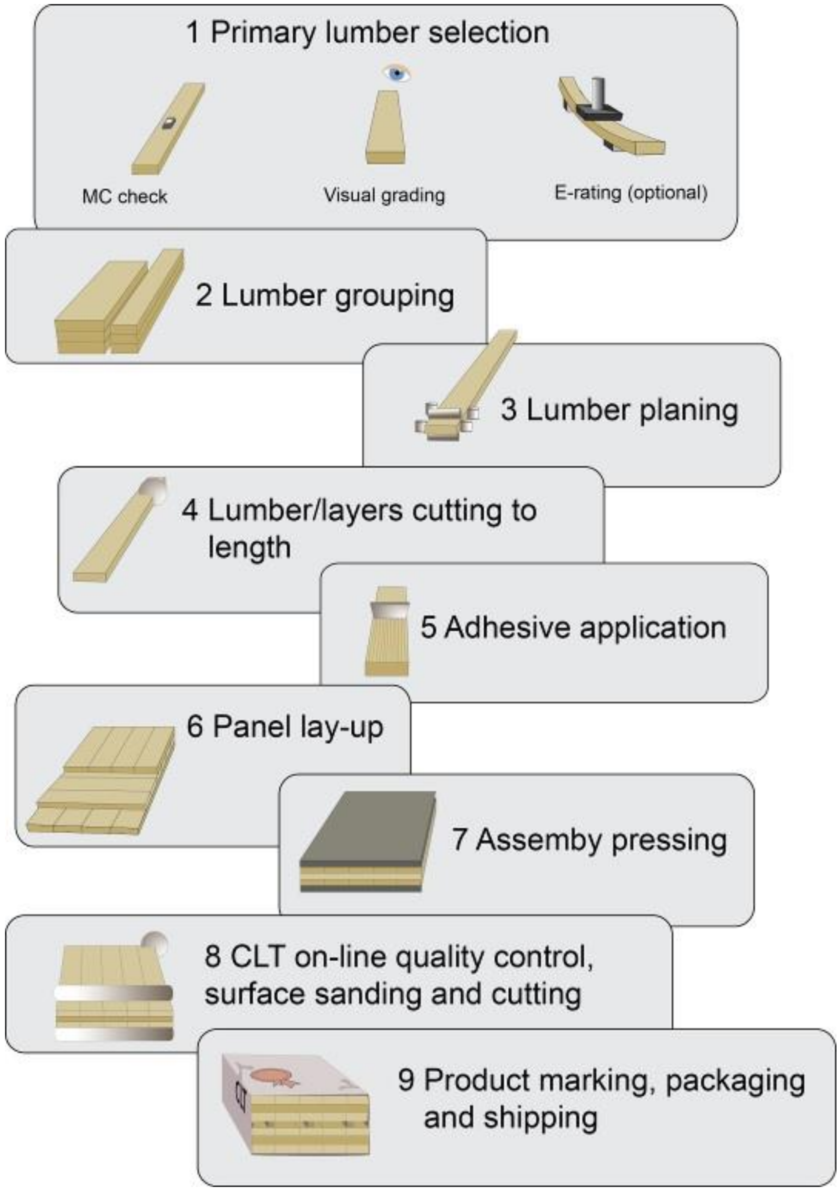


Figure 10 Manufacturing process of CLT products

As opposed to commodity products, the vast majority of CLT panels are made for a specific application. They have a specific layup, size, shape, machined sections (recesses, holes, slots, etc.), and appearance.

As such, CLT panels can take full advantage of the building information modeling (BIM) process. Information collected and shared for building projects following the methodology and principles of the BIM process is used to manufacture the panels and also provide feedback information, such as when the panels will become available. Another advantage is that it allows for just-in-time manufacturing of the panels in coordination with the erection of the building, which minimizes inventories and ensures the panels will be manufactured with the latest information available.

Each step in Figure 10 may include several sub-steps. Step 1 includes lumber moisture content (MC) verification and quality control (QC) inspection. Lumber will normally arrive graded according to the grades listed in Section 2.3.2, so the QC step generally involves further visual inspection with or without E-rating. For a CLT plant with an annual capacity below 30,000 m^3 (1 million ft^3), Step 3 is usually planing (or surfacing) the lumber on all four sides before cutting it to length for face-gluing. For a CLT plant with an annual capacity of 30,000 m^3 (1 million ft^3) or above, Step 3 could involve secondary lumber preparation [39], with the following three options: lumber end-jointing only, lumber edge-gluing only, or both lumber end-jointing and edge-gluing. Step 3 may also involve grading the laminations as well as performing quality control tests, as bonded joints would need to show compliance with the relevant criteria. According to the CCMC TG, removing more than 3% of the material in width or thickness would also call for subsequently re-grading the material. For ANSI/APA PRG 320, there is no similar restriction except for practical considerations, which are expected to be addressed in the future version of the standard.

The key to a successful CLT manufacturing process is consistency in the lumber quality and control of the parameters that affect the bonding quality of the adhesive. Much of what is described in this Section appears in the in-plant quality control manual that was approved by the approved agency.

2.4.1 Primary Lumber Selection

Lumber stock may be selected based on the grade of the CLT panel to be produced; i.e., for appearance grade CLT, the outermost layer(s) may have specific visual characteristics for aesthetic purposes. Some manufacturers may produce a so-called composite CLT, by face bonding wood composites or engineered wood products, such as oriented strand board (OSB), plywood and LVL, to the 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 bond line. This means that graded lumber, which is usually supplied surfaced on four sides (S4S), will need to be re-planed 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.

2.4.1.1 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 is not considered to be suitable for CLT manufacturing processes, despite the fact that some adhesives may perform better with a higher MC. Lumber MC must be restricted to values that are close to mid-way between the maximum and minimum MC the laminations are expected to reach in any given circumstances during their service life. As outlined in Section 2.2.1.1, the CLT panel's layered nature implies that MC changes will induce stress both in the laminations and the bond lines. It is therefore required in both AINSI/APA PRG 320 and the CCMC TG that the target MC be 12% with an acceptable range of ±3%. The targeted value falls right in the middle of the MC range in which wood shrinks and swells, minimizing the development of internal stress due to either shrinking or swelling while laminations adsorb or desorb water. It is also recommended that the maximum difference in MC between adjacent pieces that are to be joined not exceed 5 percentage points.

If SCL is used, the target MC should be 8 ± 3% at the time of CLT manufacturing. Another reason for limiting the MC variation is to ensure consistent and proper performance of the adhesives.

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 unwrapped, stickered in rows to allow air circulation and 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 during the production process. Other in-line MC meters using emerging technologies, such as 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 the microwave field into the product, leading to a more accurate MC measurement. More research and development are needed to adapt the latter to emerging technologies, to improve accuracy of in-line measurements of lumber.

Wood temperature will also affect the bond line quality and the adhesive manufacturer's recommendations should be closely 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 15ºC (60ºF). 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 bond line and panel quality is better understood, revisions can be made to the in-plant manufacturing standard to better allocate monitoring resources.

2.4.1.2 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 have an impact on the pressure that is effectively applied to the bond line 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 are allowed to display these characteristics to varying degrees. While this may be acceptable for wood frame construction, some of these characteristics need to be restricted when manufacturing CLT, in order to ensure the formation of good bond lines.

It is important that the impact of these characteristics, if permitted, be taken into account in the product manufacturing process and the expected bond line performance. In ANSI/APA PRG 320 and the CCMC TG, for instance, this is addressed by grading to achieve an “effective bond line area” of at least 80%. Consider wane as an 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 grades. The effect of wane can be accommodated by removing pieces with excessive amounts of wane and/or rearranging or reorienting the pieces with wane.

2.4.2 Lumber Grouping

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

2.4.3 Lumber Planing

Lumber planing (or surfacing) helps activate or “refresh” the wood surface by reducing oxidation, to improve gluing effectiveness. Removal of a very thin surface layer ensures better bonding [39]. 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, face and back planing only may suffice, if the width tolerance is acceptable and the lumber edges are not glued. In general, removing 2.5 mm (0.1 inch) from the thickness and

3.8 mm (0.15 inch) from the width is recommended [39]. This would however require re-grading the laminations if the CLT panels are made according to the CCMC TG, as more than 3% of the width or thickness would have been removed.

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.

2.4.4 Lumber/Layer 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 breaking 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.

2.4.5 Adhesive Application

In a typical glue application system used in a through-feed process, which is generally used for PUR and PRF adhesives, the extruder heads move and apply parallel lines/threads of adhesive in an air tight system and are supplied directly from an adhesive container. The layers may be lightly wetted with water mist to help the curing reaction when PUR adhesives are used. The production feed speed is generally between 18 - 60 m/min (60 and 200 feet/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. At a typical speed, it takes about 12 seconds to apply the adhesive to a 15 m (50-ft) long layer [39]. Adhesive application should occur shortly after planing, to overcome such issues as surface oxidation, ageing and dimensional instability of the wood, and improve wettability and bonding effectiveness.

The actual adhesive spread rate (or glue spread level) should be as per the adhesive manufacturer's specifications. The rate is affected by the quality of the wood and the application system. The amount of applied adhesive must result in uniform wetting of the wood surface. Proper spread rate is evidenced by very slight but even squeeze-out along the entire bond line. The adhesive applicator and spread rate are generally dependent on the adhesive type and manufacturer.

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. 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; these should be included in the in-plant manufacturing standard.

Edge-gluing of wood pieces that make up the CLT layers is not a common practice among manufacturers, due to the added manufacturing cost, as well as the potential for resulting dimensional stability problems. Unglued edges allow for stress relief zones between laminations and, incidentally, mitigate checking [12]. In order for edge-gluing to be effective, edge planing must have been conducted prior to gluing. As a trade-off between cost and improved product performance, edge-gluing of selected layers could be conducted.

2.4.6 CLT Panel Layup

In general, CLT panel layup is similar to plywood, with adjacent layers aligned perpendicular to each other, the only difference being that each layer of the CLT panel consists of multiple lumber pieces. A minimum “effective bonding area” of 80% is specified in ANSI/APA PRG 320. While there are a number of wood characteristics that may affect the bond area, the producer is ultimately responsible for finding the most effective way to meet the requirements. In the case of wane, this may be accomplished by orienting wood pieces such that the bark and pith areas of adjacent pieces face up, which 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 layers and the application of the target pressure to the assembly. It may be affected by the lamination temperature, which should follow the adhesive manufacturer’s recommendation. The manufacturing process and any restart after a temporary disruption should be designed to ensure that the assembly time does not exceed the maximum target set out in the adhesive specification. In some cases, such as if ambient conditions are not ideal, these may need to be more restrictive than the adhesive manufacturer’s specifications.

2.4.7 Assembly Pressing

Pressing is a critical step of the CLT manufacturing process, to achieve proper bond development and CLT quality. The applied pressure allows the lamination faces to mate adequately and also ensures proper penetration of the adhesive in the laminations.

As outlined in Section 2.2.2, an adhesive must comply with specific standards to be used for the manufacturing of CLT panels in accordance with either ANSI/APA PRG 320 or the CCMC TG. Through a qualification process, the adhesives are qualified for a specific range of pressure and should preferably be utilized within that range, to ensure that the expected performance of the adhesive may be achieved. As the glue standards dictate the types of wood on which the certification test series must be performed, the use of other wood species, the shape and size of the laminations, as well as the quality of the mating surfaces may justify the use of higher or lower pressures. A high-quality bond is generally achieved through tight fit joints. The size of the elements to be joined has a large effect on the fit of the joint. If surface roughness or a non-planar surface must be overcome by pressure to achieve a good fit, the effect of the size and compressibility of the laminations will have a strong effect on the pressure that needs to be applied [11]. Wood species that are characterized by high compressibility and/or shallow laminations will require less pressure than wood species with low compressibility and/or thick laminations to achieve good mating of the surfaces. As a result, the applied pressure on the CLT panels may have to be adjusted depending on the layups.

Two main types of presses are used for CLT manufacturing: vacuum presses (flexible membrane) and hydraulic presses (rigid platen). A vacuum press generates a theoretical maximum clamping pressure of 0.1 MPa (14.5 psi), which is typically insufficient for fulfilling the bonding requirements set forth in ANSI/APA PRG 320. Such a low pressure may not suppress sufficiently the potential warping of layers and overcome their surface irregularities in order to create intimate contact for bonding. To partially address this deficiency, lumber shrinkage reliefs can be introduced by longitudinally sawing through partial thickness of the lumber, as shown in Figure 11; this releases the stress and, in turn, reduces the chances of developing cracks when the CLT panels lose moisture. However, the relief kerfs cannot be too wide or too deep because they may reduce the bonding area, and affect the panel capacity and fire performance. It should be noted that the use of lumber shrinkage relief may affect the CLT performance and should be tested as part of the product qualification. The CCMC TG limits the shrinkage relief to less than half the lamination thickness, while ensuring that less than 10% of the lamination cross-section or 5% of the lamination width is removed.

Figure 11 Lumber shrinkage relief

A rigid hydraulic press can generate much greater vertical clamping pressures and side clamping pressures than a vacuum press. To minimize the potential gaps between the lumber pieces in the main layers, application of side clamping pressure in the range of 276 to 550 kPa (40 to 80 psi) is recommended, concomitantly with vertical pressing.

Side clamping pressure is sometimes needed to ensure that gaps between laminations in the major strength 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 side clamping 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 clamping pressure could suffice. Some vertical presses allow for multiple panels to be pressed simultaneously at high clamping pressures [39]. A lateral unloading device is generally used to unstack 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 15ºC (60ºF), since some adhesives may take longer to cure at low temperatures.

Structural cold-set adhesives such as PRF, MF, EPI, and PUR are commonly used to avoid having to heat the panels during pressing, or the laminations prior to lay-up. The pressing time required is generally from 10 minutes to several hours depending on the type of adhesive. In general, commercial PRF requires the longest pressing time, followed by MF, PUR and EPI. To shorten the pressing time, radio frequency (RF) technologies could be applied during CLT manufacturing. Preliminarily testing using RF technology in the pressing of an EPI bonded three-ply CLT assembly, resulted in a reduction in pressing times to only about 15 minutes, without sacrificing panel bond strength. It was also found that the adhesive spread rate may be reduced by more than 30% of the target specification amount. During RF pressing, arcing and burning, as generally seen when pressing with high-alkaline phenol formaldehyde (PF) adhesives, can be avoided. It is possible that the moisture in the lumber may be redistributed to help partially release internal stress and achieve high panel dimensional stability. However, there are cost issues associated with an investment and the installation of an RF press.

2.4.8 CLT Surface Sanding and Cutting

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 (+0.0004 inch). Tighter or looser tolerances may be specified depending on the building project. After sanding, CLT panels are conveyed to a machining station where a multi-axis numerically-controlled machine cuts out openings for windows and doors, splices and other required parts. The same machine can also be utilized to mill the surface to get the panel to a specific thickness. While this operation is generally slower than with a sander designed for wood composite panels, it allows the plants to use one less piece of equipment. Cutting is performed under strictly controlled conditions for maximum accuracy. Minor repairs are carried out manually at this stage of the manufacturing process.

2.4.9 Product Marking, Packaging, and Shipping

Product marking ensures that the correct product is specified, delivered, and installed. It is also an important part of the product conformity assessment, as it provides the information that allows designers, contractors, and the authority having jurisdiction to check the authenticity of the product.

CLT products represented as conforming to the ANSI/APA PRG 320 standard are required to bear the stamp of an approved agency, which either inspects the manufacturer or has tested a random sampling of the finished products in the shipment being certified for conformance with the standard.

CLT products represented as conforming to ANSI/APA PRG 320 standard are required to be identified with marks containing the following minimum information:

a) CLT grade qualified in accordance with this standard
b) CLT thickness or identification
c) Mill name or identification number
d) Approved agency name or logo
e) Symbol of *ANSI/APA PRG 320* signifying conformance to this standard
f) Any manufacturer's designations which shall be separated from the grade-marks or trademarks of the approved agency by not less than 152 mm (6 inches)
g) "Top" stamped on the top face of custom CLT panels used for roof or floor, if manufactured with an unbalanced lay-up.

Non-custom and other required marks must be placed on standard products at intervals of 2.4 m (8 feet) or less, in order to ensure that each piece cut from a longer piece will have at least one of each of the required marks. For products manufactured to meet specific job specifications (custom products), the marking may contain less information than that specified for standard CLT products. However, custom products must bear at least one mark with a required identification.

Additional markings on the panels may show the main direction (major strength) 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 (see Chapter 12).

The CCMC TG requires that the products be identified with the following information:

a) The mill name or identification number
b) The approved agency name or logo
c) The CLT thickness or identification
d) Laminate grade, type, species, and thickness in sufficient detail to derive the CLT panel capacity
e) Any special zones in the panel specifically designed to receive fasteners or treatment
f) The CCMC report number.

2.5 QUALIFICATION AND QUALITY ASSURANCE

Both the Canadian CCMC TG and the ANSI/APA PRG 320 standard stipulate the requirements for plant pre-qualification, structural performance qualification, and quality assurance.

2.5.1 Plant Pre-qualification

Plant pre-qualification is intended to ensure that the CLT plant is qualified for the various manufacturing factors, such as assembly time, lumber MC, adhesive spread rate, clamping pressure, pressing time, and wood surface temperature, prior to the normal production. The plant pre-qualification should be conducted with full-thickness CLT panels of at least 610 mm (24 inches) in the major and minor strength directions for those complying with ANSI/APA PRG 320, whereas the dimensions required for conformity to the CCMC TG are between 610 and 910 mm. Two replicated CLT panels must be manufactured for pre-qualification of each combination of factors considered. The two replicated CLT panels must not be extracted from a single full-size CLT panel. It is recommended to utilize dimensions that are larger than 610 mm for thicker panels.

Factor combinations going under the procedure should cover the expected range within which the plant aims to operate, such as temperature, pressure, adhesive spread, or any other factor. This methodology will allow the setting of boundaries between which the manufacturing facility can operate. Following that principle, the CCMC TG states that:

- When two or more layups with the same number of layers differ by only the thickness of the laminations, only the layups with the minimum and maximum overall thicknesses need be evaluated.
- When layups differ only by the nominal width of the laminations in one or more layers, only the layups with the minimum and maximum width laminations need be evaluated.

Multiple elements have been identified herein that have the potential to render the manufacturing of compliant products more challenging (e.g., thicker laminations, wood species characterized with high differential swelling and shrinking ratios, wood species with low compressibility, etc.). A thorough screening process of the considered layups is consequently recommended prior to plant pre-qualification, as well as proper sequencing of the products subjected to pre-qualification, to ensure the efficacy and the efficiency of the process.

Plant pre-qualification includes the evaluation of glue bond (block shear) and durability. Figure 12 shows the locations where the block shear and delamination specimens should be taken for the pre-qualification, to ensure good dispersion of the specimens within a sampled CLT qualification panel. Results obtained from the pre-qualification are required to be documented and serve as the basis for the manufacturing factors specified in the in-plant manufacturing standard.

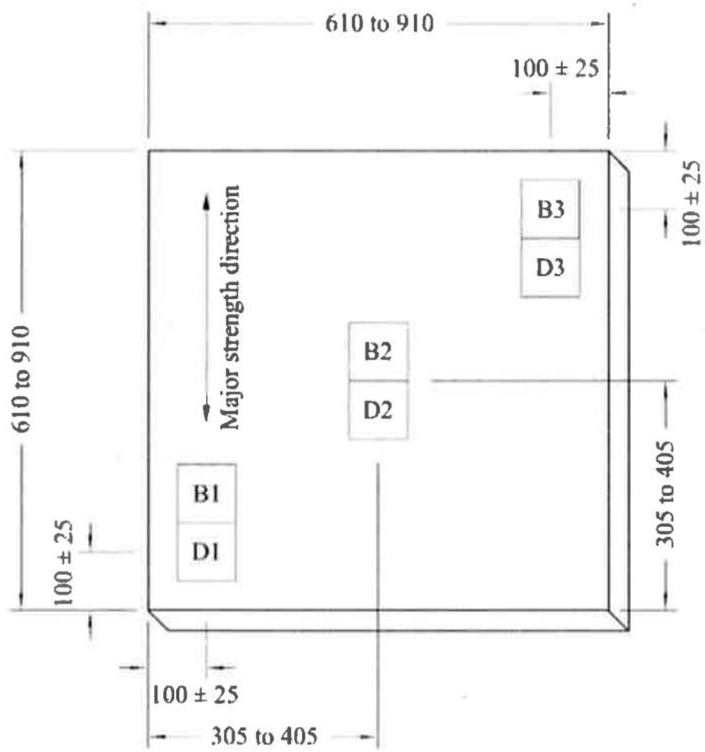


Figure 12 Location of delamination ("D") and block shear ("B") specimens within the pre-qualification panel according to ANSI/APA PRG 320 and the CCMC TG. Dimensions are in mm and are slightly different from ANSI/APA PRG 320.

2.5.2 CLT Mechanical Properties Qualification

To confirm the main CLT design properties, structural performance tests are required in accordance with ANSI/APA PRG 320 and the CCMC TG. These tests include flatwise bending strength, flatwise bending stiffness, interlaminar shear, and flatwise shear stiffness in both major and minor strength directions. In addition to those properties, the CCMC TG requires that the axial compression properties in both the major and minor directions be evaluated.

According to ANSI/APA PRG 320, the sample size for determination of bending stiffness and shear stiffness must be sufficient to estimate the population mean within 5% precision with 75% confidence, or 10 specimens, whichever is greater. The sample size for determination of bending strength and interlaminar shear must be sufficient to estimate the characteristic value with 75% confidence in accordance with ASTM D2915 [40].

For each mechanical test, sample size requirements in the CCMC TG are that it shall be sufficient to estimate the population mean within 5% precision with 75% confidence, or 10 specimens, whichever is greater. The sample size may be determined following the parametric or nonparametric procedures described in ASTM D2915.

The bending tests are required to be conducted flatwise (loads are applied perpendicular to the face layer of the CLT) in accordance with the third-point load method of ASTM D198 or ASTM D4761 [42], using a specimen width of no less than 305 mm (12 inches) and a span-to-depth ratio of approximately 30 in the major strength direction, and approximately 18 in the minor strength direction. It was agreed that a minimum specimen width of 305 mm (12 inches) is necessary to distinguish CLT from typical beam elements. The weight of the CLT panel is allowed to be included in the determination of the CLT bending strength.

The interlaminar shear tests are required to be conducted flatwise in accordance with the center-point load method of ASTM D198 or ASTM D4761, using a specimen width of no less than 305 mm (12 inches) and a span-to-depth ratio between 5 and 6. The bearing length must be sufficient to avoid bearing failure, but not greater than the specimen depth. All specimens must be cut to length without overhangs, which are known to increase the interlaminar shear strength in shear tests.

The shear stiffness tests are required to be conducted flatwise in accordance with ASTM D198, which employs at least four different span-to-depth ratios. These are slightly time-consuming tests, but the test results usually are highly dependable.

The CCMC TG requires that the axial compression properties in both the major and minor directions be evaluated in addition to the aforementioned properties. The test method is based on ASTM E72 [43], in which the specimen width shall not be less than 1.5 times the thickness of the CLT panels.

2.5.3 Qualification Due to a Process Change

When process changes occur in production, qualification tests may be required, depending on the extent of the changes and their impacts on the CLT performance. The following non-exhaustive list of examples can trigger a re-qualification process:

- Change to the press equipment
- Change to the adhesive formulation class
- Addition or substitution of wood species from a different species group or type of structural composite lumber
- Changes to the laminations cross-section such as lumber shrinkage relief profile
- Changes to the visual grade rules that reduce the effective bonding area or the effectiveness of the applied pressure (e.g. warp permitted)
- Other changes to the manufacturing process or component quality not listed above
- Change in the adhesive composition (e.g. fillers and extenders)
- Increase in panel width or length by more than 20%.

Response to these modifications of the manufacturing process or others that are not listed may lead to:

- A plant pre-qualification and structural re-evaluation
- A plant pre-qualification
- A structural re-evaluation.

2.5.4 Quality Assurance

Quality assurance is required by both the CCMC TG and ANSI/APA PRG 320 to ensure CLT product quality, through the detection of changes in properties that may adversely affect the CLT performance.

In either case, an on-going evaluation of the manufacturing process, including end, face, and edge (if used) joints in laminations, effective bonding area, lamination grade limitations, and finished product inspection, is required to be conducted by the CLT manufacturer to confirm that the product quality remains in satisfactory compliance with the product specification requirements. The production must be held pending results of the quality assurance testing on representative samples. In addition, the product quality assurance must be audited by an independent inspection or certification agency on a regular basis, in accordance with the building code requirements.

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 assurance (QA) program.

2.5.5 Quality Assurance Tests

Since the CCMC TG addresses only the evaluation of the end-products, it requires that the evaluated products be covered by a proper quality system; although it specifies the minimum number of elements that shall be included, it does not specify how those are to be evaluated. Therefore, the following subsections refer mainly to ANSI/APA PRG 320.

2.5.5.1 Delamination Tests

The ANSI/APA PRG 320 standard uses delamination testing as a means to assess the quality and moisture durability of the bond line. In the delamination test, a square (or core) specimen obtained from a pre-qualification or production panel is saturated with water and then dried, to evaluate the ability of the adhesive bond line to resist wood shrinkage and swelling stresses. The delamination test also assesses to some extent 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. Ideally, assemblies should withstand the quality control test without delamination. As localized defects in the bond line may be considered to have a negligible effect on the end-product performance, a low level of delamination is nevertheless accepted. The acceptance limits on the amount of delamination are based on the glulam manufacturing standards in either Canada or the United States.

2.5.5.2 Block Shear Test

Block shear tests are required to be conducted based on the methodology of the glulam manufacturing standards. While two results – the strength and the wood failure percentage – are analyzed when glulam products are tested, only one is relevant when testing CLT panels. Because of the orthogonally layered nature of CLT panels, strength is not relevant and, therefore, only wood failure is evaluated. For additional information on this topic, refer to the report on block shear testing of CLT panels [44].

2.5.5.3 Visual Quality of CLT

Wood shrinkage is not equal in all directions due to the anisotropic nature of wood. The main visual effect due to shrinkage is the in-between lamination gaps that open due to the sawn lumber transverse direction shrinkage (see Section 2.2.1.1). End-users should be aware that this effect is both normal and common, and that it depends on the wood moisture content relative to what it was at the time of manufacturing. In-between lamination gaps tend to be fairly constant on the surface of the panels.

Drying checks may also develop in CLT panels during storage and use if the MC of the wood at the time of manufacturing is significantly different from the equilibrium MC at the ambient conditions. The shrinkage can develop tensile stresses that could exceed the local wood strength perpendicular to the grain causing checks or cracks, or that could exceed the shear capacity of the bond lines causing localized delamination. Although the checks may partially or fully close if exposed to a higher humidity environment, they will reappear when the panel is re-dried.

Checks affect the aesthetic value of the surface, and could thus lower the product's market acceptance. 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 in relation to 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 increase the development of checks [12].

An exploratory study has been carried out to develop a procedure for quantifying the severity of, or the potential for checking [38]. The intention is for such tests to provide an indication of the appearance of these in CLT products after long-term exposure in service to dry conditions, or of the effectiveness of steps taken to minimize checking. The CCMC TG includes provisions based on this methodology to quantify checking that could be used in agreements of performance between manufacturers and clients.

Gaps at the unglued edges of adjacent laminations and checks normally will not have a significant impact on strength properties. However, some of the panel's physical properties, such as thermal conductivity, moisture diffusion, and fire performance may be affected. These properties may have an impact on the energy performance and durability of the building assembly.

2.6 STANDARDIZATION

CLT products in general, as well as ANSI/APA PRG 320 implementation have achieved various milestones since the standard was first introduced in 2011. CLT has been integrated in both Canadian and American wood design reference manuals:

- 2016 Supplement of CSA O86-14 Engineering Design in Wood (Canada)
- 2015 and 2018 National Design Specification for Wood Construction (United States)
- 2015 and 2018 International Building Code (United States)
- 2015 and 2018 International Residential Code (United States).

In 2018, a new version of the ANSI/APA PRG 320 standard was published, as well as a CCMC TG for the evaluation of commercial CLT panels in Canada, paving the way for an even broader acceptance of the products via standardization (for CSA O86-19 and 2020 NBCC in Canada, and 2020 NDS, 2021 IBC, and 2021 IRC in the United States).

It should be noted that neither the CCMC TG nor the ANSI/APA PRG 320 standard are CLT design standards and, therefore, do not address design-specific issues, such as creep, duration of load, volume effect, moisture effect, lateral-load resistance, connections, fire, energy, sound, and floor vibration. Design guides for many of those topics are provided in other chapters of this CLT Handbook.

CLT panel-based construction is in expansion in both Canada and the United States, and its use has been encouraged via national and/or provincial programs such as the Canadian Tall Wood Building Demonstration Initiative and the province of Québec's *Programme Vitrine Technologique* (which would translate as "Technology Demonstration Program"). These programs contributed to the extensive use of standardized CLT panels manufactured under ANSI/PRG 320 in the erection of the following two tall buildings:

- Brock Commons Tallwood House
  - 18-storey hybrid mass timber student residence at the University of British Columbia in Vancouver: 17-storeys of mass timber construction above a one-storey concrete podium with 2 concrete stair and elevator shafts
  - Building structure made with three Canadian mass timber products: cross-laminated timber floor panels, glue-laminated columns and parallel strand lumber columns.
  - Housing for just over 400 students
  - Prefabrication of the structural wood elements helped the building go up two months ahead of schedule
  - Project completed in May 2017; occupancy in July 2017.

- Origine Eco-Condos
    - Tallest all-wood condominium building in North America as of fall 2017
    - 13-storey condominium in Québec City: 12 storeys of mass timber on top of a one-storey concrete podium and an underground parking garage
    - Elevator and stairwell shafts made with Canadian cross-laminated timber
    - Building design based on the "*Technical Guide for the Design and Construction of Tall Wood Buildings in Canada*" (Karacabeyli and Lum) [45] published by FPInnovations, Canada's national forest research institute
    - This project helped in turn in the development of "*Mass Timber Buildings up to 12 Storeys – Directives and Explanatory Guide*" published by the Régie du bâtiment du Québec (RBQ) [46].

Continuous efforts are deployed by the ISO Technical Committee (TC) 165 on Timber Structures to develop an ISO standard for CLT products. This ISO standard is intended to harmonize the CLT standards from North America and Europe as an international standard, which will encourage the use of CLT in building construction globally.

2.7 CONCLUSIONS

CLT panel manufacturing requires a good understanding of the properties and behaviour of the various components entering in its manufacturing process and an equally good understanding of the end-product characteristics and performance. This Chapter outlined multiple key elements of the manufacturing process that need to be considered when manufacturing CLT panels, but did not cover all potential situations, which warrants manufacturers to do due diligence when making decisions concerning their manufacturing processes. The availability of the ANSI/APA PRG 320 standard and the Canadian Construction Material Centre Technical Guide (CCMC TG) for proprietary CLT panels contributes to either or both their manufacturing and evaluation.

Future development of CLT panels regarding their manufacturing may concentrate on the following elements:

- Determination of adequate manufacturing parameters for CLT panels made with SCL (surface preparation and tolerances, pressure, etc.) that would lead to the publication of guidelines and recommendations
- Harmonization of glue bond durability test requirements between Canada and the United States.

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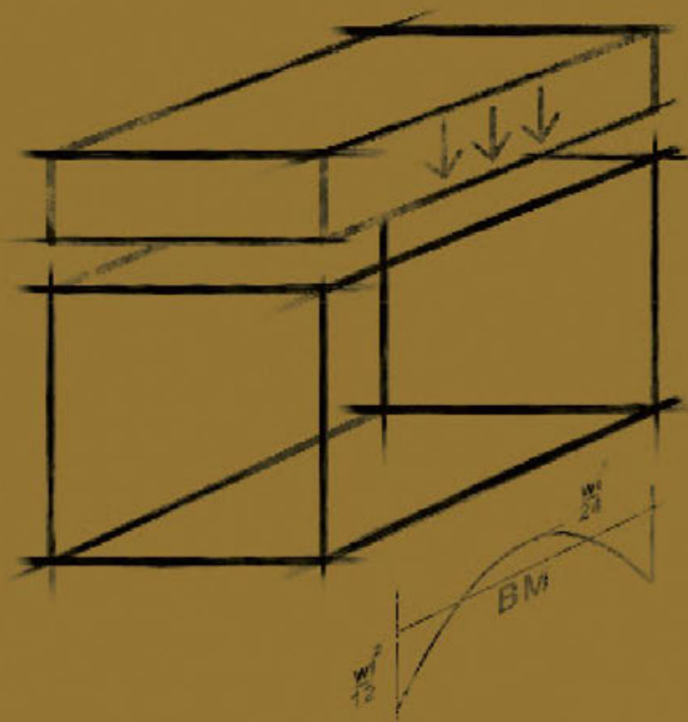
FPInnovations



CHAPTER 3

Structural design of cross-laminated timber elements

Lead Authors
Marjan Popovski
Sylvain Gagnon
Mohammad Mohammad
Zhiyong Chen

Reviewer
Erol Karacabeyli

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ABSTRACT

CLT panels can be used in a wide variety of structural applications in either all wood or wood-hybrid buildings. In the case of all wood buildings, CLT panels can be used in platform-type or balloon-type CLT systems or in buildings that use CLT in combination with other Mass Timber (MT) elements. In the case of platform-type CLT buildings, the panels are usually used for wall, floor and roof assemblies. The wall assemblies in this case carry both gravity loads and lateral (wind and seismic) loads. In balloon-type MT buildings, CLT panels can be used for the walls that run along the entire height of the buildings thus forming the main Lateral Load Resisting System (LLRS), as well as for floor panels that rest on MT beams and columns as a gravity system. Similarly, in wood-hybrid buildings, CLT panels are mostly used for floor assemblies that transfer the gravity loads to MT beams and columns, while the LLRS is either a concrete core or steel-based system. In some applications, CLT panels can also be used on edge as a beam (header) or girder members.

This Chapter provides the material properties and stress grades of CLT panels that are available in CSA O86, Engineering Design in Wood (CSA, 2016), and the ANSI/APA PRG 320 standard for Performance Rated CLT (ANSI/APA, 2018). Basic design aspects for CLT panels used as floor or roof assemblies subjected to out-of-plane bending and shear loads are also provided. In addition, design for CLT walls subjected to either pure axial or combination of axial and out-of-plane loads is included. Furthermore, the use of CLT panels in beam or lintel applications, where CLT panels develop in-plane bending and shear forces, is presented. Finally, some aspects related to deflections of CLT panels loaded out of plane are included as well as some basic examples for calculation of the main CLT panel properties and resistances. The design guidelines provided in this Chapter follow the CSA O86 design approach, where available. Information in this Chapter and in the CWC Wood Design Manual (CWC, 2017) are complementary to each other.

Finally, Appendix A of this Chapter provides information on some of the available analytical models for determining the stiffness and strength properties of CLT panels such as the γ (gamma) method, the k-method, and the Kreuzinger shear analogy method. This is important for the designers when the CLT panel layout is not implemented in the PRG 320 and CSA O86 standards. In such cases, the analytical methods presented in this Appendix can be used to determine the panel properties.

3.1 INTRODUCTION

3.1.1 Cross-Laminated Timber Panels

Cross-laminated timber (CLT) is an engineered wood product made of at least three orthogonal layers of machine stress-rated or visually stress-graded structural sawn lumber that are laminated by gluing with structural adhesives to form a solid panel (Figure 1). Most of the CLT panels currently produced in Canada and the United States meet the requirements of the ANSI/APA PRG 320 standard (ANSI/APA, 2018), and can have up to 9 layers. The narrow faces (edges) of the boards are usually not glued together, although sometimes boards positioned in the longitudinal direction of the panel can be edge-glued. Some manufacturers may also produce panels with edge-glued transverse planks. In addition, special configurations are available where consecutive board layers may be placed in the same direction, thus providing a double layer, usually at the outer longitudinal layers. Figure 2 shows examples of various CLT panel cross-sections.

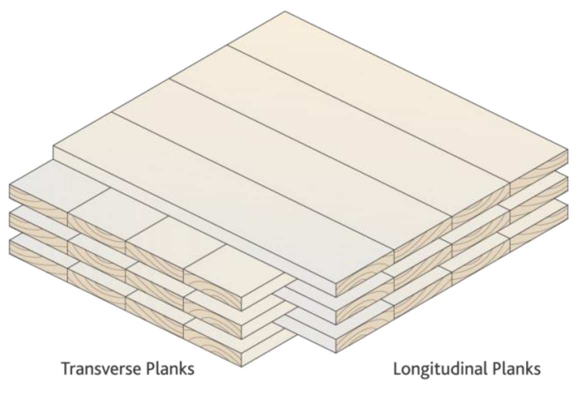


Figure 1 Typical CLT panel configuration

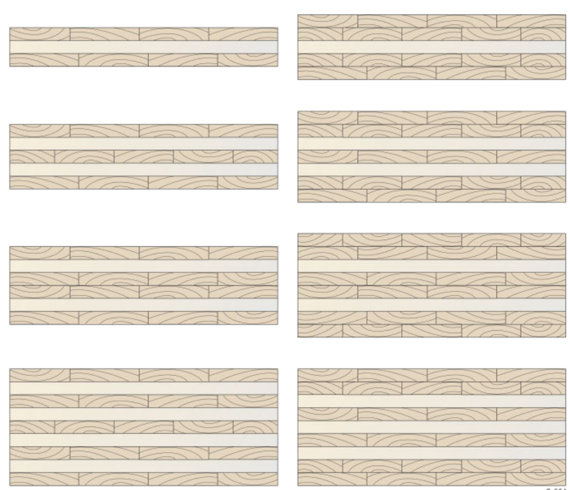


Figure 2 Examples of typical CLT panel cross-sections

Figure 3 illustrates a 5-layer CLT panel with its cross-sections. The direction of the grain (fibre) of the boards in the outer layers of a CLT panel is usually referred to as the major strength axis, while the direction of the grain in the orthogonal layers is referred to as the minor strength axis.

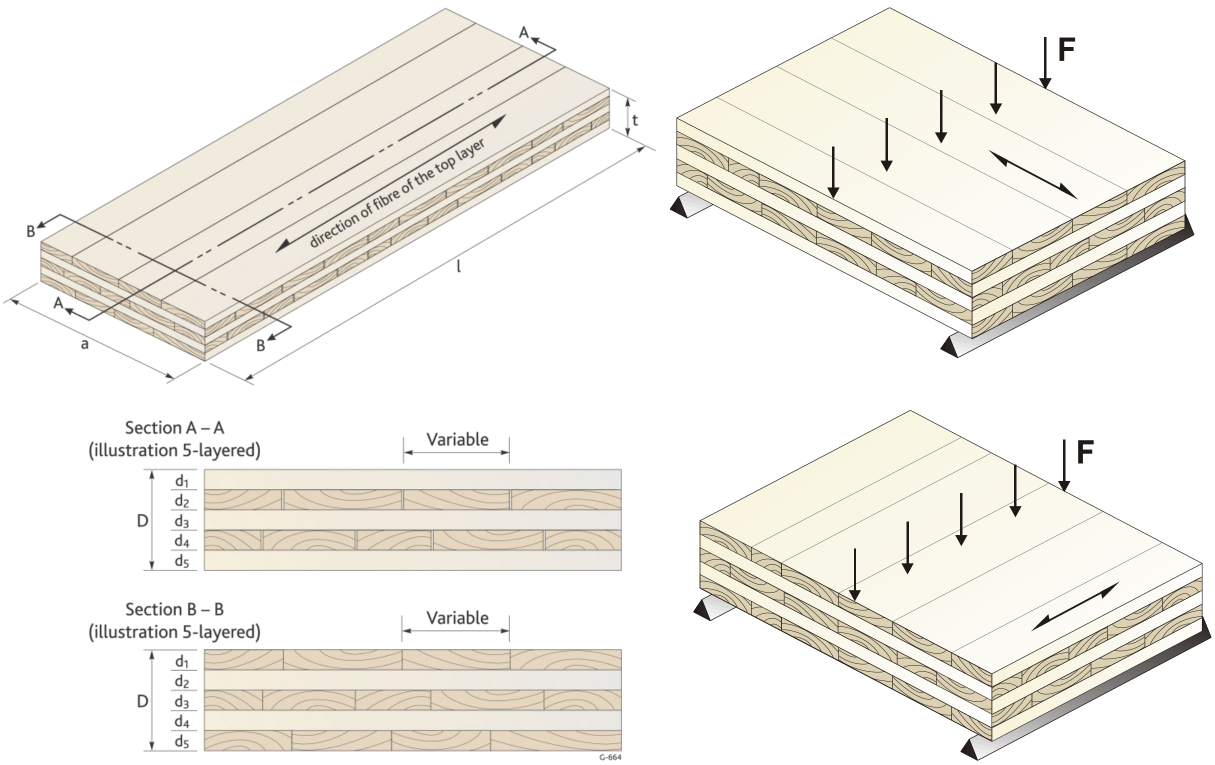
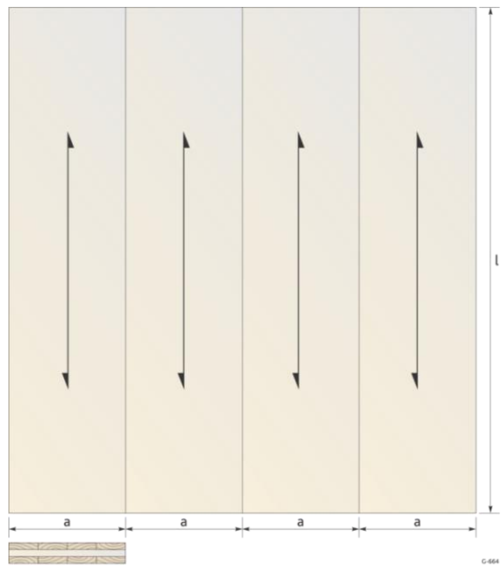


Figure 3 **Cross-sections of a 5-ply CLT panel and direction of the fibres in the top layers**

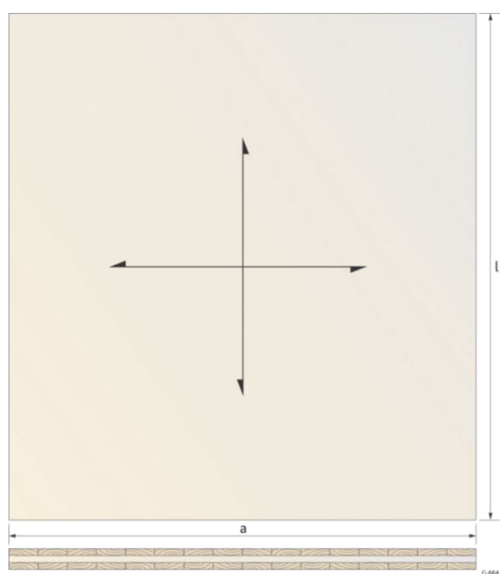
3.1.2 Structural Applications

CLT panels can be used in a wide variety of structural applications, in either all wood or wood-hybrid buildings. In all wood buildings, CLT panels can be used in platform-type or balloon-type CLT assemblies or in buildings that use CLT in combination with other Mass Timber (MT) elements. In platform-type assemblies, CLT panels are most often used for wall, floor, and roof assemblies. The wall assemblies in this case carry both gravity loads and lateral (wind and seismic) loads. In many cases, CLT panels can also be used for elevator shafts and stairwells. In balloon-type assemblies in MT buildings, CLT panels can be used for the walls that run along the entire height of the building, thus forming the main Lateral Load Resisting System (LLRS), as well as for floor panels that rest on MT beams and columns as a gravity system. Similarly, in wood-hybrid buildings, CLT panels are mostly used for floor assemblies that transfer the gravity loads to MT beams and columns, while the LLRS is either a concrete core or steel-based system. In some applications, CLT panels can also be placed on edge and used as a beam (header) or girder members.

In floor applications, CLT panels are usually placed next to each other in the same direction (Figure 4a), thus acting as single directional slabs. In some cases, CLT floors can be built with only one CLT panel acting in two directions (Figure 4b). Please note that the 3-ply panel in this figure is for illustration purposes only, as typically a 5-ply floor panel is required for a two-way action.



(a)



(b)

Figure 4 Floor assemblies made of (a) four CLT panels acting in one direction and (b) one CLT panel acting in both directions. Distance "a" depends on the manufacturer and may reach up to 3m.

3.2 MATERIAL PROPERTIES AND STRESS GRADES OF CLT ELEMENTS

Around the world, the thickness of the individual boards (layers) in CLT panels varies from 10 mm to 40 mm, while the width varies from 30 mm to 240 mm. In Canada, the layers are usually either 17 mm, 19 mm, or 35 mm thick. The boards are finger-joined using structural adhesive for longer spans. They are either visually or machine stress-rated and are usually kiln dried to achieve an average moisture content of 12% ± 3%. Basic mechanical properties of the boards used in CLT elements vary from one producer to another. Different lumber grades can be used for different layers. Layers in the minor direction are usually produced using lower grade lumber. Each Canadian producer offers several different CLT panel stress grades that are either made of visually stress-graded lumber (V grades) or machine stress-rated (MSR) lumber (E grades). The CLT panel stress grades produced are usually in line with the stress grades provided in the ANSI/APA PRG 320 standard for performance-rated CLT (ANSI/APA, 2018). It should be noted that in the PRG 320 standard, the different stress grades are called layups. The layups from the PRG 320 standard were incorporated in the design provisions for CLT that were introduced in Update No. 1 of the 2014 edition of CSA O86, Engineering Design in Wood, that was published in 2016 (CSA, 2016). Table 8.2.3 of CSA O86 contains species combinations and lamination grades for three E stress grades (E1 to E3) and two V stress grades (V1 and V2) for CLT panels made from Canadian species:

E1: 1950f-1.7E SPF MSR lumber in all layers along the major (longitudinal) axis (direction), and No. 3 SPF lumber in all layers in the minor (transverse) axis (direction);

E2: 1650f-1.5E D.fir-Larch MSR lumber in all layers along the major (longitudinal) axis (direction), and No. 3 D.fir-Larch in all layers in the minor (transverse) axis (direction);

E3: 1200f-1.2E Eastern Softwoods, Northern Species, or Western Woods MSR lumber in all layers along the major (longitudinal) axis (direction), and No. 3 Eastern Softwoods, Northern Species, or Western Woods lumber in all layers in the minor (transverse) axis (direction);

V1: No. 2 D.fir-Larch lumber in all layers along the major (longitudinal) axis (direction), and No. 3 D.fir-Larch lumber in all layers in the minor (transverse) axis (direction);

V2: No. 1/No. 2 SPF lumber in all layers along the major (longitudinal) axis (direction), and No. 3 SPF lumber in all layers in the minor (transverse) axis (direction).

CSA O86 requires that CLT layups be a balanced combination of orthogonal layers, where all laminations oriented in the same direction are made of structural sawn lumber of the same grade and species combination. CLT panels may be designed with adjacent layers oriented in the same direction, using the section properties provided by the product manufacturer.

Specified strength values and moduli of elasticity for the laminations in longitudinal and transverse layers of the primary CLT stress grades are provided in Table 8.2.4 in CSA O86 and in Table A3 in PRG 320. If any disparity between these tables arises in the future, the table in CSA O86 should be used. For convenience, these values are also given in Table 1 of this Chapter. It should be noted that the values in Table 1 are based on dry service conditions and standard-term duration of load. The specified values are taken from Table 6.3.2 for MSR lumber and Table 6.3.1A for visually stress-graded lumber, in CSA O86. The specified strength in rolling shear f_s is taken as approximately 1/3 of the specified strength in shear, f_v, for the corresponding species combination. The modulus of elasticity for design of compression members, E_{05}, should be taken from Table 6.3.1A in CSA O86 for visually stress-graded lumber and as 0.82E for MSR lumber.

Table 1 Specified strengths and moduli of elasticity of laminations for CLT stress grades [MPa]

Stress Grade	Longitudinal Layers						Transverse Layers					
	f_b	E	f_t	f_c	f_s	f_{cp}	f_b	E	f_t	f_c	f_s	f_{cp}
E1	28.2	11700	15.4	19.3	0.50	5.3	7.0	9000	3.2	9.0	0.50	5.3
E2	23.9	10300	11.4	18.1	0.63	7.0	4.6	10000	2.1	7.3	0.63	7.0
E3	17.4	8300	6.7	15.1	0.43	3.5	4.5	6500	2.0	5.2	0.43	3.5
V1	10.0	11000	5.8	14.0	0.63	7.0	4.6	10000	2.1	7.3	0.63	7.0
V2	11.8	9500	5.5	11.5	0.50	5.3	7.0	9000	3.2	9.0	0.50	5.3

where:

E = specified modulus of elasticity of the layer, MPa

f_b = specified strength in bending, MPa

f_c = specified strength in compression parallel to grain, MPa

f_{cp} = specified strength in compression perpendicular to grain, MPa

f_s = specified strength in rolling shear, MPa

f_t = specified strength in tension parallel to grain, MPa

Remanufactured lumber is considered equivalent to solid-sawn lumber when qualified in accordance with SPS 1, 2, 4, or 6 in Canada (NLGA). Proprietary lumber grades meeting or exceeding the specified mechanical properties of lumber grades specified above are also permitted for use, provided they are qualified by certification agencies.

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. 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. In the literature (Mestek et al., 2008; Bejtka and Lam, 2008), the rolling shear modulus $G_\perp$ is usually assumed to be 1/10 of the shear modulus parallel to grain, G (i.e. $G_\perp = G/10$). The same analogy has been included in CSA O86. Similarly, the transverse modulus of elasticity, $E_\perp$, was estimated as E/30, while the shear modulus, G, may be estimated as E/16. Figure 5 provides some clarification concerning the rolling shear mechanism occurring in CLT panels.

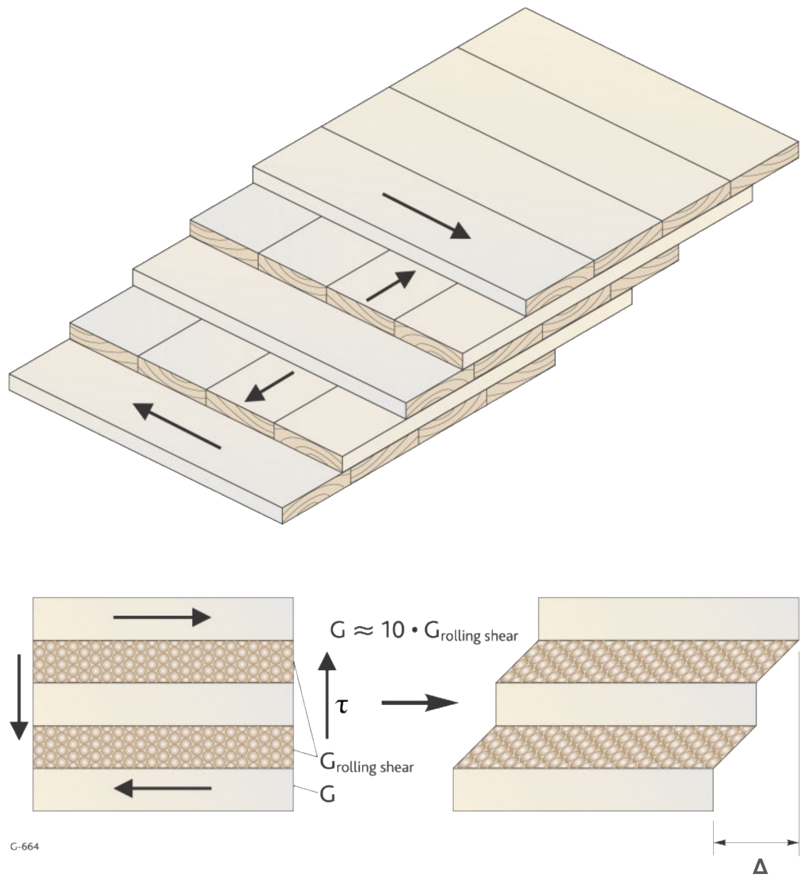


Figure 5 Rolling shear deformation of a 5-layer CLT panel

CLT panels can also be produced in grades that are outside of the PRG 320 scope. For such panels it is important that the boards forming the panel be manufactured using Canadian lumber grades 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 (CSA, 2005). The boards should have a minimum relative density of 0.35. Additionally, boards graded using in-house quality control standards may be used but shall be validated by a certification agency. It is recommended to use boards having a maximum moisture content of 12% ± 3% for pilot projects until further research in this area is conducted.

Custom CLT grades with layups that differ from the ones specified in PRG 320 and CSA O86 are possible. PRG 320 states that such layups must be specified by the product manufacturer and certified by an approved certification agency as meeting the mechanical tests requirements given in the standard. Custom stress grades may include double outer layers or unbalanced layups. Also, the Canadian Construction Materials Centre (CCMC) has developed a Technical Guide describing the technical requirements, evaluation procedure, and performance criteria for the assessment of custom and proprietary types of CLT panels (CCMC 2016). The guide also provides design provisions for use of CLT products and connections in structural applications.

Appendix X1 in ANSI/APA PRG 320 contains examples of CLT appearance classifications for reference purposes only. It is recommended that the actual CLT panel appearance requirements be agreed upon between the buyer and the seller. Appearance grades as defined in ANSI/APA PRG 320 do not affect the structural performance of the CLT panels.

3.3 ANALYTICAL METHODS

CLT components of structures such as floors, roofs, walls, etc., can be analyzed and designed using established analytical methods for determining cross sectional properties, using computer programs developed specially for CLT elements, or a general Finite Element (FE) software.

3.3.1 Analytical Methods for Determining CLT Properties

Various methods have been adopted in Europe for determination of the stiffness and strength properties of CLT panels. 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 layup, type of material, or any of the manufacturing parameters change, more testing is needed to evaluate the properties of these products. An analytical approach, once verified with 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.

During the last two decades, various types of analytical models for evaluation of the basic mechanical properties of CLT panels have been developed or existing models have been modified for use with CLT. This Section provides basic information only, for the three most commonly used methods. More detailed information on these methods is given in Appendix A. The methods can be used in determining the stiffness properties of CLT floor panels loaded perpendicular to the face of the panel. All models are suited for CLT panels that carry the loads in a single direction, so the CLT panels are treated as strips of a unit width (usually 1.0 m). It is important to mention that since CLT floor panels are relatively soft and light, the design (e.g., minimum thickness and the maximum span) is often governed by serviceability criteria (e.g., vibration, deflection and creep) rather than by their strength (e.g., bending and shear strength).

The most common analytical approach in Europe is based on the “Mechanically Jointed Beams Theory”. As the name suggests, this method was originally developed for beams (e.g., I or T beams) connected with mechanical fasteners with a certain stiffness, uniformly spaced along the length of the beams. This method, also named the γ (Gamma) method, is included in Annex B of Eurocode 5 (EN 2009). According to the 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 stiffness characteristics of the fasteners, with $\gamma = 1$ representing a completely glued member, and $\gamma = 0$ no connection at all. This approach only provides a closed (exact) solution for the differential equation 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 therefore acceptable in engineering practice (Ceccotti, 2003).

Since CLT panels are glued products with no mechanical joints present, some modifications to the theory were needed to make it applicable to CLT panels. If we assume that only the 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 the 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” (spacers) that have stiffness equal to that of the rolling shear deformation of the cross layers. This method ignores the influence of shear deformations in the longitudinal layers on the total deflection of the panel. Depending on the cross-section layup, the portion of the shear deformation may be up to 30% of the bending deformation (Wallner-Novak et al., 2014). Another disadvantage of this method is the fact that the $(EI)_{eff}$ depends on the span l and thus, is a system-dependent value. In addition, using this method for a CLT panel with a total of seven or more layers requires some modifications that render it cumbersome.

A “Shear Analogy” method (Kreuzinger, 1999) has been developed in Europe that is applicable to solid panels with cross layers. The methodology takes into account the shear deformation of the parallel and the cross layers and is not limited to a certain number of layers within a panel. Similarly to the Gamma method, this method also uses $(EI)_{eff}$ in the calculation of the bending stiffness. The shear deformation is introduced through a new shear stiffness term denoted as $(GA)_{eff}$. Although this method does not provide a “closed solution”, it is fairly accurate and adequate for CLT panels; for these reasons, this method was used to determine the stiffness properties of CLT panels loaded perpendicular to the face, in both the PRG 320 and CSA O86 standards.

The Euler-Bernoulli beam theory is used extensively in structural analysis and design, and most design guides for structural elements of any material will exclusively use this theory. The shortcoming of the Euler-Bernoulli beam theory is that it is inaccurate for deep beams, or in other words, beams where the depth is not negligible compared to the length. The Timoshenko beam theory is an extension of the Euler-Bernoulli beam theory that includes shear deformations and rotational bending effects in developing the basic equations, making it suitable for predicting the behaviour of thick beams and sandwich composites (beams/plates), such as CLT. The Timoshenko beam theory is widely used in Europe and in several specialty computer programs for CLT.

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. This method is reasonably accurate for panels with high span-to-depth ratio.

3.3.2 General Finite Elements and Specialized Software for CLT

To the best knowledge of the authors, CLT components are implemented in a limited number of general Finite Element (FE) software for structural analysis and design. Dlubal Software Products for structural analysis and design, for example, have developed a stand-alone module RF-LAMINATE as an add-on module for their RFEM structural engineering software https://www.dlubal.com/en-US/products/rfem-and-rstab-add-on-modules/others/rf-laminate. The RFEM finite element analysis program provides modeling, structural analysis, and design of 2D and 3D models consisting of beam, plate, wall, folded plate, shell, solid, or contact elements. The RF-LAMINATE module performs code independent stress design and deflection analysis of laminate surfaces, considering the laminate theory. Based on a user-defined layer structure, the module creates the local stiffness matrix for the respective surface, including CLT elements or glass fiber reinforced plastic plates. The developed stiffness matrix can then be used in the general RFEM software.

During the last decade, however, a number of specialized computer programs have been developed by major CLT manufacturers in Europe, as well as by teams in some European universities. The *CLT Designer* is a free-of-charge specialized software developed by a joint team from the *Holz.bau forschungs gmbh* and the Institute for Timber Engineering and Wood Technology of Graz University of Technology in Austria.

The software (https://www.cltdesigner.at/index.php?id=106&L=2) includes several CLT analysis and design modules based on the Timoshenko beam theory, offering a variety of verifications for ultimate and serviceability limit states for CLT slabs subjected to bending, shear, accidental fire, point or line loads, as well as in-plane loads. Although all verifications are made according to the Eurocode (CEN, 1995), they can still be useful for Canadian designers. The program also has a module that calculates the stiffness values necessary for a two-dimensional FE calculation and offers an export opportunity to the Dlubal RFEM software.

One of the major European CLT producers, Stora Enso Wood Products GmbH, has launched a free-of-charge on-line platform named *Calculatis* that can be used on almost any device, from PC or laptop to smartphone or tablet. The platform enables accurate design of CLT floors, roofs, glulam or steel columns, beams, CLT headers, supports, etc. This software module is based on the Timoshenko beam theory, which includes effects of the shear forces on the total deformations and provides reasonable design accuracy compared to all other methods mentioned in this Handbook. *Calculatis* can be accessed at: www.calculatis.clt.info.

The Canadian Wood Council is planning to include sizing of CLT elements according to the CSA O86 design provisions in the near future.

3.4 MODIFICATION FACTORS FOR STRUCTURAL DESIGN

As stipulated in CSA O86, the specified strengths and capacities of structural wood components should be multiplied by appropriate modification factors. The factors to be used for CLT are given in this Section.

3.4.1 Load Duration Factor K_D

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). Generally, in the absence of test data on the long-term performance of CLT under permanent load, the recommended approach at this time is to use the appropriate load duration factor, K_D, in accordance with Clause 5.3.2 of CSA O86. The rationale is that the design of CLT used as floor and roof elements is usually governed by deflection, and deflection falls under serviceability state design where the additional deflection due to rolling shear effects is accounted for in the deflection calculation equations given in Annex A.8.5.2 of CSA O86. See also Chapter 6, "*Duration of Load and Creep Factors for Cross-Laminated Timber Panels*" of this Handbook.

3.4.2 Service Condition Factor K_S

CLT covered in the ANSI/APA PRG 320 and the CSA O86 standards should be used in dry service conditions only. Consequently, all service condition factors should be taken as unity ($K_{Sb} = K_{Sc} = K_{Scp} = K_{St} = K_{Sv} = K_{SE} = 1.0$). For humid service conditions (i.e. protected exterior condition), please refer to Chapter 6, "*Duration of Load and Creep Factors for Cross-Laminated Timber Panels*". CLT structures may be used in wet service conditions only if specifically permitted by the manufacturer based on documented test data in accordance with CSA O86 Clause 4.3.2, and when approved by a certification organization. CLT elements must be protected to avoid direct contact with moisture for a long period of time. This can be achieved through better detailing and use of sealants, coatings and flashing, especially on the CLT panel edges. In general, CLT products carrying a trademark of PRG 320 should be used in accordance with the installation requirements and recommendations provided by the CLT manufacturer, the approving agency, and/or its trade association.

3.4.3 Treatment Factor K_T

The treatment factor K_T should be taken as equal to 1.0 for untreated CLT products. For CLT treated with fire-retardant or any other potentially strength-reducing chemicals, the strength and the stiffness should be based on the documented tests results. The tests should take into account the effects of time, temperature, and moisture content in accordance with CSA O86 Clause 5.3.4. In this case use of an appropriate value for the K_T factor that corresponds to the influence of the strength-reducing chemicals is suggested. Treatment of CLT after gluing with water-borne preservatives is currently not allowed in Canada.

3.4.4 System Factor K_H

Due to the fact that CLT panels act as orthotropic plates, it is recommended to use a system factor, K_H, equal to 1.0 for all strength properties. Further work is needed to determine if CLT construction can use the benefit of a higher value for the system factor.

3.5 FLATWISE BENDING AND SHEAR RESISTANCE OF CLT PANELS

When CLT panels are used in floor or roof applications to carry gravity loads (dead, live and/or snow loads), they are subjected to flatwise bending. Similarly, CLT panels used in wall applications can be subjected to flatwise bending from wind loads acting on the building. CLT floor and roof panels are usually designed to act in a single direction of loading. The ANSI/APA PRG 320 standard provides examples of specified resistances for specific CLT panel layups, based on the "Shear Analogy" method, for Canada and the US. The specified bending resistances for primary Canadian CLT grades with 3-, 5-, and 7-layer layups in the major and minor directions are provided in Table A4 of the standard. The resistance values were derived analytically using the "Shear Analogy" model and were validated by test results. The calculated moment capacities in the major strength direction were multiplied by a factor of 0.85, for a conservative estimate. The "Shear Analogy" method was also implemented in CSA O86 for determining the bending and shear rigidity of the CLT panels. Other analytical models are allowed to be used by the CLT manufacturers, provided that the design capacities are confirmed by qualification tests as specified in the ANSI/APA PRG 320 standard and agreed upon by an approved agency. For design purposes, the actual dry sizes of the panel rounded to the nearest millimeter (net dimension) should be used. Depending on their layup, actual sizes, and tolerances, CLT panels may deviate from those specified in ANSI/APA PRG 320; this should be accounted for in the calculations.

3.5.1 Flatwise Bending Resistance

The factored flatwise bending moment resistance of CLT panels in the major strength (longitudinal) axis, $M_{r,y}$, should be calculated according to CSA O86 (Figure 6) as follows:

$$M_{r,y} = \phi \cdot F_b \cdot S_{eff,y} \cdot K_{rb,y} \qquad [1]$$

where:

ϕ = 0.9 is the resistance factor for CLT in bending

$K_{rb,y}$ = 0.85 is the strength modification factor.

The factored bending strength can be calculated as:

$$F_b = f_b(K_D K_H K_{Sb} K_T) \qquad [2]$$

where:

f_b = specified bending strength of laminations in the longitudinal layers, (Table 1) in CSA O86, in MPa

The effective flatwise section modulus of CLT panels in the major axis $S_{eff,y}$ can be calculated as:

$$S_{eff,y} = \frac{(EI)_{eff,y}}{E} \cdot \frac{2}{h} \qquad [3]$$

where:

$(EI)_{eff,y}$ = effective bending stiffness of the panel for the major strength axis, in N•mm^2

E = specified modulus of elasticity of laminations in the longitudinal layers, (Section 3.2 or Table 1), in MPa

h = thickness of the panel, in mm (Figure 6)

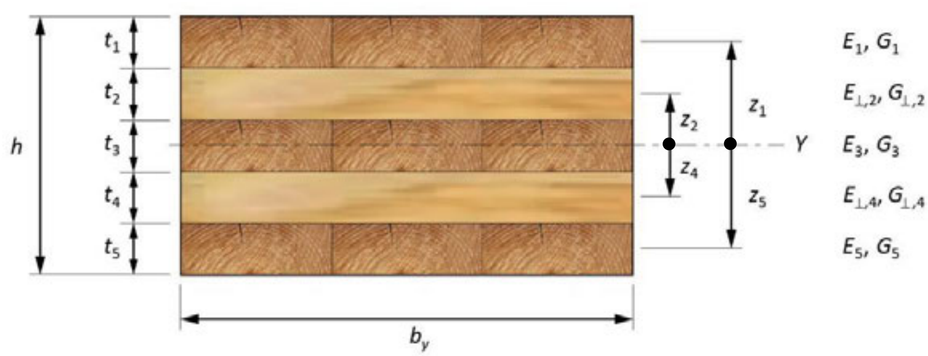


Figure 6 Cross-section properties of a 5-ply CLT panel in the major direction (CSA, 2016)

As mentioned, the Shear Analogy method was used to determine the effective flatwise bending stiffness of the panel using Equation [4] below:

$$(EI)_{eff,y} = \sum_{i=1}^{n} E_i \cdot b_y \cdot \frac{t_i^3}{12} + \sum_{i=1}^{n} E_i \cdot b_y \cdot t_i \cdot z_i^2 \qquad [4]$$

where:

b_y = width of the panel for the major strength axis, in mm

E_i = modulus of elasticity of laminations in the *i*-th layer, in MPa

= E, for laminations in the longitudinal layers

= $E_\perp$, for laminations in the transverse layers

n = number of layers in the panel

t_i = thickness of laminations in the *i*-th layer, in mm

z_i = distance between the center point of the *i*-th layer and the neutral axis, in mm (Figure 6).

The effective in-plane (planar) shear rigidity $(GA)_{eff,zy}$ of the CLT panel in the major direction can be calculated according to the shear analogy method as:

$$(GA)_{eff,zy} = \frac{(h - \frac{t_1}{2} - \frac{t_n}{2})^2}{\left[\left(\frac{t_1}{2 \cdot G_1 \cdot b_y}\right) + \left(\sum_{i=2}^{n-1} \frac{t_i}{G_i \cdot b_y}\right) + \left(\frac{t_n}{2 \cdot G_n \cdot b_y}\right)\right]} \qquad [5]$$

where:

G_i = shear modulus of laminations in the *i*-th layer, in MPa

= G, for laminations in the longitudinal layers

= $G_\perp$, for laminations in the transverse layers

h = thickness of the panel (Figure 8.4.3.2a of CSA O86).

The factored flatwise bending moment resistance for CLT panels in the minor strength direction, $M_{r,x}$, should be calculated according to CSA O86 (Figure 7) as follows:

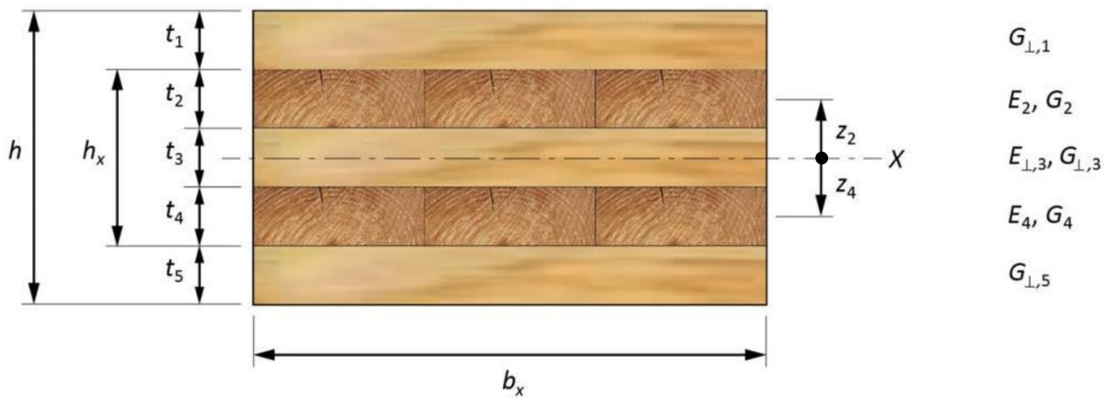


Figure 7 **Cross-section properties of a 5-ply CLT panel in the minor direction (CSA, 2016)**

$$M_{r,x} = \phi \cdot F_b \cdot S_{eff,x} \cdot K_{rb,x} \quad [6]$$

where:

ϕ = 0.9 is the resistance factor

$K_{rb,x}$ = 1.0 is the strength modification factor.

The effective flatwise section modulus of CLT panels in the minor axis $S_{eff,x}$ should be calculated as:

$$S_{eff,x} = \frac{(EI)_{eff,x}}{E} \cdot \frac{2}{h_x} \quad [7]$$

where:

$(EI)_{eff,x}$ = effective bending stiffness of the panel for the minor strength direction, in N·mm^2

E = specified modulus of elasticity of laminations in the perpendicular layers, in MPa

h_x = thickness of the panel without the outer longitudinal layers, in mm (Figure 7).

Using the shear analogy method the effective flatwise bending stiffness of the panel in the minor direction $(EI)_{eff,x}$ can be calculated using Equation [8] below:

$$(EI)_{eff,x} = \sum_{i=2}^{n-1} E_i \cdot b_x \cdot \frac{t_i^3}{12} + \sum_{i=2}^{n-1} E_i \cdot b_x \cdot t_i \cdot z_i^2 \quad [8]$$

The effective in-plane (planar) shear rigidity $(GA)_{eff,zx}$ of the CLT panel in the minor direction can be calculated according to the shear analogy method as:

$$(GA)_{eff,zx} = \frac{(h - \frac{t_1}{2} - \frac{t_n}{2})^2}{\left[\left(\frac{t_1}{2 \cdot G_1 \cdot b_x}\right) + \left(\sum_{i=2}^{n-1} \frac{t_i}{G_i \cdot b_x}\right) + \left(\frac{t_n}{2 \cdot G_n \cdot b_x}\right)\right]} \quad [9]$$

where:

b_x = width of the panel for the minor strength axis, in mm (Figure 7)

E_i = modulus of elasticity of laminations in the *i*-th layer, in MPa

= E, for laminations in the transverse layers, in MPa

= $E_\perp$, for laminations in the longitudinal layers, in MPa

G_i = shear modulus of laminations in the *i*-th layer, in MPa

= G, for laminations in the transverse layers, in MPa

= $G_\perp$, for laminations in the longitudinal layers, in MPa

n = number of layers in the panel

h = thickness of the panel, in mm (Figure 7)

t_i = thickness of laminations in the *i*-th layer, in mm

z_i = distance between the center point of the *i*-th layer and the neutral axis, in mm (Figure 7).

As mentioned before, the equations given in this Section apply to symmetrical (balanced) layups where the neutral axis is actually in the physical center of the panel cross-section. For panels with unsymmetrical cross-sections due to different layer thicknesses, different materials, etc., it is suggested that the Gamma method be used. In that case, the position of the neutral axis of the panel cross-section should be determined using the basic mechanics of material rule, i.e. that the first moment of the cross-section should be zero along its neutral axis. Once the position of the neutral axis is determined, the stresses in the CLT panel can be determined based on the distance of the particular layer (zone) from the neutral axis. In this case, the Gamma method described in Appendix A1 of this Chapter may provide a faster solution.

3.5.2 Flatwise Shear Resistance

In the calculation of the shear resistance of CLT panels loaded out-of-plane, the effect of the loads acting within a certain distance from a support equal to the thickness of the panel may not be taken into account. The factored shear resistance of the panel in the major strength direction, $V_{r,zy}$, can be calculated according to Clause 8.4.4.2 in CSA O86:

$$V_{r,zy} = \phi \, F_s \, \frac{2A_{g,zy}}{3} \tag{10}$$

where:

ϕ = 0.9 is the resistance factor;

$A_{g,zy}$ = gross cross-sectional area of the panel for the major strength axis, in mm^2, calculated as $b_y \cdot h$ from Figure 6;

F_s = factored shear resistance that can be calculated as:

$$F_s = f_s \, (K_D \, K_H \, K_{Sv} \, K_T) \tag{11}$$

where f_s is the specified strength in rolling shear of laminations in the longitudinal layers, in MPa.

Similarly, the factored shear resistance of the panel in the minor strength direction, $V_{r,zx}$ can be calculated as:

$$V_{r,zx} = \phi \, F_s \, \frac{2A_{g,zx}}{3} \tag{12}$$

where:

ϕ = 0.9 is the resistance factor

$A_{g,zx}$ = gross cross-sectional area of the panel for the minor strength axis, in mm^2, calculated as $b_x \cdot h_x$ from Figure 7, excluding the outermost longitudinal layers.

In cases where a CLT panel has multiple longitudinal outermost layers, all these layers must be excluded from the gross cross-sectional area $A_{g,zx}$.

3.5.3 Cantilevered and Statically Indeterminate Elements

The analytical design procedure developed for CLT panels generally assumes that the floor/roof elements are simply supported with a span of "l". For cantilever CLT slabs, it is suggested that the length l in the calculations be taken as two times the cantilever length "l_c" when calculating the $(EI)_{eff}$ with the Gamma method.

To determine the effective bending stiffness $(EI)_{eff}$ with the Gamma method in continuous multi-span beams, two approaches are suggested: a simplified procedure, and an iterative procedure. According to the simplified procedure, the span in the calculations should be taken as $0.8 \cdot l$. 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 redo the analysis again. Usually after only a few iterations a stable solution for $(EI)_{eff}$ can be obtained.

3.5.4 Two-Way Slabs

CLT elements used in floor or roof assemblies are usually simply supported on walls or beams on both sides and will generally act in the principal direction when loaded perpendicular to the plane. These panels can then be either free or connected to another panel along the other two edges. In some cases, CLT panels can be supported on three or even four sides, as there are some panels on the market that have a width of up to 4 meters. This can also be the case when beamless (flat plate) systems are used, such as in the Brock Commons tall wood building in Vancouver, where CLT panels are supported directly by the columns (i.e., point supported). In these cases, the two-way behaviour of the CLT slab system should be taken into account. Evaluation of the two-way action has to include the influence of the support conditions, as different support conditions may influence the relative effective stiffness of the panels at the supports. 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 (CAC, 2017) and CSA A23.3 Design of Concrete Structures (CSA, 2014), 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 the 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 the centrelines of successive columns.

Figure 8 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 less than or equal to 2 (b/a ≤ 2).

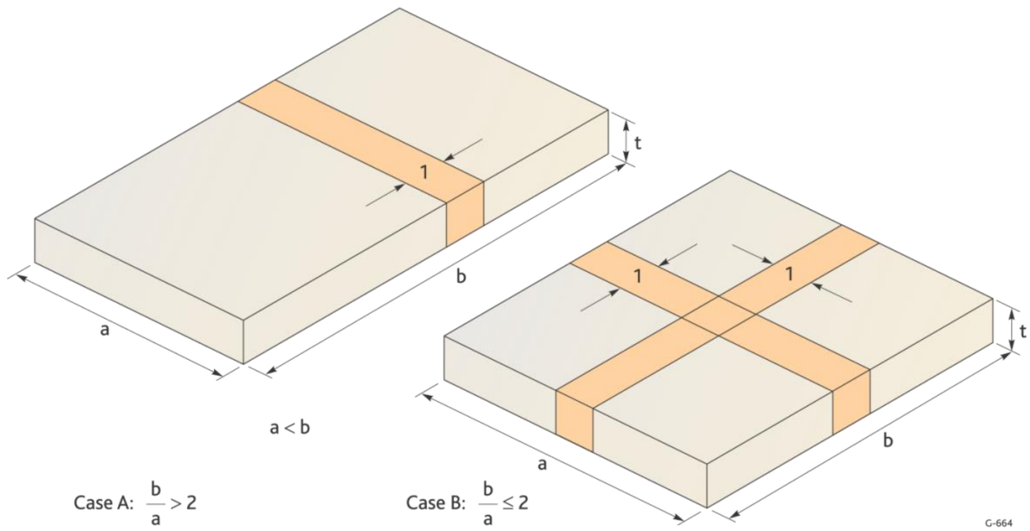


Figure 8 Cases of CLT panels with different size ratios

Based on the theory of plates (Timoshenko, 1959) and on the details presented above, it is suggested that CLT panels supported on four sides should be designed in one direction (i.e. in the shorter direction *a*) when b/a > 2, where *a* and *b* represent the panel dimensions. In that case, the length *L* used in the design should be *a,* as shown in Figure 8. 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 panels 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 MOE parallel and perpendicular to the action of the load, rolling shear in both directions, torsional stiffness of the panel, 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 panel 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.

When using finite element software, the two-way acting CLT panels can be modeled as orthotropic plates that have bending stiffness $EI_{eff,\,x}$ and $EI_{eff,\,y}$ in each orthogonal direction. In most finite element software, the torsional stiffness of the panel is calculated automatically, based on the input for the bending stiffness in both directions. These torsional stiffness values have been found to be on the high side and it has been suggested that they be reduced to 50% of the calculated value for 3-ply panels and to only 25% of the value for 5-ply panels (Wallner et al., 2014).

3.6 RESISTANCE OF CLT WALLS

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

a) Axial in-plane compression loads from gravity and live loads, with or without eccentricity "*e*"

b) Lateral in-plane loading from wind and earthquake loads

c) Lateral out-of-plane loading from wind loads.

When CLT walls are under out-of-plane wind loading only, they should be analysed in the same way as floor systems under vertical loads (Section 3.5). For walls loaded in-plane, please refer to Section 3.8 of this Chapter, as well as to Chapter 4 *"Lateral design of cross-laminated timber buildings"* of this Handbook.

3.6.1 Pure Axial Loads

When CLT panels are loaded under in-plane axial loads, only the layers with laminations oriented parallel to the applied axial load should be assumed to carry that load. For optimum design, CLT wall panels should normally be placed with the outer layers parallel to the gravity loads.

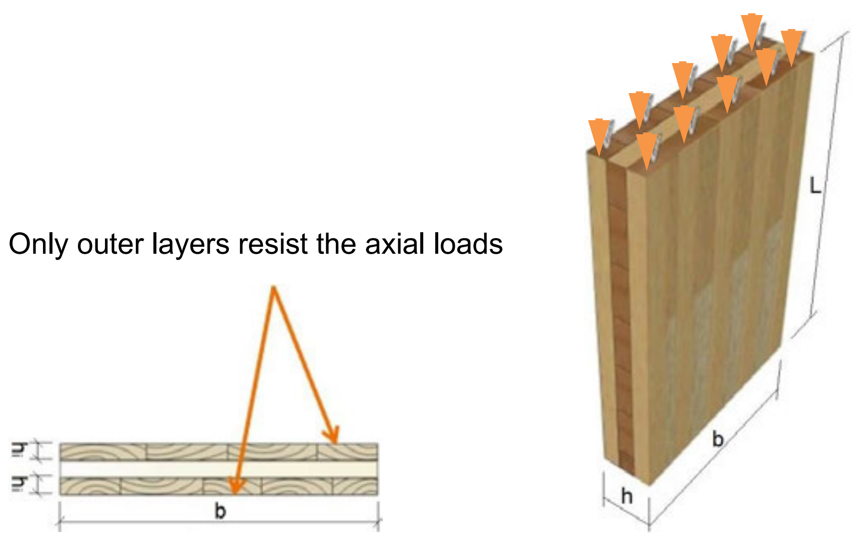


Figure 9 Axially loaded 3-ply CLT panel

The factored compressive resistance of a CLT panel under axial loads P_r should be calculated as:

$$P_r = \phi F_c A_{eff} K_{Zc} K_C \quad [13]$$

where $\phi = 0.8$ and:

A_{eff} = effective cross-sectional area of the panel accounting only for the layers with laminations oriented parallel to the axial load, as given in Equation [15], in mm^2

K_{zc} = Size factor for compression, as given in Equation [16]

K_c = Slenderness factor for compression members, as given in Equation [17]

$$F_c = f_c \left(K_D K_H K_{Sc} K_T\right) \quad [14]$$

$$A_{eff} = b \cdot h_{eff} = b \cdot \sum_i h_i \quad [15]$$

$$K_{Zc} = 6.3 \left(2\sqrt{3} \cdot r_{eff} \cdot L\right)^{-0.13} \leq 1.3 \quad [16]$$

$$K_C = \left[1.0 + \frac{F_c K_{Zc} C_c^{\,3}}{35 E_{05} (K_{SE} K_T)}\right]^{-1} \quad [17]$$

where:

E_{05} = the modulus of elasticity for design of compression members, only for the laminations oriented parallel to the axial load, in MPa

L = length of the panel, in mm

r_{eff} = the radius of gyration as given in Equation [18] below:

$$r_{eff} = \sqrt{\frac{I_{eff}}{A_{eff}}} \quad [18]$$

where I_{eff} is the effective out-of-plane moment of inertia of the panel accounting only for the layers with laminations oriented parallel to the axial load, in mm^4.

The slenderness ratio, C_c, of CLT panels of constant rectangular cross-section shall not exceed 43:

$$C_c = \frac{L_e}{\sqrt{12} \cdot r_{eff}} \leq 43 \qquad [19]$$

Or in other words:

$$\frac{L_e}{r_{eff}} \leq \; 150 \qquad [20]$$

Tabulated values for r_{eff}, A_{eff}, and I_{eff} for 3- to 9-ply CLT panels are given in the CWC Wood Design Manual (CWC, 2017).

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

CLT panels subject to combined out-of-plane bending and compressive axial load shall be designed to satisfy the interaction equation given below:

$$\frac{P_f}{P_r} + \frac{M_f}{M_r}\left[\frac{1}{1-\frac{P_f}{P_{E\,v}}}\right] \leq 1 \qquad [21]$$

where:

P_f = factored compressive axial load

P_r = factored compressive resistance under axial load, calculated in accordance with Equation [13]

M_f = factored bending moment

M_r = factored bending moment resistance, calculated in accordance with Equation [1]

$P_{E,v}$ = Euler buckling load in the plane of the applied bending moment adjusted for shear deformation, calculated according to Equation [22] below:

$$P_{E,v} = \frac{P_E}{1 + \frac{\kappa \cdot P_E}{(GA)_{eff}}} \qquad [22]$$

where:

P_E = Euler buckling load in the plane of the applied bending moment in accordance with Clause 7.5.12 of CSA O86, where I_{eff} and E_{05} are determined, accounting for the layers with laminations (Table 1) oriented parallel to the axial load only and as given in Equation [23]

κ *(kappa)* = shear form factor

$(GA)_{eff}$ = effective planar shear rigidity, accounting for all layers according to Equation [6], in N

$$P_E = \frac{\pi^2 E_{05} I_{eff}}{(K_e \cdot L)^2} \qquad [23]$$

The shear correction coefficient κ *(kappa)* for CLT is influenced by the shear-flexible transverse layers and should be taken as 1.0 when the Kreuzinger method is used.

An example calculation for a CLT wall under axial and out-of-plane loads is given in Section 3.10.4 of this Chapter. The CWC Wood Design Manual (CWC, 2017) contains tables for the resistance of CLT walls subjected to axial and out-of-plane wind loads. The tables provide combinations of the factored compressive resistance P'_r and maximum factored lateral wind resistance w'_r that satisfy interaction Equation [23]. The tables cover 3-ply to 9-ply CLT walls with 35-mm thick layers subjected to short term loads, with different heights, stress grades, and eccentricity of the axial loads from $e = 0$ to $e = d/2$. The eccentric axial load values are appropriate for situations where the effect of the eccentric load and the effect of the wind load cause bending in the same direction. The designer is responsible for selecting the most appropriate eccentric load value to use. The concentric axial load case may be used where the effect of the wind load and the effect of the eccentric axial load produce opposing bending.

3.7 COMPRESSION RESISTANCE PERPENDICULAR TO THE PANELS

During design, the factored bearing forces should be checked against the factored compressive resistance of CLT panels perpendicular to grain (Figure 10). According to Clause 8.4.7.2 of CSA O86, the factored compressive resistance perpendicular to grain, Q_r, of a CLT panel under the effect of all applied loads should be calculated as per Equation [24] below:

$$Q_r = \phi F_{cp} A_b K_B K_{Zcp} \qquad [24]$$

where ϕ = 0.8 and:

$$F_{cp} = f_{cp}(K_D K_{Scp} K_T) \qquad [25]$$

where:

f_{cp} = specified strength in compression perpendicular to grain of laminations in the outer layers (Table 1), in MPa

A_b = bearing area, in mm^2

K_B = length of bearing factor (1.75 for L = 12.5mm, 1.0 for $L \geq 150$mm)

K_{Zcp} = size factor for bearing = 1.0

Figure 10 Compression failure perpendicular to the CLT panel at a support

When the applied loads act within a distance from the centre of the support equal to the depth of the CLT panel, the factored compressive resistance perpendicular to grain, Q'_r shall be calculated as per Equation [26] below:

$$Q'_r = \frac{2}{3} \phi \, F_{cp} \; A'_b \; K_B K_{Zcp} \qquad [26]$$

where A'_b is the average bearing area, in mm^2, that is calculated according to Clause 8.4.7.3.2 of CSA O86. The same equation can be used when there are unequal bearing areas on the opposite surfaces of the panels.

The factored compressive resistance at an angle to the face of the CLT panel should be calculated in the same fashion as for lumber, using Clause 6.5.8 of CSA O86; the factored resistance values for CLT loaded axially should be obtained according to Equation [13] with $K_{Zc} = K_C = 1.0$, while Equations [24] and [26] should be used for CLT loaded on the face.

3.8 CLT USED AS A BEAM OR LINTEL

While CLT elements are mostly used in floor and wall applications, they can also be efficient in resisting loads as beam or lintel members loaded on the edge. Beams made of CLT offer some advantages over solid or glued laminated timber beams, due to their layup of orthogonally bonded layers. One major advantage of CLT is the high tensile strength perpendicular to the beam length that is provided by the orthogonal (vertical) layers, which makes CLT beams less susceptible to cracks. Consequently, the use of CLT in beam applications can provide additional robustness. Figure 11 illustrates two 5-layer CLT beams under in-plane bending loads. The beam on the left has its outer layers parallel to the beam length, while the one on the right has its outer layers perpendicular to the beam length (i.e., parallel to the applied load). The same configurations are possible for 3- and 7-layer CLT panels.

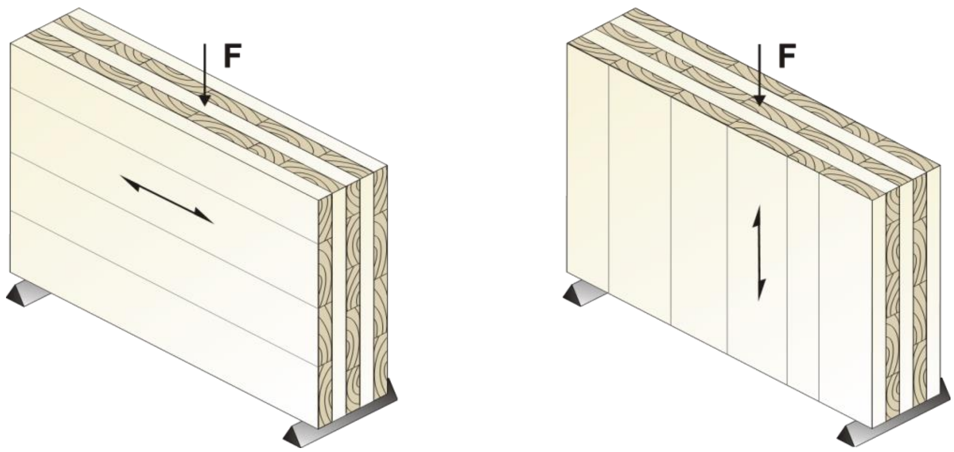


Figure 11 CLT panels with different orientations used as beams or lintels under in-plane loads

A significant amount of research has been conducted for CLT under out-of-plane loading, and the design approaches and the strength values are well understood and agreed upon. For CLT under in-plane loading, however, only a handful of research studies have been conducted, and uniformly accepted design methods are currently under development. Due to the typical composition of CLT, the stress state when a CLT is used as a beam is complex and several failure modes need to be considered in the design. Besides the bending stresses, there are three different shear failure modes that need to be taken into consideration.

3.8.1 Bending Strength

When calculating the bending resistance, it is suggested that only the effects of the layers that are parallel to the length of the CLT beam be taken into account. Keeping that in mind, the bending stress σ_x may be expressed as:

$$\sigma_x = M \cdot y \cdot \frac{(E_{mean})}{(EI)_{eff}} \qquad [27]$$

where x is the direction along the beam and y is the vertical direction. The maximum stress, therefore, will occur for $y=H/2$, where H is the beam depth; Equation [27] can then be expressed as:

$$\sigma_{\max} = M \cdot 0.5H \cdot \frac{(E_{mean})}{(EI)_{eff}} \qquad [28]$$

Using the design analogy presented in CSA O86, the factored bending resistance should be verified against the maximum stress:

$$\sigma_{\max} \leq \phi \cdot F_b \qquad [29]$$

The factored moment bending resistance, M_r, is determined in terms of the specified bending strength, F_b, as:

$$M_r = \phi \cdot F_b \cdot \frac{(EI)_{eff}}{E_{mean}} \cdot \frac{1}{0.5H} \qquad [30]$$

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

When the moduli of elasticity of all longitudinal layers are equal, Equation [30] can be expressed as:

$$M_r = \phi \cdot F_b \cdot \frac{I_{eff}}{0.5H} \qquad [31]$$

and I_{eff} can be calculated as:

$$I_{eff} = \frac{t_{eff} \cdot H^3}{12} = \frac{H^3}{12} \cdot \sum_i t_i \qquad [32]$$

where H is the CLT beam depth and t_i is the thickness of the boards in the longitudinal direction (i.e. the effective boards for that orientation).

It should be noted that this method assumes a perfect composite action between effective longitudinal boards. Consequently, using Equation [31] with Equation [32] to estimate the bending strength of a deep beam or a lintel may not lead to a conservative solution, due to possible size effect phenomena and flexibility between the individual longitudinal boards. A (much) more conservative way to evaluate the I_{eff} would be to calculate it as a sum of the moments of inertia of all individual longitudinal (effective) boards.

3.8.2 Shear Failure Modes and Stresses

3.8.2.1 General

In CLT beams, like in solid materials, transversal forces acting in-plane cause shear stresses. The shear stress distribution can be assumed to be constant over the element thickness. In CLT beams where adjacent lamellas (boards) within individual layers are not edge-glued, the thickness is not constant throughout the height of the CLT beam. In cross-sections at unglued joints between neighbouring lamellas, the shear forces can only be transferred by lamellas in the perpendicular direction. Consequently, the shear stresses in these so-called net cross-sections are higher than in the gross cross-sections (between unglued joints). The transfer of shear forces between longitudinal and transversal lamellas also causes shear stresses in the crossing areas of orthogonally bonded lamellas. By considering the shear stresses in the lamellas and in the crossing areas, three different failure modes exist in CLT beams subjected to shear stresses, as shown in Figure 12 (Flaig and Blass, 2013).

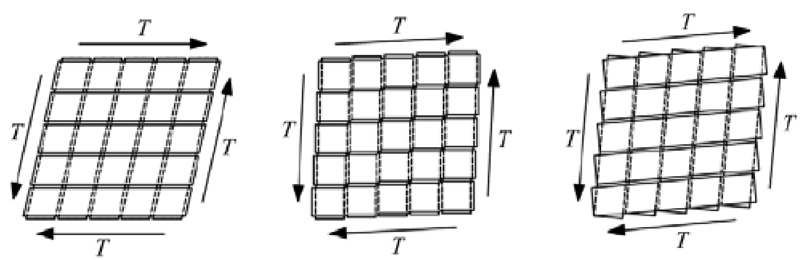


Figure 12 Shear failure modes I, II and III in CLT beams subjected to in-plane transversal forces (from left to right)

Failure mode I is characterised by shear failure parallel to the grain in the gross cross-section of the beam. This failure occurs in sections between unglued joints with equal shear stresses in longitudinal layers and transversal layers. Failure mode II is characterised by shear failure perpendicular to the grain in the net cross-section of the beam. This failure occurs in sections coinciding with unglued joints with shear stresses only in lamellas perpendicular to the joints. Failure mode III is characterised by shear failure within the crossing areas between the orthogonally glued boards (lamellas). This failure mode is caused by torsional and unidirectional shear stresses resulting from the transfer of the shear forces between adjacent layers. Using a composite beam model, the shear stresses in the crossing area can be further characterized as: (a) shear stress parallel to the beam axis $\tau_{x,z}$, (b) shear stress perpendicular to the beam axis $\tau_{y,z}$, and (c) torsional shear stress τ_{tor}.

3.8.2.2 *Failure Modes I and II*

Research results (Flaig and Blass, 2013) have shown that the shear strength of a CLT beam is affected not only by the basic material strength and the gross cross-sectional dimensions, but also by the ratio of the thickness of the longitudinal and the transversal layers, by the dimensions of the cross-sections of the individual laminations, and by the number of laminations. Many of the geometry parameters such as the thickness of the longitudinal and perpendicular layers are defined by the CLT producers, but parameters such dimensions of individual laminations and lamination placement with respect to the edges of the CLT beam are often not known to the design engineer. The CLT beams are usually cut from CLT panels without consideration to the location of the beam element edges with respect to the edges of the individual laminations. Main aspects of these topics will be discussed below. Since this topic has not been codified in Canada, the designations for the dimensions and stresses are the same as in the research sources. For more detailed information please refer to Flaig and Blass, 2013, Danielsson et al. 2017a, Danielsson et al. 2017b, Brandner et al. 2013, Brandner et al. 2015, and Jeleč et al. 2016.

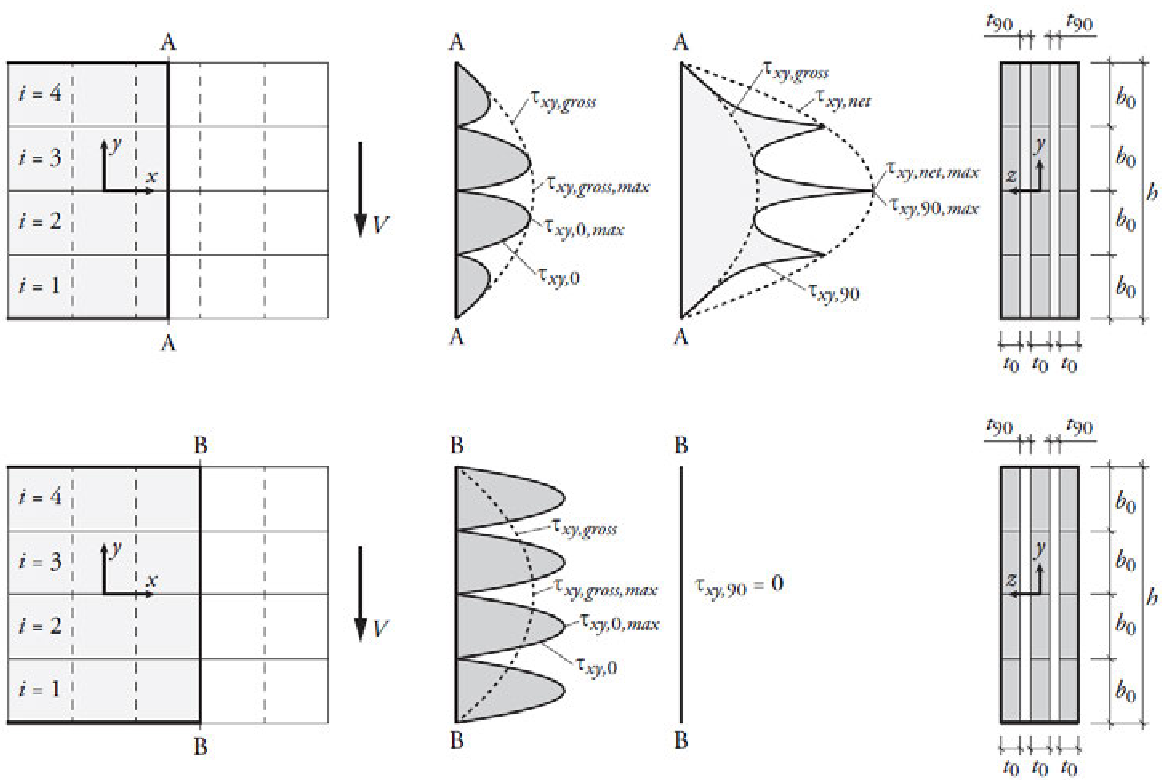


Figure 13 **CLT panels (beams or lintels) under in-plane loads. Shear stress distribution in the CLT beam cross-section A (upper part of the figure), and in cross-section B (lower part) (Danielsson et al., 2017b).**

Shear stress distributions in the longitudinal and transversal layers of a CLT header or lintel beam are shown in Figure 13 (Danielsson et al., 2017b), which is based on the CLT beam theory developed by Flaig and Blass, 2013. The shear stress in the longitudinal layers is denoted as $\tau_{xy;0}$ and the shear stress in the transversal layers is denoted as $\tau_{xy;90}$. The shear stress within each lamination is assumed to be constant with respect to the CLT beam thickness (*z* direction). All boards in the CLT beam are assumed not to be edge-glued, meaning that the shear stress must be zero at the interface between two individual laminations within the same layer (Figure 13). Also, it is assumed that there is no friction between adjacent laminations of the same layer. The CLT beam in Figure 13 is made of longitudinal layers that have four identical longitudinal boards ($m = 4$) with identical lamination width ($b = b_0$) along the beam height. The thickness of the boards (laminations) in the longitudinal layers is assumed to be twice that of the transversal ones, i.e. $t_0 = 2\ t_{90}$. Section A-A in the CLT beam is located through the middle of a transversal lamination, while section B-B is located on the interface between adjacent transversal laminations.

According to the beam theory, the maximum value of the gross shear stress related to shear failure mode I (Figure 12) can be calculated as:

$$\tau_{xy,gross,max} = \frac{3\ V}{2\ h\ t_{gross}} \qquad [33]$$

where V is the shear force, h is the beam height, and t_{gross} is the total thickness of the CLT beam.

To quantify the stresses related to failure mode II we have to look in detail at Figure 13. For section A-A that corresponds to a location through the center of a transversal lamination, the shear stress $\tau_{xy;0}$ in the two outermost longitudinal laminations (i = 1 and i = 4) and the shear stress $\tau_{xy;90}$ in the transversal laminations, follow a parabola defined by the gross shear stress $\tau_{xy;gross}$. For locations along the CLT beam height that correspond to the interfaces between adjacent longitudinal laminations ($y=b_0$, $y=0$, or $y=b_0$), the shear stress in the longitudinal layers must be zero to maintain equilibrium and the entire shear flow in the beam must instead be carried by the transversal laminations only. The values of the shear stress $\tau_{xy;90}$ in the transversal layers at these locations can be found on a parabola defined by the net shear stress $\tau_{xy;net}$ with the maximum stress being:

$$\tau_{xy,90,max} = \tau_{xy,net,max} = \frac{3\ V}{2\ h\ t_{net,90}} \qquad [34]$$

where $t_{net,90}$ is the total thickness of all perpendicular layers, which in the case of Figure 13 is equal to $2{\cdot}t_{90}$.

In section B-B, at the interface between adjacent transversal laminations within the same layer, the entire shear force must be carried by the longitudinal laminations. If we assume that the total shear force V is divided evenly between the longitudinal laminations (i.e. $V_i = V/m$ where m is the number of horizontal laminations), and that the shear stress distribution is parabolic within each lamination, the maximum shear stress in the longitudinal laminations is given by:

$$\tau_{xy,0,max} = \frac{3\,V}{2\,h\,t_{net,0}} \qquad [35]$$

where $t_{net,0}$ is the beam net cross-section thickness when considering longitudinal layers only, i.e. $3\,t_0$. It should be noted that at this location, the shear stress in the transversal laminations is zero due to the assumptions of no edge-gluing and zero friction between adjacent laminations.

When the CLT beam has an even number of laminations m, and for the assumed stress distributions, the maximum value of the shear stress in the transversal laminations is exactly equal to the maximum value of the net shear stress, i.e. $\tau_{xy;90,\,max} = \tau_{xy;net,\,max}$. At locations corresponding to section A-A, the maximum value of the shear stress in the longitudinal laminations is approximately, but not exactly, equal to the maximum value of the gross shear stress, i.e. $\tau_{xy;0,max} \approx \tau_{xy;gross,\,max}$. For CLT beams with an uneven number of longitudinal layers (for example with $m = 3$), the situation is slightly different. For uneven m, the maximum value of the shear stress in the longitudinal laminations is at section A-A and exactly equals the maximum value of the gross shear stress, i.e. $\tau_{xy;0,\,max} = \tau_{xy;gross,max}$. The value of the maximum shear stress in the transversal layers is considered to be approximately equal to the maximum value of net shear stress, i.e. $\tau_{xy;90,max} \approx \tau_{xy;net,max}$. The maximum value of the shear stress in the longitudinal layers in section B-B however, is the same irrespective of whether m is even or uneven, according to the assumed stress distributions.

Shear failure modes I and II are commonly referred to as *gross shear failure* and *net shear failure,* respectively. The stress components can be calculated using Equation [33] for failure mode I and using Equations [34] and [35] for failure mode II.

3.8.2.3 Failure Mode III

When a CLT beam is loaded in-plane, shear stresses τ_{xz} and τ_{yz} are developed in the crossing areas between the transversal and longitudinal laminations, in addition to the shear stress τ_{xy} that is present in both the longitudinal and transversal layers. Using a composite beam model as suggested by Flaig and Blas, 2013, the shear stresses acting in the crossing area can be categorized as: (a) shear stress parallel to the beam axis τ_{xz} , (b) shear stress perpendicular to the beam axis τ_{yz} and (c) torsional shear stress τ_{tor} (Figure 14). These shear stress components in the crossing areas simultaneously represent both the longitudinal and the rolling shear stress components in the longitudinal and transversal laminations. Detailed information about how the equations for these stresses are developed can be found in Flaig and Blass (2013), and Danielsson et al. (2017b).

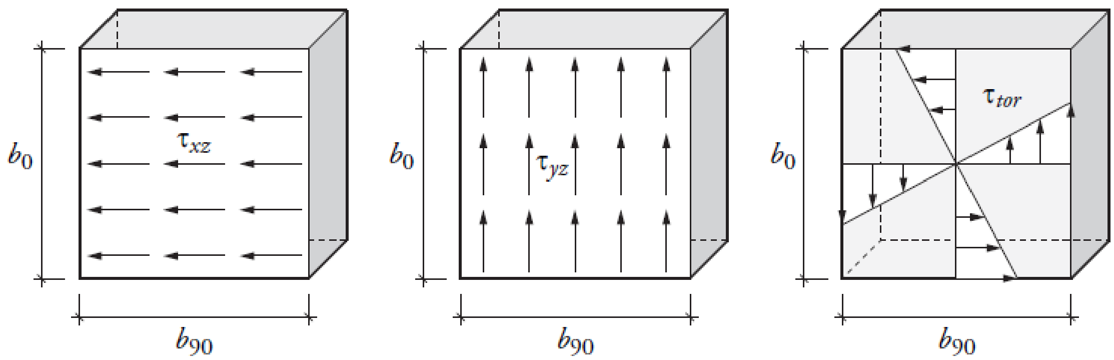


Figure 14 Assumed shear stress distribution in a crossing area (Danielsson et al., 2017b).

According to the assumed model given in Figure 15, the maximum shear stress parallel to the CLT beam length is found in the crossing areas of the uppermost or the lowest longitudinal laminations of the CLT beam, i.e. for lamella *i = 1* and lamella *i = m*, where *i* = 1, 2…*m* represents the position of the longitudinal laminations along the beam height (y-direction) and *m* is the total number of lamellas. The maximum value also depends on layup dimensions such as the thickness of the longitudinal layers $t_{0,k}$, where *k* =1,2… and shows the position of the longitudinal layers along the beam thickness (z-direction). The most favorable stress situation is obtained for layups with a constant value of $t_{0,k}/n_{ca,k}$ for all longitudinal layers, where $n_{ca,k}$ is the number of crossing areas that the longitudinal lamination *i,k* shares with the adjacent transversal laminations ($n_{ca,k}$ = 1 for all external layers and $n_{ca,k}$ = 2 for all internal layers). This condition is always fulfilled for a 3-layer CLT beam with symmetric layup, while for 5- and 7-ply CLT elements, it is only fulfilled when the internal layers are twice as thick as the external layers.

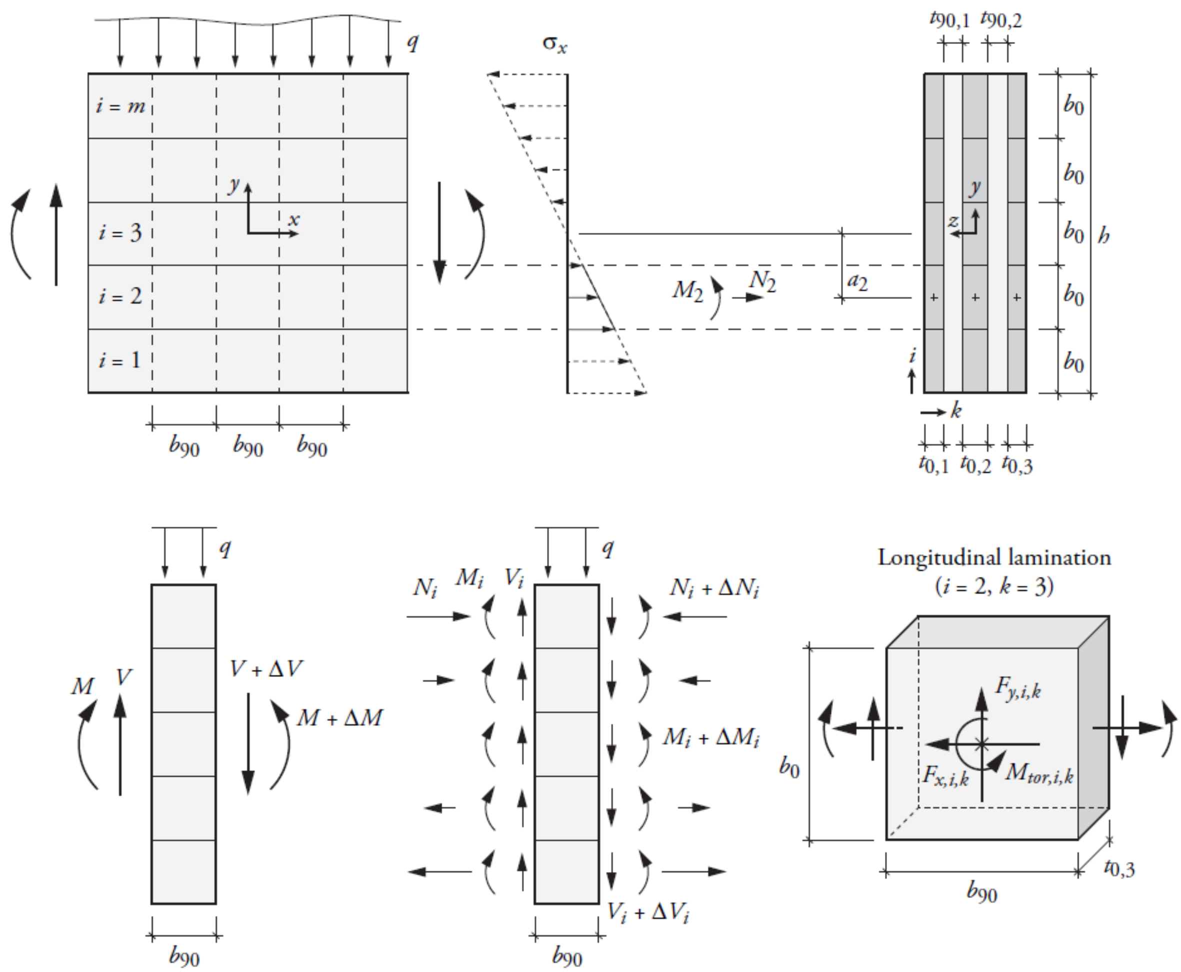


Figure 15 **Composite beam model used for derivation of the shear stresses in the crossing areas (Danielsson et al., 2017b).**

According to Flaig and Blass, 2013, if we assume that the width of all longitudinal boards in the CLT beam is a constant value b_0 (Figure 15), then the maximum longitudinal shear stress can be estimated as:

$$\tau_{xz,max} = \frac{6\,V}{b_0^2\,n_{ca}}\left(\frac{1}{m^2} - \frac{1}{m^3}\right) \qquad [36]$$

where m is the number of horizontal boards in the CLT beam or $m=h/b_0$. It should be noted that Flaig and Blas 2013, have developed this equation for a reference case where $t_{0,2} / t_{0,1} = t_{0,2} / t_{0,3} = 2.0$, or in other words, for a case when the thickness of the middle layer of the CLT beam is twice the thickness of all other layers (Figure 15). They also reported that Equation [36] can provide a good estimate for a wide variety of CLT beam configurations with different thicknesses. Danielsson *et al*. 2017a, and 2017b, however, have reported that an increase in the maximum shear stress $\tau_{xz,max}$ in the range of 33% is present for regular cases where all longitudinal layers have an equal thickness. This increase can be as high as 60% for cases where the external layers have twice the thickness of the internal ones.

Shear stresses perpendicular to the CLT beam length, τ_{yz} , arise due to transverse loading of the beam by externally applied loads and support reaction forces. The transverse loads are assumed to be introduced in the transversal layers, due to the large difference in stiffness between the parallel- and the perpendicular-to-grain loading directions. Shear stress perpendicular to the beam axis may also arise due to internal redistribution of forces caused by irregularities in the geometrical shape of the CLT beam, e.g. in the vicinity of a hole or a notch. All these stresses (caused by a force $F_{y,i,k}$ in Figure 15) are assumed to be evenly distributed over the crossing area between a longitudinal and a transversal lamination. Consequently, these stresses, due to vertical load q, can be calculated as:

$$\tau_{yz} = \frac{q}{b_0 m\, n_{ca}} = \frac{q}{h\, n_{ca}} \qquad [37]$$

where n_{ca} is the number of crossing areas throughout the CLT beam thickness.

Finally, torsional stresses in the crossing areas τ_{tor} occur due to relative rigid body rotation in a shear compliant material (medium). The torsional moment M_{tor} and the related torsional shear stress τ_{tor} acting in the crossing areas between longitudinal and transversal laminations can be derived according to the models shown in Figures 14 and 15, and can be found in Flaig and Blass (2013), Danielsson et al. (2017a and 2017b). If we assume that all layers have the same thickness b (i.e. $b_0 = b_{90} = b$) and that the ratio of the layer thickness vs. the number of crossing areas is constant for all longitudinal layers ($t_{0,k}$ / $n_{ca,k}$ = constant), then the torsional stress can be calculated as:

$$\tau_{tor} = \frac{3\,V}{b^2\, n_{ca}}\left(\frac{1}{m} - \frac{1}{m^3}\right) \qquad [38]$$

If the width of the boards in the layers is not the same, then the torsional stress in Equation [38] has to be multiplied by a factor K_b that can be obtained according Equation [39]:

$$k_b = \frac{b_{max}}{b_0}\left(\frac{2\,b_0^2}{b_0^2 + b_{90}^2}\right) \qquad [39]$$

where b_{max} is the maximum value for the width of either the longitudinal boards (b_0) or the transversal boards (b_{90}) i.e. $b_{max} = max\{b_0, b_{90}\}$.

Research information on the performance of CLT beams with notches and holes can be found in Danielsson et al. (2017a and 2017b).

3.8.2.4 Checking the Shear Stress States

For verification with respect to gross shear failure (mode I), the specified in-plane shear strength given by the CLT producer (usually around 5 MPa) should be used. For the net shear failure (mode II), a characteristic shear strength of 8.0 MPa is suggested in Europe by Flaig (2015). To obtain a conservative solution, this value can be assumed to be the same as for mode I.

When checking the stress state for failure mode III (shear failure in the crossing areas), a stress interaction criterion needs to be chosen, since shear stresses in two directions are present. The three shear stress components that were mentioned represent either shear stress in the direction parallel to the beam length (τ_{xz}), shear stress in the direction perpendicular to the beam length (τ_{yz}), or shear stress in both directions, i.e. parallel and perpendicular to the beam (τ_{tor}), as illustrated in Figure 14. For any specific point in a single crossing area that bonds a longitudinal and a transversal board (lamination), these three shear stress components present themselves as a longitudinal shear, a rolling shear, or a combination of both. A shear stress component giving pure longitudinal shear in the longitudinal lamination will produce pure rolling shear in the transversal laminations, and vice versa.

A compilation of several possible stress interaction criteria is presented by Flaig (2015). The criteria were evaluated based on the results obtained in experimental tests conducted on a single crossing area. All considered stress interaction criteria were based either on linear or quadratic interaction of the three stress components and by comparing them either to the rolling shear strength f_s (for the τ_{xz} and the τ_{yz}), or to the torsion shear strength f_{tor} (for τ_{tor}). Assuming a constant ratio of f_{tor} / f_s of 2.33, the most appropriate criteria were found to be:

$$\frac{\tau_{tor}}{f_{tor}} + \frac{\tau_{xz}}{f_s} \leq 1.0 \qquad [40]$$

and

$$\frac{\tau_{tor}}{f_{tor}} + \frac{\tau_{yz}}{f_s} \leq 1.0 \qquad [41]$$

Additional research is needed in the area of CLT used as beams and lintels in general. Until further research information is available, the failure checks above have to be considered with caution.

3.8.3 Modification Factors (K-factors)

3.8.3.1 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 lateral stability factor K_L. Some design provisions in CSA O86 could be used by designers as guidance. In particular, Sections 6.5.4.2, 7.5.6.4, and 8.4.5 of CSA O86 could be helpful.

3.8.3.2 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 that the design provisions given in Section 6.4.5 of CSA O86 be followed when calculating K_{Zbg}.

3.8.3.3 Curvature Factor K_X and Radial Resistance K_R

This Chapter does not cover curved CLT products.

3.8.4 Creep Behaviour of CLT in Bending

Duration of load and creep behaviour have to be taken into account in the design of structural elements made of CLT. Various options to address creep and duration of load effects for CLT panels are presented in Chapter 6 "*Duration of Load and Creep Factors for Cross-Laminated Timber Panels*" of this Handbook.

3.9 DEFLECTIONS AND VIBRATION OF CLT PANELS

Out-of-plane deflection and vibration characteristics of CLT panels in floor applications can often be a limiting factor in determining the maximum span. When calculating deflections of CLT floors, the shear deformations have to be taken into account. Deflection Δ of a CLT floor panel with length l, loaded with uniformly distributed load w, can be calculated according to Equation [42], which has a bending and a shear component.

$$\Delta = \frac{5wl^4}{384(EI)_{eff}} + \frac{kwl^2}{8(GA)_{eff}} \quad [42]$$

where:

κ *(kappa)* = shear coefficient form factor (set to 1.0 in this Handbook. Please see Chapter 6 for details).

Laboratory tests performed by FPInnovations on various floor systems have shown that the vibration behaviour of CLT floors is different from lightweight joisted wood floors and heavy concrete slab floors. CLT floors are heavier than conventional joisted wood floors and lighter than concrete slab floors. FPInnovations has proposed a design method for controlling vibrations in CLT floors and the method has now been implemented in CSA O86. More details on vibration properties of CLT floor panels are provided in Chapter 7 *"Vibration Controlled Designs for Mass Timber Floors and Tall Wood Buildings"* of this Handbook.

3.10 SIMPLE DESIGN EXAMPLES

The main purpose of the simple examples provided here is to illustrate methods for calculating the basic stiffness and strength properties of CLT panels used in structural applications. It should be noted that not all the necessary steps and checks are included in each of the examples. An example that includes a more complex design for an 8-storey mass timber building is included in Chapter 13 *"Design Example"* of this Handbook.

More detailed design examples and selection tables for CLT panels used as floors, roofs, or walls are provided in the 2017 Edition of the Wood Design Manual (CWC, 2017):

1) A design example and selection tables for CLT floors or roofs under the factored moment resistance M_r, the factored shear resistance V_r, and maximum spans determined in accordance with floor vibration criteria and deflection limit

2) A design example and selection tables for CLT walls under the factored compressive resistance P_r, with major or minor strength axis parallel to the applied load

3) A design example and selection tables for CLT walls under the factored compressive resistance P'_r and maximum factored lateral wind resistance w'_r that satisfy the interaction Equation [21].

3.10.1 Effective Flatwise Bending Stiffness ($(EI)_{eff}$) and Bending Strength

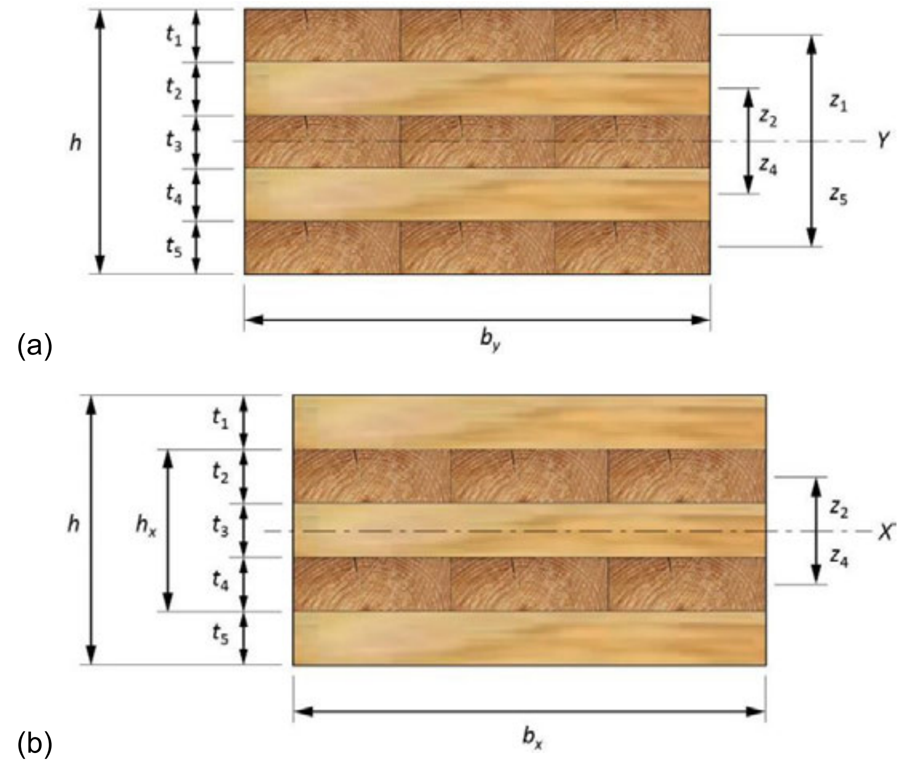


Figure 16 Cross-section of a 5-layer CLT panel in the major direction (a) and minor direction (b)

Typical cross-section details of a CLT floor panel in the major and minor directions are shown in Figure 16a and 16b. If we assume that the panels are of E1 stress grade (Table 1) and the thickness of the layers are:

$$t_1 = t_3 = t_5 = 34\,\text{mm} \qquad t_2 = t_4 = 30\,\text{mm} \qquad h = 162\,\text{mm}$$

the effective bending stiffness in the major direction $(EI)_{eff}$ can be calculated as:

$$(EI)_{eff,y} = \sum_{i=1}^{n} E_i \cdot b_y \cdot \frac{t_i^3}{12} + \sum_{i=1}^{n} E_i \cdot b_y \cdot t_i \cdot z_i^2 = 3.4 \times 10^{12}\ \text{N} \cdot \text{mm}^4$$

Assuming a strip with a 1-m width ($b_y = 1\,m$), the section modulus in the major direction $S_{eff,y}$ would be:

$$S_{eff,y} = \frac{(EI)_{eff,y}}{E_1} \cdot \frac{2}{h} = 3.6 \times 10^6\ \text{mm}^3$$

Assuming standard service conditions ($K_D = K_S = K_T = K_H = 1$), the bending resistance $M_{r,y}$ can be calculated as:

$$M_{r,y} = \phi \cdot F_b \cdot S_{eff,y} \cdot K_{rb,y} = 7.7 \times 10^7 \text{ N·mm}$$

Similarly, for the minor strength direction:

$$(EI)_{eff,x} = \sum_{i=2}^{n-1} E_i \cdot b_x \cdot \frac{t_i^3}{12} + \sum_{i=2}^{n-1} E_i \cdot b_x \cdot t_i \cdot z_i^2 = 5.9 \times 10^{11} \text{ N·mm}^4$$

If we assume $b_x = 1\,m$

$$S_{eff,x} = \frac{(EI)_{eff,x}}{E} \cdot \frac{2}{h_x} = 1.4 \times 10^6 \text{ mm}^3$$

$$M_{r,x} = \phi \cdot F_b \cdot S_{eff,x} \cdot K_{rb,x} = 8.9 \times 10^6 \text{ N·mm}$$

3.10.2 Effective Flatwise Shear Stiffness $(GA)_{eff}$ and Shear Strength

For the same CLT panel cross-section, the effective shear stiffness in the major direction can be calculated as:

$$(GA)_{eff,zy} = \frac{(h - \frac{t_1}{2} - \frac{t_n}{2})^2}{\left[\left(\frac{t_1}{2 \cdot G_{1,zy} \cdot b_y}\right) + \left(\sum_{i=2}^{n-1} \frac{t_i}{G_{i,zy} \cdot b_y}\right) + \left(\frac{t_n}{2 \cdot G_{n,zy} \cdot b_y}\right)\right]} = 1.4 \times 10^7 N$$

The shear strength in the major direction $V_{r,zy}$ can be calculated as:

$$V_{r,zy} = \phi F_s \frac{2A_{g,zy}}{3} = 4.9 \times 10^4 N$$

Similarly, the shear stiffness in the minor direction can be calculated as:

$$(GA)_{eff,zx} = \frac{(h - \frac{t_1}{2} - \frac{t_n}{2})^2}{\left[\left(\frac{t_1}{2 \cdot G_{1,zx} \cdot b_x}\right) + \left(\sum_{i=2}^{n-1} \frac{t_i}{G_{i,zx} \cdot b_x}\right) + \left(\frac{t_n}{2 \cdot G_{n,zx} \cdot b_x}\right)\right]} = 1.6 \times 10^7 N$$

The shear strength in the minor direction $V_{r,zx}$ can be calculated as:

$$V_{r,zx} = \phi F_s \frac{2A_{g,zx}}{3} = 2.8 \times 10^4\, N$$

3.10.3 Compression Strength of a CLT Wall

If we assume that the same CLT panel used in Sections 3.10.1 and 3.10.2 is now used in a wall application where the panel width is 1 m (*b=1000mm*) and the height of the simple supported wall (panel length) is *L=3 m* , the effective thickness, effective cross-section area, and the effective out-of-plane moment of inertia can be calculated as:

$$h_{eff} = \sum_{i=1}^{(n+1)/2} t_{2n-1} = 102 \text{ mm}$$

$$A_{eff} = b \cdot h_{eff} = 1.0 \times 10^5 \text{ mm}^2$$

$$I_{eff,y} = \frac{(EI)_{eff,y}}{E_1} = 2.9 \times 10^8 \text{ mm}^4$$

Calculations of the effective radius of gyration r_{eff}, the slenderness ratio C_c, the size factor for compression K_{zc}, the slenderness factor for compression K_c, and finally the factored compression resistance P_r are as given below:

$$r_{eff} = \sqrt{\frac{I_{eff}}{A_{eff}}} = 53.2 \text{ mm}$$

$$C_c = \frac{L_e}{\sqrt{12} \cdot r_{eff}} = 16.3 \le 43$$

$$K_{Zc} = 6.3\left(2\sqrt{3} \cdot r_{eff} \cdot L\right)^{-0.13} = 1.1 \le 1.3$$

$$K_C = \left[1.0 + \frac{F_c K_{Zc} C_c^{\ 3}}{35 E_{05} (K_{SE} K_T)}\right]^{-1} = 0.78$$

$$P_r = \phi\, F_c\, A_{eff}\, K_{Zc} K_C = 1.4 \times 10^6\, N$$

3.10.4 CLT Wall Subjected to a Combination of Axial and Bending Loads

If we assume that the same 1-m wide, 5-ply CLT panel used in Section 3.10.3 is now used in a wall application subjected to an in-plane axial load P_f = 7.5 x 10^5 N, and out-of-plane bending moment $M_{f,y}$ = 5.0x10^6 N·mm, the calculation checks will be as given below:

$$M_{r,y} = \phi \cdot F_b \cdot S_{eff,y} \cdot K_{rb,y} = 7.7 \times 10^7 \text{ N·mm}$$

$$P_r = \phi\, F_c\ A_{eff}\, K_{Zc} K_C = 1.4 \times 10^6\, N$$

$$(GA)_{eff,zy} = 1.4 \times 10^7\, N$$

$$P_E = \frac{\pi^2 E_{05} I_{eff}}{(K_e \cdot L)^2} = 3.0 \times 10^6$$

$$P_{E,v} = \frac{P_E}{1 + \dfrac{\kappa \cdot P_E}{(GA)_{eff}}} = 2.5 \times 10^6$$

$$\frac{P_f}{P_r} + \frac{M_{f,y}}{M_{r,y}} \left(\frac{1}{1 - \dfrac{P_f}{P_{E,v}}} \right) = 0.83 < 1$$

3.10.5 Deflection of CLT Panels

If we assume that the same 1-m wide, 5-ply CLT panel is used in a floor application with a span of 4.5 m and subjected to a uniformly distributed live load w = 1.9 kPa, the deflection of the panel Δ_L can be calculated as shown below:

$$(EI)_{eff,y} = 3.4 \times 10^{12} \text{ N·mm}^4$$

$$(GA)_{eff,zy} = 1.4 \times 10^7\, N$$

$$\Delta_L = \frac{5wl^4}{384(EI)_{eff,y}} + \frac{\kappa w l^2}{8(GA)_{eff,zy}} = 3.3\text{mm}$$

3.10.6 Effective Flatwise Bending Stiffness $(EI)_{eff}$ and Bending Strength Using the Gamma Method

As mentioned, the Gamma method (Mechanically Jointed Beams Theory) can also be used to calculate the effective bending stiffness and resistance of CLT panels. This method can also be used effectively in cases when there are unbalanced CLT cross-sections. More information about this method is provided in Appendix 3A.1 of this Chapter. For example, if we use the same CLT panel as used in Sections 3.10.1 and 3.10.2, and assuming the span of the CLT panel is 6 m, the connection efficiency factor for layer 1, 3, and 5 can be calculated as:

$$\gamma_1 = \frac{1}{1 + (\pi^2 \frac{E_1 \cdot A_1}{l^2} \frac{t_2}{G_{y2} \cdot b_y})} = 0.945$$

$$\gamma_3 = 1$$

$$\gamma_5 = \frac{1}{1 + (\pi^2 \frac{E_5 \cdot A_5}{l^2} \frac{t_4}{G_{y4} \cdot b_y})} = 0.945$$

where A_i represents the cross-section area of the ith layer in the CLT panel. The effective bending stiffness in the major direction *$(EI)_{eff}$* can be calculated as:

$$(EI)_{eff,y} = \sum_{i=1}^{n} \left(E_{yi} \cdot I_{yi} + \gamma_i \cdot E_{yi} \cdot A_i \cdot z_i^2 \right) = 3.2 \times 10^{12} \text{ N·mm}^4$$

We can find that:

$$I_{eff,y} = \frac{(EI)_{eff,y}}{E_1} = 2.7 \times 10^8 \text{ mm}^4$$

Since the modulus of elasticity of all longitudinal layers is equal, the bending resistance $M_{r,y}$ can be calculated as:

$$M_{r,y} = \phi \cdot F_b \cdot \frac{I_{eff,y}}{\gamma_1 z_1 + 0.5 t_1} = 8.9 \times 10^7 \text{ N·mm}$$

As can be seen, the bending resistance in this case is 15.6% higher when compared to the result obtained using the shear analogy method shown in Section 3.10.1. This was due to the introduction of the strength modification factor, $K_{rb,y}$= 0.85, in ANSI/APA PRG 320 and CSA O86 when determining the bending strength of CLT in the major strength axis.

3.11 CONCLUDING REMARKS

CLT panels have been used as structural products in Canada for less than 10 years. In this relatively short amount of time, significant research and codification efforts have taken place. Several editions of the North American Standard for Performance Rated CLT, ANSI/APA PRG 320, have been published, the latest one in 2018. The use of CLT as a structural product has been implemented in CSA O86 along with design approaches for CLT elements used in floor, roof, and wall applications.

This Chapter of the CLT Handbook provides guidance on the design of CLT panels subjected to flatwise bending and shear loads, out-of-plane bearing compression loads, in-plane axial loads, or a combination of axial and out-of-plane bending loads; it also presents design information for CLT used as beams or lintels. The design guidelines provided in this Chapter follow the CSA O86 design approach where available.

Appendix A of this Chapter provides information on some of the available analytical models for determining the stiffness and strength properties of CLT panels such as the γ (Gamma) method, the k-method, and the Kreuzinger shear analogy method. This information is important for designers in cases when CLT panel layout is different than what is covered in the PRG 320 and CSA O86 standards, as these analytical procedures can be used to determine the panel properties.

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APPENDIX A

Analytical Methods for CLT Floor Panels

Although CSA O86 already contains the equations for determining the stiffness of CLT panels loaded perpendicular to the face of the panel, giving a short background on the available analytical models is important for designers in cases when the CLT panel layup is outside the scope of the PRG 320 and CSA O86 standards. In such cases, the analytical procedures presented in this Appendix can be used to determine the stiffness properties of the panels.

Over the last two decades, various methods have been adopted in Europe for the determination of the strength and stiffness properties of CLT. Some of these methods are experimental, while others are analytical. Some involve a combination of both empirical and analytical approaches based on model testing. An analytical approach, once confirmed by 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.

Four analytical approaches are most commonly used for determining the bending properties of CLT floor systems. The first one is based on the mechanically jointed beams theory, available in Annex B of Eurocode 5 (EN, 2009). In this method, often referred to as the “Gamma method”, 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 completely glued members, and $\gamma=0$ no connection at all. The second method (often referred to as the k-method) was developed by Blass and Fellmoser (2004) and applies the “Composite Theory” to predict flexural properties of CLT. This method, like the “Gamma method”, does not account for shear deformation in the longitudinal layers. The third method called “Shear Analogy” or “Kreuzinger Method” (Kreuzinger, 1999) 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 is accurate and adequate for the prediction of stiffness properties of CLT panels, and as such, has been used in determining the stiffness properties of the CLT panels in the ANSI/APA PRG 320 (ANSI/APA, 2018) Standard for Performance Rated CLT and in the 2016 supplement of the CSA O86 (CSA, 2016), the Canadian Standard for Engineering Design in Wood. The fourth method is the Timoshenko beam theory. This method is an extension of the classic Euler-Bernoulli beam theory and includes shear deformations and rotational bending effects in developing the basic equations, making it suitable for predicting the behaviour of thick beams and beams/plates sandwich composite, such as CLT.

This appendix describes the first three design methods mentioned above for CLT floor systems. Information on the Timoshenko beam method is available in textbooks that cover this topic and is not included here. For simplicity, the notation for the methods presented here is the same as that used in the original literature and may differ from the notation implemented in CSA O86.

3A.1 MECHANICALLY JOINTED BEAMS THEORY (GAMMA METHOD)

The Mechanically Jointed Beams Theory is included in Annex B of Eurocode 5 (EN, 2009). 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 beam. 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 $(EI)_{eff}$, which is dependent on the section properties of the beams and the connection efficiency factor γ. The γ factor depends on the slip characteristics of the fasteners (also called 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 to the theory were needed 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 [3A.1]

$$\frac{s}{K_i} = \frac{\bar{h}_i}{G_R \cdot b} \qquad [3A.1]$$

where:

G_R = shear modulus perpendicular to the grain (rolling shear modulus)

$\bar{h}_i$ = thickness of the board layers in the 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.

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 used in the calculations be equal to 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. 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 to be equal to 0.8 l, in the calculations. 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 or spreadsheet 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 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 can also be obtained from the CLT manufacturer.

The equations and examples of calculation of the effective bending stiffness $(EI)_{eff}$ of CLT panels with five and seven layers are given in Section 3.3 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.

3A.1.1 Bending Stiffness and Strength

The bending strength of the slab is usually defined in relation to the effective section modulus S_{eff} of the CLT element. The bending strength shall then be calculated from test results and using the effective section modulus. The expression for the effective section modulus is shown in Equation [3A.2]:

$$S_{eff} = \frac{2 \times I_{eff}}{h_{tot}} = \frac{I_{eff}}{0.5 \times h_{tot}} \qquad [3A.2]$$

where:

S_{eff} = effective section modulus

I_{eff} = effective moment of inertia (see Figure 6 and Section 3.5)

h_{tot} = total depth of the panel

$$(EI)_{eff} = \sum_{i=1}^{n} (E_i I_i + \gamma_i E_i A_i a_i^2) \qquad [3A.3]$$

where the coefficient "gamma" for the different layers can be calculated as:

$$\gamma_i = \frac{1}{1 + \frac{\pi^2 E_i A_i}{l^2} \frac{s}{K_i}} \qquad [3A.4]$$

The coefficient γ can vary from $0 < \gamma \leq 1$, with $\gamma = 1$ for a rigid connection and $\gamma = 0$ for no connection at all. In real applications, γ may typically vary from 0.85 to 0.9.

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:

$$\sigma_{\max} = \sigma_{global} + \sigma_{local} \qquad [3A.5]$$

where σ_{local} is the stress in the outside layer as a consequence of the 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 Equations [3A.6] and [3A.7].

$$\sigma_{global} = \frac{\gamma_1 E_1 a_1 M}{(EI)_{eff}} \qquad [3A.6]$$

$$\sigma_{local} = \frac{0.5 E_1 h_1 M}{(EI)_{eff}} \qquad [3A.7]$$

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 Figure 3A.1). Keeping Equations [3A.6] and [3A.7] in mind, the maximum bending stress can be expressed as:

$$\sigma_{\max} = \frac{\gamma_1 E_1 a_1 M}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M}{(EI)_{eff}} \qquad [3A.8]$$

or in other words:

$$\sigma_{\max} = \frac{M E_1}{(EI)_{eff}} \cdot (\gamma_1 a_1 + 0.5 h_1) \qquad [3A.9]$$

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:

$$\sigma_{\max} = \frac{M}{I_{eff}} \cdot (\gamma_1 a_1 + 0.5 h_1) \qquad [3A.10]$$

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 included to find the total bending stress in any layer (see Equation B.7 from Section B.3 in Eurocode 5).

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

$$\sigma_{\max} = \sigma_{global} + \sigma_{local} \leq \phi \cdot F_b \quad [3A.11]$$

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

$$M_r = \phi \cdot F_b \cdot \frac{I_{eff}}{(\gamma_1 a_1 + 0.5 h_1)} \quad [3A.12]$$

Equation [3A.11] is valid when the modulus of elasticity of all longitudinal layers is equal.

3A.1.2 Shear Strength

Experimental methods are normally used for assessing the shear strength of a structural glued product. 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:

$$\tau = \frac{1.5 \times V}{A_{gross}} \quad [3A.13]$$

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

$$\tau = \frac{V \cdot (EQ)}{(EI)_{eff} \cdot b} \quad [3A.14]$$

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 (Figure A1), the static moment of area, *Q*, for that part of the section above the centroid axis, can be calculated as:

$$(EQ) = \gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 + E'_1 \cdot A'_1 \cdot a'_1 + \gamma_2 \cdot E_2 \cdot \frac{A_2}{2} \cdot \frac{h_2}{4} \quad [3A.15]$$

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

$$\tau \leq \phi \cdot F_v \quad [3A.16]$$

Keeping in mind Equations [3A.14] to [3A.16], the factored longitudinal shear resistance, $V_{rL,}$ can be expressed in terms of the specified shear strength, F_v, as:

$$V_{rL} = \frac{\phi \cdot F_v \cdot (EI_{eff}) \cdot b}{\gamma_1 \cdot E_1 \cdot A_1 \cdot a_1 + E'_1 \cdot A'_1 \cdot a'_1 + \gamma_2 \cdot E_2 \cdot \frac{A_2}{2} \cdot \frac{h_2}{4}} \quad [3A.17]$$

In a similar way, with the appropriate modifications, 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 by the rolling shear strength F_{vR}.

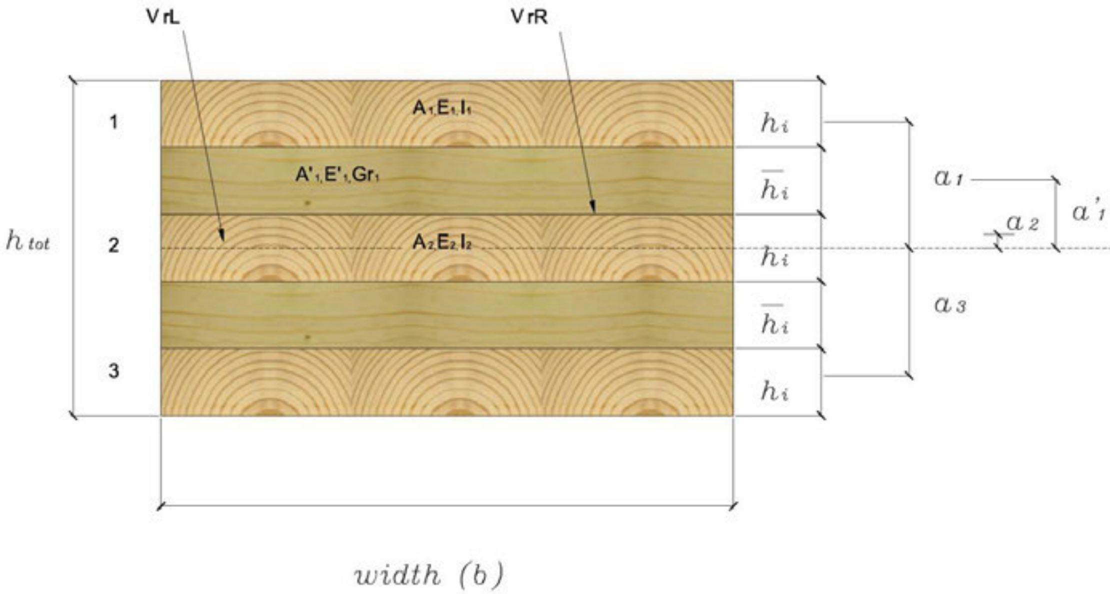


Figure 3A.1 Cross-section of CLT panel with five layers

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:

$$(EQ) = \gamma_1 E_1 A_1 (a_1 - \frac{h_2}{2}) + E'_1\, A'_1\, (a'_1 - \frac{h_2}{2}) \quad [3A.18]$$

The factored rolling shear resistance, V_{rR}, can be expressed in terms of the specified rolling shear strength, F_{vR}, according to Equation [3A.19] below:

$$V_{rR} = \frac{\phi \cdot F_{vR} \cdot (EI_{eff}) \cdot b}{\gamma_1 E_1 A_1 (a_1 - \frac{h_2}{2}) + E'_1\, A'_1\, (a'_1 - \frac{h_2}{2})} \quad [3A.19]$$

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 [3A.20] below:

$$V_r = \min\ (V_{rL}\ or\ V_{rR}) \quad [3A.20]$$

3A.2 COMPOSITE THEORY – K METHOD

This design method is well-known in the plywood industry. In the original version of this method, the plies of the plywood panel stressed perpendicular to the grain is 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 i.e. the layers loaded parallel to the grain and the cross layers loaded perpendicular to the grain. The stiffness of the 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. $l/h \geq 30$);
- Composition factors are determined for certain loading configurations (see Table A1).

Table A1 provides the equations to evaluate the composition factors k_i for certain configurations of loading with respect to the panel orientation. For instance, the factor k_1 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 3A.2 gives the effective values of strength and stiffness for solid wood panels with cross layers (Blass and Fellmoser, 2004).

Table 3A.1 Composition factors "k" for solid wood panels with cross layers (Blass, 2004)

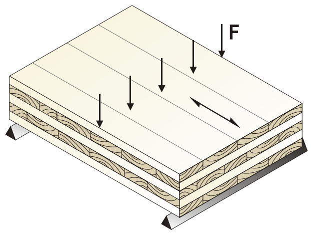
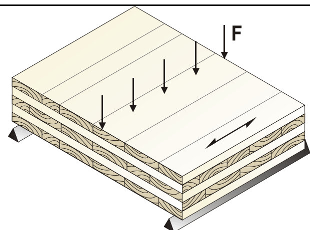
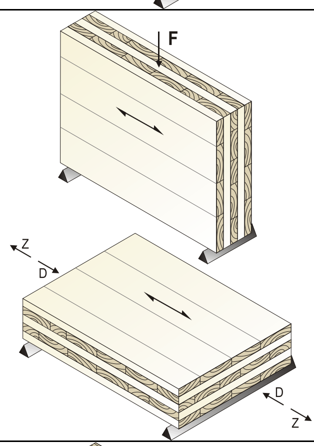
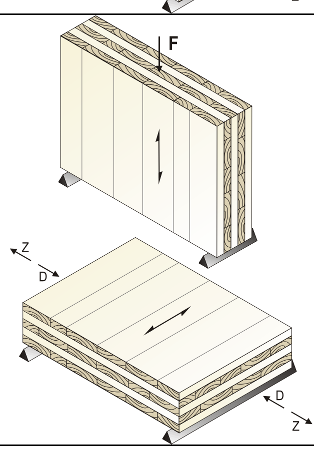
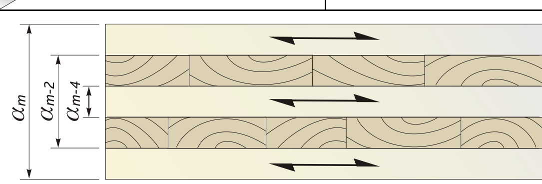
Load Configuration	k_i
	$k_1 = 1 - \left(1 - \frac{E_{90}}{E_0}\right) \cdot \frac{a_{m-2}^3 - a_{m-4}^3 + \ldots \pm a_1^3}{a_m^3}$
	$k_2 = \frac{E_{90}}{E_0} + \left(1 - \frac{E_{90}}{E_0}\right) \cdot \frac{a_{m-2}^3 - a_{m-4}^3 + \ldots \pm a_1^3}{a_m^3}$
	$k_3 = 1 - \left(1 - \frac{E_{90}}{E_0}\right)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

Table 3A.2 Effective values of strength and stiffness for solid wood panels with cross layers (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	$f_{c,90,eff} = f_{c,0} \cdot k_4$	$E_{c,90,eff} = E_0 \cdot k_4$

3A.2.1 Bending Strength and Stiffness

The maximum bending stress may be expressed as:

$$\sigma_{\max} = \frac{M}{S} \qquad [3A.21]$$

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

$$\sigma_{\max} \le \phi \cdot F_{b,eff} \qquad [3A.22]$$

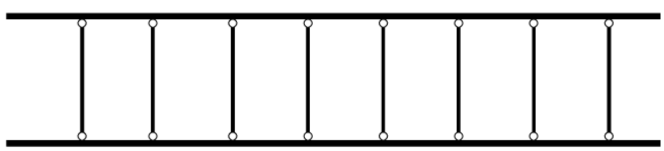
where $F_{b,eff}$ is the effective bending strength value $f_{b,0,eff}$ obtained from Tables 3A.1 and 3A.2.

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

$$M_r = \phi \cdot F_{b,eff} \cdot S_{gross} \qquad [3A.23]$$

3A.3 SHEAR ANALOGY METHOD OR KREUZINGER METHOD

This calculation method is the most precise design method for CLT (Blass and Fellmoser, 2004). 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 3A.2).



Beam A (bending stiffness *$(EI)_A$* = B_A and shear stiffness *$(GA)_A$ = S_A~ ∞)*

Web members with infinite axial rigidity

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

Figure 3A.2 Beam differentiation using the shear analogy method

3A.3.1 Bending Stiffness

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 [3A.24]

$$B_A = \sum_{i=1}^{n} E_i \cdot I_i = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} \qquad [3A.24]$$

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):

$$B_B = \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2 \qquad [3A.25]$$

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:

$$\frac{1}{S_B} = \frac{1}{a^2} \cdot \left[\sum_{i=1}^{n-1} \frac{1}{k_i} + \frac{h_1}{2 \cdot G_1 \cdot b_1} + \sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i} + \frac{h_n}{2 \cdot G_n \cdot b_n} \right] \qquad [3A.26]$$

where:

$$k_i = \frac{K_i}{s_i} \qquad [3A.27]$$

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 it is suggested that $E_{90} = E_0/30$ be used for the cross layers. Also, in the same equations, the shear modulus for the longitudinal layers 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 Equations [3A.28] and [3A.29] respectively.

$$M_{A,i} = \frac{E_i \cdot I_i}{B_A} \cdot M_A \qquad [3A.28]$$

$$V_{A,i} = \frac{E_i \cdot I_i}{B_A} \cdot V_A \qquad [3A.29]$$

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

The bending stresses $\sigma_{A,i}$ and shear stresses $\tau_{A,i}$ of each individual layer of beam A can be obtained using Equations [3A.30] and [3A.31] respectively:

$$\sigma_{A,i} = \pm \frac{M_{A,i}}{I_i} \cdot \frac{h_i}{2} \qquad [3A.30]$$

$$\tau_{A,i} = \frac{E_i \cdot I_i}{B_A} \cdot 1.5 \cdot \frac{V_A}{b \cdot h_i} \qquad [3A.31]$$

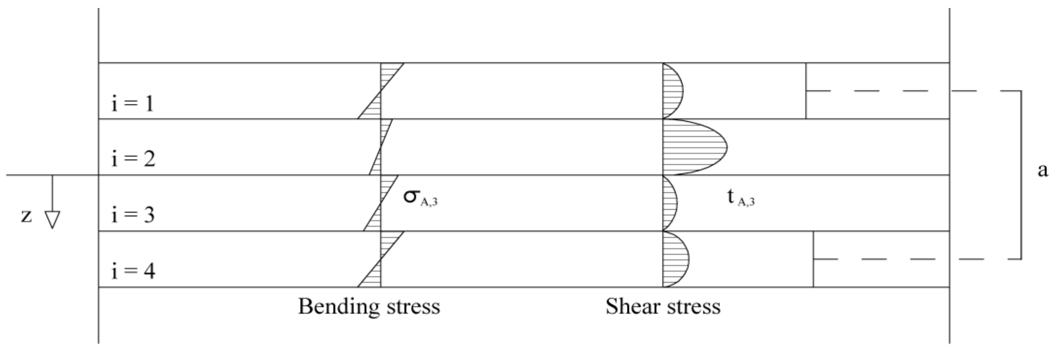


Figure 3A.3 Bending and shear stresses in beam A using the shear analogy method (Kreuzinger, 1995)

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 Equations [3A.32], [3A.33]. and [3A.34] respectively:

$$N_{B,i} = \frac{E_i \cdot A_i \cdot z_i}{B_B} \cdot M_B \qquad [3A.32]$$

$$\sigma_{B,i} = \frac{N_{B,i}}{b_i \cdot h_i} = \frac{E_i \cdot z_i}{B_B} \cdot M_B \qquad [3A.33]$$

$$t_{Bi,i+1} = \frac{V_B}{B_B} \cdot \sum_{j=i+1}^{n} E_j \cdot A_j \cdot z_j \qquad [3A.34]$$

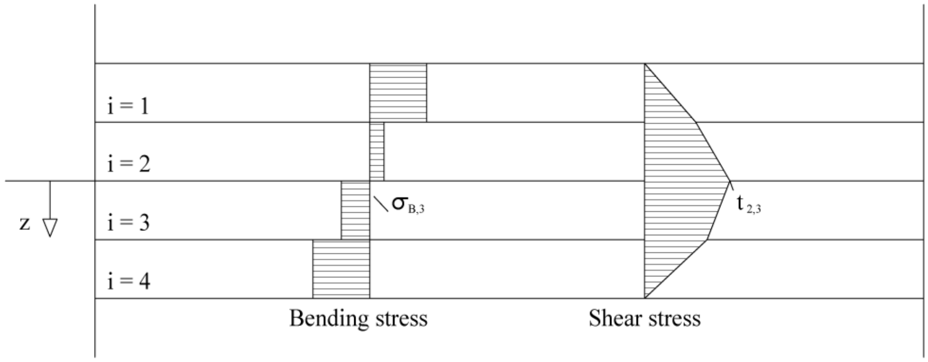


Figure 3A.4 Normal and shear stresses in beam B using the shear analogy method (Kreuzinger, 1995)

The final stress distribution obtained from the superposition of the results from beams A and B is shown in Figure 3A.5. It should be noted that the shear distribution in Figure 3A.5 includes the influence of the connector devices that will not be existent for a CLT panel.

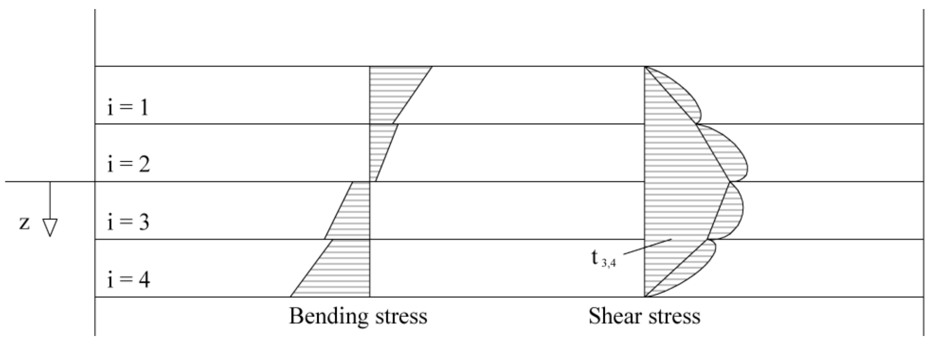


Figure 3A.5 Final stress distribution obtained from the superposition of the results from beams A and B (Kreuzinger, 1995)

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 the sum of the contribution due to bending and to shear:

$$u_{\max} = \frac{5}{384} \cdot \frac{qL^4}{(EI)_{eff}} + \frac{1}{8} \cdot \frac{qL^2\kappa}{(GA)_{eff}} \qquad [3A.35]$$

or in other terms:

$$u_{\max} = \frac{5}{384} \cdot \frac{qL^4}{(EI)_{eff}} \cdot \left(1 + \frac{48(EI)_{eff}\,\kappa}{5(GA)_{eff}\,L^2}\right) \qquad [3A.36]$$

which can be expressed as:

$$u_{\max} = \frac{5}{384} \cdot \frac{qL^4}{(EI)_{eff}} (\alpha + \beta) \quad [3A.37]$$

where α = *1.0* and β can be expressed according to Equation [3A.38] below, where κ (kappa) is the shear coefficient form factor and is equal to 1.0 (please refer to Chapter 6 for details).

$$\beta = \frac{48(EI)_{eff}\,\kappa}{5(GA)_{eff}\,L^2} \quad [3A.38]$$

The effective bending stiffness can be obtained using Equation [3A.39]:

$$(EI)_{eff} = B_A + B_B = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2 \quad [3A.39]$$

The effective shear stiffness can be obtained using Equation [3A.40]:

$$(GA)_{eff} = \frac{a^2}{\left[\left(\frac{h_1}{2 \cdot G_1 \cdot b}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i}\right) + \left(\frac{h_n}{2 \cdot G_n \cdot b}\right)\right]} \quad [3A.40]$$

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:

$$u_{\max} = \frac{1}{48} \cdot \frac{PL^3}{(EI)_{eff}} + \frac{1}{4} \cdot \frac{PL}{(GA)_{eff} / \kappa} = \frac{1}{48} \cdot \frac{PL^3}{(EI)_{eff}} \left(1 + \frac{12 \cdot (EI)_{eff}\,\kappa}{(GA)_{eff}\,L^2}\right) \quad [3A.41]$$

which can be expressed as:

$$u_{\max} = \frac{1}{48} \cdot \frac{PL^3}{(EI)_{eff}} (\alpha + \beta) \quad [3A.42]$$

where α = 1.0 and β can be expressed according to Equation [3A.43] below, where κ (kappa) is the shear coefficient form factor and is equal to 1.0 (please refer to Chapter 6 for details).

$$\beta = \frac{12(EI)_{eff}\,\kappa}{(GA)_{eff}\,L^2} \quad [3A.43]$$

3A.3.2 BENDING STRENGTH

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.

The bending stress σ may be expressed as:

$$\sigma = M \cdot y \cdot \frac{(E_1)}{(EI)_{eff}} \tag{3A.44}$$

The maximum stress will occur for $y = \frac{h_{tot}}{2}$, so Equation [3A.44] can be expressed as:

$$\sigma_{\max} = M \cdot 0.5 h_{tot} \cdot \frac{(E_1)}{(EI)_{eff}} \tag{3A.45}$$

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

$$\sigma_{\max} \leq \phi \cdot F_b \tag{3A.46}$$

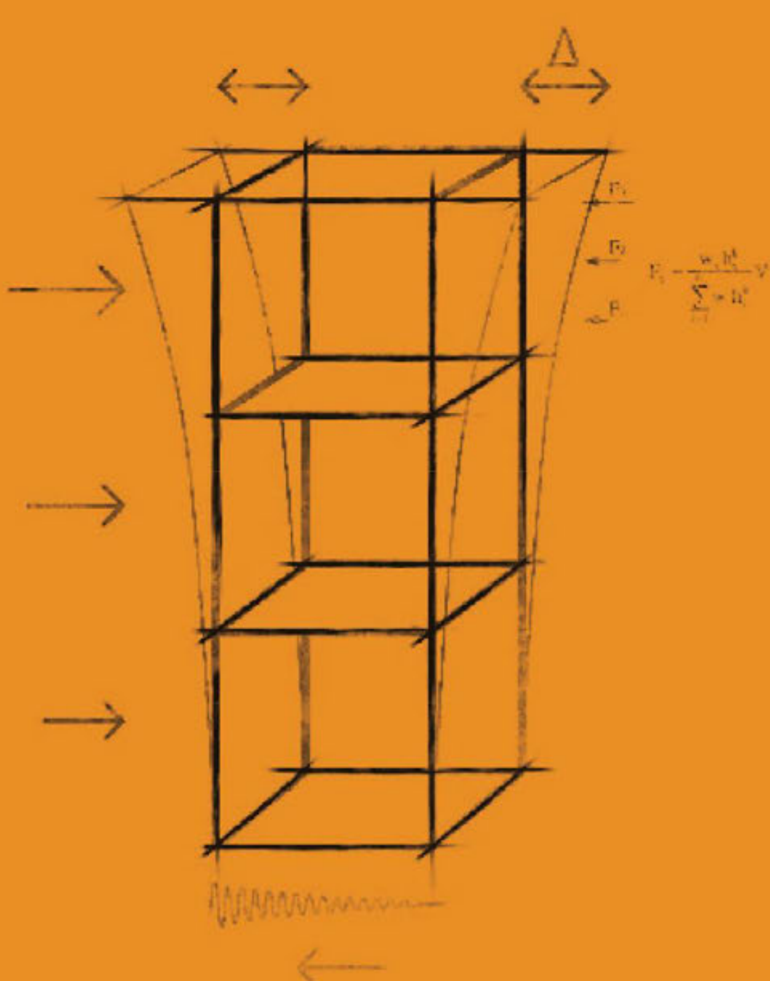
and determine the factored moment bending resistance M_r in terms of the specified bending strength F_b as:

$$M_r = \phi \cdot F_b \cdot \frac{(EI)_{eff}}{E_1} \cdot \frac{2}{h_{tot}} \tag{3A.47}$$

where E_1 is the modulus of elasticity of the outer longitudinal layer in tension and $(EI)_{eff}$ is determined according to Sections 3A.1 to 3A.3.

When the modulus of elasticity of all longitudinal layers is equal, then Equation [3A.47] can be expressed as:

$$M_r = \phi \cdot F_b \cdot \frac{2I_{eff}}{h_{tot}} \tag{3A.48}$$



FPInnovations

CHAPTER 4

Lateral design of cross-laminated timber buildings

Lead Authors
Marjan Popovski
Erol Karacabeyli

Reviewers
John van de Lindt
Robert Jonkman
David Moses
Chun Ni

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ABSTRACT

This Chapter covers the performance and design of CLT structures under lateral loads such as earthquakes or winds. Since the release of the 2011 Edition of the Canadian CLT Handbook, lateral design provisions for CLT have been incorporated in the CSA Standard O86-14 Update 1 (CSA, 2016). The design recommendations presented in this Chapter are based on the CSA Standard O86-14 Update 1 (CSA, 2016), CWC Wood Design Manual (CWC, 2017), and the general requirements of the National Building Code of Canada (NBCC, 2015). Where justified by the state-of-the-art research, this Chapter also includes practices that have been implemented in the latest Edition of CSA Standard O86 (2019).

Since most of the research conducted on this topic around the world to date is related to platform-type CLT buildings, the design recommendations that are provided in this Chapter are mostly related to that type of structural system.

4.1 INTRODUCTION

Structures made of cross-laminated timber (CLT) are well suited for use in a wide variety of structural applications, from low-rise to high-rise residential and non-residential buildings. A number of tall wood buildings as high as 24 storeys have already been built around the world. In structural applications, CLT can be used for wall and floor panels in either platform- or balloon-type applications. The typical platform-type CLT building is robust in resisting gravity and lateral loads due to the large number of walls present in both orthogonal directions. While the design of CLT buildings under gravity loads is relatively straightforward, the behaviour of CLT structures under lateral loads generated by high winds or earthquakes is more complex.

Until recently, no design provisions for CLT structures under seismic loads were available in any national or international code or material standard. In Canada, design provisions for CLT shear walls and diaphragms as a part of the CLT-based lateral load resisting system (LLRS) were introduced in Clause 11.9 of the Canadian Standards Association (CSA) Standard O86-14 Update 1 (CSA, 2016). This standard, its commentary, and design tables are included in the Canadian Wood Council Wood Design Manual (CWC, 2017). These provisions are generally used as the basis for the suggested design concepts presented in this Chapter. Where justified by state-of-the-art research, this Chapter also includes practices that are expected to be implemented in future editions of codes and standards. In fact, the 2019 Edition of the CSA O86 standard (CSA, 2019) incorporates those practices as lateral design provisions for CLT.

Section 4.2 of this Chapter briefly introduces CLT as a structural system and provides a summary of the behaviour of CLT structures under lateral loads. Section 4.3 includes pertinent information related to the design of CLT platform-type systems under lateral loads. Section 4.4 provides a brief introduction to modelling and analysis of CLT structures, while Section 4.5 discusses the research information on the seismic performance of CLT structures. Additional information may be obtained from the list of references provided at the end of this Chapter.

4.2 CLT AS A LATERAL LOAD RESISTING SYSTEM

CLT buildings can be constructed using two types of construction: platform type and balloon type. In the platform-type construction, the floor platform of each storey is used as a base for erecting the CLT walls of the next storey. The height of the CLT walls is therefore equal to the height of the storey. At each storey, the CLT walls transfer the gravity loads from the storey above to the CLT floor panels underneath. Because the gravity loads are cumulative, the maximum height of these buildings is usually limited by the compression perpendicular to the grain resistance of the CLT floor panels on the lowest floor. This type of construction, however, generally has a large number of walls that can also be used to resist seismic loads, thus allowing for high redundancy (Figure 1). Most CLT buildings erected to date in Europe are of this type of construction, as well as a few in Canada and the United States. In addition, most of the research conducted on the seismic performance of CLT structures around the world is based on this type of construction.

Figure 1 Typical platform-type CLT construction

In balloon-type CLT structures, CLT walls are erected along the entire height of the building, and the floor panels are attached (suspended) to the walls at each storey. In such cases, the LLRS of the building consists of a limited number of walls in the floor plan. Several buildings of this type have already been built in Canada, such as the 13-storey Origine building in Quebec City (Figure 2), the 8-storey buildings of the Arbora complex in Montreal, and the 30 m high Wood Innovation and Design Centre in Prince George, B.C. Additional information on these buildings can be found in the case studies available on the www.cwc.ca and www.thinkwood.com websites.

Figure 2 Typical balloon-type CLT construction used in the 13-storey Origine building in Quebec City (courtesy of Nordic Structures)

4.2.1 A summary of the Seismic Behaviour of CLT Structures

The initial research on CLT as a seismic force resisting system (SFRS) on component and structural levels was conducted predominantly in Europe. Although significant research in this field is ongoing in Europe, researchers in Canada, the USA, Japan, and New Zealand have made valuable contributions to clarify the seismic behaviour of CLT structures. A brief description of some of these studies and their main findings are provided in Section 4.5 of this Chapter.

Experimental studies to quantify the performance of connections in CLT, CLT wall assemblies, and entire CLT buildings over the past decade have provided valuable information for the analysis and design of CLT structures. A CLT SFRS develops its ductility primarily through the deformation of connections, while the CLT wall panels themselves remain almost linear elastic, with minimal localized crushing at the corners. Consequently, the behaviour of the connections affects the behaviour of the entire wall and will thus have a large influence on the behaviour of the CLT structure. Where nails or slender screws connect CLT wall panels to steel brackets, a ductile failure mode of the connections was observed.

CLT wall segments with higher aspect ratios (height-to-length ratio) connected by lap joints that use ductile fasteners tend to rock about their corners under lateral loads when the boundary conditions allow. This rocking engages the typical bracket connections at the bottom of the walls in a combined uplift and shear. On the other hand, very long wall segments (with low aspect ratios) tend to have a primary sliding motion that engages the bottom bracket connections primarily in shear. The boundary conditions and vertical loads all contribute to the strength and stiffness of CLT shear walls. Numerical studies have verified that component-based modelling of CLT wall assemblies using connection test data is a reasonable approach, with the panel modelled using elastic shell or block elements. Modelling of the non-linear effects, such as connection behaviour, should be carefully considered as it influences the behaviour of the model significantly. Test data is still critical for the validation of a CLT system model. Non-linear behaviour and any damage in case to CLT wall assemblies are concentrated in the connections.

Full scale tests have shown that CLT buildings are robust against collapse. A complete collapse of a CLT building has not been realized experimentally to this point. It is feasible to design a multi-storey CLT building using an existing force-based design methodology to protect life safety, as long as the appropriate capacity-based seismic design procedures are used. Floor acceleration on the higher levels of a multi-storey CLT building should be mitigated for the comfort of the occupants. A performance-based seismic design philosophy should be adopted for tall CLT buildings in high seismic regions to ensure resilience.

CLT construction can be effectively used as a part of an innovative SFRS that can achieve improved seismic performance and resilience under major earthquakes. Such systems usually include the use of post-tensioning devices, energy dissipators, or other innovative solutions. Significant research in this area has already been conducted in New Zealand, while additional information to fit the design demands in North America is currently being developed in Canada and the United States. Section 4.5 offers additional information.

4.3 DESIGN OF CLT STRUCTURES UNDER LATERAL LOADS

4.3.1 Background

The main sources of lateral loads on buildings are strong winds and earthquakes. These loads are resisted by the LLRS of the building (referred to as the SFRS in buildings designed to resist earthquakes). The main parts of an LLRS (or SFRS) in buildings are the floor and roof diaphragms, which are referred to as the horizontal elements, and walls or frames, which are the vertical elements. The diaphragms usually transfer the lateral loads from each floor level to the vertical systems below.

As mentioned before, in platform-type CLT construction, the floor platform of each storey is used as a base for erecting the CLT walls of the next storey. Consequently, the CLT floors of each storey level (including the roof) act as diaphragms; these collect the lateral loads and transfer them to the shear walls below. Platform-type CLT construction usually involves a large number of CLT walls that can be used to resist lateral loads, thus allowing for relatively high redundancy.

Generally, CLT floor panels are connected to the shear walls below using long self-tapping screws. CLT panels that are part of a diaphragm are usually connected by lap or spline joints and self-tapping screws. CLT shear wall panels that are part of the same wall are connected using lap joints and self-tapping screws, while perpendicular walls are usually connected using long self-tapping screws. CLT shear walls are connected to the floor panels underneath using brackets or other connectors with nails or wood screws. Numerous experimental studies (Dujic et al., 2006a; Popovski et al., 2010; Popovski and Karacabeyli, 2012a; 2012b; Gavric, 2012; Gavric, 2015a; 2015b) on CLT shear walls loaded in-plane have shown that the connections between CLT wall panels and the foundation (or the floor below) and the connections between the vertical joints of adjacent CLT wall segments (in multi-panel walls) are the main contributors to shear wall deformation, while the CLT panels themselves behave mostly as rigid bodies. For that reason, the shear resistance of CLT shear walls is governed by the resistance of the connections between the shear walls and the foundation or floor underneath, and the connections between the individual wall panels. Similarly, the in-plane resistance of the diaphragms is governed by the resistance of the connections between the diaphragms and the supporting structure, and the connections between the individual diaphragm panels. For calculation of the nominal shear resistance of CLT shear walls and diaphragms, designers may use a suitable method of mechanics, assuming that each individual panel acts as a rigid body.

The design recommendations presented in this Section are based on the latest research findings, the CSA Standard O86-14 Update 1 (CSA, 2016), and the information provided in CWC's Wood Design Manual (CWC, 2017). Where justified by state-of-the-art research, this Chapter also includes practices that are expected to be implemented in future editions of codes and standards. In fact, the 2019 Edition of the CSA O86 standard (CSA, 2019) incorporates those practices as lateral design provisions for CLT. Unless otherwise stated, the design guidelines in this Section apply to platform-type CLT buildings only.

4.3.2 Height Limitations

In CSA O86-14 Update 1 (CSA, 2016) as well as in CSA (2019), the height of the CLT SFRS (designed as an "acceptable solution," according to building codes) is limited to 30 m (about 10 storeys) for low and moderate seismic zones. As a conservative measure, the height limit for high seismic zones is 20 m (about 6 storeys), the same as for nailed wood-based shear wall systems. High seismic zones refer to the zones where the product I_E F_a $S_a(0.2)$ is higher than 0.75, in accordance with the National Building Code of Canada (NBCC) (Canadian Commission on Building and Fire Codes [CCBFC], 2015). Buildings exceeding these limits have already been designed and built in Canada as alternative solutions, in accordance with Clause 4.3.2 of CSA O86-14 and Clauses 1.2.1.1.1(b) (Division A) and 4.1.8 (Division B) of the 2015 NBCC.

4.3.3 Capacity Design Principles

The concept of capacity design is of major importance in seismic design. Capacity design is widely used for the seismic design of concrete, steel, and masonry structures, and must be used in the seismic design of tall wood buildings. This design approach is based on the simple understanding of the way a structure sustains large deformations during severe earthquakes. By selecting certain modes of deformation of the SFRS, certain parts of it are chosen to be designed and suitably detailed to yield and dissipate energy under the imposed severe deformations. These critical regions of the SFRS, often called "plastic hinges", or "dissipative zones", act as energy dissipators to control the level of force in the structure. All other structural elements are then designed to be protected against actions that could cause failure, by providing them with strength greater than the one that corresponds to the development of maximum feasible strength in the potential dissipative zones. In other words, non-ductile elements, resisting actions originating from plastic hinges, must be designed with over-strength rather than the code-specified factored strength (resistance) that is used to determine the strengths required of hinge regions. This "capacity" design procedure ensures that the chosen means of energy dissipation can be maintained.

CLT structures that resist seismic loads should be designed using capacity design principles with at least moderately ductile connections for energy dissipation at specified locations, while all other connections should be designed as non-dissipative connections and should have sufficient over-strength. Research results from Pei et al. (2013a; 2013b) and Popovski et al. (2013, 2014) have shown that for CLT structures designed using capacity design principles and certain wall (or wall segment) aspect ratios, the appropriate values for the ductility and over-strength related force modification factors are $R_d \leq 2.0$ and $R_o = 1.5$. These values have also been implemented in CSA O86-14 Update 1 (CSA, 2016) and CSA (2019).

Chapter 13 of this Handbook includes a design example of an 8-storey CLT building that is designed with these factors.

4.3.3.1 Energy Dissipative Connections

All CLT structures need to be designed using the capacity-based design principles regardless of the seismic zone they are located in, the ductility-based force-modification factors used for the seismic design (R_d=2.0, R_o=1.5), or the type of construction (platform- or balloon-type). According to capacity-based design principles, energy-dissipative connections should have sufficient ductility and deformability. Since there is no universally agreed-upon classification of connections based on their ductility at this point (Munoz et al., 2008), a moderately ductile connection in CLT structures should demonstrate that it fulfils all of the following requirements:

- the resistance to be governed by the yielding failure mode;
- to be at least moderately ductile in all non-restricted directions of the CLT panels' kinematic modes; and
- possesses sufficient deformation capacity to allow the CLT panels to develop a predominantly rocking deformation.

While CSA Standard O86-14 Update 1 (CSA, 2016) includes pure rocking and a combination of rocking and sliding as desirable kinematic modes for CLT wall panels, this Handbook and CSA (2019) recommend that rocking be chosen as the predominant deformation mode of panels, with minimal or non-existent sliding (see Section 4.4.1 for the recommended aspect ratios for shear wall segments to promote the rocking mechanism over sliding). This will help minimize the concerns that a sliding mechanism may lead to undesirable seismic response of the building that may result in permanent deformation at the end of the earthquake response.

Connections tested under cyclic loading in accordance with ASTM Standard E2126 (ASTM International, 2011), and having a minimum ductility ratio of 3.0 – determined using the equivalent energy elastic-plastic methodology as defined in the ASTM E2126 – may be considered moderately ductile. For further details about connections with moderate ductility, refer to Section 5.4.5 in Chapter 5 of this Handbook.

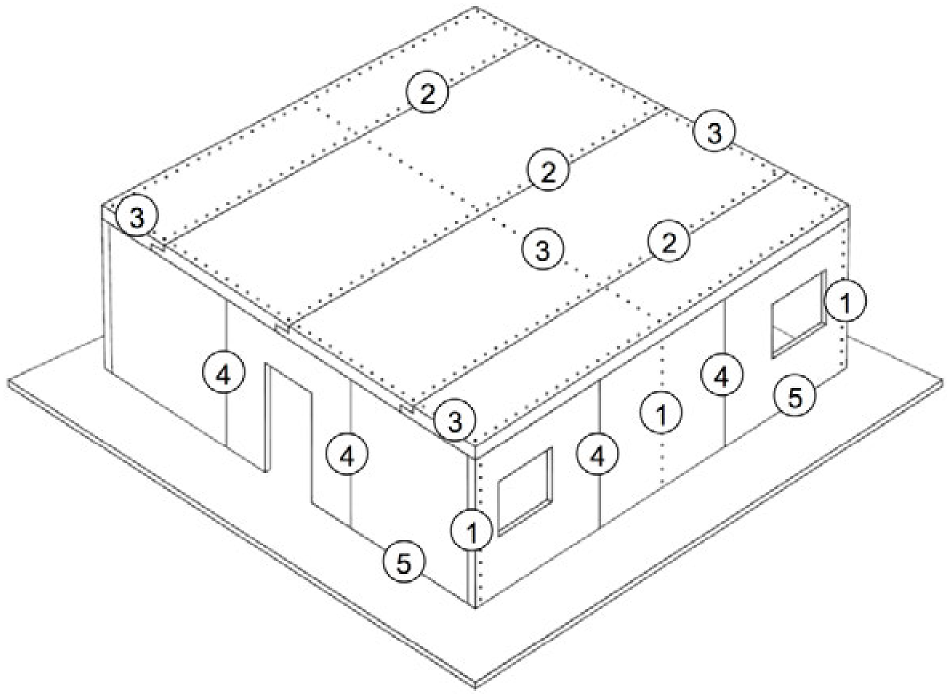


Figure 3 **Details of a typical platform-type CLT construction with different connections and their locations**

Research results (Fragiacomo et al., 2011; Gavric, 2013; Popovski and Gavric, 2014; Follesa, 2015) have shown that all non-linear deformations and energy dissipation for moderately ductile platform-type CLT structures should occur in the following connections:

- wall-to-foundation or wall-to-floor panels below (connections 5 in Figure 3);
- vertical joints between wall panels (connections 4 in Figure 3); and
- discrete hold-downs, when designed for energy dissipation.

While CSA O86-14 Update 1 (CSA, 2016) allows for discontinuous hold-down devices (discrete hold-downs in each storey) to be part of the primary energy-dissipating mechanism of the structure, it is recommended in this Handbook that the discrete hold-downs be designed using connections that fail in yielding mode, but with factored resistance 20% greater than the forces developed in them when the vertical joints between the wall panels (connections 4 in Figure 3) reach their nominal yielding resistance (resistance calculated with ϕ=1.0). This way, the hold-downs can still provide the important function of load transfer and not be subjected to a brittle failure (and even slightly contribute to energy dissipation) during an extreme event. Continuous steel rods along each storey of the building should always be designed to remain linear elastic.

As sliding is proposed to be minimized as a kinematic mode, non-linear deformations should be allowed in the uplift directions only in connections that connect the CLT walls to the floor panels below (connections 5 in Figure 3). The hierarchy of yielding should consist of the vertical joints between the wall panels (connections 4 in Figure 3) yielding first or at the same time as the connections connecting the walls to the foundation or the floor panels underneath working in uplift (connections 5 in Figure 3).

Another form of energy-dissipative connection may be a "fuse" (a connection that usually relies on the yielding of a metallic component) that is specifically designed to dissipate the energy induced by an earthquake. Such fuses can be used as dissipative connections between CLT panels or at the bottom of the walls.

4.3.3.2 Non-Dissipative Connections

According to capacity design principles, non-dissipative connections (i.e., those that are not expected to undergo plastic deformations) should be capacity-protected with sufficient over-strength to remain linear elastic. To ensure adequate over-strength for non-dissipative connections, CSA O86-14 Update 1 (CSA, 2016) requires that the factored resistance of non-dissipative connections be higher than the strength demand that is induced on them when the energy-dissipative connections reach their 95th percentile of ultimate resistance under reversible cyclic loading. Since displacement compatibility is also an important design aspect, non-dissipative connections should also be able to tolerate displacement demands when the energy-dissipative connections reach the target displacement.

Since it is difficult to determine the 95th percentile of ultimate resistance for ductile connections, it is recommended that the 95th percentile resistance be obtained by testing. The 95th percentile can be estimated as the mean ultimate resistance multiplied by a factor of $(1+ k \cdot COV)$, where k is a factor that depends on the number of samples. For example, see the k factor values for a one-sided 95% tolerance limit with 75% confidence for a normal distribution in Table X5.3 of ASTM D5055-16 (ASTM International, 2019).

Among the connections shown in Figure 3, the non-dissipative ones are the following:

- perpendicular walls (connections 1);
- floor panels (connections 2); and
- roofs/floors and the walls below (connections 3).

In accordance with the NBCC, the maximum seismic design force for non-dissipative connections need not exceed the force developed in them when the CLT structure is designed using the seismic force modification factors of $R_d R_o$=1.3.

Where innovative or fuse-type connections are used, the connections between the fuse and the timber components must be designed to remain in the elastic range. The fuse capacity is usually well defined, and its coefficient of variation is small (in the order of 5%), making it easier to determine its 95th percentile of ultimate resistance.

4.3.3.3 CLT Panels and Shear Walls

According to capacity-based design principles, CLT panels that are part of a shear wall or diaphragm should also be capacity-protected to ensure that there will be no failure in the CLT material itself before there is a ductile failure in the energy-dissipative connections. Consequently, the factored in-plane shear resistance of CLT panels that are part of an LFRS should be higher than the seismic forces that develop in the panels when the energy-dissipative connections in the shear walls reach the 95th percentile of their ultimate resistance (i.e., their strength). Such forces need not exceed the forces determined using the seismic force modification factors of R_d R_o=1.3. The factored in-plane resistance of the panels should also be higher than the nominal resistance of the non-dissipative connections to ensure that failure of the panels does not occur. The in-plane shear resistance of CLT panels should be provided by the product manufacturer. Net section effects and openings are to be accounted for when checking for shear resistance.

While CLT walls comprised of long segments (those with a low height-to-length aspect ratio) may be used to resist wind loads, the use of such walls to resist seismic loads is not recommended due to the high likelihood of sliding. Research studies on CLT components and CLT buildings (Dujic et al. 2006a, b; Ceccotti, 2008; Ceccotti et al., 2013; Popovski et al., 2014; Popovski and Gavric, 2015; Tomasi and Smith, 2015) have shown that pure rocking or a combination of rocking and sliding are preferable response mechanisms for seismic performance for platform-type CLT structures. To encourage a predominant rocking response of a building, low aspect-ratio walls that resist seismic loads should be divided into several wall segments that are joined by energy-dissipative connections.

As CLT wall segments act primarily in rocking during the seismic response of a building, the compressive resistance at both ends of each CLT panel should be checked. This is especially important for the lower storeys of the building, which carry the highest vertical loads. Limited information is available on the lengths of walls that should be used in the calculations at both ends of the wall, so designers should use good engineering judgement. Under no circumstances should any buckling be allowed in CLT walls. In all storeys where CLT walls rest on a CLT floor slab, the compression resistance in the CLT floor perpendicular to the grain should also be checked. It should be noted that some localized crushing of the floor slab is inevitable during the rocking motion of CLT wall panels. In such cases, the reference strength against which the stresses perpendicular to the grain are checked can be taken to be at least double the value found in CSA O86-14 Update 1 (CSA, 2016).

4.3.4 Other Design Aspects

4.3.4.1 CLT Panels Aspect Ratio

CSA Standard O86-14 Update 1 (CSA, 2016) requires that in order to apply the ductility and over-strength modification factors $R_d R_o$=3.0, the aspect ratios of the CLT panels in each seismic force–resisting shear wall (or wall segment) must be between 1:1 and 4:1. This is important for encouraging the desirable rocking mechanism of the CLT segments during a seismic event. The rocking mechanism engages the dissipative connections and provides the necessary deformation and ductility properties of the system. To further promote the rocking mechanism over the sliding one, it is recommended in this Handbook that the lower bound on the aspect ratio for the walls and wall segments be increased to 2:1. If the aspect ratio of a wall or wall segment is higher than 4:1, it should not be relied upon to take lateral loads. Connections between wall segments (connections 4 in Figure 3), along with the connections connecting the shear wall panels to the floor underneath (connections 5 in Figure 3) must be designed as yielding elements (dissipative zones) to allow for the CLT panels to rotate. Structures that contain wall segments with an aspect ratio of less than 2:1, or ones that act in sliding only, should be designed using R_d R_o=1.3.

The in-plane shear resistance of CLT panels should be provided by the product manufacturer. Net section effects and openings are to be accounted for when checking for shear stiffness and resistance of the panels. Some guidelines for considering the effect of the openings on the stiffness of CLT wall panels is given in Shahnewaz (2016; 2017). Although wood failure modes in the connections in CLT are rare when fasteners are used on the face of the panels, it is a good engineering practice to use larger fastener spacing in connections to help avoid stress concentration in a small area of the CLT panel. Minimal fastener spacing provisions specific to CLT are provided in Clause 12 of CSA O86-14 Update 1 (CSA, 2016) and in Chapter 5 of this Handbook.

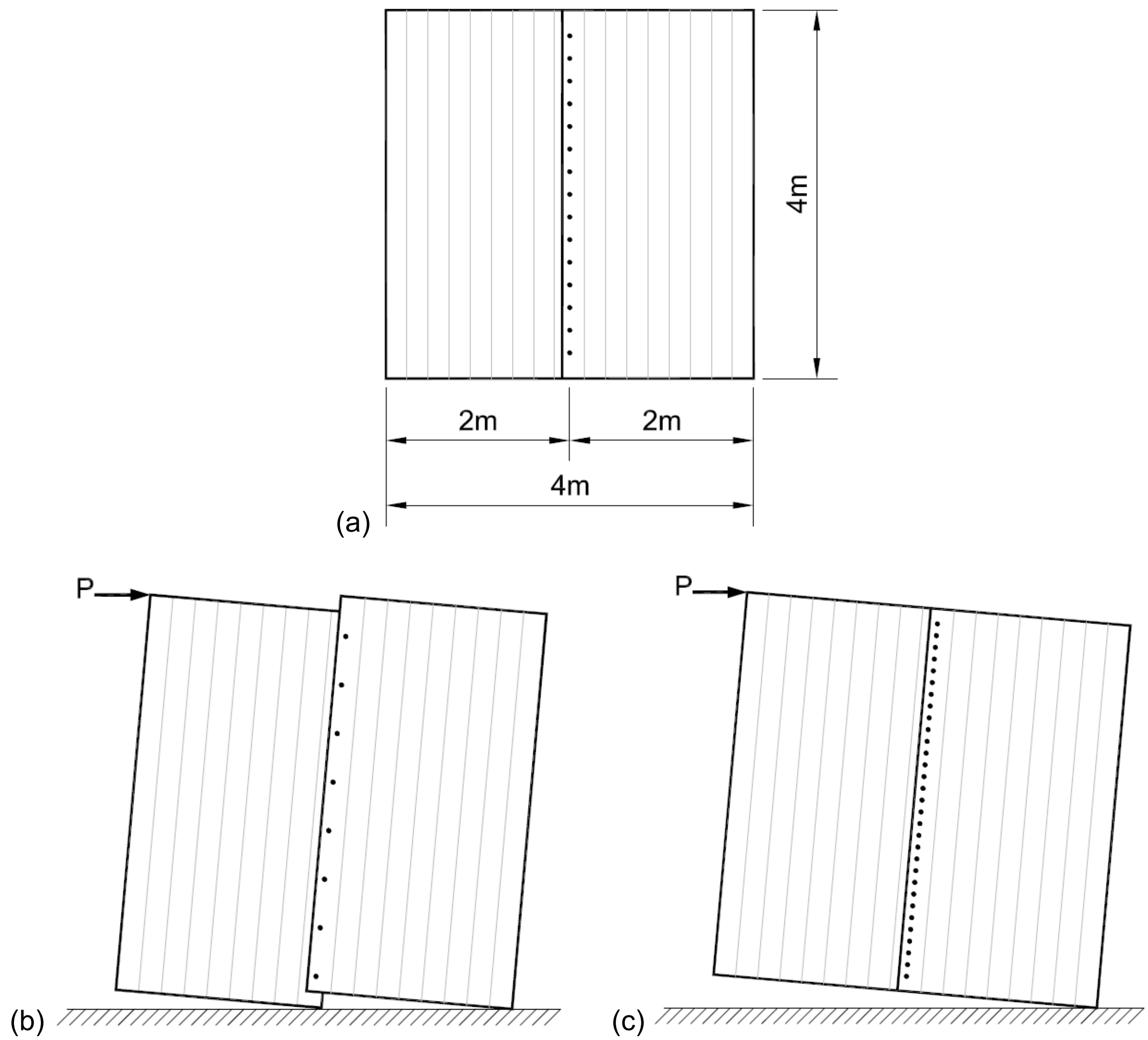


Figure 4 (a) CLT shear wall made of two vertical panels connected to each other with a half-lap joint; (b) shear wall panels acting as two segments; and (c) shear wall panels behaving as one segment (a rigid connection)

A shear wall segment refers to a section of a shear wall with uniform construction that forms a structural unit designed to resist lateral forces parallel to the plane of the wall. Length is defined as the dimension of the segment parallel to the lateral force, and height is the dimension of the segment perpendicular to the lateral force. As opposed to a physical “panel”, a “segment” simply defines the aspect ratio. These concepts are illustrated in Figures 4a to 4c.

When two CLT panels are placed vertically (Figure 4a) and the connection between the two allows them to slide relative to each other, the shear wall should be considered to have two segments (Figure 4b), each with an aspect ratio of 2 (4 m / 2 m). Connections between the two panels should be designed with strength and deformation properties that allow independent rocking of each segment and sliding between the segments. Examples of such connections are half-lapped and spline joints (refer to Chapter 5 for examples) using screws that are adequately spaced to allow for connection deformation.

When two CLT panels are joined with very strong and rigid connections (Figure 4c), the two panels rotate (rock) as one rigid body, and thus the CLT wall should be considered one segment with an aspect ratio of 1 (4 m / 2 panels x 2 m). One example of a rigid connection between panels is a series of closely spaced self-tapping screws placed at an angle to the panel surface and at an angle to the vertical interface between the panels, so that the load is transferred mostly through axial resistance of the self-tapping screws.

Research related to defining a design method for platform-type CLT structures (Flatscher and Schickhofer, 2016) suggests that friction can also be taken into account when determining the lateral resistance of CLT shear walls. However, due to possible vertical accelerations expected during an earthquake, the friction component should not be relied upon in lateral resistance of the shear walls.

4.3.4.2 Brackets and Hold-downs

Angle brackets (connectors connecting the CLT shear wall to the floor) are often used to transfer the horizontal shear forces from CLT shear walls to the floors below. Experimental tests on CLT brackets with small-diameter fasteners, such as nails or wood screws, have shown that angle brackets may contribute significantly in the vertical (uplift) direction as well. Therefore, 2016 O86-14 states that when assuming a rocking and sliding kinematic mode of any wall segment, the designer should consider that the angle brackets resist both shear and uplift forces (shear-uplift interaction of the load-carrying capacity of the brackets). Due to combined (two-direction) loading, there is a reduction in capacity, as described in the interaction Equations [1] and [2], which assume the interaction of the uplift and the shear resistance of the brackets according to a circular domain (Figure 5) (European Organisation for Technical Approvals, 2016). Equations [1] and [2] were suggested for use in connections in CLT shear walls as they represent the actual connection resistance (Popovski and Gavric, 2014; Gavric and Popovski, 2014).

$$\left(\frac{N_{i,x}}{N_{RB}}\right)^2 + \left(\frac{N_{i,y}}{N_{RB}}\right)^2 \leq 1 \qquad [1]$$

or

$$N_{i,x}^2 + N_{i,y}^2 \leq N_{RB}^2 \qquad [2]$$

where:

$N_{i,x}$ = factored horizontal shear force in bracket *i,* $N_{i,y}$ = factored vertical uplift force in bracket *i*, N_{RB} = factored lateral resistance of a bracket in a single direction. The horizontal resistance of all angle brackets in a shear wall segment may be taken as equal. More information on the interaction properties of brackets is provided in Izzi et al. (2018) and Pozza et al. (2018).

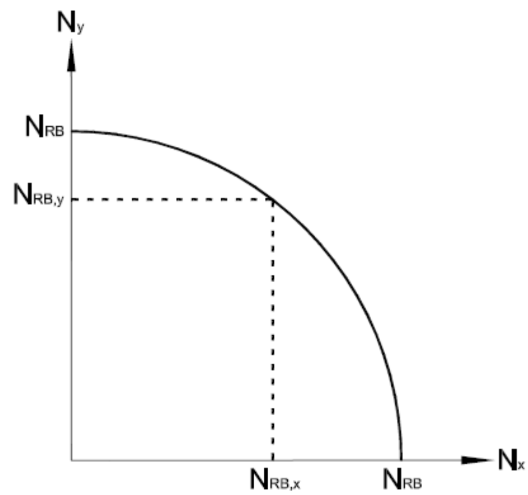


Figure 5 Circular domain for interaction of shear and uplift resistance of bracketed connections

As the current thinking (as well as the latest Edition of CSA Standard O86 (CSA, 2019)) moves toward minimizing the sliding motion and promoting rocking motion through adopting aspect ratios of not less than 2:1, the shear-uplift interaction of the horizontal shear connections may be omitted for shear wall segments with these aspect ratios. To further minimize (or eliminate) sliding, the use of shear keys or wall stops can be considered.

Hold-down connections are required to resist uplift forces and transfer them through a continuous load path to the foundation. If continuous steel rods are used, they are required to be designed to remain elastic at all times and allow the assumed rigid body rotation. Discrete hold-downs (Figure 6) should be connected to the hold-downs on the floor above or below using a steel rod.

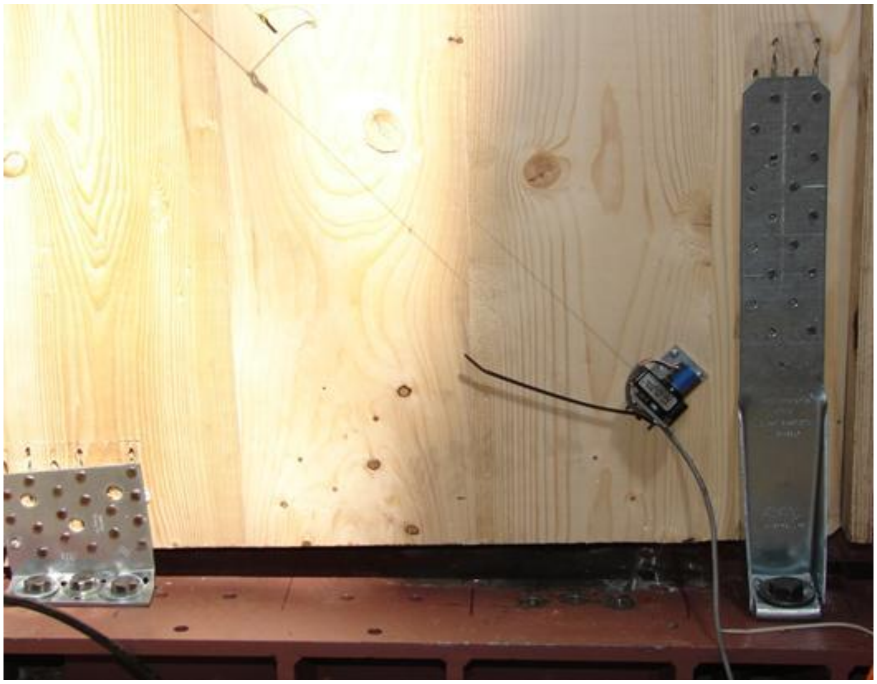


Figure 6 CLT shear wall with discrete hold-down designed for uplift (right) and horizontal shear-resisting angle bracket (left) during testing

Research results have shown that hold-downs have a negligible shear resistance (Gavric, 2012; Liu and Lam, 2016). Consequently, hold-downs may be assumed to resist no horizontal (shear) loads. It should be noted that if the wall has continuous hold-downs, they should be able to deform and allow for the kinematic deformation of wall segments to occur without yielding.

While CSA O86-14 Update 1 (CSA, 2016) allows for hold-down devices (discrete hold-downs in each storey) to be part of the primary energy-dissipating mechanism of the structure, it is recommended in this Handbook (as well as the CSA O86 standard (CSA, 2019)) that discrete hold-downs are not part of that mechanism. Hold-downs, however, should be still designed using connections that fail in yielding mode, but with higher loads, as described in Section 4.3.3.

4.3.4.3 Effect of Perpendicular Walls

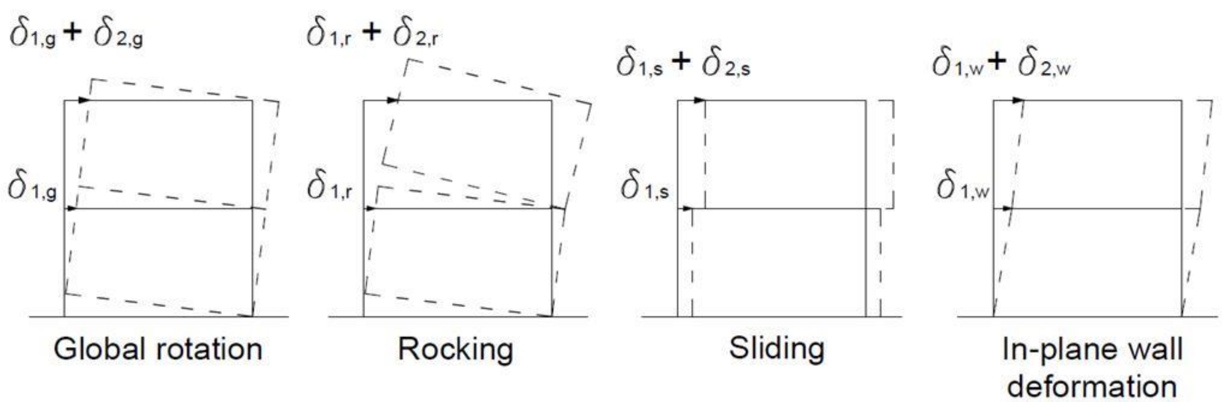
Research on the effect of shear walls perpendicular to the direction of loading on seismic performance of CLT walls is limited. There is some evidence that perpendicular walls may act as hold-downs and have a significant effect on the lateral-load resistance and deformability of CLT structures (Popovski and Gavric, 2014, 2015). However, until these effects are fully understood and quantified, it is suggested that the effects of the walls perpendicular to the direction of loading are not taken into account when calculating the resistance of the CLT structure. Connections between perpendicular walls (connections 1 in Figure 3) should not be considered dissipative zones and should be capacity-protected (non-dissipative). This approach simplifies the seismic design procedure and provides the structure an additional level of robustness, and may reduce damage during strong earthquakes.

4.3.4.4 Irregularities in the SFRS

Limited research has been conducted to date on the effect of irregularities of CLT SFRSs in platform-type construction. Consequently, CSA O86-14 Update 1 (CSA, 2016) prohibits the use of type 4 and 5 irregularities (in-plane discontinuities and out-of-plane offsets), as defined in the NBCC. It is prudent to also avoid irregularity type 6, 8, and 9 irregularities (weak-storey, non-orthogonal systems, and gravity-induced, lateral-demand) until research information is available. For all other irregularities, a dynamic analysis should be carried out in accordance with the NBCC to determine the design loads and seismic performance of the building.

4.3.4.5 Deflections

Like in any other structure, the deflection of CLT structures should be determined and considered during the design process. Computer modelling or established methods of mechanics are to be used for analysis of deflections. Calculations must account for the main sources of shearwall deformations, such as panel rocking, global rotation, sliding (if present), deformation of connections, and in-plane wall deformation (Figure 7). It is recommended in this Handbook that rocking is chosen as a predominant deformation mode of the CLT panels, while sliding is minimized or completely eliminated. The CLT panels that form diaphragms or shear walls may be assumed to act as rigid bodies. Example of a simplified procedure for calculating deflections is presented in Shahnewaz et al. (2018).



Note: Deformation contribution of supports at each storey is not shown

Figure 7 An example of the main deflection components in a two-storey platform-type CLT structure

It is highly important to ensure that the gravity load-carrying system in a building can accommodate the lateral drifts associated with the seismic response of the building. The building drifts could also produce secondary forces and moments in the gravity system, which also have to be taken into account in the design. The larger and stiffer the gravity system is, the more it will interact with the SFRS, especially in taller buildings. The entire structural system should also be designed to sustain the anticipated P-δ effects during the seismic response.

4.3.4.6 CLT as a Gravity System Used With Non-Wood SFRSs

When CLT or other mass timber systems are used in the gravity system only, and the lateral loads are resisted by an SFRS of another material, the gravity system must be able to "go along for the ride" with the SFRS, as per Clause 4.1.8.3 (5) of the NBCC. Unless the building is located where I_E F_a $S_a(0.2)$ is less than or equal to 0.35, as defined in the NBCC (CCBFC, 2015), all CLT structural elements and their connections not considered part of the SFRS should be designed to ensure they behave elastically or have sufficient non-linear capacity to support their gravity loads while undergoing earthquake-induced deformations. In other words, the designer should ensure that the gravity load-carrying system in the building can accommodate the lateral drifts associated with the seismic response of the building.

The building drifts would also produce secondary forces and moments in the gravity system that also have to be taken into account in the design. The larger and stiffer the gravity system is, the more it will interact with the SFRS, especially in taller buildings. The entire structural system should also be designed to sustain the anticipated P-δ effects during the seismic response.

Clauses 4.1.8.3 (6) and (7) of the NBCC should also be taken into consideration when analyzing and designing a building or calculating its period. CLT elements that are not considered part of an SFRS should either be separated from all structural elements of the SFRS, so that no interaction takes place as the building deforms due to earthquake effects, or be made part of the SFRS and satisfy the system design requirements.

According to Clause 4.1.8.3 (7) of the NBCC, the stiffness imparted to the structure from elements that are not part of the SFRS should not be used to resist earthquake deflections, but should be accounted for when:

(a) calculating the period of the structure for determining forces if the added stiffness decreases the fundamental period by more than 15%;

(b) determining the irregularity of the structure, except that the additional stiffness shall not be used to make an irregular SFRS regular or to reduce the effects of torsion; and

(c) designing the SFRS if inclusion of the elements not part of the SFRS in the analysis has an adverse effect on the SFRS. Adverse effects may change the load path and cause some parts of the SFRS to be subject to higher forces and/or deformations than would otherwise be the case.

When CLT is used as a floor diaphragm or roof in systems that use non-wood SFRS, they should be able to act as a diaphragm and should be able to transfer the seismic forces to the non-wood SFRS. Various means of connecting wood-based diaphragms to non-wood-based SFRSs have been around for centuries, and with only slight modifications, they can be used to connect CLT floors to the supports below. These connections should be non-dissipative connections and should be designed to remain elastic under the force and displacement demands that are induced in them when transferring the seismic loads to the SFRS.

4.3.4.7 CLT Diaphragms

Floor and roof diaphragms are important horizontal structural elements in wood buildings to carry vertical and lateral loads. The inertia forces caused by earthquakes or lateral forces from high winds 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 CLT wall systems, which will affect the design. CLT as a structural material has high in-plane rigidity. Consequently, most CLT diaphragms in platform-type residential buildings in Europe and around the world so far have been designed using the rigid diaphragm approach. It should be noted that the connections between CLT floor panels and those between the floor panels and the walls below also influence diaphragm performance. In addition, as the spans increase, this assumption may not be valid. For conservative reasons, it is suggested that CLT buildings with CLT diaphragms be designed using the International Building Code (IBC) analogy (IBC, 2015). The structure should first be designed using the flexible diaphragm assumption and then using the rigid diaphragm approach. The more critical solution should then govern the final design.

Where CLT is used for large floor or roof diaphragms (such as big box stores), or where large openings are present, the CLT diaphragms may fall into the flexible category, and designers would need to take into account the potential flexibility of the entire diaphragm.

CLT panels in the diaphragms should be capacity-protected, as described in Section 4.3.3.3. Shear connections that connect the diaphragms to the walls beneath and the connections between adjacent diaphragm panels shall be non-dissipative and designed in accordance with Section 4.3.3.2.

The main force transfer elements of diaphragms, such as the chords, struts, and parts around any openings, should be capacity-protected to properly collect and transfer forces. They should stay elastic under the force and displacement demands that are induced in them when the energy-dissipative connections that are connecting them to the SFRS reach the 95th percentile of their ultimate resistance or the design target displacement. As per the NBCC, the seismic design force need not exceed the force determined using R_d R_o=1.3.

4.3.4.8 Balloon-type CLT Structures

Although beyond the scope of this Chapter, some basics about use of CLT walls as LLRSs in balloon-type construction are included in this Section. Balloon CLT walls can be used in elevator shafts and stairwells or at other chosen places in the floor plan, and they go along the entire height of the building. Compared to the large number of shear walls present in platform-type CLT structures, the number of shear walls in balloon-type structures is usually lower. Similar to concrete shear walls, the ductility and non-linear deformations demand in balloon-type CLT walls is concentrated at the bottom of the walls. Due to their high aspect ratios, CLT shear walls in balloon-type applications are usually more flexible under lateral loading and may have a significantly higher bending component than walls in platform-type construction. Since CLT panels can be produced only with certain maximum lengths due to production or transportation limits, CLT panels used in balloon walls often need to be connected to reach the entire height of the building. These connections usually need to be capacity-protected in the design to allow for the continuation of the panel properties along the entire height of the wall. Similarly, as CLT panels are produced with widths of up to 4 m, it may be necessary to connect together two CLT panels in their own plane to form one wall of the desired aspect ratio, especially in taller buildings. Other ways of designing balloon-type CLT buildings than those presented here are also possible.

There is virtually no research related to quantifying the force modification factors (R-factors) for balloon-type CLT systems that can be used in the equivalent static seismic design procedure per the NBCC. For that reason, linear or even non-linear dynamic analysis would be the design approach of choice.

For structures consisting of a combination of platform- and balloon-type CLT applications, the designer may decide to use the R_d R_o included in CSA O86 for platform-type construction for the entire structure, provided that:

a) there are adequate connections between both systems to carry the force and deformation demands without failure;

b) the balloon part of the construction carries a small portion of the seismic load (20% or less); and

c) the balloon-type portion of the structure has deformation compatibility with the platform-type part of the structure, with no significant reduction in capacity.

4.4 METHODS OF ANALYSIS

As platform-type CLT structures have a relatively high number of shear walls to resist the lateral loads at each storey in both orthogonal directions (particularly in residential buildings), they are relatively stiff buildings. If the equivalent static force procedure is used for seismic design, the period of the structure may be determined using the NBCC equation for “shear walls and other structures.” For tall wood building applications, additional information related to analysis and design for gravity, earthquake, and wind loads may be found in the Technical Guide for the Design and Construction of Tall Wood Buildings in Canada (Karacabeyli and Lum, 2014). When linear dynamic analysis is used, all connections in the building should be modelled using adequate stiffness that is either determined by testing or provided by the manufacturer. CLT panels can be assumed as plate elements with their own stiffness properties in all directions, as provided in the American National Standards Institute/APA – The Engineered Wood Association (ANSI/APA) PRG 320 manufacturing standard (APA/ANSI, 2018), or as per information provided from the panel manufacturers. Information on quantifying the influence of the openings on the in-plane stiffness of CLT panels is provided in Dujic et al. (2007) and Shahnewaz et al. (2016). Information on dynamic analysis of CLT structures is provided in a number of research papers, among them Rinaldin and Fragiacomo (2016), Sustersic et al. (2016), and Amini et al. (2018).

Chapter 13 of this Handbook includes a design example of an 8-storey CLT building where the path followed in terms of methods of analysis can be found.

4.5 RESEARCH ON SEISMIC BEHAVIOUR OF CLT STRUCTURES

The initial research on CLT as an SFRS on component and structural levels was conducted mainly in Europe. Although a significant amount of research in this field is still conducted in Europe, researchers in Canada, the United States, Japan, New Zealand and some other countries have made valuable contributions to clarifying the seismic behaviour of CLT structures in the last few years. This Section provides snapshot of information on some of the studies conducted in this field and their main findings. Some of the research findings presented in this Chapter will be useful to the designers not only in terms of performance of an SFRS, but also on modelling and analysis procedures. The intention of the discussion provided here is by no means meant to be all-inclusive. Further information can be found in Pei et al. (2014; 2016) and Izzi et al. (2018a).

4.5.1 Research in Europe and Main Findings

The first study to determine the seismic behaviour of 2-D CLT wall panels was conducted at the University of Ljubljana, Slovenia. This project included numerous monotonic and cyclic tests on CLT 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 a constant vertical load combined with either monotonic or cyclic horizontal loads. Wall panels were tested with various boundary conditions that enabled the development of load versus wall deformation relations from cantilever to pure shear wall behaviour. Test results revealed that the load-bearing capacity of CLT shear walls was governed by the connections and the hold-downs in most cases. The influence of boundary conditions, the magnitude of vertical load, and the types of anchoring systems were also investigated (Dujic et al., 2005; 2006a). It was found that boundary conditions have a significant impact on the lateral resistance of CLT shear walls.

Differences in mechanical properties between monotonic and cyclic responses were also studied (Dujic and Zarnic, 2006), as was the influence of openings on the properties of shear walls (Dujic et al., 2006c; 2007). Mathematical formulas describing the relationship between shear strength and stiffness of CLT wall panels with and without openings were developed (Dujic et al., 2008). Results of the parametric study conducted have shown that CLT walls with openings as large as 30% of the wall area did not significantly reduce the shear resistance of the walls. The stiffness of the walls, however, was more significantly affected, with a reduction of up to 50% in the initial stiffness. As a follow-up study, shake table tests were conducted on two single storey box CLT models at the Institute of Earthquake Engineering and Engineering Seismology (IZIIS) in Skopje, Macedonia (Hristovski et al., 2013). Results have determined the dynamic characteristics of the tested specimens and have shown that CLT panels acted almost as rigid bodies during dynamic excitation, with the vertical half-lap joints in multi-segmented CLT walls significantly contributing to the ductility of the system.

One of the most comprehensive studies to quantify seismic behaviour of low- and mid-rise CLT construction was undertaken by the Trees and Timber Institute of the National Research Council of Italy (CNR-IVALSA), in collaboration with the National Research Institute for Earth Science and Disaster Resilience (NIED), Shizuoka University, and the Building Research Institute 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., 2006a), pseudo-dynamic tests on a one-storey 3D specimen in three different layouts (Lauriola and Sandhaas, 2006), shake table tests on a three-storey building (7 m x 7 m in plan and 10 m high) building under different strengths of earthquakes (Ceccotti and Follesa, 2006), and finally a series of full-scale shaking table tests on a seven-storey CLT building conducted at the Hyogo Earthquake Engineering Research Center in Miki, Japan.

Results from quasi-static tests on CLT wall panels confirmed that the behaviour of panelized CLT walls was strongly influenced by the connection properties and by their layout (Ceccotti et al., 2006b). Hysteresis loops were found to have an average equivalent viscous damping of 12%. Similar to the cyclic tests, the pseudo-dynamic tests on single storey 3D CLT specimens showed that CLT as a construction system is very stiff, but relatively good levels of ductility could be obtained (Lauriola and Sandhaas, 2006). It was also found that the initial stiffness of the 3D specimen with asymmetric configuration (openings of 4.0 m on one side and 2.25 m on the other) was not significantly different than that of the symmetric configuration (openings of 2.25 m on both sides). This suggests that although the larger opening on one side slightly reduced the stiffness of the wall panel, the shear deformation of the panel itself was still relatively small compared to the deformation of the connections.

Figure 8 Three-storey CLT house tested at the NIED laboratory in Tsukuba, Japan

Shake table tests on the three-storey house were conducted in the laboratories of the NIED in Tsukuba, Japan (Figure 8). These tests showed that the CLT structure survived 15 strong earthquakes without collapse or severe damage (Ceccotti & Follesa, 2006). The collapse state, defined as a failure of one or more hold-downs, was reached only during the last test that used the Nocera Umbra earthquake record with a peak ground acceleration (PGA) of 1.2g. This study once again highlighted the fact that most of the damage to CLT buildings during earthquakes will be concentrated at the connections, while the CLT wall and floor panels will remain mostly elastic. It also highlighted that significant uplift demands can be expected on the lower floor anchoring system.

The next series of shake table tests from the Sistema Costruttivo Fiemme (SOFIE) project was conducted in October 2007 at the Hyogo Earthquake Engineering Research Center in Miki, Japan. A seven storey CLT building with a floor plan of 13.5 m x 7.5 m and height of 23.5 m was tested (Figure 9). Walls consisted of several 2.5 m long segments connected together with lap joints and screws. The 142 mm thick CLT floors were connected to the walls with screws and steel brackets. The equivalent static force procedure according to Eurocode 8: Design of structures to earthquake resistance (Eurocode 8) (European Committee for Standardization [CEN], 2013) was used to design the building using a q factor of 3.0 and an importance factor of 1.5. During the testing program, the building was subjected to a total of 10 three-directional earthquake ground motions with increasing intensity levels, with the highest one being that of the JMA Kobe record (PGA=0.82g).

Figure 9 Seven-storey CLT house tested at the Hyogo Earthquake Engineering Research Center in Miki, Japan

The structure withstood all tests without any significant damage. The only damage observed was in the hold-down elements at the corners of the lower storeys, where uplifts of over 10 mm were observed, indicating the importance of providing an adequate overturning resistance load path in a multi-storey CLT building. The maximum first-storey drift was 38 mm (1.3%) in the shorter y direction and 29 mm (1%) in the x direction. The total deflection at the top of the building was 175 mm and 287 mm, respectively. The natural frequency of the building was measured between each test and showed a moderate decrease from the initial frequency (17% to 24%). During the strongest earthquake, high accelerations (approximately 3g) were recorded at the top storeys. Thus, the authors suggested adding ductile elements to CLT building design to help improve building performance. After 10 major earthquakes, the building was able to return to its original equilibrium position with no residual displacement. Based on the observations of the building's performance, Ceccotti (2008) suggested that CLT would be a good candidate for the design of high-performance buildings. Such buildings can be designed using the so-called No Damage Design (NDD) philosophy, which can be categorized as a special case of performance-based seismic design (Pei et al., 2014).

Fragiacomo et al. (2011) conducted a study to investigate the appropriate design over-strength factor for CLT connections based on connector test data. The study indicated that the main source of ductility in CLT buildings is from the ductility in the connections, which should be used as the basis for calculating over-strength for all other parts of the building if ductile failure of the system is desired. The authors suggested designing the floor panel connections using over-strength, thus limiting diaphragm damage during earthquakes. Based on a limited number of cyclic loading tests of connectors with nails and screws, an over-strength factor of 1.3 was suggested for the hold-downs and angle brackets of CLT walls. As a part of this study, non-linear static analyses were conducted on a four-storey CLT building using SAP2000. CLT panels were modelled using shell elements, while non-linear springs were used for the connections. The analyses revealed that proper modelling of the properties of the ductile connections is critical to the proper quantification of the building period. While modelling of all connections with rigid links can greatly underestimate the natural period of the building and its displacements during earthquakes, and over-predict its base shear, using only ductile springs to connect the panels may result in over-estimation of the building period. A push-over analysis following the N2 procedure recommended in Eurocode 8 was conducted using the non-linear model and proved that using ductile connectors can increase system ductility and seismic performance.

Following the seven-storey test from the SOFIE project, Dujic et al. (2010) conducted a numerical study to predict the dynamic response of the building. The trial models with SAP2000-non-linear analysis used shell elements for the CLT panels and multi-linear spring elements for connections and hold-downs. The authors reported that the numerical integration did not converge due to the use of non-linear springs, with a descending branch in the envelope curves. A modification was made to use equivalent linear springs to capture the secant stiffness of the connection, with a 15% artificial viscous damping. Using this simplification, the model was able to reproduce the measured inter-storey drifts of the structure with reasonable accuracy.

Sustersic et al. (2016) conducted a parametric study to investigate the effects of friction and vertical load on the dynamic behaviour of panelized CLT structures. Similar to earlier studies, the numerical model was developed in SAP2000 using shell elements for the walls, rigid plates for the diaphragms, and non-linear springs and gap elements for the connections. The modelled building was a 4-storey CLT building with a relatively simple floor plan (6.5 m x 8.5 m) and a storey height of 2.8 m (total building height of 11.2 m). A friction element was used between the panels when friction was considered (with a friction coefficient of 0.4). The models were subjected to earthquake excitation with varying connector and friction parameters. The results showed that both friction and vertical load can have a significant impact on the response of the CLT system as modelled. The study highlighted the modelling uncertainty that could be associated with a CLT system when the friction mechanism within the system is not fully understood. It also indicated that neglecting vertical acceleration during non-linear time history analysis may considerably affect system response.

The Technical University of Graz (Austria) carried out an extensive testing program as part of the Seismic Engineering Research Infrastructures for European Synergies (SERIES) project. The tests analyzed the performance of typical connections and wall systems (Hummel et al., 2013; Flatscher et al., 2015); furthermore, full-scale shake table tests were performed on a three-storey CLT building (Flatscher and Schickhofer, 2015). Compared to the structures tested in the framework of the SOFIE project, the CLT building used in the SERIES project was assembled using long, continuous walls rather than segmented, narrow panels with vertical lap joints. Furthermore, fully threaded screws were primarily used as fasteners, rather than partially threaded screws. Consequently, uncoupled movements of the walls did not occur during the tests, and the inter-storey drifts were significantly smaller compared to those measured before. The SERIES project (Piazza, 2015), which also involved the University of Trento (Italy), the University of Minho (Portugal), and the Laboratório Nacional de Engenharia Civil of Lisbon (Portugal), also provided a comprehensive overview of the seismic behaviour of timber buildings.

Rinaldin et al. (2013) developed a numerical model to estimate the dissipative capacity of a CLT building, focusing on detailed calibration of the non-linear hysteretic and pinching behaviour of the connectors. Analytical models of CLT shear walls with connectors were developed in Abaqus using external subroutines for the connection elements. By comparing the numerical simulation results with the results obtained from panelized wall tests and the single storey building tests conducted previously in Italy, the accuracy of the model to predict the wall/building response and energy dissipation under reverse cyclic tests was confirmed.

Izzi et al. (2018) proposed a numerical model capable of predicting the mechanical behaviour and the failure mechanism of typical wall-to-floor connections for CLT structures. The CLT elements were modelled as 3D solid bodies, while the steel-to-timber joints were simulated as non-linear hysteretic springs. Numerical simulations led to accurate predictions of the mechanical behaviour of the connections and wall elements. The performance of connections when lateral and axial loads are applied simultaneously was also investigated. Results showed that a quadratic interaction relationship exists between shear and axial tension loads.

Pozza et al. (2018) studied the effect of lateral deformation on the axial behaviour of typical hold-down connections in CLT structures. Despite the current assumption that hold-downs do not carry any horizontal (shear) loads, the results of 15 tests have confirmed that stiffness, yielding displacement, and load-carrying capacity of hold-downs are to some extent affected by the axial-shear load interaction. The axial-shear interaction was found to be small, up to 7.5 mm of lateral displacement (when shear brackets are still in the elastic range). At this point, the peak force decrease was less than 10% compared to loads obtained from a uniaxial configuration, while the yielding force reduction was less than 20%. For higher values of lateral displacement, the force decrease was more significant, about 50% for the yielding force and 25% for the peak force. The value of the axial displacement when peak force was reached was not affected by the imposed lateral displacement. The yielding displacement, however, was strongly affected by the lateral displacement, with a 30% reduction for a lateral displacement of 7.5 mm. A significant increase in the value of the elastic axial stiffness was also registered in case of axial-shear coupling.

Initial research to quantify the seismic modification factors for platform-based CLT buildings was also first conducted in Europe. These factors are used in equivalent static seismic design procedures in most national and international building codes around the world, and they account for the capability of the structure to undergo ductile non-linear response and to dissipate energy, while also having adequate over-strength. For example, in the NBCC (CCBFC, 2015), the elastic seismic design load is reduced by two types of R factors: the R_o factor, related to the over-strength of the system, and the R_d factor, related to the ductility of the structure. In the *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7) (American Society of Civil Engineers, 2016) in the United States, there is only one R factor, called the response modification coefficient, and Eurocode 8 (CEN, 2013) also uses only one factor, the q factor, for reduction of the seismic design force. Although every model code should be considered a separate calibrated entity, as the loading code is different in different regions, it is still useful to compare the product of R_d R_o in Canada with the R factor in the United States and with the q factor in Europe for the same seismic hazard probability.

Estimates of appropriate q factors for CLT as a platform-type structural system were made based on the results of incremental non-linear dynamic analyses on verified building models. Ceccotti (2008) has shown that for the analyzed three-storey CLT structure (Figure 8), the q factor was greater than 3.0 for seven of eight earthquakes, and was even greater than 4.0 in two cases, with an average q factor of 3.4. Pozza (2009) showed q factor values approximately 20% lower for similar three-storey buildings with different masses, with q=3.0 still being acceptable. Pozza et al. (2016) investigated the influence of CLT wall configuration, connection arrangement, and presence and type of vertical lap joints in CLT walls on the q factor. The numerical models for the study were developed based on the results from the cyclic tests of full-scale walls, performed at CNR-IVALSA during the SOFIE project (Gavric et al., 2015a). It was found that ductility, displacement capacity, and the q factor varied (from 2.5 to 3.25) among the analyzed wall configurations. It was also found that the q factor increased when the number of vertical joints in the CLT walls was increased.

The effects of the building and wall geometry on the q factor can also be obtained by comparing the available results from shake table tests of two three-storey buildings. The first building had a number of vertical panel-to-panel joints in the walls (Ceccotti, 2008), while the second had full-length walls without vertical wall joints (Flatscher and Schickhofer, 2015). Flatscher and Schickhofer (2015) obtained a lower q factor lower than Ceccotti (2008) (average q=2.8 versus q=3.4) and concluded that this was due to the absence of vertical wall joints. Furthermore, Pozza and Trutalli (2017) conducted a parametric study by means of incremental dynamic analyses to correlate the q factor to some CLT building configurations, such as the density of panel-to-panel joints and building slenderness. Results have shown that a value of q=2 can be assumed for squat-type CLT structures with single monolithic wall panels. This value, however, is highly conservative for slender CLT structures with narrow wall panels, connected with the traditional angle brackets that are able to deform and dissipate seismic energy.

Provisions on the seismic design of CLT structures are expected to be implemented in Eurocode 8 in 2020. Details about these new provisions can be found in Follesa et al. (2015; 2018).

4.5.2 Research in North America

Research on seismic performance of CLT systems was initiated in Canada by Popovski et al. (2010), who conducted a series of 32 quasi-static tests on CLT shear wall specimens with different aspect ratios, openings, multi-panel combinations, and number of storeys. A wide range of connectors (spiral nails, ring nails, screws, and timber rivets) and two types of brackets were used to connect the walls to the foundation. Various types of screws were also used in the lap joints between the panels in multi-panel wall specimens. Results from these tests showed that inter-panel lap joints and metal brackets were the main sources of ductility in CLT walls. CLT walls can have adequate seismic performance when common nails, spiral nails, or screws are used with steel brackets. Use of hold-downs with nails on the ends of the walls improved their seismic performance. Use of diagonally placed screws to connect the CLT walls to the floor below is not recommended in high seismic zones due to less ductile wall behaviour. Use of lap joints in longer walls can be an effective solution not only to reduce 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, but further research in this field was suggested to clarify the use of timber rivets in CLT. These tests provided a good data set that jump-started the follow-up seismic research on CLT in North America.

Using FPInnovations' set of test data on CLT walls, Schneider et al. (2012) applied an energy-based damage index to quantify the damage to CLT shear walls. Different failure modes for metal bracket connections were identified. Connection tests were analyzed using the energy-based index initially developed for concrete buildings (Kratzig et al., 1989) to obtain the relationship between the damage observed during the test and the damage index calculated. This is one of the few studies that attempted to quantify the amount of damage to CLT connections. This type of research is needed as the design community moves toward performance-based seismic design.

Figure 10 Two-storey CLT house tested at FPInnovations

With the objective of investigating the 3D system behaviour of CLT structures subjected to lateral loads, Popovski and Gavric (2014) tested a two-storey, full-scale model of a CLT house (Figure 10) under quasi-static monotonic and cyclic loading at FPInnovations. The house was 6.0 m x 4.8 m in plan, with a height of 4.9 m. A total of five quasi-static tests (one pushover and four cyclic) were performed one at a time in both directions. Parameters such as the direction of loading, number of hold-downs, and number of screws in perpendicular wall-to-wall connections were varied during the tests. The CLT structure performed according to the design objectives, with ultimate resistance being almost identical in both directions. Perpendicular walls had a significant influence on the lateral load resistance of the house. Failure mechanisms were similar in all tests; shear failure of nails occurred in the brackets in the first storey as a result of sliding and rocking of the CLT wall panels. The lap joints between the CLT wall panels allowed for relative slip during their rocking, as predicted in the design. Despite the rigid connection between the floor panels and wall panels, rocking of the wall panels was not fully restricted by the floor panels above. Relative slip between CLT floor panels in the diaphragms was negligible, suggesting that the connections between the diaphragm panels were properly designed. The deformation in the middle of the floor and roof diaphragms was only 14% of that at the supports, suggesting that the CLT floor panels acted as rigid diaphragms. Maximum storey drift of 3.2% (inclusive of sliding) was reached during test 5 in the north-south direction, suggesting that CLT structures can achieve relatively large storey drifts when properly designed. The types of connections used, their positioning, and the defining of their resistance based on the kinematic behaviour of the structure are crucial for a proper design of the structure.

An investigation of high-capacity hold-downs for tall CLT and mass timber buildings has been conducted in Canada (Zhang et al., 2016). It was found that proprietary hold-downs consisting of HSK inserted perforated steel plates that are glued in the mass timber element can provide hold-downs with high strength and moderate to high ductility. Nolet et al. (2017) developed an analytical methodology to predict the elasto-plastic behaviour of multi-panel CLT shear walls.

Pei et al. (2012) proposed a concept of alternating rigid CLT shear walls (using long panels) and ductile CLT shear walls (using short panel segments) in a multi-storey building at different storey levels. A ten-storey CLT building was designed using the equivalent force procedure from ASCE 7 with an R factor of 2.0. Then, three selected stories were replaced with ductile shear walls. A numerical model was built using SAPWood (Pei and van de Lindt, 2007) and subjected to multiple earthquake ground motions. It was confirmed that lateral deformation was concentrated at the ductile storeys, and acceleration at higher storeys was reduced significantly. Similarly, Dolan et al. (2014) proposed adding ductile components in tall CLT buildings to improve the resilience of the system during large earthquakes. The concept of inter-storey isolation was applied to tall CLT construction, detailing the CLT floor diaphragms to be deformable with a slip plane with stiffness and damping elements. Through numerical simulation, it was found that for a 10-storey building, one or two layers of deformable diaphragm at selected storeys would effectively reduce floor accelerations and the force demands on CLT connections.

Pei et al. (2012; 2013a; 2013b) undertook a study to identify suitable seismic design factors for CLT buildings in North America for both the United States and Canada. The approach of the two studies was similar, but it differed in the use of the corresponding codified design methods (ASCE 7 for the United States, and NBCC for Canada). “Nominal” design capacity tables for CLT shear walls of different lengths and bracket configurations were developed in this study. The idea was to enable an engineer to design a multi-storey CLT building following ASCE 7 or NBCC equivalent static force procedures. A prototype building was designed using a range of R factors and modelled in SAPWood. Performance expectations were outlined to determine the adequate R_d factor in NBCC, with the R_o factor set to 1.5. In order to find the appropriate R factor for the US code, the building backbone curves designed using different values of R were compared with those obtained if the building were designed using direct displacement design (Pang et al., 2010), which is a performance-based design approach that was previously validated for light-frame wood buildings. Based on the comparison, it was recommended that a preliminary value of R=4.5 may be adequate for ASCE 7 in the United States, while R_o=1.5 and R_d=2.0 can be used for the NBCC. The later result was consistent with the earlier estimates by Popovski and Karacabeyli (2012a). It should be noted that these studies were limited by their scope in that the results were based on a single building configuration and limited shear wall component test data.

Supported by the U.S. Department of Agriculture through the Forest Products Laboratory, a North American research team led by Colorado State University (Amini et al., 2014; 2016) initiated a FEMA P-695 (Federal Emergency Management Agency [FEMA], 2009) study to identify the appropriate seismic performance factors (R, C_d, and Ω_o) for platform-type CLT shear walls in ASCE 7. The project completed a three-component program that included: (1) approximately 30 CLT wall tests with various configurations; (2) tests on 3D box-type structures; and (3) tests on connectors, to help facilitate FEMA P-795 (FEMA, 2010) application for various connector manufacturers following the project. This project will also propose CLT as an addition to the response modification factors table within ASCE 7 as a culmination of the effort (van de Lindt et al., 2018a; 2018b).

The latest research efforts in Canada and the United States have dealt with resilient mass timber structural systems. Mass timber and CLT rocking shear walls are emerging as an effective solution for lateral load resisting systems in tall and mid-rise buildings. Rocking shear walls create a system to effectively dissipate seismic energy while reducing or eliminating the potential for damage to the building's superstructure. They also minimize the permanent displacements that occur after an earthquake. One such post-tensioned mass timber rocking wall system is Pres-Lam, which was originally developed in New Zealand. FPInnovations currently holds the licensing rights for this system in Canada and the United States and has initiated a multi-year research program to determine the behaviour of this system with U.S. engineered wood products such as CLT (Chen, Popovski and Symons, 2018). Work in this area has also been conducted by Kovacs and Wiebe (2016).

Oregon is emerging as a leader in North America in the use of mass timber rocking shear walls. The system has already been adopted in the 12-storey building in Portland called Framework, and the Oregon State University's 4-storey Forest Science Complex (both buildings currently under construction). The proposed mass timber parking garage project in Springfield, Oregon, will also use this system. Rocking wall systems are well suited for office, education, and other non-residential structures where open floor spaces are required.

The National Science Foundation (NSF) in the United States sponsored a two-year Network for Earthquake Engineering Simulation (NEES) CLT planning project (Pei et al., 2014), which was completed in 2016. The NEES-CLT project targeted the development and experimental validation of innovative CLT systems by 2016, with a vision to enable tall CLT buildings in the Pacific Northwest by 2020. The Pres-Lam system was selected as one of the best candidates, and variations of this system were investigated by Pei et al. (2012) and Dolan et al. (2014).

In July 2017, the NSF also funded a preliminary two-storey CLT rocking shear wall test at the University of California San Diego. After four separate shake table tests, no significant damage was detected in the 2-storey 7.7 m (22-foot) tall structure (Pei et al., 2018a; 2018b). This testing also assisted in validating the systems and methodology for future testing on a larger structure. A team of researchers from a number of U.S. universities, FPInnovations, designers, and architects, will test a 10-storey CLT rocking wall structure by 2020 on the nation's largest shake table to simulate earthquake conditions and verify the performance of mass timber in seismic regions.

Finally, Bezabeh et al. (2018a; 2018b; 2018c) investigated the serviceability performance of wind-excited tall mass- timber buildings. Wind tunnel testing of a model of a tall wood building and dynamic analyses of tall wood buildings were conducted. A probabilistic performance assessment of tall mass timber buildings subjected to stochastic wind loads was also conducted. Further, the lateral stability of multi-storey mass timber buildings subjected to tornado-like wind fields was examined. It was found that wind-induced vibrations may be the governing factors for the design for lateral loads in many cases of tall mass timber buildings.

4.5.3 Research in Japan and New Zealand

CLT structural systems were first investigated in Japan at Shizuoka University in 2012. The research studies focused on the assessment of simplified formulas capable of predicting the failure mechanism of wall systems with vertical lap joints (Yasumura, 2012). Furthermore, since defects in CLT panels are less critical than in other timber-based products (e.g., sawn timber or glue-laminated timber), a significant effort was devoted to develop CLT elements using the locally grown sugi (*Cryptomeria japonica*) instead of a European tree species (e.g., *Picea abies*). To this aim, the hysteretic behaviour of wall systems assembled with panels of Japanese sugi was analyzed (Okabe et al., 2012). In addition, the racking resistance of monolithic and segmented wall systems with an opening was examined (Kawai et al., 2014; Yasumura and Ito, 2014; Yasumura, Kobayashi, and Okabe, 2016a). Finally, shake table tests of multi-storey CLT prototypes (two-, three-, and five-storeys high) demonstrated satisfactory seismic performance (Tsuchimoto et al., 2014; Kawai et al., 2016; Yasumura et al., 2016b) (Figure 11).

Figure 11 Five-storey CLT house tested at E-Defense Laboratory in Miki, Japan (Yasumura et al., 2016b).

A significant amount of research has been conducted in New Zealand on the use of CLT and other mass timber products in innovative, seismically resilient timber systems. Researchers at the University of Christchurch have been working for the last 15 years on quantifying the seismic performance of post-tensioned rocking walls with seismic dissipators. Multiple research papers have been produced, and only few are referenced here (Iqbal et al., 2016; Sarti et al., 2016; Moroder et al., 2014; Dunbar et al., 2014). The mass timber systems with post-tensioning have been proven to provide adequate seismic response without any resilient deformations after the seismic response. Researchers at the University of Auckland have been working on development of resilient slip friction joints and their implementation in mass timber walls (Hashemi et al., 2016; 2017). It was found that these joints used as hold-downs provided adequate seismic behaviour of the walls and added self-centering characteristics. The joints also allowed for damage-free deformation and ductility of the system.

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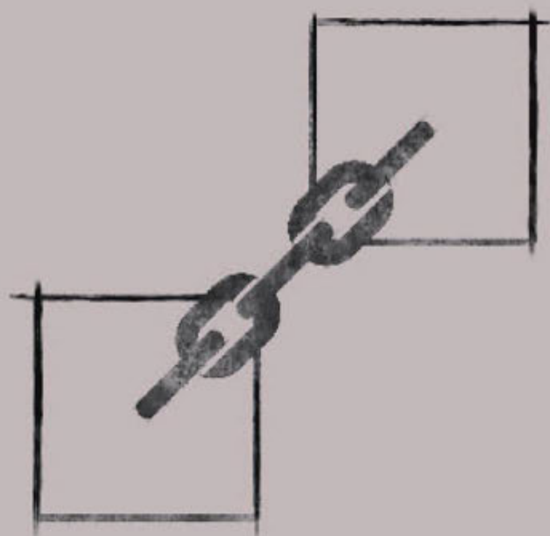

CHAPTER 5

Connections in cross-laminated timber buildings

Lead Authors
Mohammad Mohammad
Chun Ni
Williams Munoz

Reviewers
Alexander Salenikovich
Pierre Quenneville
David Moses
Erol Karacabeyli

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ABSTRACT

Connections play an important role in maintaining the integrity of the structure and in providing strength, stiffness, stability and ductility. For example, the structural efficiency of the floor system acting as a diaphragm and that of walls in resisting lateral loads depend on the efficiency of the fastening systems and connection details used to interconnect individual panels and assemblies together. Long self-tapping screws are typically recommended by the 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 also be used in CLT assemblies.

This Chapter focuses on connector systems that reflect present-day practices, some being conventional, others being proprietary. The Canadian design approach, which is based on the European and Canadian research, is presented for traditional fasteners in CLT such as bolts, dowels, nails and wood screws. Three typical connection design examples are provided at the end of the Chapter.

5.1 INTRODUCTION

Connections in mass 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 a timber structure caused by the presence of connections may result in a decrease in the overall strength and stiffness of the structure if not properly designed.

Joints between structural members using fasteners or other hardware are referred to as “mechanical connections”. Large fastener spacing is 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 structural materials such as steel or concrete. This is particularly important for multi-storey heavy timber and mixed structures where CLT is used alone or in combination with light wood-frame or steel or concrete.

The use of CLT panels enables a high degree of prefabrication at the plant. The dimensional stability of CLT due to the use of kiln-dried (KD) source material and the use of CNC (Computer Numerical Control) technology to profile the panels facilitates the installation of conventional or innovative connections with a high degree of accuracy and efficiency.

5.1.1 Commonly Used Connections in CLT Structures

A wide variety of fasteners and types of joints can be used for roof-to-wall, wall-to-floor, and inter-storey connections in CLT and mixed structures. While long self-tapping screws are typically recommended by CLT manufacturers and are commonly used for connecting floor panels and walls to floors, standard dowel-type fasteners such as wood screws, nails, rivets, lag screws, bolts and dowels have also been effectively used in connecting panel elements together in many projects in Canada and beyond. Other traditional fasteners, including timber connectors, such as split rings and shear plates, spikes and tooth plates, may have some potential; however, their use is expected to be limited to applications where high loads are involved. There is also a high potential in the CLT construction market for innovative connections, especially those that employ a high degree of prefabrication using CNC machining technology. Fortunately, major CLT panel and glulam plants are equipped with CNC technology which facilitates the rapid adoption of such connections. The choice of the type of connection depends largely on the type of assemblies (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. Some applications of these fasteners are presented in Section 5.2.

5.1.1.1 Screws

Screws are extensively used for the assembly of CLT panels (Figure 1). The ease of installation and the high lateral and withdrawal capacity of screws, as well as their ability to resist combined axial and lateral loads make them quite common in construction. The screws come in a variety of sizes and special features. Self-tapping screws are available in diameters that range from 4 mm to 12 mm and in lengths up to 600 mm (Augustin, 2008). In most cases, predrilling is not required for installation, unlike traditional wood screws or lag screws, which usually require predrilled holes, the size of which depends on the screw diameter and the wood density. The design capacity of screws in CLT depends on gaps between unglued cross plies and grooves common in CLT fabrication as will be discussed in the following sections.

Figure 1 Self-tapping screws used in CLT connections

5.1.1.2 Nails, spikes and timber rivets

Nails, spikes and timber rivets are often used in spline connections, or connections with perforated metal plates and brackets, as shown in Figure 2. Most timber design standards do not allow the use of nails in the end grain of wood-based products for withdrawal forces. Therefore, smooth-shank fasteners such as nails should not be driven in the edge of CLT panels to resist withdrawal forces unless it can be ensured that the nail is driven in the side grain of the CLT lamination. However, the use of nails in the end grain is permitted for lateral resistance, which is reduced with the end grain factor in most timber design standards, including CSA O86-14 Update 1 (CSA, 2016). No reduction factor is applied when nails and rivets are driven into the face of CLT panels.

Figure 2 Power-driven nails used in combination with perforated metal plates

5.1.1.3 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 (mostly in hold-downs of shearwalls). Care must be taken during the design and installation, especially in CLT panels with unglued lamination edges. The resistance may be compromised if the fasteners are driven in the gaps.

5.1.1.4 Split ring and shear plate connectors

While split ring and shear plate connectors are commonly used in structures with 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. These connectors can be used in certain locations depending on the position of the fasteners with respect to the CLT layers and the type of design loads. One of the drawbacks of these connections is the need for special tools and precise profiling for installation.

5.1.2 Innovative Fasteners and Connectors

A new generation of fasteners and connectors has been developed specifically for CLT mainly by European producers. This is driven by the increased use of the CNC technology in the production of wood materials and the desire for high level of prefabrication to reduce the assembly time and cost. The main advantage of the new fasteners and connectors is that they facilitate quick assembly and dismantling of structures. Moreover, these connections are often concealed inside the CLT panels and, therefore, have a better fire resistance. Glued-in rods can be used for connections subjected to high axial and lateral forces to reduce the splitting potential (Augustin, 2008). Slip-friction connectors (Hashemi et al., 2017) can be used in CLT shearwalls to allow CLT panels to develop the rocking motion. Some examples on the next generation connections and their suitability for CLT assemblies are shown in Section 5.2. It should be noted that the trade names that have been included are only for the purpose of providing examples, without any intention to promote specific products or manufacturers.

5.2 CONNECTIONS IN STRUCTURES WITH CLT

This Section is focused on providing general information and schematics of traditional and innovative connections for structures with CLT. Figure 3 shows locations of connections in a multi-storey CLT building. While most of the commonly used fasteners and those with some potential for use in CLT assemblies are described below, the list is not comprehensive. Other types of innovative proprietary fasteners, not mentioned in this Section, could also be suitable.

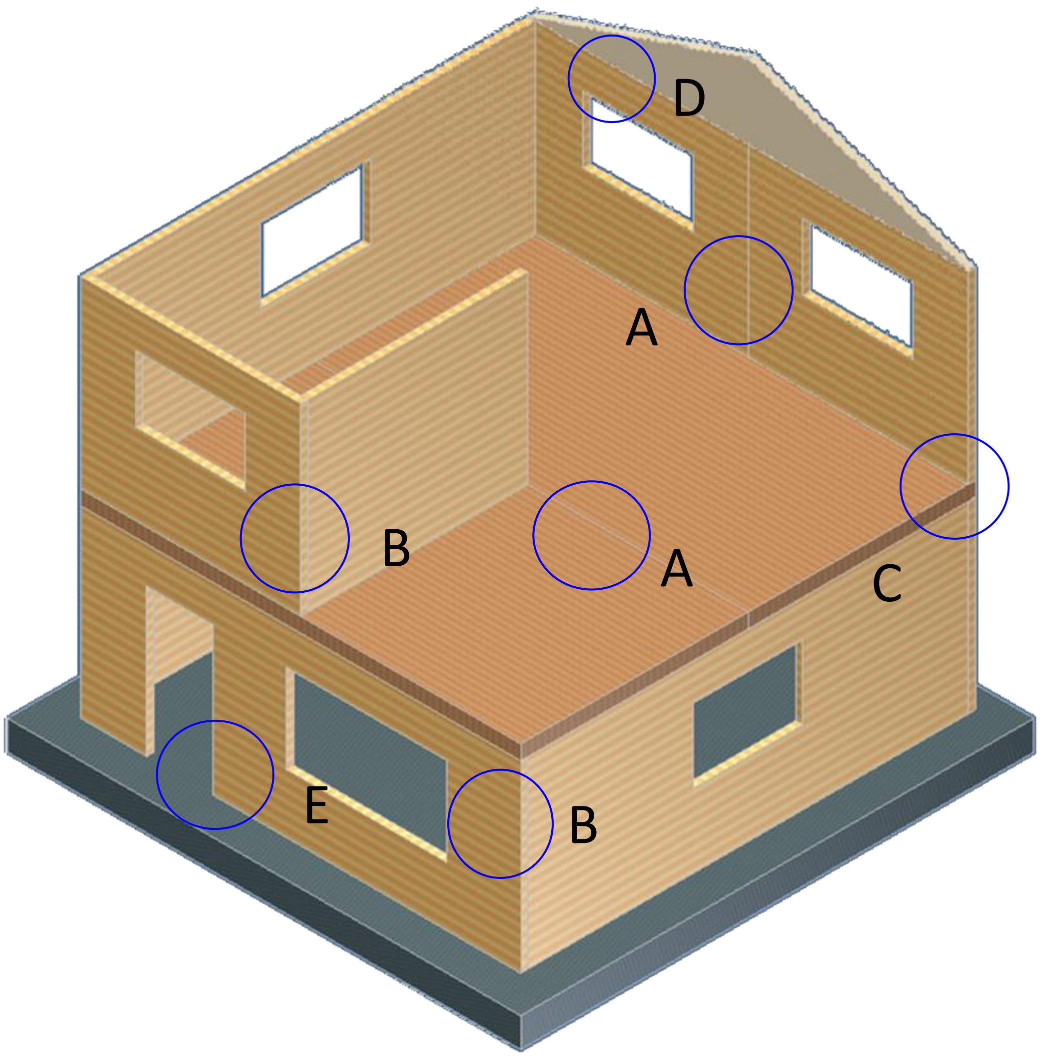


Figure 3 Typical 2-storey CLT building showing various locations requiring connections

5.2.1 In-Plane Connections (Detail A)

This is the basic connection along the panel edges in walls or floors oriented in one plane. Due to production and transport limitations related to the size of the panel that can be delivered to building sites, panels are usually joined on site. Therefore, it is desirable that the connections allow easy and quick erection of the structure. The panel-to-panel connections transfer forces through the walls or floors. In walls, the connection must be designed to resist in-plane shear and out-of-plane bending. In floors acting as diaphragms, the connections must be designed to transfer in-plane shear forces and maintain the integrity of the diaphragms. Several possible in-plane connections are described below.

5.2.1.1 Internal Spline

A single or a double spline/strip made of plywood (SCL such as LVL or lumber is also possible if longitudinal shear of the spline is not critical) could be used to form this connection. Profiling of the panel at the plant is necessary prior to delivery on site. The spline between the panel edges can be attached using self-tapping screws, wood screws, nails or spikes. The fabrication requires accurate profiling and it can be challenging to fit the parts on site. However, an advantage of this connection is the high lateral resistance because the fasteners are loaded in double shear. There are also other advantages regarding the resistance to normal or out-of-plane forces. Structural adhesive could also be applied in addition to the mechanical fasteners to provide more rigidity to the connection, if needed. A schematic of a single internal spline is shown in Figure 4.

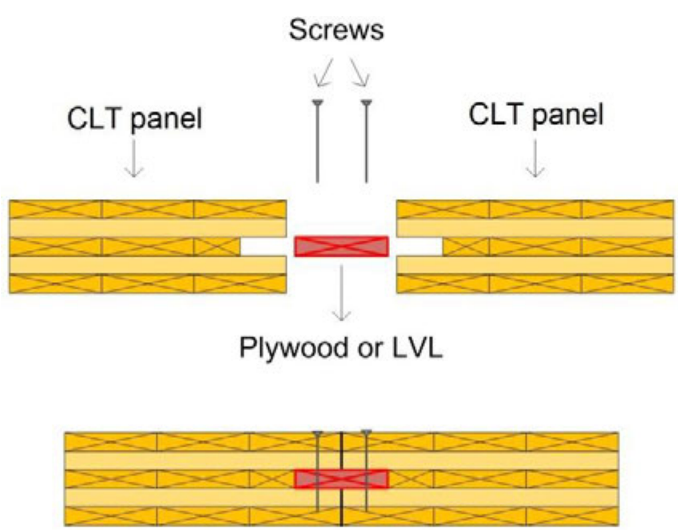


Figure 4 Internal spline

5.2.1.2 Single Surface Spline

This is a rather simple connection that can be assembled quickly on site. At the plant, CLT panels are profiled along the edges to accommodate a strip/spline of plywood (lumber or SCL such as LVL or laminated strand lumber (LSL) is also possible if longitudinal shear of the spline is not critical) (Figure 5). Self-tapping screws, long wood screws, nails, spikes or a combination of these are installed on site. As the fasteners work in single-shear, the lateral resistance of this connection is typically inferior to the internal spline described in Section 5.2.1.1. However, due to its simplicity, it is preferred by designers and builders. This simple connection is often used to connect floor panels. Structural adhesive could also be used in this connection for higher rigidity.

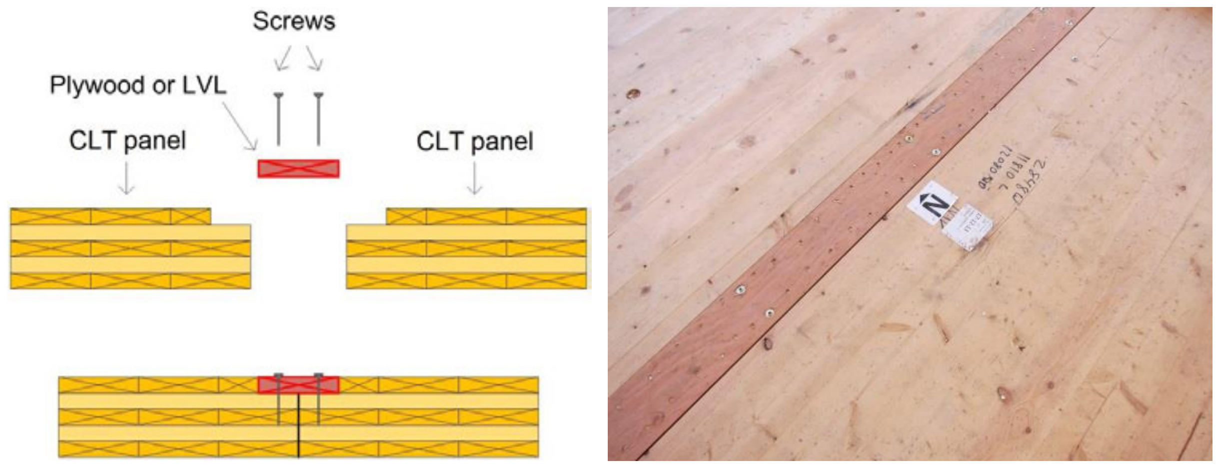


Figure 5 Single surface spline

5.2.1.3 Double Surface Spline

This connection is similar to the single surface spline described in Section 5.2.1.2, except that a second spline is added on the opposite face of the panels to increase the connection strength and stiffness (Figure 6). Since two sets of fasteners are used, which results in doubling the number of shear planes resisting the load, a higher resistance can be achieved. However, this connection requires more machining and more time for erection since there is a need to fasten the splines on both sides of the panels. If plywood is used for the splines, then the joint could be designed to resist moment for out-of-plane loading. Structural adhesives could also be used in this connection for higher rigidity.

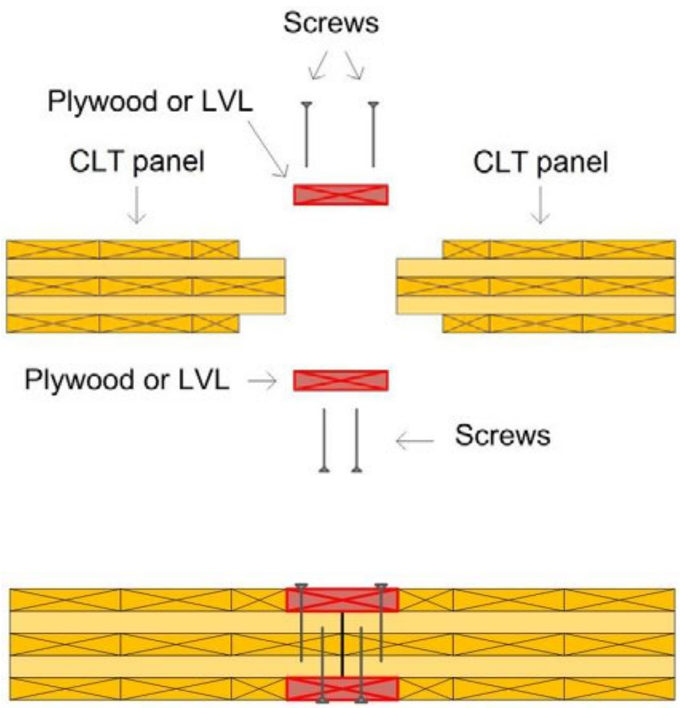


Figure 6 Double surface spline

5.2.1.4 Half-Lap Joint

This joint is commonly used for in-plane connections in walls and floors (Figure 7). The half laps are milled along the panel edges at the plant and long self-tapping screws are commonly used to connect the panels on site. The joint can resist in-plane shear and normal forces, but is not considered to be a moment resisting connection (Augustin, 2008). While this is a very simple connection that facilitates quick assembly, 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 risky if uneven loading on the floor elements may occur (Augustin, 2008). Another disadvantage is the loss of fiber and the reduced width of the panel in comparison with other types of connections (i.e., surface or internal splines).

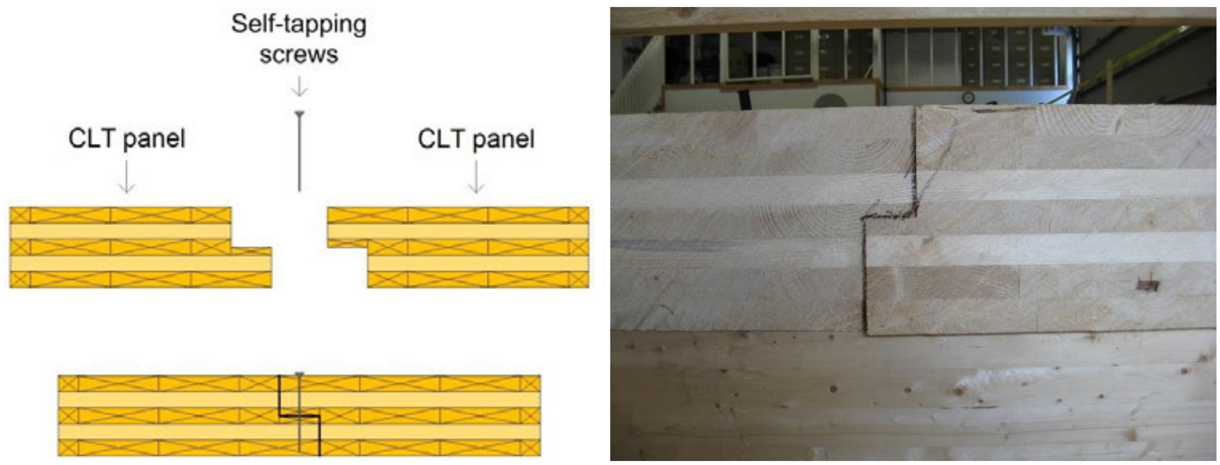


Figure 7 Half-lap joint

5.2.1.5 Proprietary Connectors

Innovative connectors with good potential for use in CLT structures are presented in this Section.

The ***tube connector*** has been developed and studied by G. Traetta (2007) at the Building Research Center in Graz, Austria. It incorporates slotted steel tubes, which are inserted in predrilled holes along the panel edges so that the glued-in or screwed-in threaded rods driven into the edges of the panels fit in the slots and then the joint is tightened with steel nuts (Figure 8). Usually no edge profiling along the panels is needed for this connector as it principally relies on the pullout and the shear resistance of the threaded rods.

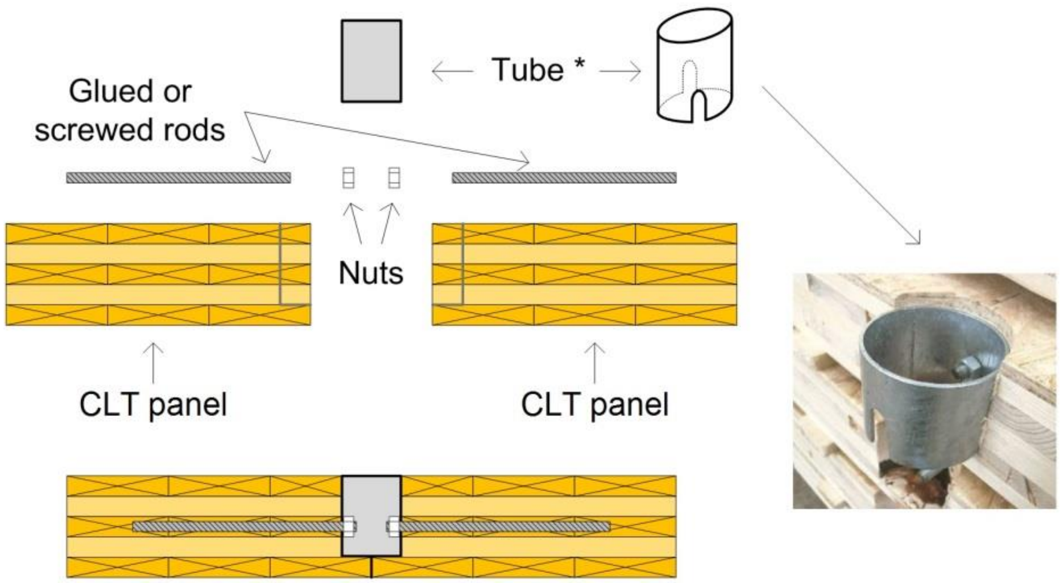


Figure 8 Tube connector

The ***shear key connector*** was developed and used by Nordic Structures to maximize the resistance of CLT shearwalls in multi-storey buildings. The shear keys shown in Figure 9 take advantage of compression resistance parallel to grain and the shear resistance of steel plates. This connection offers high in-plane stiffness and resistance for shearwalls under wind loading combined with significant ductility under the seismic loading. This simple and versatile concept can be used to join CLT panels as well as CLT and glulam members.

Figure 9 Shear keys for shearwalls (photos courtesy of Nordic Structures)

The ***Knapp® connector*** was developed in Germany for wood-based panels to facilitate quick erection using the concept of a male/female attachment (Figure 10) (Knapp, 2015). It is mainly used for panel-to-panel connections along the edges. Knapp® brackets are usually attached to CLT panels with wood screws. They provide in-plane and out-of-plane resistance as well as uplift resistance. The Knapp® brackets are equipped with a self-locking mechanism that enables the wall to be tightly locked with the adjacent wall quickly and easily. However, it might be relatively complicated to assemble or dismantle structures with complex plans with several intersecting wall segments.

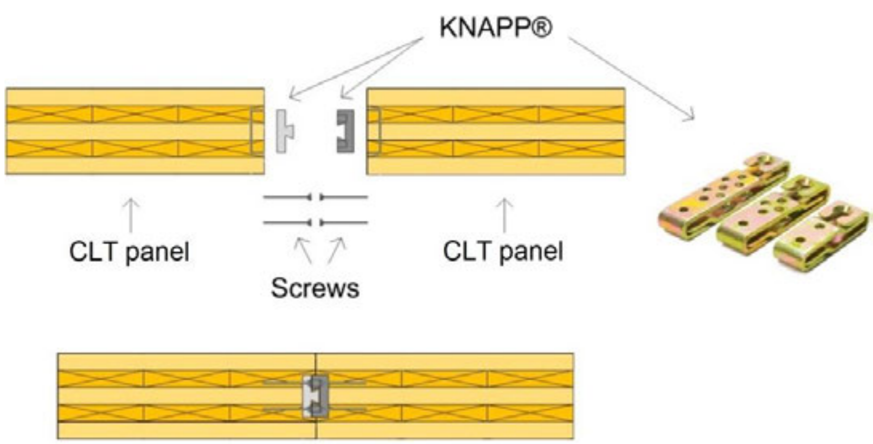


Figure 10 Knapp® connector

5.2.2 Corner Connections (Detail B)

In-plane wall connections (Detail A) are covered in Section 5.2.1. This Section covers connections for walls positioned at an angle, such as wall corners and junctions of partitions and exterior walls. In the following paragraphs, commonly used fasteners for CLT walls are presented as well as several innovative connectors for use in this application.

5.2.2.1 Self-Tapping Screws

Self-tapping screws provide the simplest means to connect transverse walls (Figures 11 and 12). However, caution should be exercised when the screws are driven in the panel edge to avoid installation of the screws in the end grain of the cross layers or in the gaps between laminations, particularly for the transfer of high wind and seismic loads. To avoid end-grain installation, self-tapping screws could be driven at an angle (inclined) as shown in Figure 13. When using inclined screws (toe-screwing), caution should be exercised to avoid chipping-off panel corners due to tension perpendicular to grain.

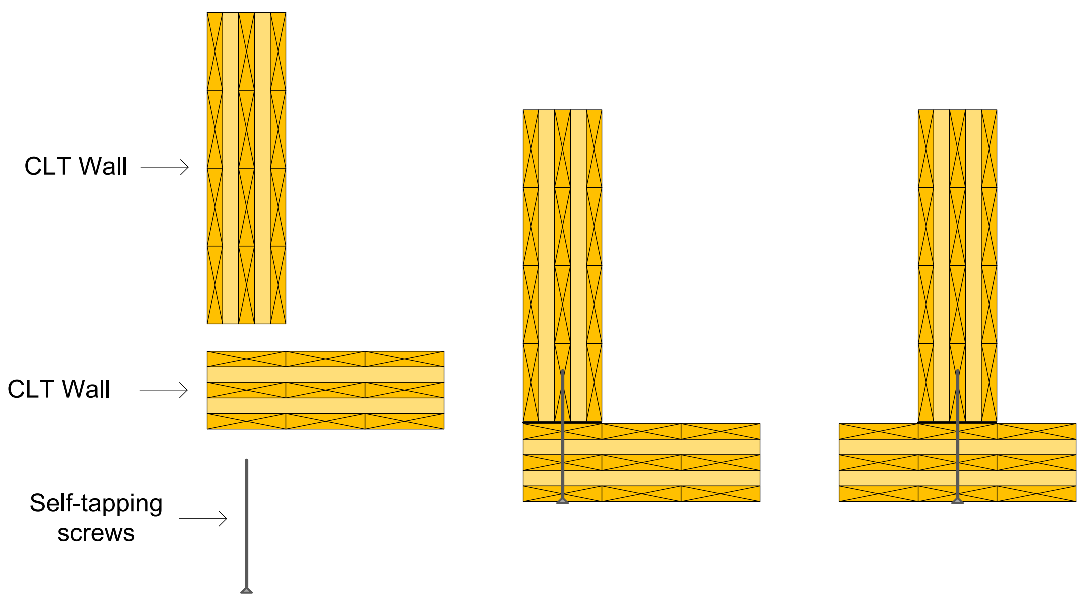


Figure 11 Self-tapping screws connections driven from the exterior (plan view)

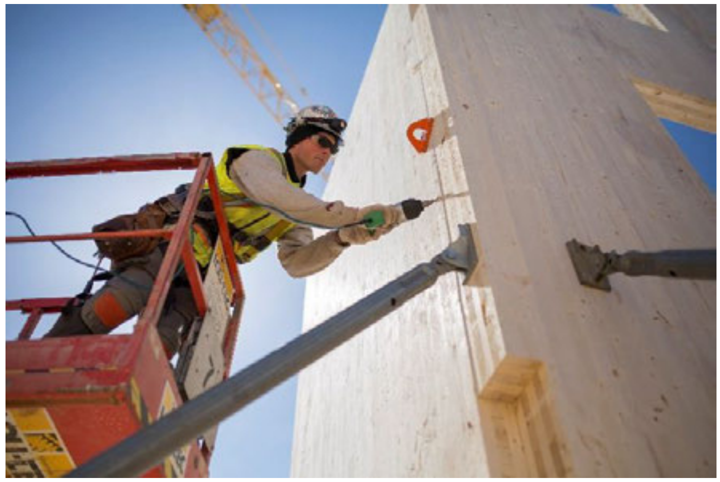


Figure 12 **Installation of self-tapping screws from the exterior (right: photo courtesy of Stephane Groleau)**

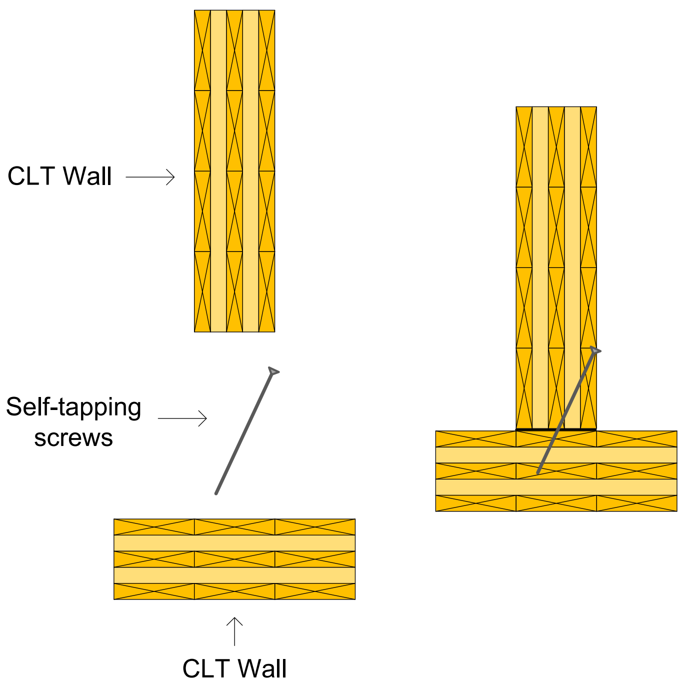


Figure 13 **Self-tapping screws driven at an angle (toe screwing) from the interior (plan view)**

5.2.2.2 Metal Brackets

Another simple means of connecting transverse walls is the use of metal brackets with screws, nails or glulam rivets (Figure 14). While this connection is one of the simplest and most efficient types of connections in terms of strength resulting from fastening in the direction perpendicular to the plane of the panels, architects normally do not prefer this connection because the metal plates are exposed and have less fire resistance than concealed connections. Designers may choose to hide plates by profiling the wall panel at the locations of the brackets (recessing) and then covering the metal hardware with finishing materials or wood caps.

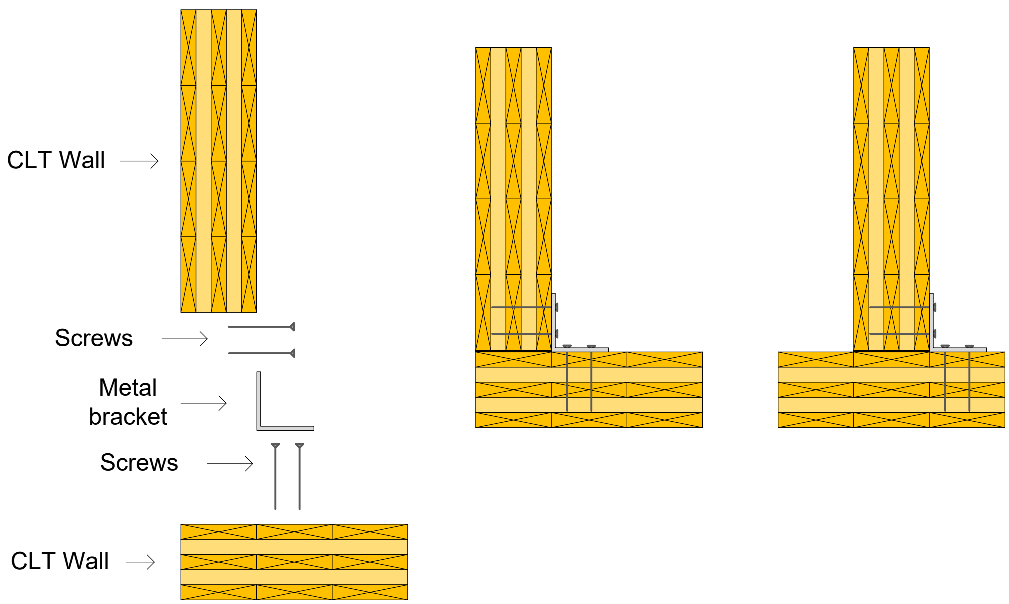


Figure 14 Interior metal bracket (plan view)

5.2.2.3 Proprietary Connectors

Several proprietary connectors can be used for CLT walls.

The ***X-RAD*** from Rothoblaas is a compact connector composed of several hardware components (Rothoblaas, 2015). Certain components are pre-assembled at the plant using self-tapping screws and thus both floor and wall panels are delivered to the construction site furnished with the connection system. The X-RAD connection can also be used as a lifting hook for onsite handling of panels. On site, wall panels are assembled using metal brackets with predrilled holes (Figure 15). Bolts then connect the pre-assembled components to the steel brackets turning the assembly into a steel-to-steel connection. The X-RAD system allows quick connection to reinforced concrete foundation slabs and walls before the installation of the panels. The connection, while complex, enables quick on-site assembly. But it also requires precise profiling and fitting on site.

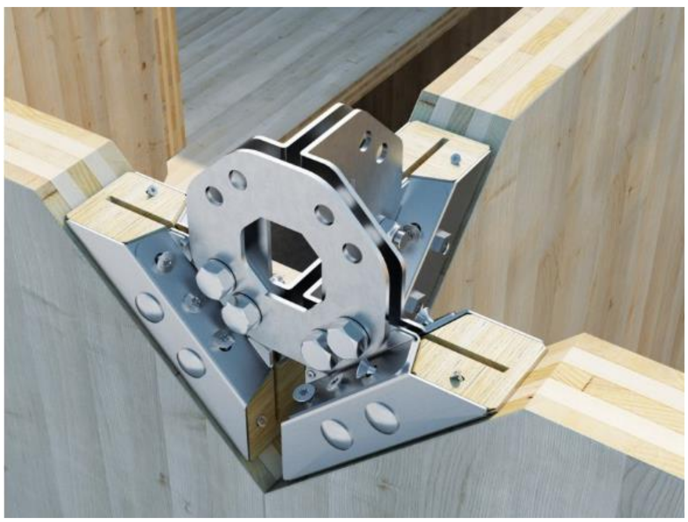


Figure 15 X-RAD connector from Rothoblaas (photo courtesy of Rothoblaas)

The ***Dovetail metal brackets*** can be used to connect wall panels (Figure 16). Several forms of a male/female type of connection can be designed to resist in-plane and out-of-plane loads. The continuous or intermittent metal brackets are attached to the panel edges at the plant using regular wood screws or self-tapping screws. The panel is slide into place, which speeds the erection of the walls on site.

The ***Knapp® connectors*** and ***hook joints*** use the clip-on method (Figures 17 and 18) (Knapp, 2015). Wood screws are typically used to fasten the metal brackets to CLT panels.

It should be noted that all these connections require tight fabrication tolerances to facilitate site installation. Measures should be taken to ensure that wall panels are properly aligned and snugly fit.

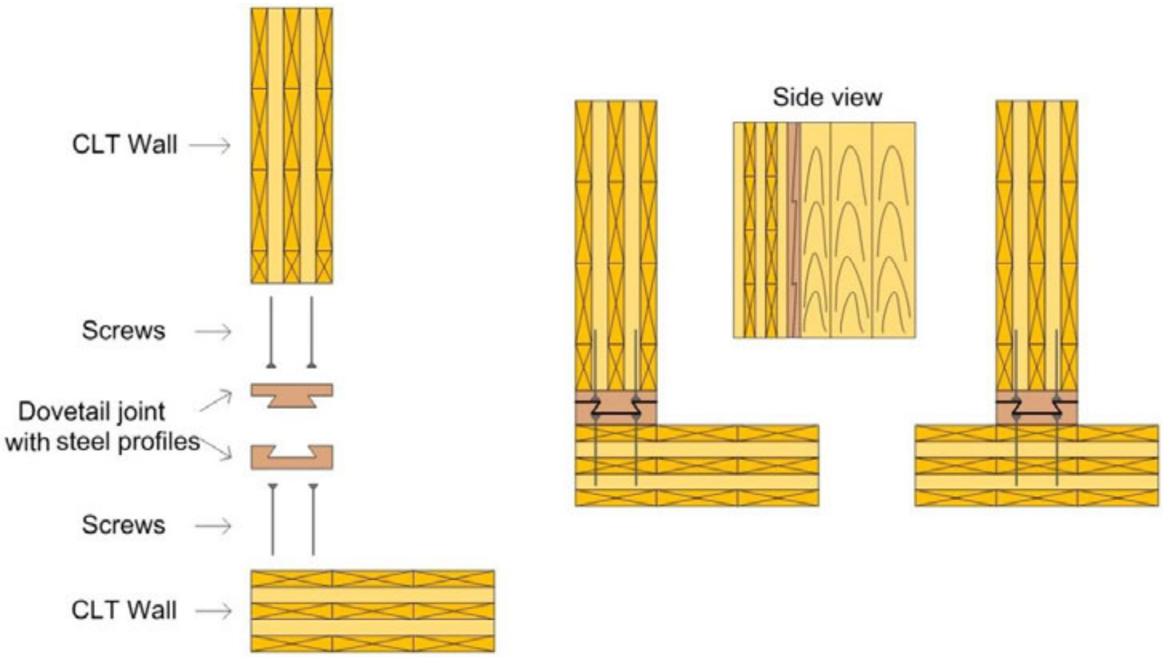


Figure 16 Dovetail joint (plan view)

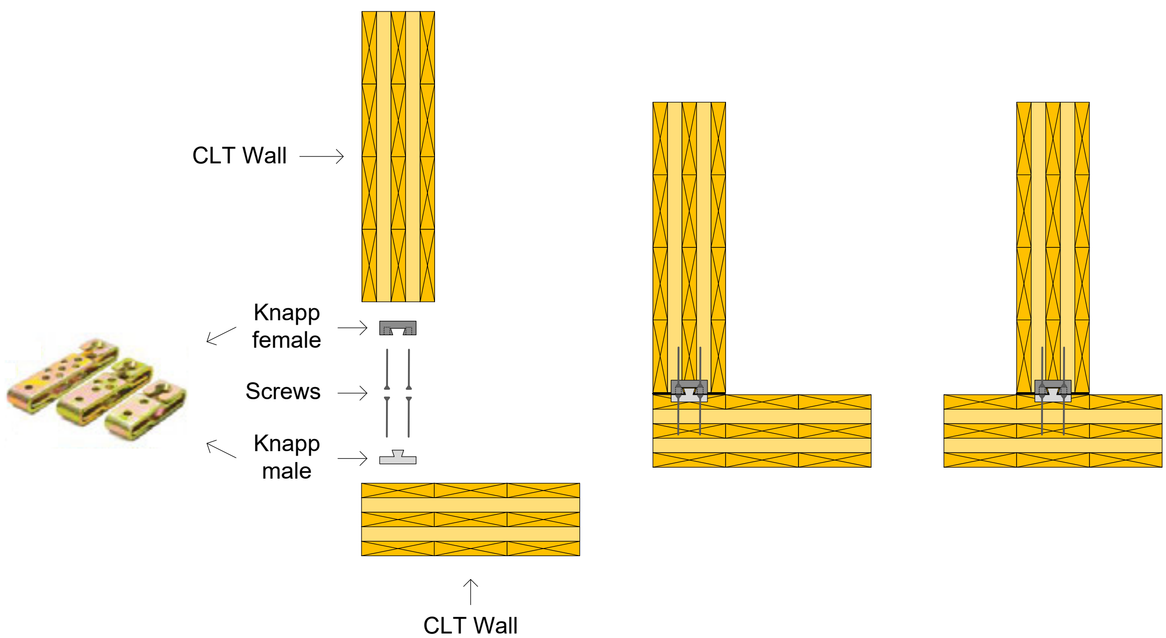


Figure 17 Knapp® connector (plan view)

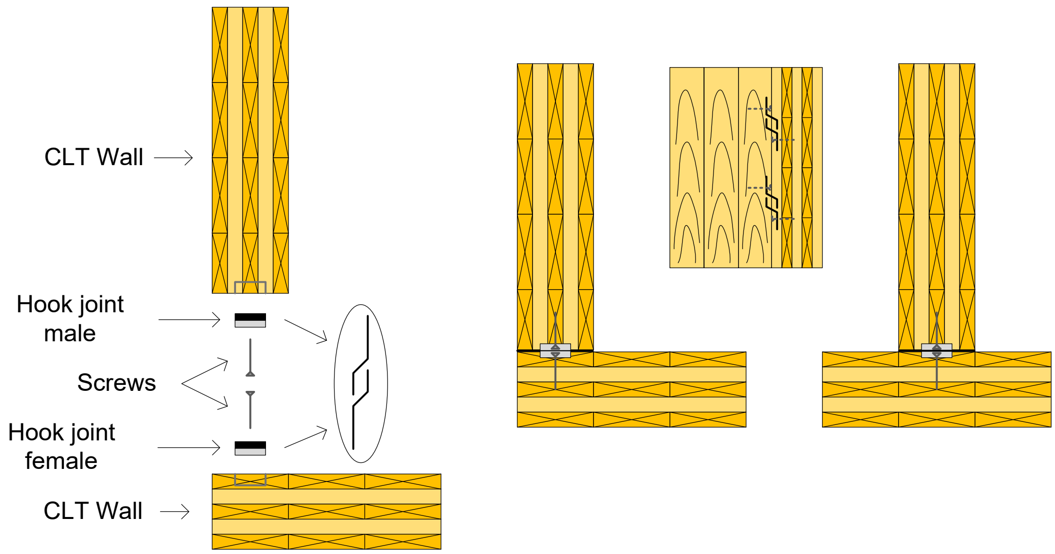


Figure 18 Hook joint (plan view)

5.2.2.4 Concealed Metal Plates

Concealed perforated or non-perforated metal plates can also be used to connect CLT walls in the transverse direction using power driven nails or self-tapping screws. Metal plate thickness typically ranges from 6 mm up to 12 mm depending on the loads. While these connectors have advantages over exposed plates and brackets, especially in fire performance, they require precise profiling at the plant using CNC technology (Figures 19 and 20). Proprietary self-drilling screws that can penetrate through wood and steel such as those produced by SFS Intec (Figure 21) can be used with plates that are not pre-drilled to reduce on-site alignment issues.

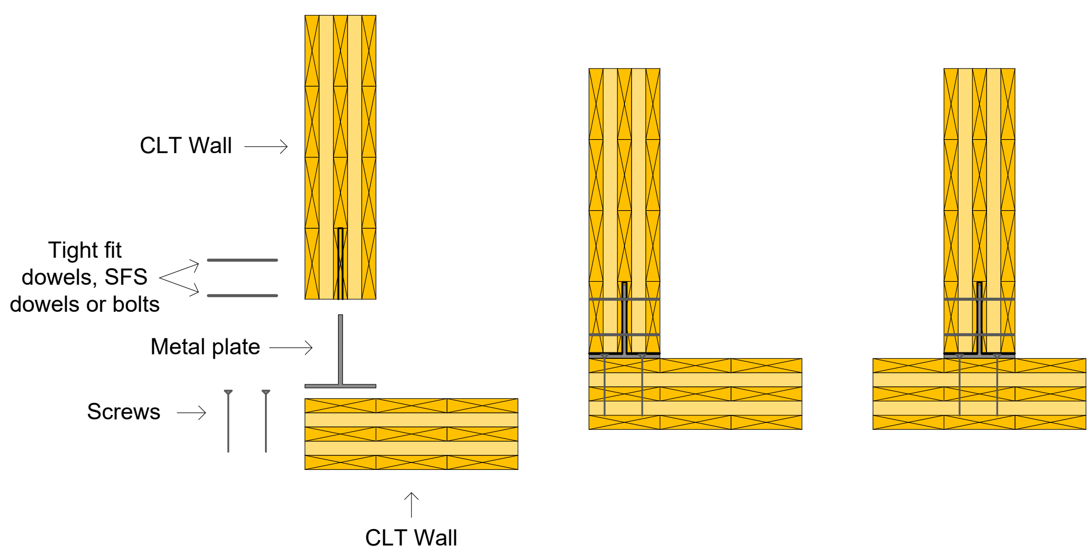


Figure 19 Concealed metal plate (plan view)

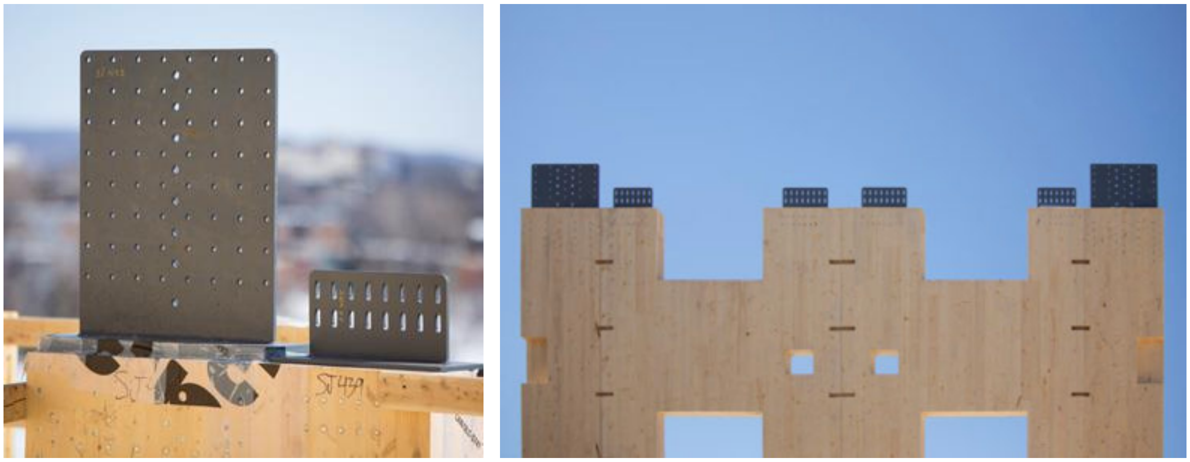


Figure 20 Perforated metal plates (photos courtesy of Stephane Groleau)

Figure 21 Self-drilling screw for use through steel and wood

5.2.3 Wall-to-Floor Connections (Detail C)

Several possibilities exist for connecting walls to the floors above and below the floor level. These depend on the type of structural systems (i.e. platform or balloon), availability of fasteners and the degree of prefabrication.

5.2.3.1 Platform Construction

5.2.3.1.1 Self-Tapping Screws

To connect a floor or a roof to walls below, the simplest method is to use long self-tapping screws driven from above directly into the wall edge, as shown in Figure 22. Self-tapping screws can also be driven at an angle to maximize the fastening capacity in the panel edge. The same principle can be applied to connecting walls above to floors below, where self-tapping screws are driven at an angle (toe-screwed) in the wall near the junction with the floor. Depending on the angle and the length, the self-tapping screws could reach the wall below, further reinforcing the connection.

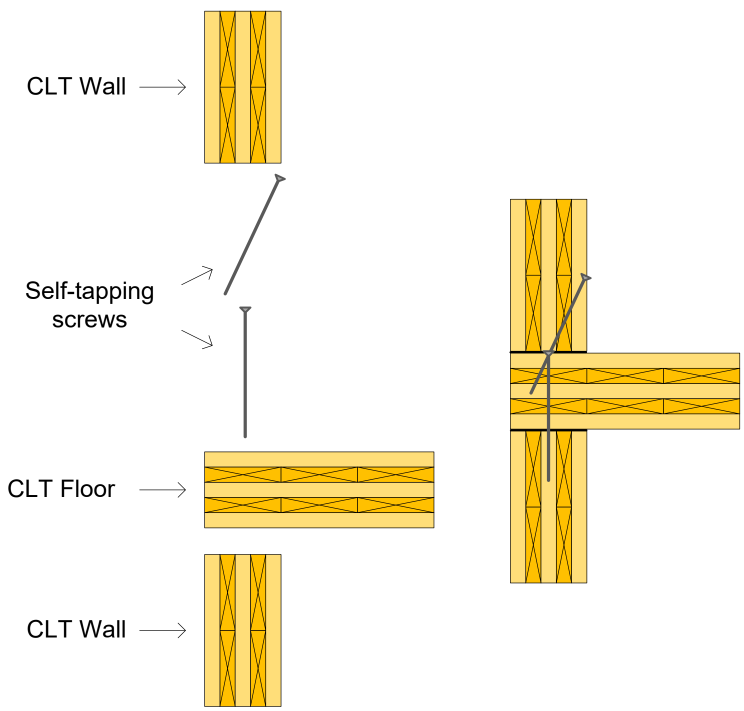


Figure 22 Self-tapping screws (elevation view)

5.2.3.1.2 Metal Brackets

Metal brackets are commonly used to connect floors to walls above and below. They are also used for connecting roofs to walls. Self-tapping screws, nails, glulam rivets and wood screws can be used to attach metal brackets to CLT panels (Figures 23 and 24). Some bracket designs may include vertical slots on the wall side of the bracket to further facilitate the rocking motion for shearwalls subjected to seismic loading.

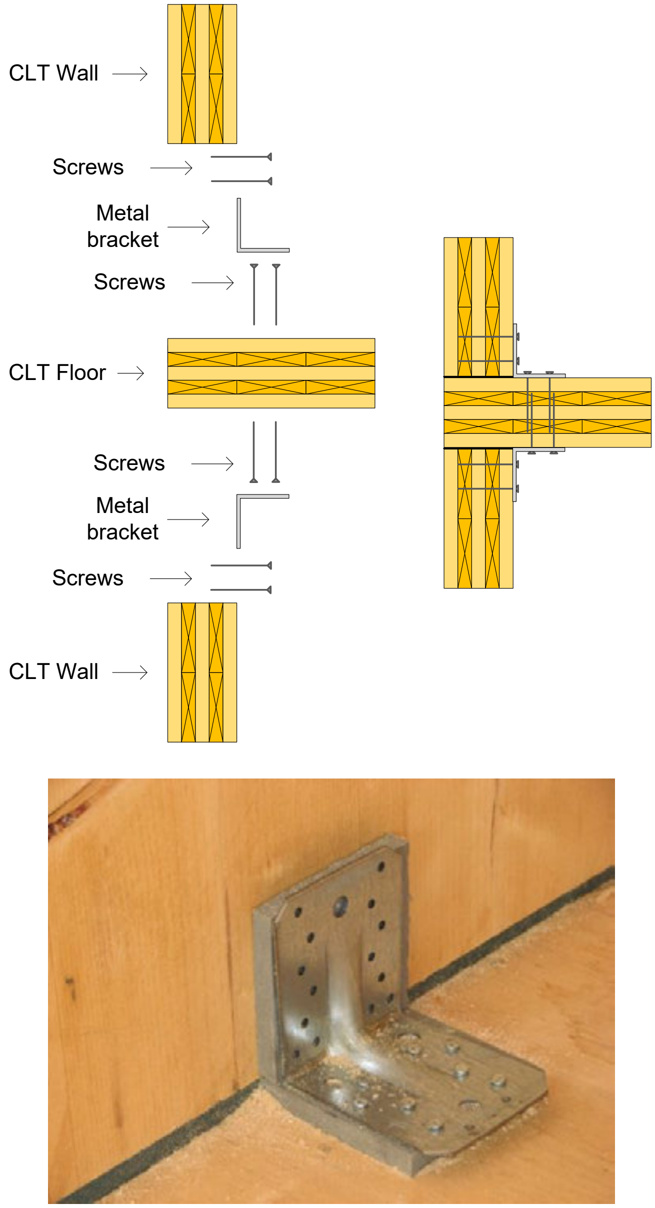


Figure 23 Metal brackets (plan view)

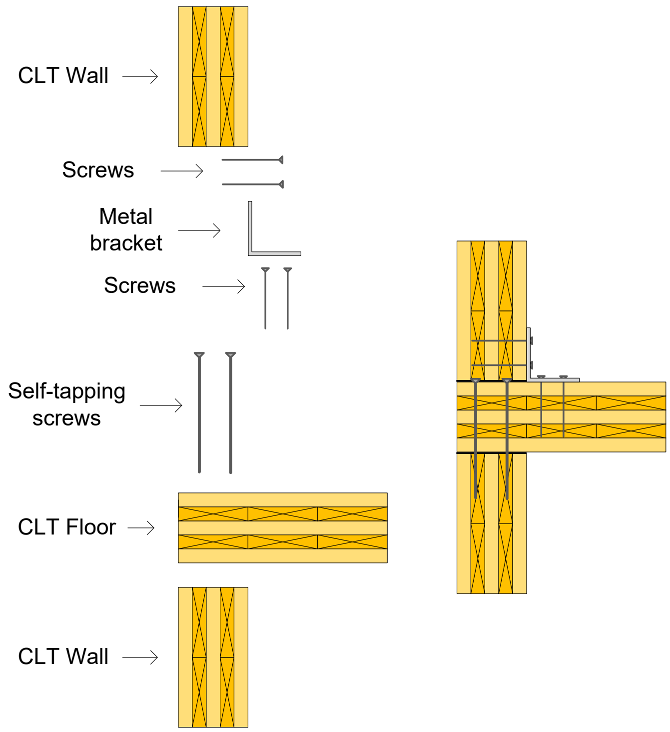


Figure 24 Metal bracket and self-tapping screws (plan view)

An efficient alternative bracket to the metal brackets shown above in situations where the shearwall is rocking is to profile the holes so that the rocking motion is not prevented and the bolts or dowels are always in contact with the edge of the hole, as shown in Figure 25. This bracket configuration (named the shear key) was successfully used in a CLT shearwall experiment (Hashemi, 2017). The profiles of holes are determined based on the position of the shear key in relation to the shearwall center of rotation.

a) Shear key with profiled holes to allow CLT shearwall rocking motion

b) Shear key in CLT shearwall

Figure 25 Shear key used in CLT shearwall (photos courtesy of Quenneville)

5.2.3.1.3 Proprietary Connectors

This Section covers innovative fastenings described above including Knapp® connectors and slip-friction connectors (Figures 26 and 27) (Knapp, 2015). Some of these connections, such as Knapp®, have a self-locking mechanism to resist against uplift.

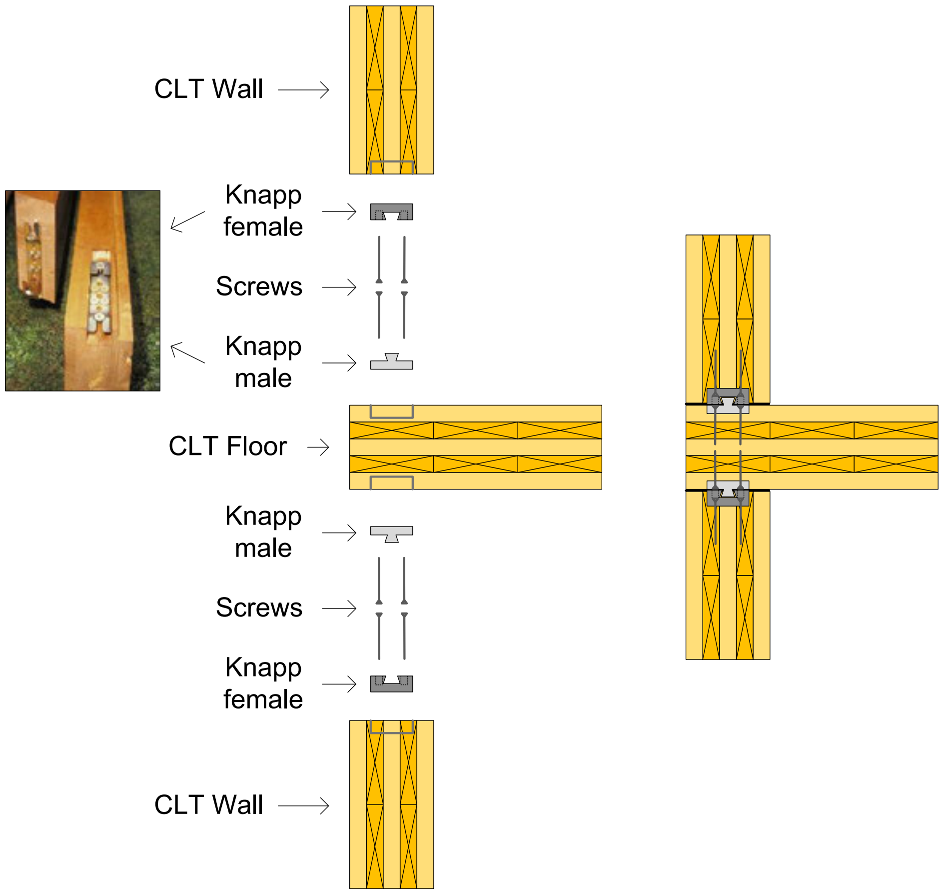


Figure 26 Knapp® connector (plan view)

Another proprietary system is the ***Resilient Slip-Friction Joint (RSFJ)*** which is a friction-based energy-dissipative device that is used as a hold-down in CLT construction. Its configuration is such that its behavior is geometrically non-linear, but the material remains elastic and, thus, does not require any repairs or replacement following an earthquake or after-shock. The RSFJ provides damping (up to 20%) and self-centering for rocking CLT shearwalls. It is connected to the CLT shearwall using bolts, screws or rivets.

Figure 27 Resilient slip-friction joints as hold-downs (photos courtesy of Tectonus)

5.2.3.1.4 Concealed Metal Plates

Concealed T-shape metal plates can be used in wall-to-floor connections (Figure 28). As previously discussed, while this connection has advantages over the exposed plates and brackets, especially for fire performance, it requires precise profiling at the plant using CNC technology.

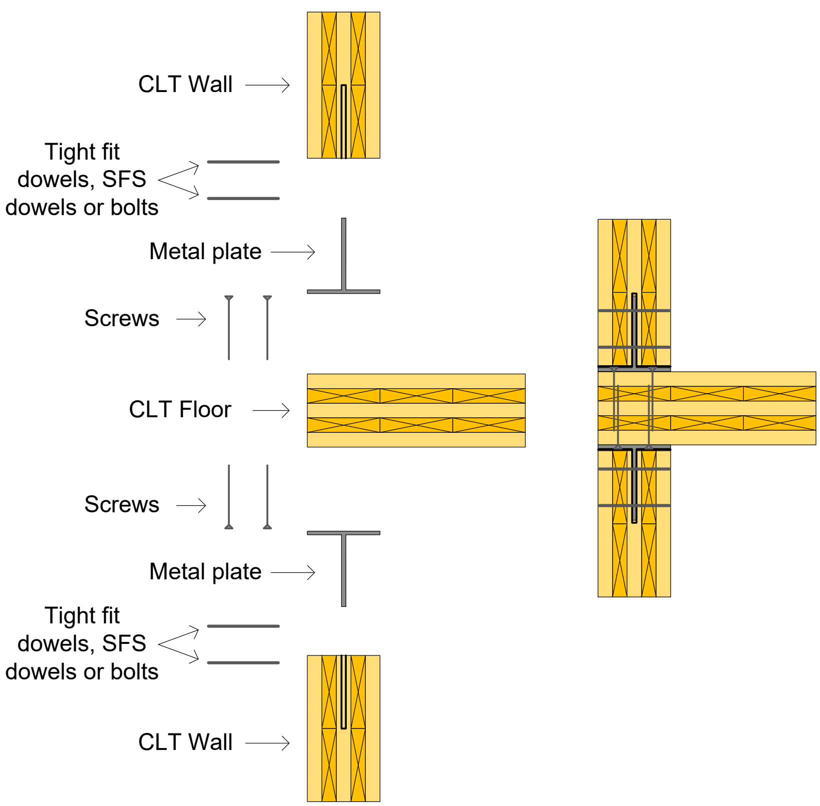


Figure 28 Concealed metal plates (plan view)

5.2.3.2 Balcony Details

5.2.3.2.1 Balcony in Cantilever

For situations where a balcony is designed by extending the floor or roof panel as a cantilever (Figure 29), the wall and the balcony floor panel can be connected using self-tapping screws or metal brackets. In this case, the panels forming the balcony should be installed with the major strength axis extending outward. 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 30 and 31). Inclined (toe-screwed) self-tapping screws are preferred for improved performance (Figure 30a) rather than the screws driven straight into the edge of the supporting wall panel (Figure 30b). However, caution should be exercised in design of CLT panels as a cantilever because of potential issues related to water infiltration.

Figure 29 Balcony in cantilever

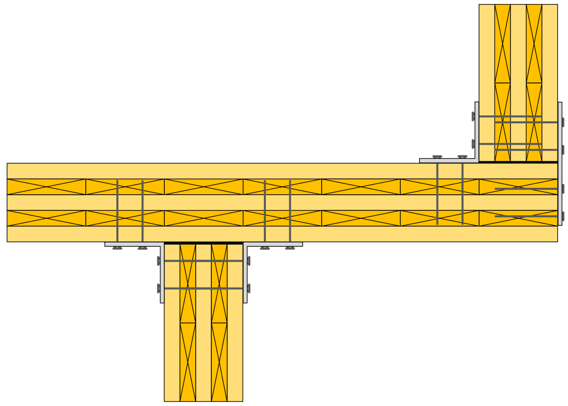
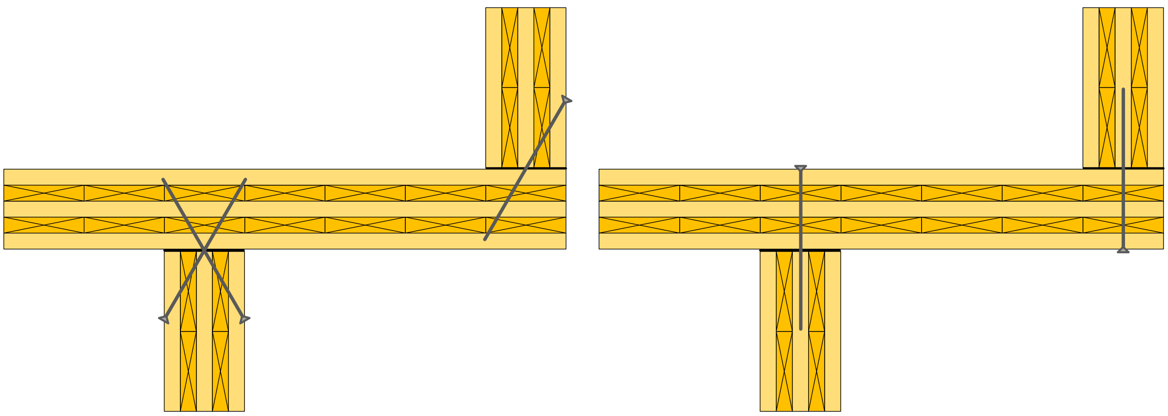


Figure 30 Metal brackets in balcony design



(a) (b)

Figure 31 Self-tapping screws in balcony design

5.2.3.2.2 Supported Balconies

In some cases, the balcony can be attached to the main CLT structure using simple fastenings that allow for easy installation and dismantling (i.e. in case of any potential modification to the configuration of the building in the future). Several buildings in North America and Europe have been constructed with this type of balcony (Figure 32). A combination of metal plates and hinges can be employed to secure the balcony to the main structure at four points as can be seen in Figure 33. The connection is equipped with metal brackets which are attached to the CLT floors at the top and bottom floors using self-tapping screws or lag screws. The balcony is prefabricated on the ground as a rigid box using screws and metal brackets and then lifted and attached to the building using bolts. Other types of fastenings could also be used. The gap between the building and the balcony is closed with cladding materials to maintain the integrity of the building envelope. Flashings should be installed to divert rain water away from the wall and avoid water accumulation.

Alternatively, balconies can be built into the building structure. This concept has been used in the design of the Murray Groove building in London, UK (Figure 34, left side), and the Arbora building complex in Montreal, Canada, where several corner balconies were introduced as part of the main structure. Other concepts involve designing and constructing an external structure (e.g. posts) to support the ends of the balcony, while the back side of the balcony is supported by the main structure. This is also common in low-rise projects that have been built recently in Europe. Additional information about detailing for durability and energy efficiency may be found in Chapter 10 of this Handbook, *Building Enclosure Design and Construction Moisture Management of Cross-Laminated Timber*.

Figure 32 Balcony supported by the main structure

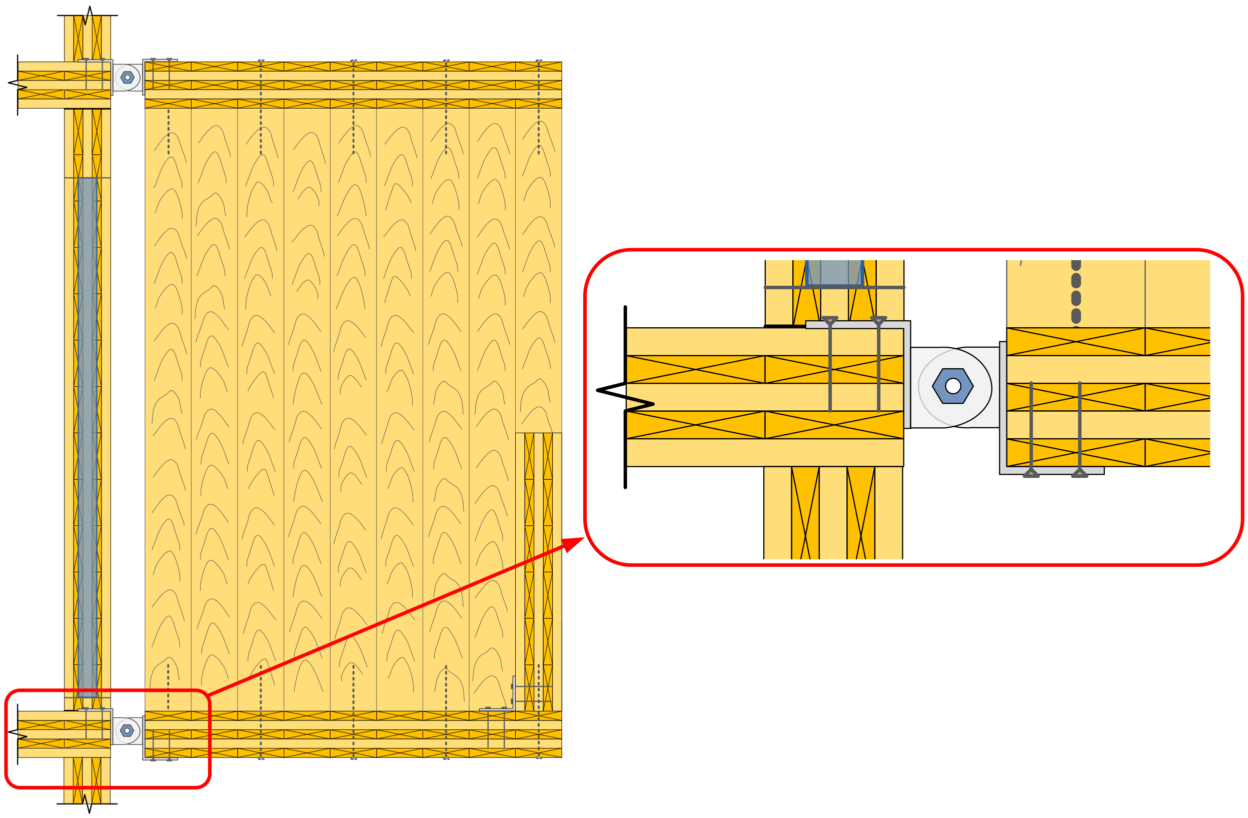


Figure 33 Balcony attached to the platform construction

Figure 34 Examples of European CLT projects with built-in balconies

Another example of balcony design in CLT buildings includes the use of prefabricated concrete slabs supported by metal brackets fastened to the CLT walls. This system was used in the "Condos Origine", a 13-storey CLT building in Quebec City, and was a requirement for the acceptance by the building authorities in Quebec. Four metal brackets were used to support the concrete prefabricated slab as can be seen in Figure 35.

Figure 35 Balconies in the Origine building, Quebec City

5.2.3.3 Balloon Construction

The platform-type of CLT construction in Europe is predominant 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 floor, which is attached to the side of the wall. Mezzanine floors are often located between the ground floor and the first floor but are not unusual in the upper floors of a building. The balloon-type construction is common in design of low-, mid- and high-rise CLT buildings, where shearwalls run continuously from the concrete base to the upper floor (Figure 36).

Figure 36 Metal plates used to ensure continuity of the shearwalls in balloon construction

The simplest floor-to-wall attachment in the balloon-type structure includes a wooden ledger providing a continuous bearing support to the floor panels (Figure 37). The ledgers are typically made of SCL such as LVL, LSL or PSL. Glulam or CLT ledgers could also be used. Another type of attachment is established with the use of metal brackets (Figure 38). The ledgers or brackets are fastened using self-tapping screws or lag screws.

Note that acoustic separation should be considered at the interfaces between the floors and walls.

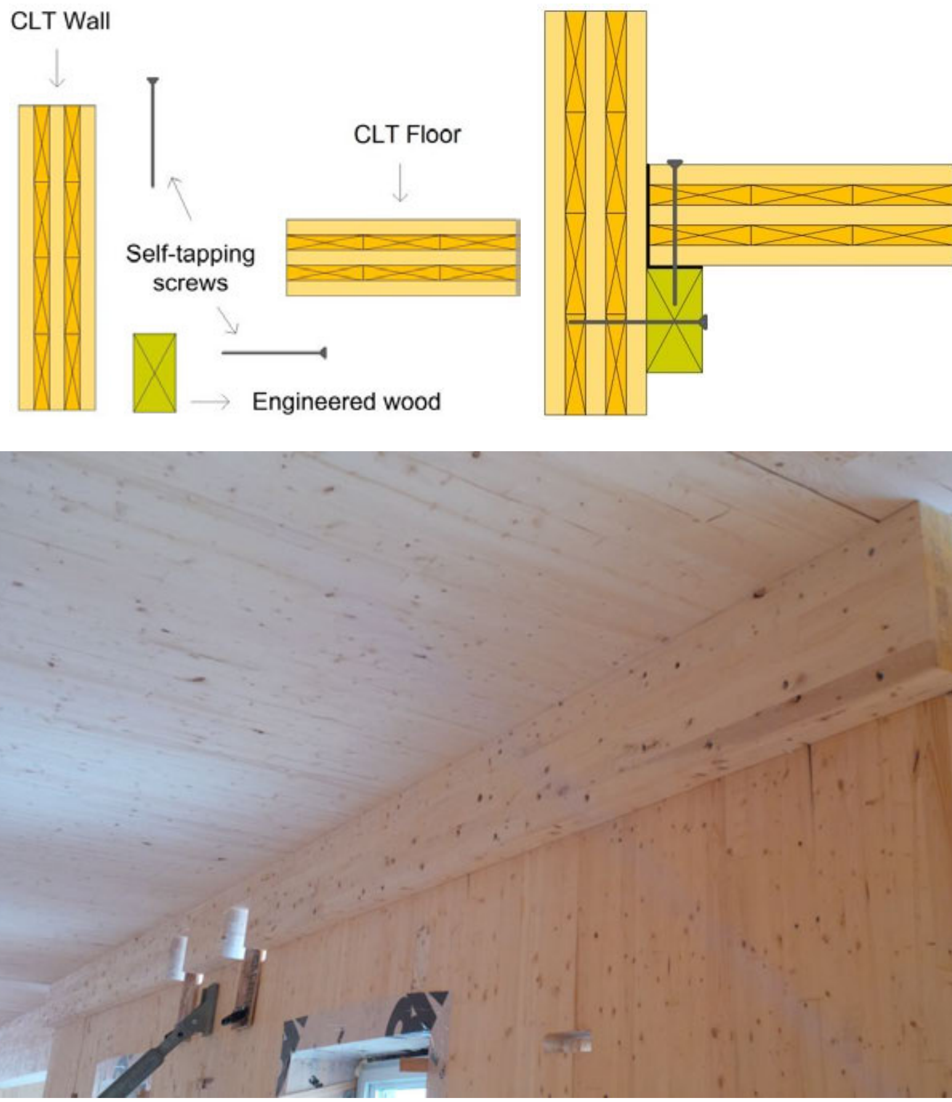


Figure 37 Glulam ledger for floor support

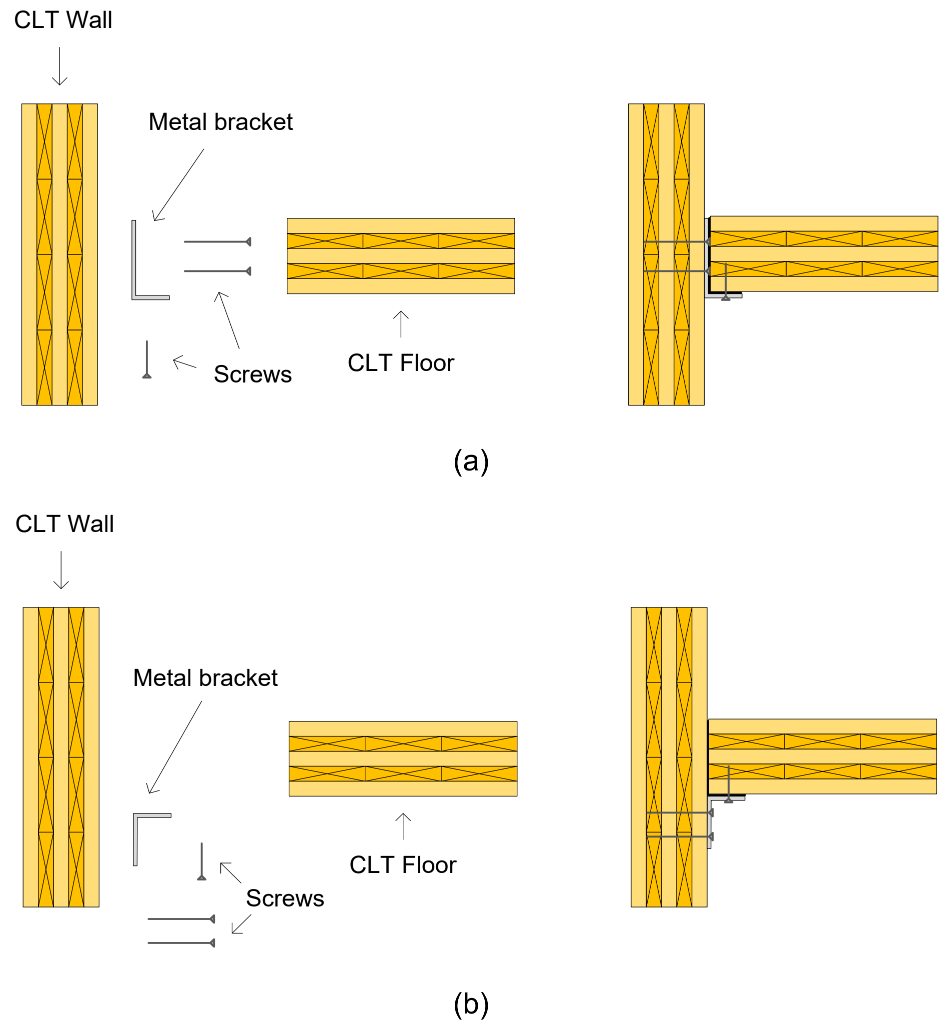


Figure 38 Metal brackets for floor support (adapted from TRADA 2009)

5.2.4 Wall-to-Roof Connections (Detail D)

For walls to sloping or flat roof connections (Figure 39), self-tapping screws and metal brackets are most commonly used similar to those for attaching floors to walls (Figures 40 and 41).

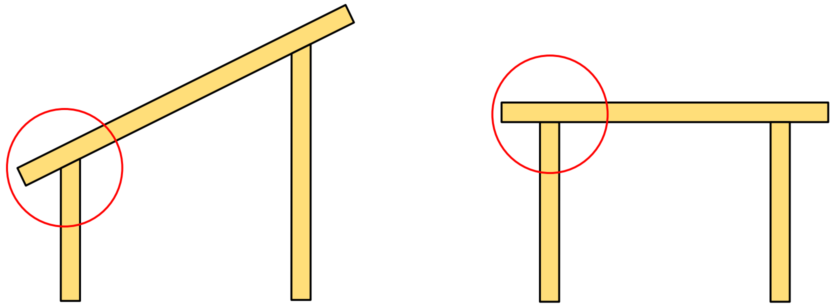
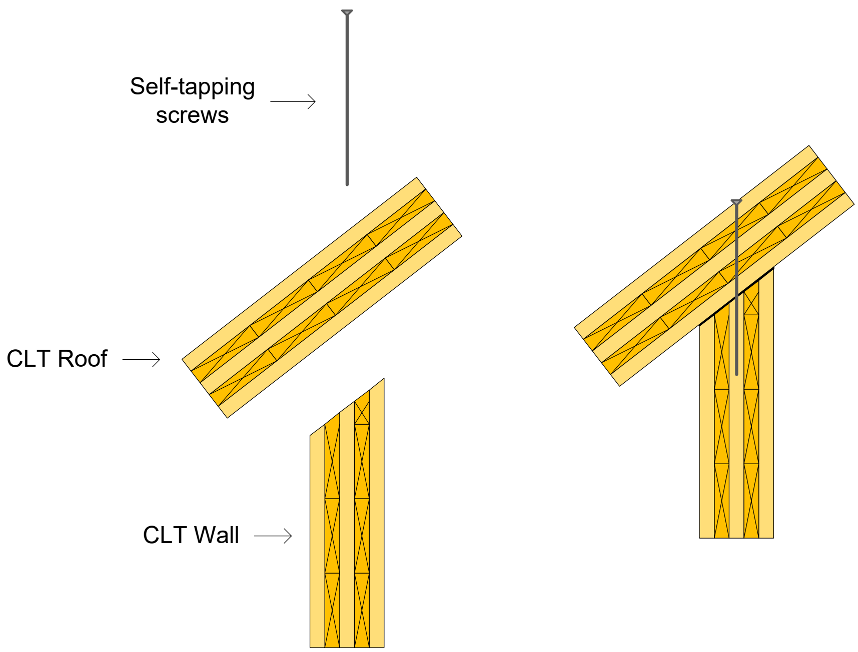


Figure 39 Possible roof to wall joints configurations



(a)

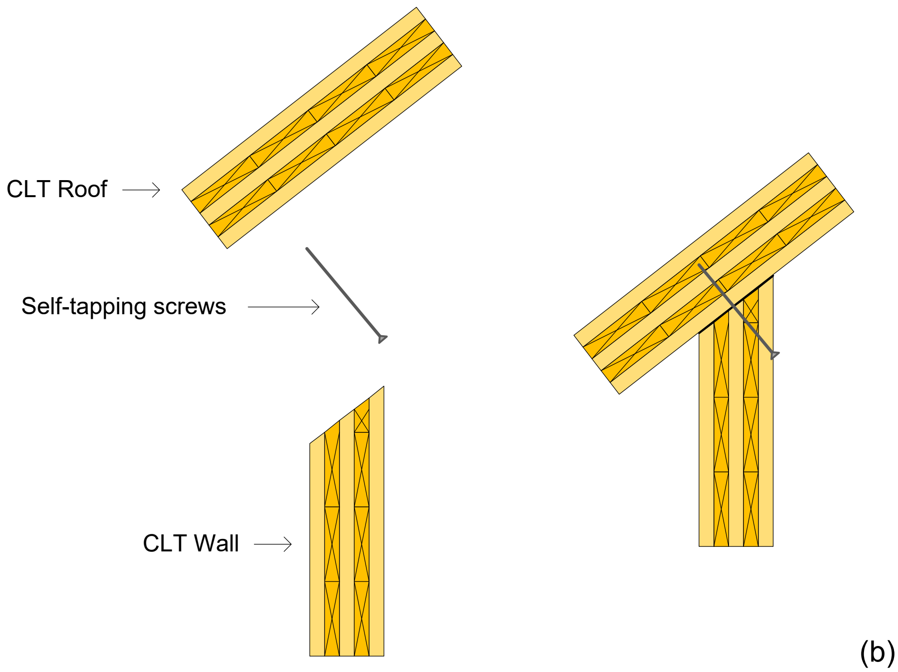


Figure 40 Self-tapping screws

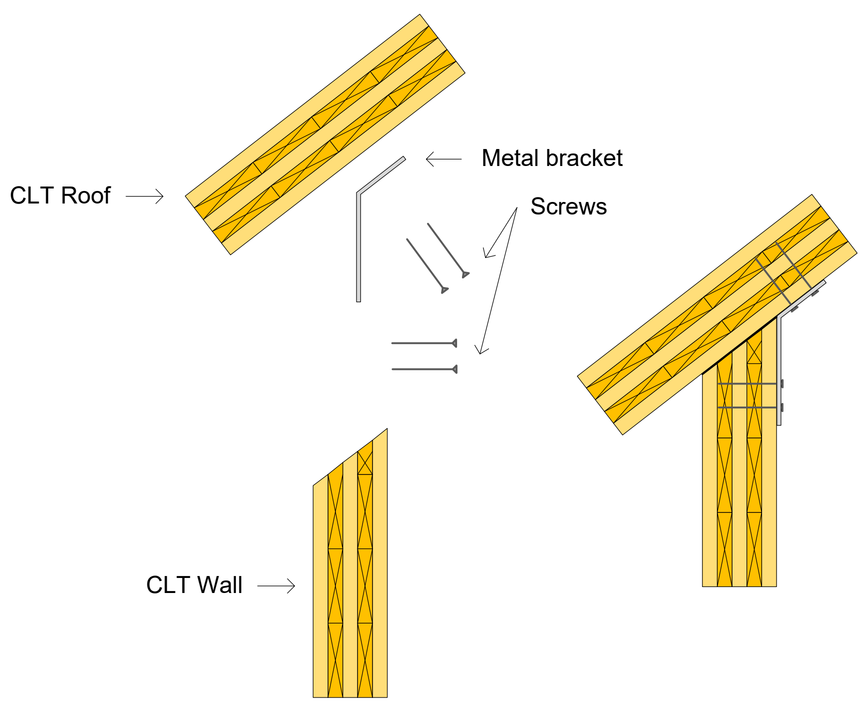


Figure 41 Metal bracket

5.2.5 Wall-to-Foundation Connections (Detail E)

5.2.5.1 Visible/Exposed Plates

Various fastenings, such as exterior metal plates and brackets, are readily available on the market for connecting CLT wall panels to concrete foundations and podiums or to steel beams. Exposed steel brackets, like those shown in Figure 42, are probably the most commonly used in Europe due to their simplicity of installation in buildings with low lateral load demands. Metal plates can be installed from outside, such as shown in Figure 43. The placement of metal plates or brackets depends on the required load capacity and ductility. The type of connection also depends on whether the CLT wall is designed as a shearwall with heavy hold-downs or not and the magnitude of the uplift and base shear loads. Lag screws or powder-actuated fasteners can be used to connect the metal plates and brackets to the concrete foundation/slab, while nails, lag screws or self-drilling screws are used to connect them to the CLT panel.

Note that a damp-proof membrane is recommended to prevent direct contact between CLT and concrete.

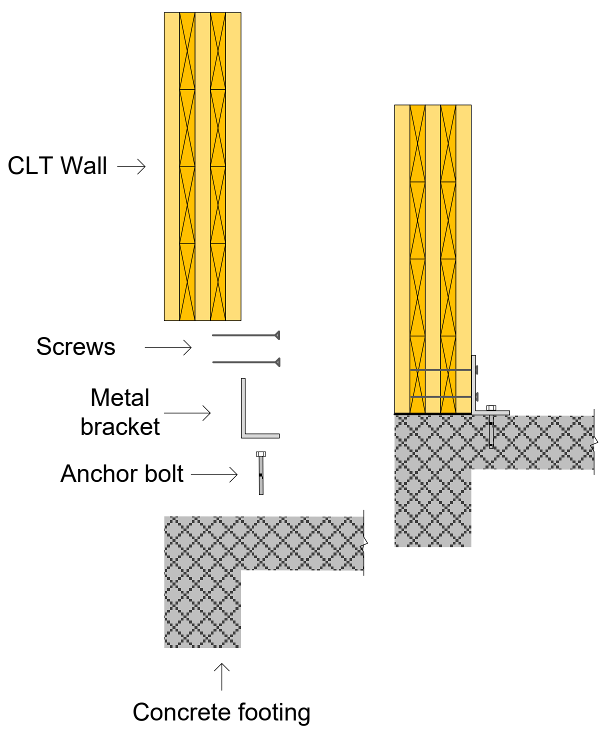


Figure 42 Metal brackets

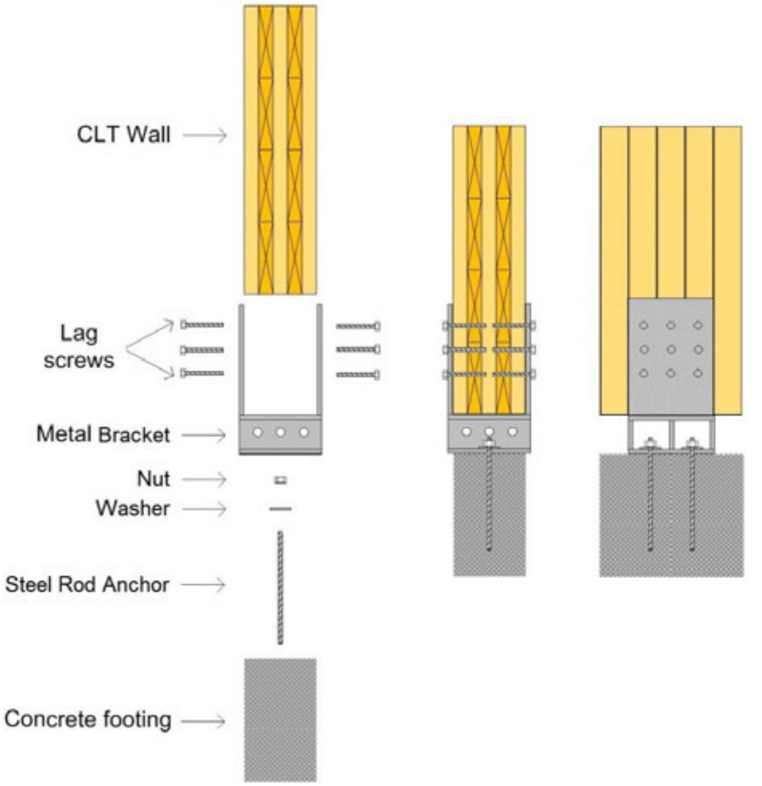


Figure 43 Exterior metal plate

5.2.5.2 Concealed Hardware

To achieve better fire performance and improve aesthetics, designers prefer concealed connections. Hidden metal plates like those shown in Figure 44 can be used, but they require some machining to produce the grooves in the CLT panel to accommodate the metal plates. Tight-fit 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 SFS Intec do not require any predrilling).

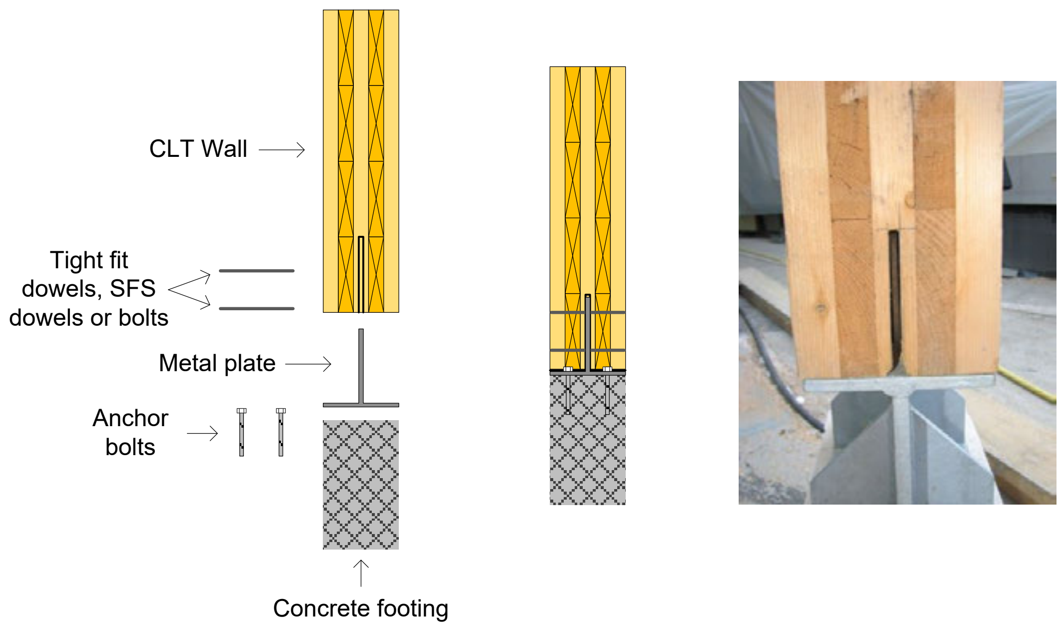


Figure 44 Concealed metal plate

5.3 CONNECTIONS IN MIXED CLT CONSTRUCTION

Mixing CLT with other types of wood-based materials such as glued-laminated timber (glulam) is common. Mixed systems are becoming increasingly popular in North America and Europe to optimize the design by capitalizing on the useful attributes of the various products. Mixing CLT with other types of construction materials such as steel, concrete and masonry or mixing different types of structural forms is also common especially in non-residential and mid- and high-rise CLT buildings.

5.3.1 CLT Mixed with Other Wood-Based Materials and Systems

Mixing CLT with different wood-based materials and structural systems is done to optimize the design and to meet certain performance requirements. Therefore, it is not unusual to combine CLT walls with sawn lumber, glulam, wood I-joists, metal plated wood trusses or other engineered wood elements as the main floor support system, with either wood-based decking such as wood boards or structural panels. The following paragraphs provide examples of CLT connections in mixed structures of platform- and balloon-type construction.

5.3.1.1 Platform Construction

In platform-type construction, the main structural elements of the floor rest on top of the walls at every level. In mixed construction, a typical joisted floor is placed on top of CLT walls, as shown in Figure 45.

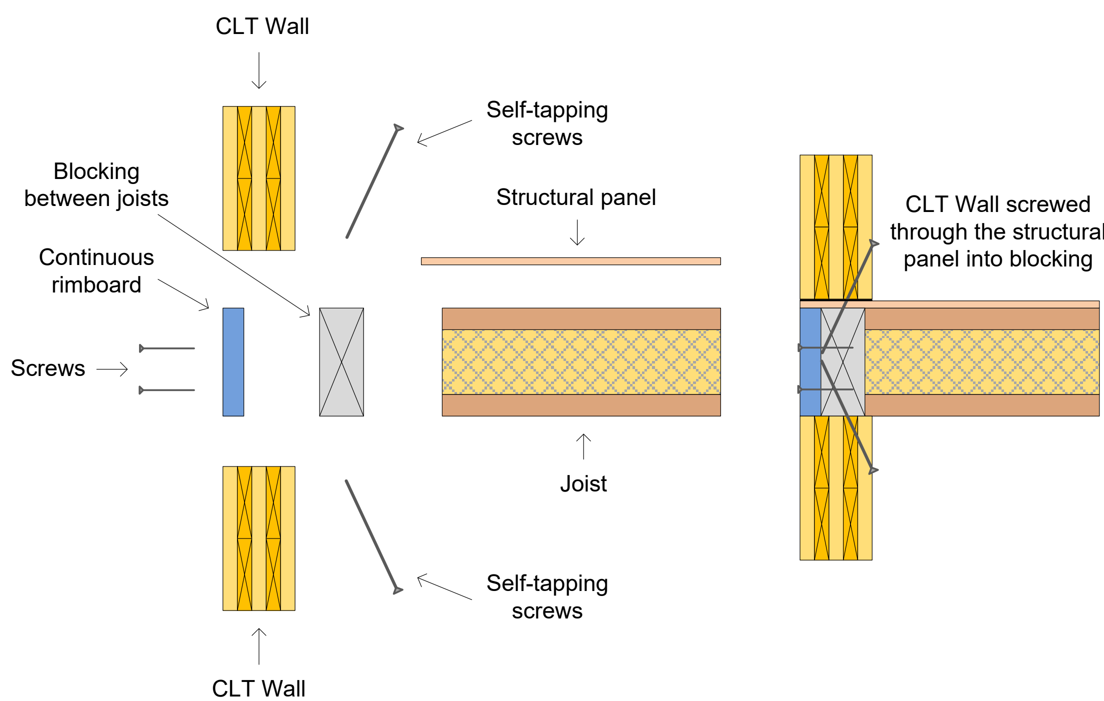


Figure 45 CLT Wall – I-joist (adapted from TRADA 2009)

Sawn lumber, glulam, SCL or wood I-joists could be used as the main structural members supporting the floor. 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 if the upper storey walls are resting completely on the rimboard and the blocking elements. In the case of wood floor trusses, it is necessary to provide wood-based blocking to prevent crushing of truss top chords and to provide a uniform stress distribution along the wall perimeter (Figure 46). The blocking is usually made of SCL for dimensional stability. Self-tapping screws driven at an angle or other metal hardware described above is used for fastening.

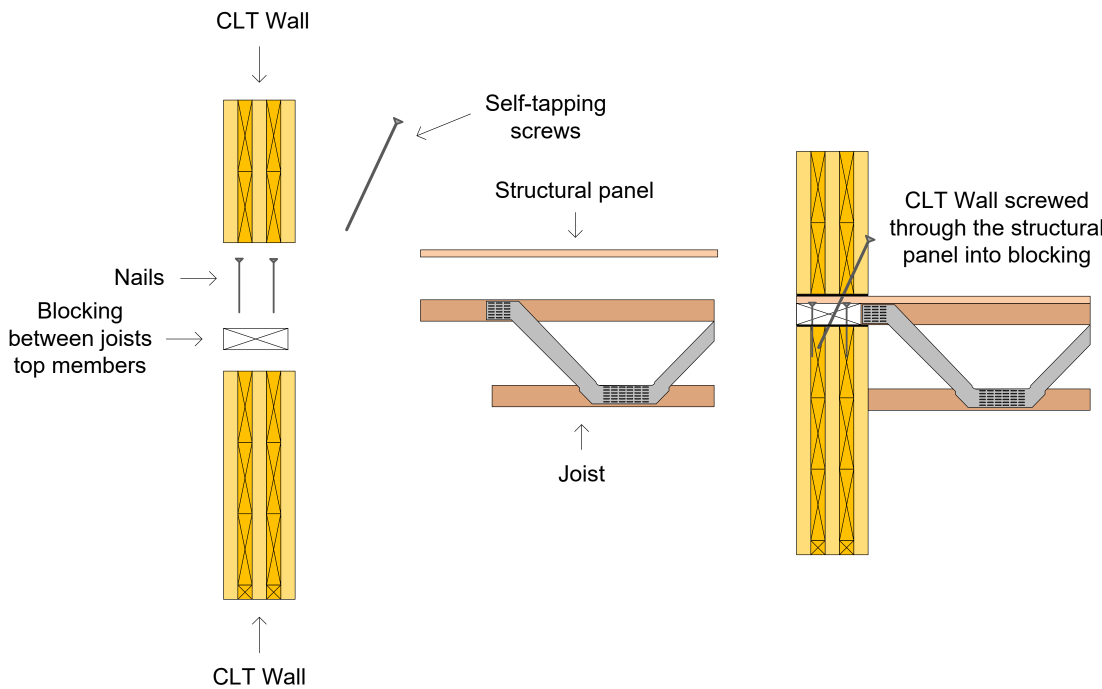


Figure 46 CLT Wall – Engineered floor (adapted from TRADA 2009)

5.3.1.2 Balloon Construction

In balloon-type of construction, wall panels are continuous, and the floor system is attached to the side of the wall. The joists of sawn lumber, glulam, SCL or wood I-joists can be attached to the CLT walls using traditional metal hangers commonly used in light frame and heavy post-and-beam timber construction (Figures 47). Alternatively, EWP ledgers or metal brackets supporting the joists could be attached to the CLT walls. Self-tapping screws and traditional fasteners are used to attach the hardware to the wall.

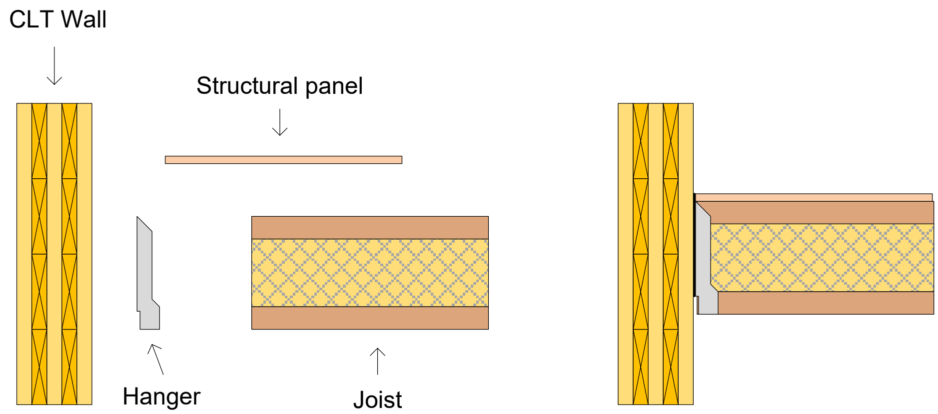


Figure 47 CLT Wall – I-joist (adapted from TRADA 2009)

5.3.2 CLT Mixed with Steel or Concrete Systems

Open-web steel joists with metal decks typically used in floor and roof applications in non-residential construction, can also be mixed with CLT structures (Figure 48). The open-web steel joists are typically supported by the CLT walls through holes in the walls where the top flanges are bearing directly on the walls and fastened with screws or nails. This efficient and cost-effective mixed system has been used in several projects in Canada.

Figure 48 Roof with open-web steel joists supported by CLT walls

5.4 ANALYSIS AND DESIGN OF CLT CONNECTIONS

The design information required for the analysis and design of CLT connections is summarized in Figure 49.

Connections in wood construction can be grouped into two categories: 1) generic connections, and 2) proprietary connections. Generic connections are made using standard fasteners such as bolts and dowels, nails and spikes, timber rivets, lag screws and wood screws which are manufactured according to the standards recognized in Canada. These connections are designed using CSA O86 standard (Engineering design in Wood) and, if needed, other design CSA standards (e.g., CSA S16 for steel hardware). Proprietary connections are made using proprietary fasteners, such as self-tapping screws, and/or other specialty hardware. The design information for proprietary connections is not covered in CSA standards and can be found in the product evaluation reports issued by approved evaluation agencies, such as Canadian Construction Materials Centre (CCMC), or manufacturers' design brochures. Design information for some proprietary connections from research literature is included in Appendix A.

CLT connections resisting gravity and wind loads, and connections required to be non-dissipative under the seismic loads are designed to remain elastic. In this case, only the strength and stiffness of these connections are required for the design. According to CSA O86-14 Update 1 (CSA, 2016) standard, non-dissipative connections under seismic load must resist the force and displacement demands that are induced on them when the energy-dissipative connections reach the 95th percentile of their ultimate resistance or target displacement.

Energy-dissipative connections under the seismic loads must be designed to meet all the following requirements:

(a) connection resistance shall be governed by a yielding failure mode or a proven friction-based resistance;

(b) connections shall be at least moderately ductile in the directions of the assumed rigid body motions of CLT panels; and

(c) connections shall possess sufficient deformation capacity to allow the CLT shearwalls to develop their rocking motion.

Discrete hold-down connections or continuous steel rods are generally used to resist the overturning moment in shearwalls. It is recommended that in the situation where a CLT shearwall is composed of more than one segment, the yield resistance of discrete hold-down connections be 20% greater than the forces developed in them when the connections between CLT segments reach their yielding resistance. Additional details on continuous rod and hold-down design can be found in Chapter 4 of this Handbook (Lateral Design).

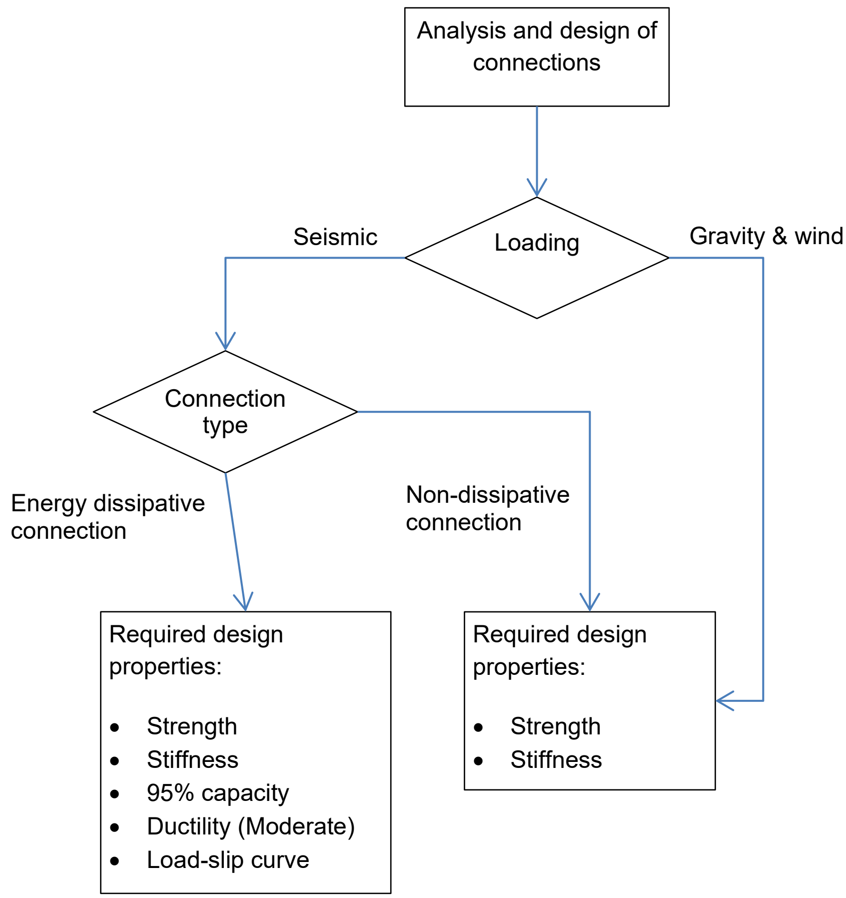


Figure 49 **Design information required for analysis and design of connections**

5.4.1 Lateral Resistance

Comprehensive research on the lateral resistance of dowel-type fasteners installed in the face and in the edge of CLT panels was conducted by Uibel and Blaβ (2006; 2007) with the intent of developing a design methodology. Embedment tests were conducted using different CLT products and fasteners. Empirical models expressed as a function of the fastener diameter, wood density and loading angle relative to the grain direction of the outer layer were developed based on test results to establish the embedment strength under lateral load.

The results of these tests were used to establish the lateral-load resistance of connections with CLT in Europe. In Canada, to simplify the provisions related to connections design in CLT keeping the existing CSA O86 format, it was decided to adjust the embedment strength equations by a factor, J_X, taking into account CLT features such as unglued edges and gaps between laminations. The values of J_X were calibrated to match the results obtained using European equations.

5.4.1.1 Fasteners driven perpendicular to the plane of the CLT panel

For connections with bolts, dowels and lag screws, the embedment strengths for a fastener in the two primary loading directions are as follows (MPa):

$$f_{iP} = 50G(1 - 0.01\, d_F)\, J_X \quad \text{parallel to face grain of CLT} \quad [1]$$

$$f_{iQ} = 22G(1 - 0.01\, d_F) \quad \text{perpendicular to face grain of CLT} \quad [2]$$

where

G = mean relative density

d_F = fastener diameter

J_x = adjustment factor for connections in CLT

= 0.9 for fastener bearing parallel to face grain of CLT

Although J_x could be greater than 1.0 for fastener bearing perpendicular to face grain of CLT to match the results obtained using the European equations, the Technical Committee that is in charge of CSA Standard O86 decided to conservatively use J_x = 1.0.

For connections with nails, spikes and wood screws in CLT, the embedment strength is independent of the grain direction and, like for other wood products, and Equation [1] is used for f_1 and f_2 and Equation [3] is used for the embedment strength of point-side member where failure mode is fastener yielding, f_3 (MPa):

$$f_3 = 110G^{1.8}(1 - 0.01\, d_F)\, J_X \quad [3]$$

where

J_x = adjustment factor for connections in CLT

= 0.9 for CLT

5.4.1.2 Fasteners driven in the panel edge of CLT

In case of the panel edge of CLT, the fasteners are not necessarily inserted in the end grain and even when they are, there is supporting action from the adjacent cross laminations. Nevertheless, the design values for fasteners installed in the panel edge, the following adjustments are applied.

The embedment strength for bolts and dowels in any loading direction is limited to 0.6 f_{iQ} of the fasteners installed perpendicular to the panel face. This would yield largely conservative lateral resistance values in comparison with Uibel and Blaβ (2007).

Due to the lack of test data, the existing end grain factors are used for lag screws installed parallel to grain in the end grain of wood members. When more information becomes available on the performance of lag screws in the panel edge of CLT, further revisions may be proposed.

For nails, spikes and wood screws, the existing end grain factor is used for fasteners installed in the end grain of wood members. This would yield conservative lateral resistance values in comparison with Uibel and Blaβ (2007).

5.4.2 Withdrawal resistance

Withdrawal strength of self-tapping screws inserted perpendicular to the plane of CLT panel and in the panel edge, was investigated by Uibel and Blaβ (2007). The withdrawal resistance was derived from tests on self-tapping screws with diameters ranging from 6 mm to 12 mm. The screws were deliberately installed at the edge joints between laminations to study the effect of gaps on the withdrawal resistance. The withdrawal resistance was expressed as a function of the screw diameter, wood density and the screw length of penetration.

5.4.2.1 Fasteners driven perpendicular to the plane of the CLT panel

In Canada, the withdrawal resistance of threaded fasteners in CLT was derived from tests. Like for the embedment strength equations, the existing basic withdrawal resistance in CSA O86 was adjusted with the factor, J_X. The values of J_X were calibrated to match the results obtained using the European equations.

In CSA O86-14 Update 1 (CSA, 2016), the basic withdrawal resistance of lag screws and wood screws in CLT is as follows (N/mm):

$$y_w = 59\, {d_F}^{0.82}\, G^{1.77}\, J_X \qquad [4]$$

where

J_x = 0.9 for CLT

Comparison indicates that the withdrawal resistance of denser species (D Fir-L) is slightly over predicted, while the predictions for woods of lower density (SPF and Northern Species) are under predicted relative to the European model (Salenikovich and Mohammad, 2015).

5.4.2.2 Fasteners driven in the panel edge of CLT

In CSA O86-14 Update 1 (CSA, 2016), the withdrawal resistance of lag screws installed in the end grain of wood is reduced by the end grain factor J_E = 0.75. Given that the withdrawal resistance of self-tapping screws driven into the panel edge of CLT is two-thirds of those driven into the panel face in the European model, J_E = 0.67 is used for lag screws installed in panel edge of CLT.

According to CSA O86-14 Update 1 (CSA, 2016), nails, spikes and wood screws driven through the end grain shall not be considered to carry load in withdrawal. Where designs rely on withdrawal resistance of fasteners in the panel edge of CLT, precaution shall be taken to ensure that side grain penetration occurs.

5.4.3 Elastic Stiffness

The elastic stiffness of a connection is determined by its load/slip ratio in the elastic region of the load-slip relationship. In CSA O86-14 Update 1 Annex A (CSA, 2016), the load-slip relationships in the elastic region are provided for connections with lag screws, wood screws, nails and spikes. For connections with other types of fasteners such as bolts and dowels, the stiffness values are not provided in CSA O86 and should be determined from tests, modelling or from other available resources (other design codes or research literature).

In Eurocode 5 (CEN, 2004), the elastic stiffness, referred to as slip modulus, is given for the serviceability limit state (SLS), K_{ser}, and the ultimate limit state (ULS), K_u. The stiffness for design at the SLS is taken as the secant modulus of the load-slip curve at a load level of approximately 40% of the maximum load, while the stiffness at the ULS is taken as the secant modulus of the load-slip curve at a load level of approximately 60-70% of the maximum load and is expressed as follows:

$$K_u = \frac{2}{3} K_{ser} \quad [5]$$

Formulas for the slip modulus, K_{ser}, per shear plane per fastener under serviceability limit state for different metal dowel type fasteners and split-ring and shear plate connectors are shown in Table 1. In the formulas, d (mm) is the fastener diameter and ρ_m (kg/m^3) is the mean density of the wood material. Where the connection includes members of different densities, ρ_{m1} and ρ_{m2}, ρ_m is taken as:

$$\rho_m = \sqrt{\rho_{m1} \cdot \rho_{m2}} \quad [6]$$

For steel-to-timber or concrete-to-timber connections, the slip modulus, K_{ser}, should be based on ρ_m for the wood member and may be multiplied by 2.0. These formulas may underestimate the real stiffness of the connections with steel plates ignoring the initial slack and rotation of the fasteners in pre-drilled holes.

The Eurocode 5 formulas adjusted to the CSA O86 format are shown in the last column of Table 1. The adjustment includes conversion from the mean density at 12% MC to the mean relative density on oven-dry mass and volume basis (1000/0.957 = 1045) provided in CSA O86-14 Update 1 (CSA, 2016), Table A.12.1 for calculating resistance of connections at 15% MC.

Table 1 Elastic stiffness of timber-to-timber and wood-based panel-to-timber connections per shear plane per fastener (N/mm)

Fastener type	K_{ser}	K
Dowels Bolts with or without clearance [a] Screws Nails (with pre-drilling)	$\frac{{\rho_m}^{1.5} d}{23}$	$1470 G^{1.5} d$
Nails (without pre-drilling)	$\frac{{\rho_m}^{1.5} d^{0.8}}{30}$	$1125 G^{1.5} d^{0.8}$
Split-ring connectors type A according to EN 912 Shear-plate connector type B according to EN 912	$\frac{\rho_m d_c}{2}$	$520 G d_c$

[a] The clearance should be added separately to the deformation.

Figure 50 illustrates the load-slip curve and stiffness of a typical nail and spike connection at SLS and ULS. In connections with bolts, there will be an initial slip caused by the allowance for an oversize hole up to 2 mm in CSA O86, as shown in Figure 51. If the clearance (tolerance) provided is c (mm), the slip of a connection will be:

$$\Delta = \frac{P}{K} + c \qquad [7]$$

where *P* is the load on the connection, and *K* is the elastic stiffness of the connection calculated without consideration of the initial slip.

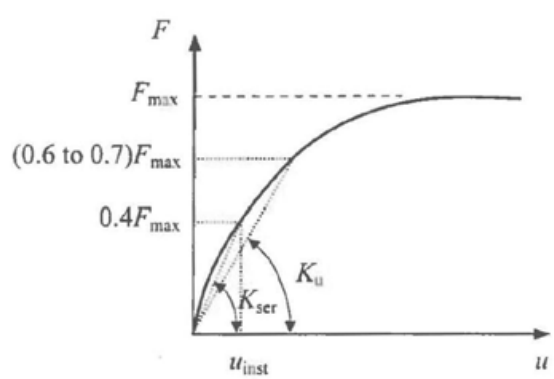


Figure 50 Load-slip curve of a typical nail or spike connection

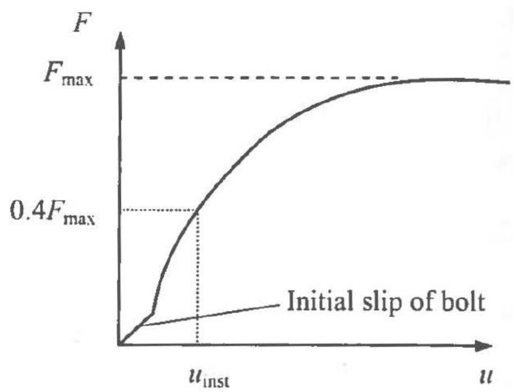


Figure 51 **Load-slip curve of a typical bolted connection**

The elastic stiffness and initial slip of connections are especially important for the prediction of the stress distribution between the members in hyper-static structures. Furthermore, where the connections have different creep behaviour, it will affect the long-term deformation and distribution of forces in the structure, which also need to be addressed in the analysis.

5.4.4 Deformation

In seismic design, engineers must use the connections that possess sufficient deformation capacity to allow the CLT structures to develop their assumed deformation behaviour. For connections that are designed to remain elastic under force and displacement demands, the connection deformation should not be greater than the yield displacement, Δ_y, which can be determined either by testing or by calculation as below:

$$\Delta_y = \frac{P_y}{K} \quad [8]$$

where P_y is the yield strength of the connection, and K is the stiffness of the connection.

The energy-dissipative connections under the seismic load, must be designed to develop yielding failure modes, be at least moderately ductile and to allow the rocking motion of shearwalls before reaching the ultimate displacement. The ultimate displacement of the connection can be determined from the load-slip curve according to ASTM Standard E2126 (ASTM, 2011).

Examples of load-deformation curves and strength and stiffness values for two types of discrete hold-downs and two types of brackets with various fasteners are provided in Appendix A.

5.4.5 Ductility

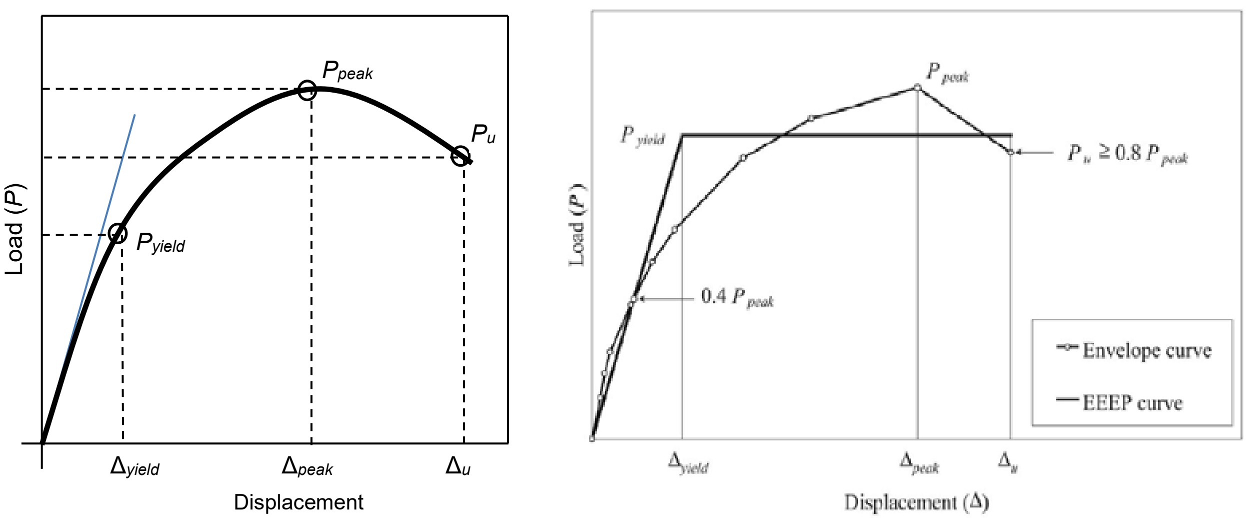
Ductility is the ability of a material to plastically deform under stress without substantial reduction in strength. It can also be applied to connections to express their ability to dissipate energy. It can be expressed as the ductility ratio, which is defined as the ratio of the displacement at the maximum (peak) or failure (ultimate) load to that at the yield load (Figure 52) as follows:

$$\mu = \frac{\Delta_u}{\Delta_{yield}} \quad [9]$$

where

Δ_u = displacement at failure load, P_u

Δ_{yield} = displacement at yield load, P_{yield}

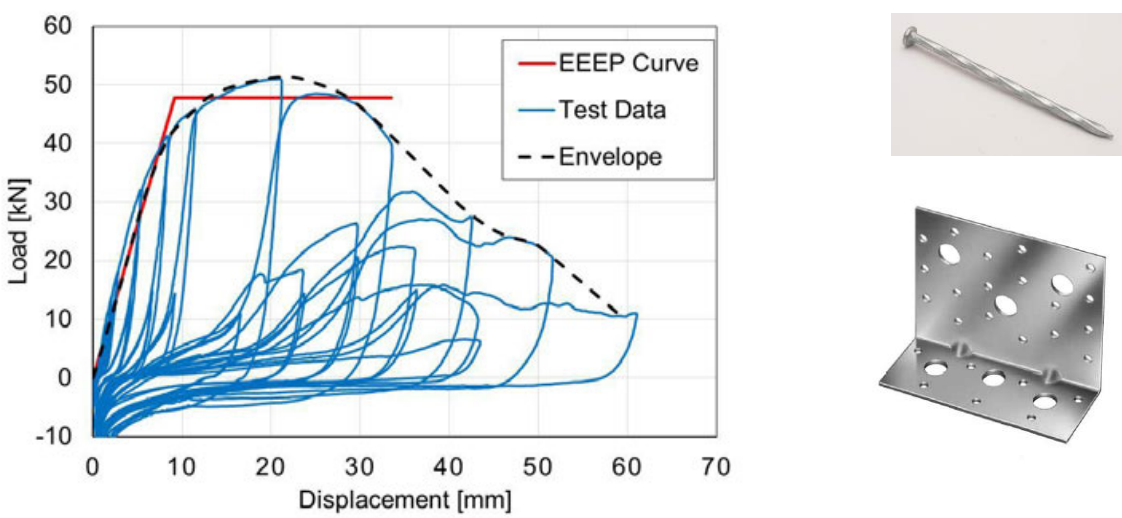


(a) Typical load-displacement curve ***(b) EEEP curve (ASTM E2126)***

Figure 52 Definitions of the ductility ratio

According to CSA O86-14 Update 1 (CSA, 2016), energy dissipative connections in CLT structures must be at least moderately ductile. Connections tested under cyclic loading in accordance with ASTM standard E2126 (ASTM, 2011), and having a minimum ductility ratio of 3.0 determined using the Equivalent Energy Elastic-Plastic (EEEP) methodology as defined in the ASTM E2126, may be considered to be moderately ductile (Figure 52b). Figure 53 shows the hysteresis loop from a non-reversible cyclic test of a typical light-gauge steel bracket connection with 18 spiral nails, loaded in uplift parallel to the grain of the face CLT lamination. Figure 54 shows the hysteresis loop of the same connection in sliding under reversed cyclic load perpendicular to the grain of the face CLT lamination (CWC, 2017).

Connections using steel brackets or steel side plates that fail in fastener yielding modes (d), (e), or (g), where plastic hinges form in the fastener, for nails or wood screws per CSA O86, driven into the face of the CLT panel and loaded parallel or perpendicular-to-grain, may be considered moderately ductile, and assumed to have a ductility ratio of 3.0 (Gavric, 2012; Schneider, 2015; Schneider et al., 2014).



a) b)

Figure 53 (a) Load-displacement curve of a bracket connection in CLT loaded in uplift; (b) the tested bracket with 18 spiral nails d=4.2mm L=89mm

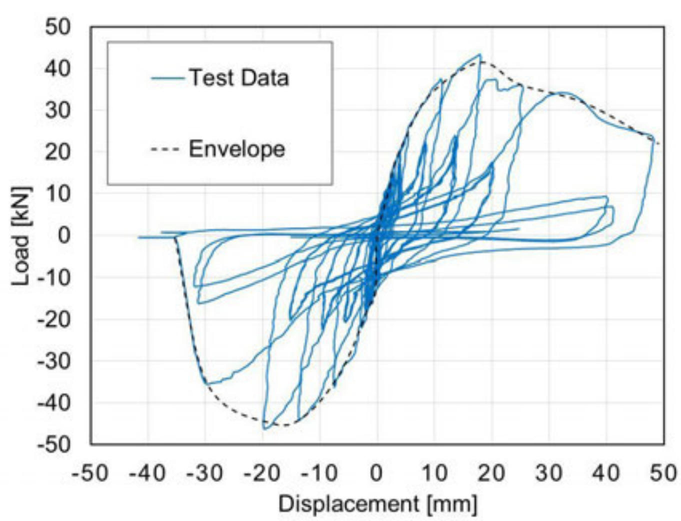


Figure 54 Load-displacement curve of a bracket connection in CLT loaded in sliding (18 spiral nails d=4.2mm L=89mm)

Mild steel dowel-type fasteners such as bolts or dowels driven perpendicular to the face of the CLT panel, that use inserted or side steel plates, and fail in fastener yielding modes (d), (e), or (g), as per CSA O86-14 Update 1 (CSA, 2016), can be considered to be moderately ductile and have a ductility ratio of 3.0 or more if the fasteners have a slenderness ratio $t_2/d_F \geq 10$. In this ratio, t_2 is the CLT member thickness for three-member connections, where fasteners develop one or two plastic hinges per shear plane, or the fastener penetration length into the main member for two-member connections where fasteners develop single or two plastic hinges, while d_F is the diameter of the fastener. In all cases the fastener diameter should be 19mm or less.

Nailed connections with or without splice plates, such as shown in Sections 5.2.1.1 to 5.2.1.3, are also considered moderately ductile. In case of connections with self-tapping screws (e.g. in half-lap joints), fastener manufacturers should be contacted to obtain the performance data.

Using many small diameter fasteners rather than fewer large diameter ones ensures yielding failure modes that include one or two plastic hinges within the fasteners and within each shear plane. Yielding failure modes that rely solely on the embedment failure (crushing) of the timber will result in severe pinching of the hysteretic loops. This is the reason for the requirement that the yielding failure mode (d), (e), or (g) should govern the behaviour.

All other connections should be tested under cyclic load according to the ASTM Standard E2126 to confirm that they satisfy the ductility and displacement capacity requirements. The ductility ratio should be calculated according to ASTM E2126 using the EEEP curve (Figure 54). The strength reduction during the first and the third cycle of testing for the ductile connections should not be more than 20% if the connection is tested using method B of ASTM E2126. Connections comprised of screws that act in withdrawal should be considered non-dissipative. Additional information regarding the cyclic behaviour of typical metal connectors (hold-downs and angle brackets) along with recommendations for better mechanical performance can be found in Gavric et al., 2015.

5.4.6 Placement of fasteners in joints

The minimum spacing requirements for the fasteners installed in panel face of CLT are the same as those in the CSA O86 for other wood products. These requirements are applied conservatively to fasteners driven perpendicular to the plane of CLT, because it is assumed that the cross-lamination tends to reinforce the wood against splitting.

New clauses were introduced in CSA O86-14 Update 1 (CSA, 2016)to provide requirements on the minimum spacing of fasteners installed in the panel edge of CLT. These requirements are intended to limit splitting of wood and are based on the recommendations of Uibel and Blaß (2007) for dowel-type fasteners and European technical approvals (e.g., ETA-11/0030). For bolts, dowels and lag screws in panel edge of CLT, the minimum spacing of fasteners shall be in accordance with Table 2, as shown in Figure 55.

Table 2 Placement of bolts and dowels in the panel edge of CLT*

Symbol	Dimension	Minimum spacing
S_R	Spacing parallel to the load direction	$3d_F$
S_C	Spacing perpendicular to the load direction	$3d_F$
a	End distance	maximum ($4d_F$ or 50 mm)
a_P	Unloaded end distance	maximum ($4d_F$ or 50 mm)
a_L	Loaded end distance	maximum ($4d_F$ or 50 mm)
e	Edge distance	$1.5d_F$
e_P	Unloaded edge distance	$1.5d_F$
e_Q	Loaded edge distance	$5d_F$

* See Figure 55

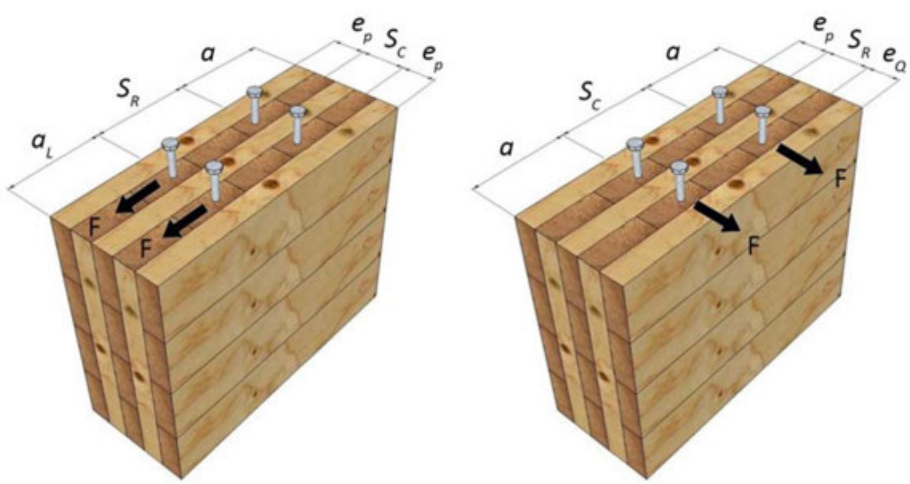


Figure 55 Placement of fasteners in panel edge of CLT

For nails and wood screws installed in panel edge of CLT, the minimum spacing of fasteners shall be in accordance with Table 3, as shown in Figure 55.

Table 3 Placement of nails and spikes in the panel edge of CLT*

Symbol	Dimension	Minimum spacing
S_R	Spacing parallel to the load direction	$10d_F$
S_C	Spacing perpendicular to the load direction	$4d_F$
a	End distance	$7d_F$
a_P	Unloaded end distance	$7d_F$
a_L	Loaded end distance	$12d_F$
e	Edge distance	$3d_F$
e_P	Unloaded edge distance	$3d_F$
e_Q	Loaded edge distance	$6d_F$

* See Figure 55

5.4.7 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 (Chapter 8), sound insulation (Chapter 9), air tightness, durability (Chapter 10), and vibration (Chapter 7). 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 56). To ensure tight fit between individual panels at the construction site, special devices like shown in Figure 57 (i.e., beam grip with ratchet and hooks) have been developed by the various CLT manufacturers to facilitate the on-site assembly of floor and wall panels – see Chapter 12, *Lifting and Handling of Cross-Laminated Elements*, for more details.

Shrinkage and swelling in CLT due to seasonal changes in the ambient environmental conditions need to be considered when designing connections. This is particularly important when other sealant products and membranes are incorporated as the connections 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 considered at the design and detailing stages due to potential shrinkage-induced stresses that could undermine the connection resistance.

Figure 56 Acoustic membrane inserted between walls and floors to provide air tightness (in exterior walls) and improve sound insulation

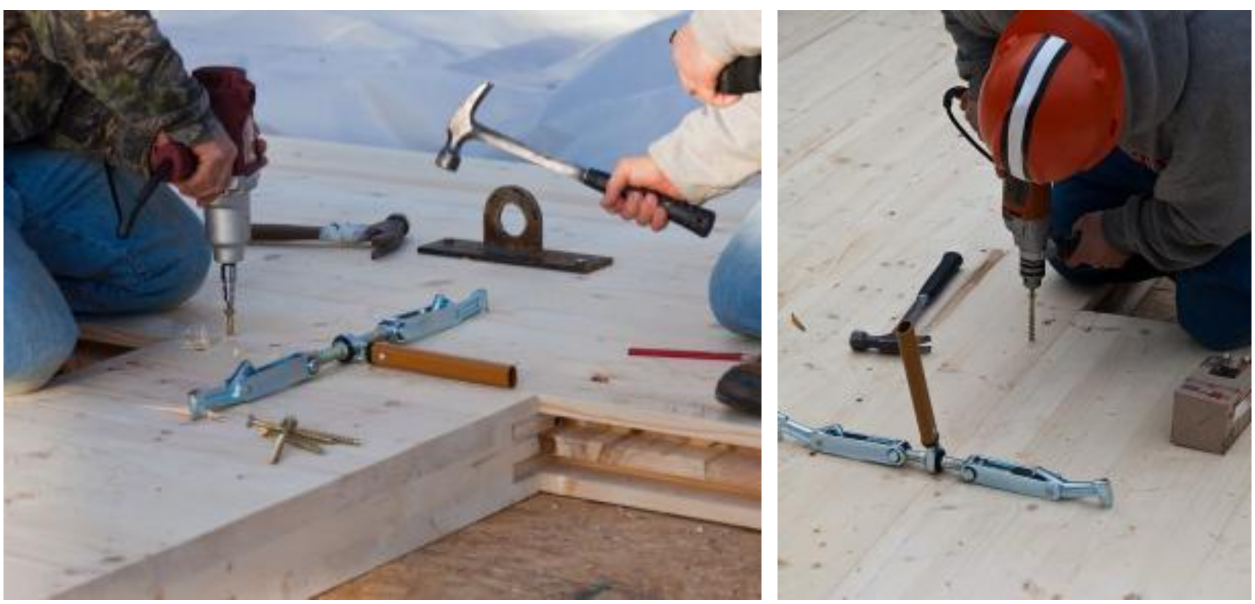


Figure 57 Tight fit between individual panels is ensured using special installation devices (photo courtesy of US WoodWorks)

5.5 DESIGN EXAMPLES

The design examples given in this Section followed the provisions in CSA O86-14 Update 1 (CSA, 2016). In the meanwhile, the 2019 Edition of CSA O86-19 (CSA, 2019) was published. The design examples in this Section conform to both editions of CSA O86.

5.5.1 Example 1: Design of Floor-to-Wall Joint

Materials:

5-ply grade E1 CLT (35 mm x 5 = 175 mm), relative density = 0.42
2 lag screws, nominal diameter = 9.42 mm (0.371”), length = 279 mm (11.0”), thread length = 152 mm (6.0”), tip length = 7.9 mm (5/16”), meeting SAE J429 Grade 1
K_D = 1.0, K_{SF} = 1.0, K_T = 1.0

Lateral Resistance:

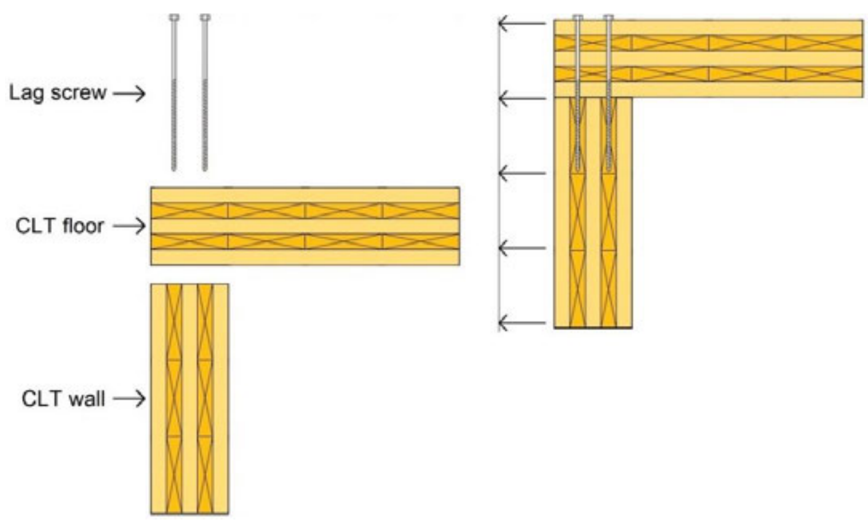


Figure 58 Lateral resistance of floor-to-wall joint

In this example, the grain orientation of the side member is parallel to the face-grain of the CLT; therefore, loading of the lag screw at the shear plane will be parallel to grain. For main member, the lag screw is driven to the panel edge of CLT.

From Clause 12.6.6 of CSA O86-14 Update 1 (CSA, 2016):

t_1 = 175 mm
f_1 = $50G\,(1\text{-}0.01d_F)\,J_x = 50 \times 0.42 \times (1 - 0.01 \times 9.42) \times 0.9 = 17.1$ MPa
t_2 = $(279 – 7.9) – 175 = 96.1$ mm
f_2 = $22G\,(1\text{-}0.01d_F)\,(2/3) = 22 \times 0.42 \times (1 - 0.01 \times 9.42) \times (2/3) = 5.6$ MPa
d_F = 9.42 mm
f_y = 310 MPa

The unit lateral strength resistance will be taken as the smallest value determined as follows:

Mode *a*: $f_1 d_F t_1$ = 28.22 kN

Mode *b*: $f_2 d_F t_2$ = 5.05 kN

Mode *c*: $f_1 d_F{}^2 \left(\sqrt{\frac{1}{6} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_1}{d_F} \right)$ = 6.95 kN

Mode *d*: $f_1 d_F{}^2 \left(\sqrt{\frac{1}{6} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_2}{d_F} \right)$ = 4.41 kN

Mode *e*: $f_1 d_F{}^2 \frac{1}{5} \left(\frac{t_1}{d_F} + \frac{f_2}{f_1} \frac{t_2}{d_F} \right)$ = 6.65 kN

Mode *f*: $f_1 d_F{}^2 \left(\sqrt{\frac{2}{3} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}} \right)$ = 2.62 kN

$P_u = p_u(K_D K_{SF} K_T)$ = 2.62 × (1 × 1 × 1) = 2.62
n_F = 2
J_G = 1.0, based on Table 12.2.2.3.4A
J_{PL} = 1.0, as the penetration length is greater than 75.36 mm (8d_F)

The factored lateral resistance of the lag screw joints is obtained as follows:

$P_r = \phi P_u n_F J_G J_{PL}$ = 0.6 × 2.62 × 2 × 1 × 1 = 3.14 kN

Withdrawal Resistance:

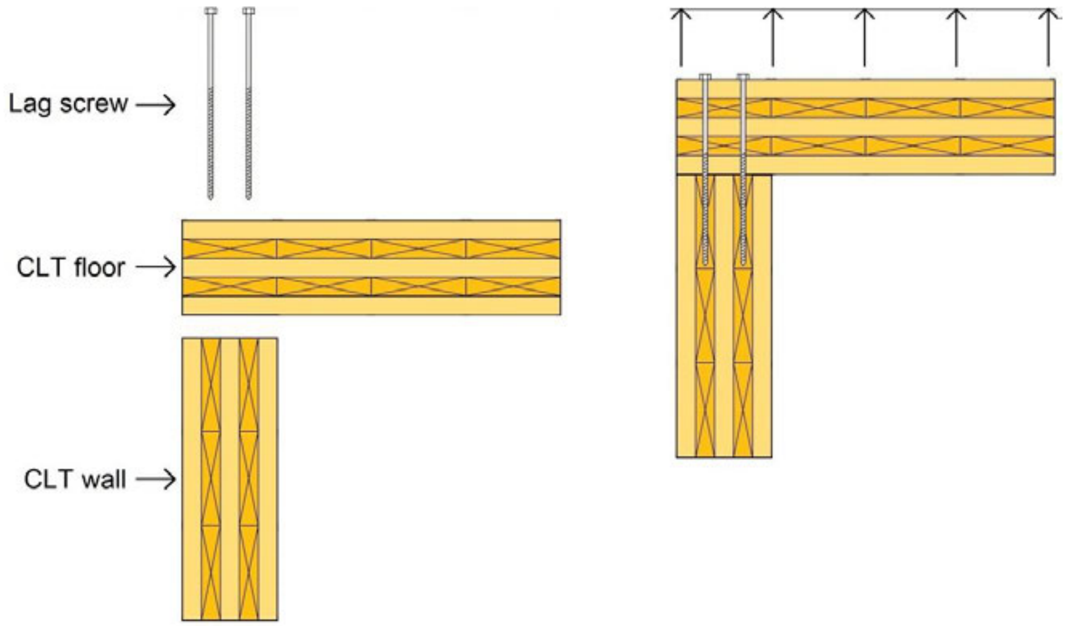


Figure 59 Withdrawal resistance of floor-to-wall joint

From Clause 12.6.5 of CSA O86-14 Update 1 (CSA, 2016):

L_t = (279 – 7.9) – 175 = 96.1 mm
J_E = 0.67, lag screw driven in panel edge of CLT
$y_w = 59 d_F^{0.82} G^{1.77} J_x = 59 \times 9.42^{0.82} \times 0.42^{1.77} \times 0.9 = 71.94$ N/mm

The factored withdrawal resistance of the lag screw joint is obtained as follows:

$P_{rw} = \phi Y_w L_t n_F J_E = 0.6 \times 71.94 \times 96.1 \times 2 \times 0.67 = 5.56$ kN

5.5.2 Example 2: Design of Metal Bracket Joint

<u>Materials:</u>

3-ply grade E1 CLT (35 mm x 3 = 105 mm), relative density = 0.42.
Metal bracket, thickness = 6 mm, CSA G40.21 steel, Grades 300W and 350W, f_u = 450 MPa
6 @ 20d common nails (4.88 mm x 102 mm)
Design for standard-term loading, K_D = 1.0
K_{SF} = 1.0, K_T = 1.0

Note that the design for the nailed connection in this example is focused on the wall connection. The nailed connection between the metal bracket and the CLT floor follows almost the same procedure.

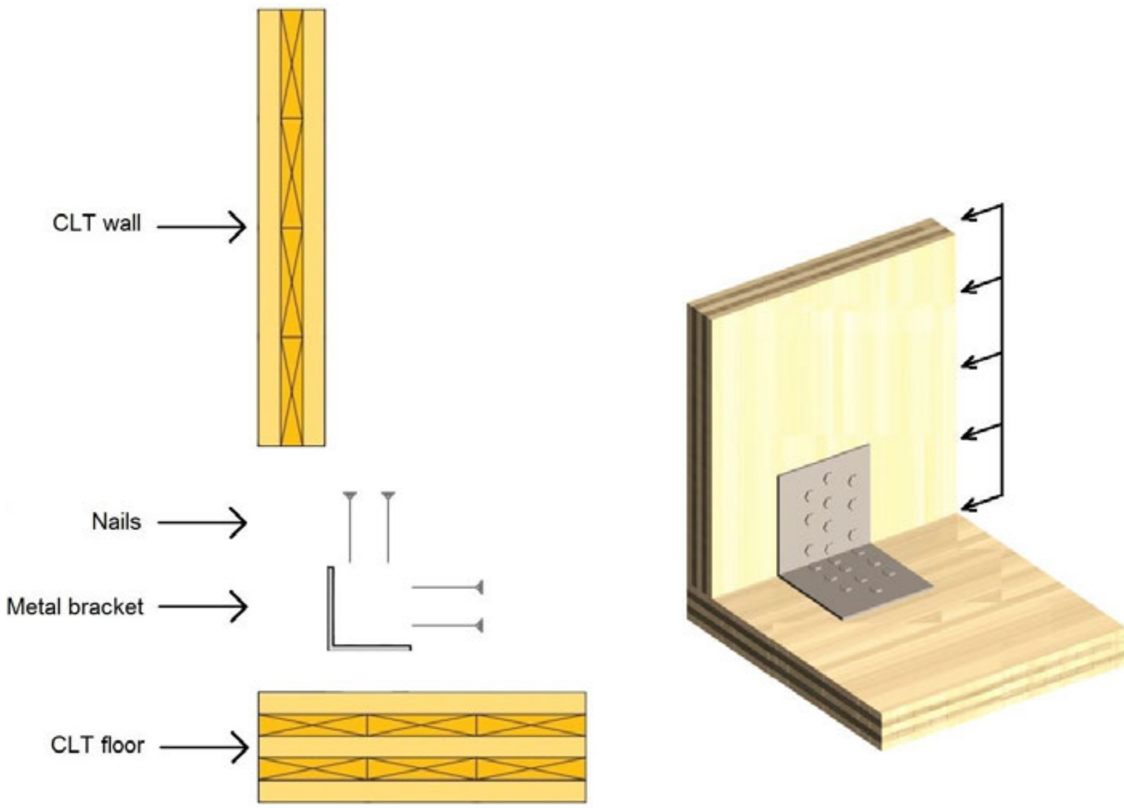


Figure 60 Metal bracket joint

Lateral Resistance:

In this example, the lateral resistance of metal bracket joint to CLT wall is calculated. Based on Figure 60, the grain orientation at the joint is perpendicular to the face-grain of the CLT wall; therefore, loading of the nail at the shear plane will be perpendicular to grain.

From Clause 12.9.4 of CSA O86-14 Update 1 (CSA, 2016):

t_1 = 6 mm
$f_1 = K_{sp}\ (\phi_{steel}/\phi_{wood})\ f_u$ = 3.0 × (0.8 / 0.8) × 450 = 1350 MPa
t_2 = 96 mm
$f_2 = 50G\ (1\text{-}0.01d_F)\ J_x$ = 50 × 0.42 × (1 - 0.01 × 4.88) × 0.9 = 18 MPa
$f_3 = 110G^{1.8}\ (1\text{-}0.01d_F)\ J_x$ = 50 × 0.42 × (1 - 0.01 × 4.88) × 0.9 = 19.8 MPa
d_F = 4.88 mm
$f_y = 50\ (16\text{-}d_F)$ = 50 × (16 – 4.88) = 556 MPa

The unit lateral strength resistance will be taken as the smallest value determined as follows:

Mode a: $f_1 d_F t_1$ = 39.53 kN

Mode b: $f_2 d_F t_2$ = 8.42 kN

Mode d: $f_1 d_F{}^2 \left(\sqrt{\frac{1}{6} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_1}{d_F} \right)$ = 8.92 kN

Mode e: $f_1 d_F{}^2 \left(\sqrt{\frac{1}{6} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_2}{d_F} \right)$ = 127.5 kN

Mode f: $f_1 d_F{}^2 \frac{1}{5} \left(\frac{t_1}{d_F} + \frac{f_2}{f_1} \frac{t_2}{d_F} \right)$ = 9.59 kN

Mode g: $f_1 d_F{}^2 \left(\sqrt{\frac{2}{3} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}} \right)$ = 2.02 kN

$N_u = n_u(K_D K_{SF} K_T)$ = 2.02 × (1 × 1 × 1) = 2.02
n_F = 6
n_s = 1
$J_F = J_E J_A J_B J_D$ = 1 × 1 × 1 × 1 = 1

The factored lateral strength resistance of the nailed metal bracket joint in the CLT wall is obtained as follows:

$N_r = \phi N_u n_F n_s J_F$ = 0.8 × 2.02 × 6 × 1 × 1 = 9.70 kN.

5.5.3 Example 3: Design of Half-Lap Joint

Materials:

3-ply grade E1 CLT (35 mm x 3 = 105 mm), relative density = 0.42
Gauge 12 wood screw, spaced @ 200 mm on center, diameter = 5.48 mm, length = 102 mm, f_y = 550 MPa

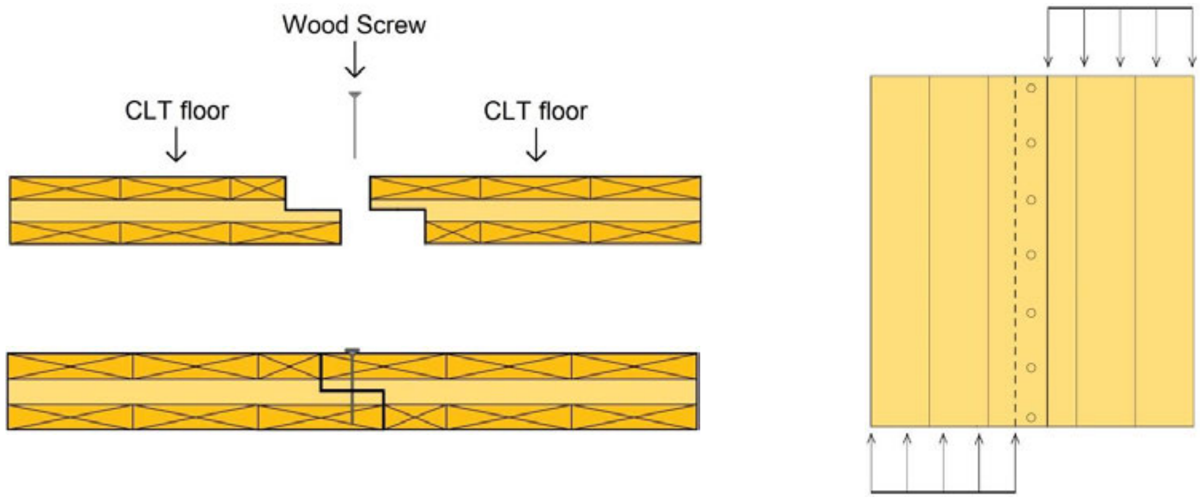


Figure 61 Half-lapped CLT joint

Lateral Resistance:

In this example, the grain orientation at the joint is parallel to the face grain of the CLT; therefore, loading of the wood screw at the shear plane will be parallel to grain.

From Clause 12.11.4 of CSA O86-14 Update 1 (CSA, 2016):

d_F = 5.48 mm
t_1 = 52.5 mm
$f_1 = 50G\,(1\text{-}0.01d_F)\,J_x = 50 \times 0.42 \times (1 - 0.01 \times 5.48) \times 0.9 = 17.9$ MPa
t_2 = 49.5 mm
$f_2 = 50G\,(1\text{-}0.01d_F)\,J_x = 50 \times 0.42 \times (1 - 0.01 \times 5.48) \times 0.9 = 17.9$ MPa
$f_3 = 110G^{1.8}\,(1\text{-}0.01d_F)\,J_x = 110 \times 0.42^{1.8} \times (1 - 0.01 \times 5.48) \times 0.9 = 19.6$ MPa
f_y = 550 MPa

The unit lateral strength resistance will be taken as the smallest value determined as follows:

Mode *a*: $f_1 d_F t_1$ = 5.14 kN

Mode *b*: $f_2 d_F t_2$ = 4.85 kN

Mode *d*: $f_1 d_F{}^2 \left(\sqrt{\frac{1}{6} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_1}{d_F} \right)$ = 1.91 kN

Mode *e*: $f_1 d_F{}^2 \left(\sqrt{\frac{1}{6} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}} + \frac{1}{5} \frac{t_2}{d_F} \right)$ = 1.85 kN

Mode *f*: $f_1 d_F{}^2 \frac{1}{5} \left(\frac{t_1}{d_F} + \frac{f_2}{f_1} \frac{t_2}{d_F} \right)$ = 2.0 kN

Mode *g*: $f_1 d_F{}^2 \left(\sqrt{\frac{2}{3} \frac{f_2}{(f_1 + f_2)} \frac{f_y}{f_1}} \right)$ = 1.76 kN

$N_u = n_u(K_D K_{SF} K_T)$ = 1.76 × (1 × 1 × 1) = 1.76
n_F = 1 / 0.2 = 5 per meter
n_s = 1
J_A = 1
J_E = 1

The factored lateral strength resistance of the wood screw joint is obtained as follows:

$N_r = \phi N_u n_F n_s J_A J_E$ = 0.8 × 1.76 × 5 × 1 × 1 × 1 = 7.04 kN / m

5.5.4 Additional Design Examples

Chapter 13 of this Handbook contains design example of an 8-storey mass timber building. This example includes the designs of CLT floor-panel-to-beam connection, drawings of glulam-beam-to-column and column-to-column connections, designs of glulam-column-to-concrete connection, shearwall hold-down and shear connectors, floor-panel-to shearwall panel below connection, floor-panel-to-shearwall panel above connection, shearwall-panel-to-panel vertical connection, and floor-panel-to-panel connection in that building.

Additional design examples and the selection tables are provided in the 2017 Edition of the Wood Design Manual (CWC, 2017) for the following connections:

1) Selection tables for a side splice plate (D Fir-L plywood, CSP, OSB and steel plate) in single shear with CLT as the main member with nails and spikes;
2) Design example and selection tables for a CLT spline (D Fir-L plywood, CSP, OSB, and steel plate) connection with wood screws;
3) Design example and selection tables for single and double shear CLT connections with steel plates and with bolts and dowels;
4) Design example and selection tables for CLT half-lapped joints with lag screws; and
5) Selection table for butt-connected CLT members with a steel plate and lag screws.

5.6 CONCLUSION

Connections play an important role in maintaining the integrity of the structure and in providing strength, stiffness, stability and ductility. Consequently, they require detailed attention by designers. Traditional and innovative connections used in CLT structures in Europe and North America are presented in this Chapter.

Researchers in Europe developed design procedures for connections in CLT, including dowels, wood screws and nails which are commonly used. The design procedures presented in this Handbook are primarily based on the European experience and are in alignment with the current design provisions in the Canadian wood design standard (CSA O86) with some exceptions that were recommended based on the most recent research findings.

The European equations for CLT were used to establish the lateral and withdrawal resistance of connections with CLT in CSA O86. To simplify the revisions related to the design of connections in CLT and keep the existing CSA O86 format, the embedment strength and basic withdrawal resistance equations have been adjusted with factor, J_X, considering differences in performance due to CLT features such as unglued edges and gaps between laminations. The values of J_X were calibrated to match the European equations conservatively.

Due to the reinforcing effect of cross-lamination in CLT, the minimum spacing requirements of fasteners specified in CSA O86 for solid timber and glulam can be safely applied to the fasteners installed in the face of CLT. New requirements were introduced in CSA O86 on the minimum spacing of fasteners installed in the panel edge of CLT. These requirements are intended to limit splitting of wood and are based on the European recommendations for dowel-type fasteners.

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APPENDIX A

Strength, Stiffness, and Deformation of Selected Connections

5A.1 AVERAGE EMC OF WOOD UNDER DIFFERENT TEMPERATURE AND RELATIVE HUMIDITY CONDITIONS (DATA FROM U.S. FPL, 2010)

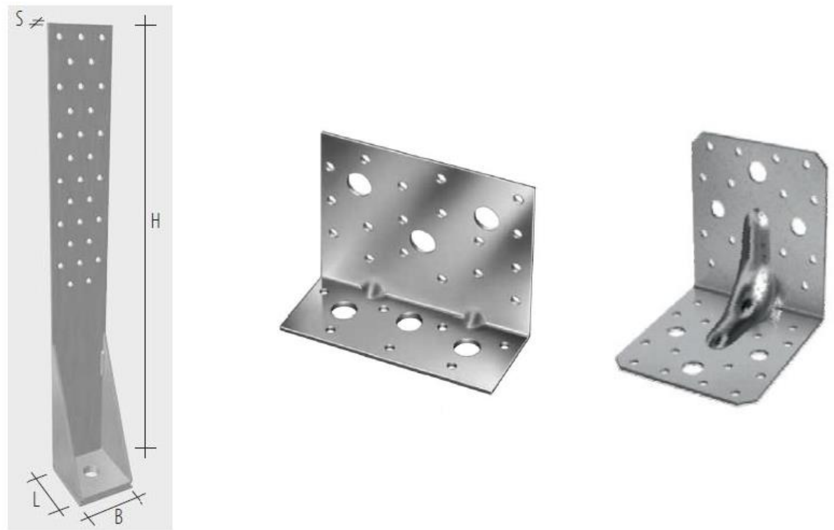
Table 5A.1 Strength, stiffness and deformability data obtained from tests on hold-down and brackets loaded in uplift (Gavric, 2012)

Tension Tests Parallel to grain of the Face Lamina									
Connector (Figure 5A.1)	**Fastener (Figure 5A.2)**	**Value type**	**Values from the EEEP Curve**						
			P_{peak} [kN]	**d_{peak} [mm]**	**P_y [kN]**	**d_y [mm]**	**d_u [mm]**	**μ**	**K_e [kN/mm]**
HD HTT 22 *	12-RN 4x60mm	Average	47.8	20.8	39.8	8.3	24.5	3.0	4.9
		CV [%]	6		8				18
HD HTT 16 *	9-RN 4x60mm	Average	36.2	21.4	29.9	10.2	22.5	2.2	3.0
		CV [%]	5		6				17
Bracket BMF 90x116x48x3mm**	11-RN 4x60mm	Average	23.5	17.7	19.8	7.5	22.5	3.0	2.7
		CV [%]	4		7				10
Bracket BMF 100x100x90x3mm***	8-RN 4x60mm	Average	12.6	7.4	11.2	4.7	22.4	4.8	2.4
		CV [%]	8		8				16

*These Simpson Strong Tie hold-downs may be superseded by newer products such as HTT5HDG.
** This BMF bracket is similar to Simpson Strong Tie bracket ABR 105.
*** This BMF bracket is identical to Simpson Strong Tie bracket AE 116.
Note: RN = annular ring nail.

where:

P_{peak} = maximum load
d_{peak} = displacement at maximum load
d_y = yield displacement
P_y = yield load
d_u = ultimate displacement
μ = ductility ratio
K_e = elastic stiffness.



(a) (b) (c)

Figure 5A.1 Connectors used in the study: (a) HTT 22 Hold-down connector; (b) BMF 90×116×48×3 angle bracket; (c) BMF 100×100×90×3mm angle bracket

Table 5A.2 Strength, stiffness and deformability data obtained from tests on brackets loaded in shear (Gavric, 2012)

Shear Tests Perpendicular to grain of the CLT Face Lamina										
Connector (Figure 5A.1)	**Fastener (Figure 5A.2)**	**Value type**	**Values from the EEEP Curve**							
			P_{peak} [kN]	d_{peak} [mm]	P_y [kN]	d_y [mm]	d_u [mm]	μ	K_e [kN/mm]
Bracket BMF 90x116x48x3mm*	11-RN 4x60mm	Average	26.8	27.7	24.3	13.6	40.3	3.0	1.8
		CV [%]	3		2				14
Bracket BMF 100x100x90x3mm**	8-RN 4x60mm	Average	19.9	29.7	17.3	13.4	47.1	3.5	1.3
		CV [%]	7		7				12

* This BMF bracket is identical to Simpson Strong Tie bracket AE 116.
** This BMF bracket is similar to Simpson Strong Tie bracket ABR 105.
Note: RN = annular ring nails.

where:

P_{peak} = maximum load
d_{peak} = displacement at maximum load
d_y = yield displacement
P_y = yield load
d_u = ultimate displacement
μ = ductility ratio
K_e = elastic stiffness.

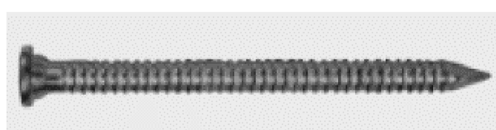


Figure 5A.2 Annular ring nail 4 x 60 mm used in the study

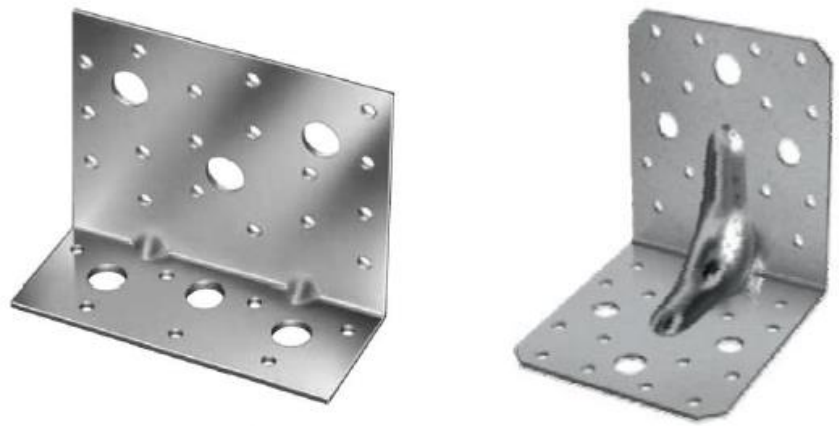
Table 5A.3 Strength, stiffness and deformability data obtained from tests on hold-down and brackets loaded in uplift (Schneider, 2015)

Tension Tests Parallel to grain of the CLT Face Lamina										
Connector (Figure 5A.3)	**Fastener (Figure 5A.4)**	**Value type**	**Values from the EEEP Curve**							
			P_{peak} [kN]	**d_{peak} [mm]**	**P_y [kN]**	**d_y [mm]**	**d_u [mm]**	**μ**	**K_e [kN/mm]**
Bracket BMF 100x100x90x3mm*	10-SN 4.2x89mm	Average	26.2	25.8	23.2	6.5	35.6	5.4	3.7
		CV [%]	14		14				19
Bracket BMF 90x116x48x3mm**	12-RN 3.8x76mm	Average	42.1	17.4	35.8	5.0	26.8	5.4	7.7
		CV [%]	8		8				31
Bracket BMF 90x116x48x3mm**	12-RN 4.2x60mm	Average	34.8	13.6	30.9	4.6	24.4	5.6	7.2
		CV [%]	3		2				30
Bracket BMF 90x116x48x3mm**	9-SFS 5x90mm	Average	46.0	19.9	39.7	7.9	27.3	3.5	5.1
		CV [%]	11		11				10
Bracket BMF 90x116x48x3mm**	18-SFS 4x70mm	Average	48.2	18.5	40.6	7.7	25.3	3.4	5.5
		CV [%]	18		20				29
Bracket BMF 90x116x48x3mm**	18-SN 4.2x89mm	Average	49.5	20.4	44.4	5.4	32.4	6.2	8.7
		CV [%]	9		7				22

* This BMF bracket is similar to Simpson Strong Tie bracket ABR 105.
** This BMF bracket is identical to Simpson Strong Tie bracket AE 116.
RN = annular ring nails; SN = Spiral nails; SFS = Wood screw produced by SFS.

where:

P_{peak} = maximum load
d_{peak} = displacement at maximum load
d_y = yield displacement
P_y = yield load
d_u = ultimate displacement
μ = ductility ratio
K_e = elastic stiffness.



(a) (b)

Figure 5A.3 Connectors used in the study: (a) BMF 90x116x48x3mm angle bracket; (b) BMF 100×100×90×3mm angle bracket

Table 5A.4 Strength, stiffness and deformability data obtained from tests on hold-down and brackets loaded in shear (Schneider, 2015)

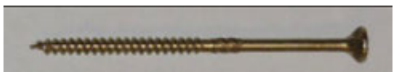
Shear Tests Perpendicular to grain of the CLT Face Lamina										
Connector (Figure 5A.3)	Fastener (Table 5A.5)	Value type	Values from the EEEP Curve							
			P_{peak} [kN]	d_{peak} [mm]	P_y [kN]	d_y [mm]	d_u [mm]	μ	K_e [kN/mm]
Bracket BMF 100x100x90x3mm*	10-SN 4.2x89mm	Average	27.9	26.9	24.3	7.6	40.4	5.3	3.2
		CV [%]	8		8				18
Bracket BMF 90x116x48x3mm**	12-RN 3.8x76mm	Average	46.1	23.0	39.3	7.6	29.7	4.0	5.3
		CV [%]	8		10				16
Bracket BMF 90x116x48x3mm**	12-RN 4.2x60mm	Average	42.7	21.0	37.2	7.0	29.3	4.4	5.5
		CV [%]	6		8				19
Bracket BMF 90x116x48x3mm**	9-SFS 5x90mm	Average	51.0	24.6	43.3	8.6	32.2	3.7	5.0
		CV [%]	3		3				9
Bracket BMF 90x116x48x3mm**	18-SFS 4x70mm	Average	56.1	25.1	48.4	8.8	33.3	3.8	5.6
		CV [%]	3		4				14
Bracket BMF 90x116x48x3mm**	18-SN 4.2x89mm	Average	49.9	25.0	44.1	7.4	35.7	4.8	6.0
		CV [%]	8		10				7

* This BMF bracket is similar to Simpson Strong Tie bracket ABR 105.
** This BMF bracket is identical to Simpson Strong Tie bracket AE 116.
RN = annular ring nails; SN = Spiral nails; SFS = Wood screw produced by SFS.

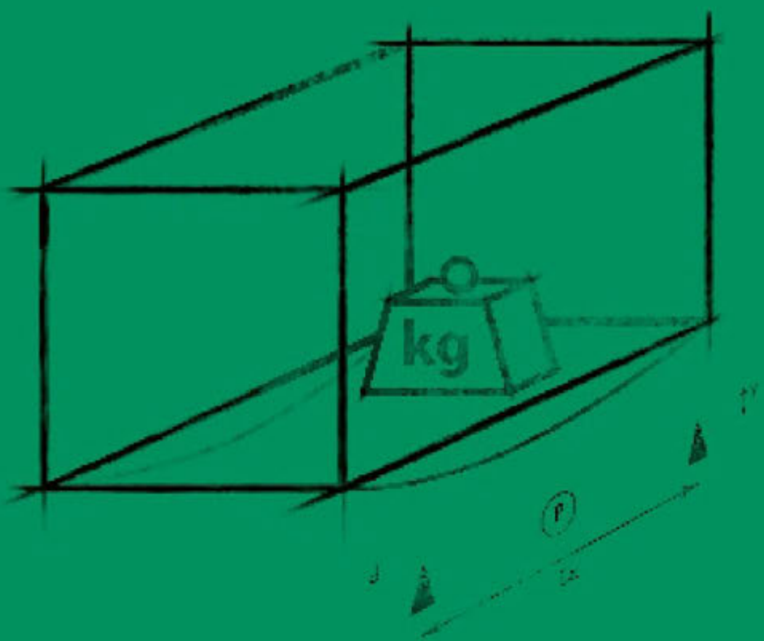
where:

P_{peak} = maximum load
d_{peak} = displacement at maximum load
d_y = yield displacement
P_y = yield load
d_u = ultimate displacement
μ = ductility ratio
K_e = elastic stiffness.

Table 5A.5 Fasteners used with the brackets in Schneider, 2015

Fastener	Description
	Spiral Nail 4.2x89mm (16d x 3 ½ in)
	Ring Shank Nail 3.8x76mm (10d x 3 in)
	Ring Shank Nail 4.2x60mm (16d x 2 5/16 in)
	SFS Wood screw 5 x 90 mm
	SFS Wood screw 4 x 70 mm

FPInnovations



CHAPTER 6

Duration of load and creep factors for cross-laminated timber panels

Lead Authors
Ghasan Doudak
Erol Karacabeyli

Reviewer
Dr. techn. Gerhard Schickhofer

SP-532E (Ch.6)

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ABSTRACT

Cross-laminated timber (CLT) products are used as load-carrying slab, wall elements and beams 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 that are 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. The Canadian Standard on Engineering Design in Wood (CSA O86) includes provisions that take into account the creep effects of CLT in the total out of plane deflection calculation.

This Chapter also contains a discussion on different parameters that may affect the duration of load and creep effects including the effect of adhesive, edge-gluing and release grooves.

Mechanically fastened CLT is outside the scope of the CSA standard and the CLT Handbook and research has found that these CLT products may deflect and creep significantly more than adhesively-bonded CLT.

6.1 OVERVIEW

This Chapter aims to describe how the duration of load[1] and creep[2] effects are taken into account in the design of CLT structures. The provisions presented herein are based on engineering principles, are consistent with the format in other standards, and are based on the test observations and current serviceability requirements in CSA Standard O86-14 Update 1 (2016), and also in CSA Standard O86-19 (2019).

6.2 DURATION OF LOAD EFFECTS

6.2.1 Load Duration Factor in CSA O86-14 Update 1

The load duration factor, K_D, is specified in Clause 5.3.2 of CSA O86-14 Update 1 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, 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 loading. The capacity design values in CSA O86-14 Update 1 are given for standard-term load duration. The load duration factors are given in Table 1.

Table 1 Load duration factor, K_D (Table 5.3.2.2, CSA O86-14)

Duration of Loading	K_D
Short term	1.15
Standard term	1.00
Long term	0.65

Clause 5.3.2.3 of CSA O86-14 Update 1 provides an equation for calculating the duration of the 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:

$$K_D = 1.0 - 0.50 \log (P_L/P_S) \geq 0.65 \quad [1]$$

[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 O86-14, Update 1 2016).
[2] Creep is defined as a slow deformation of a material in time under constant loading.

where:

P_L = specified long-term load

P_S = specified standard-term load based on S and L loads acting alone or in combination; this is equal to S, L, S+0.5L, or 0.5S+L, and is determined using importance factors equal to 1.0.

6.2.2 Service Condition Factor in CSA O86-14 Update 1

CSA O86-14 Update 1 defines dry service as a climatic condition for 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 provides service condition factors, K_S, where the specified strength is multiplied by the appropriate service condition factor.

For CLT, only dry service condition is allowed in the CSA O86-14 Update 1 and, as such, all service conditions modification factors are assumed to be equal to unity. It should be noted that CLT structures may be used in wet service conditions only if specifically permitted by the manufacturer based on documented test data and if approved by the certification organization.

6.3 CREEP EFFECTS

6.3.1 Background

The CSA O86 Update 1 requires that the total out-of-plane deflection of a CLT panel be calculated while taking into account the creep effects of CLT. The provisions are based on engineering principles and test observations. The total deflection consists of two components related to the long-term load and short-term or standard-term load component. The creep factor is applied only on the deflection due to long-term loading.

The long-term behaviour of CLT panels under out-of-plane loading has been compared to that of other laminated wood-based products such as plywood (Jöbstl and Schickhofer, 2007). The creep factor given in the National Design Specification (NDS, 2015) for plywood used in dry service conditions is 2.0. Based on this rationale, a creep factor of 2.0 was adopted in the CSA O86 standard for CLT used in dry service condition.

Research has also reported 30% - 40% larger creep values for CLT than glulam after one year loading in bending; this is attributed to the crosswise layers in CLT (Jöbstl and Schickhofer, 2007).

6.3.2 Calculating Total Deflection Including Creep Effects

The maximum deflection under a specified load acting perpendicular to the plane of the panel can be calculated as the sum of the deflections due to moment and shear, with consideration for creep effects.

The total deflection can be expressed as the sum of deflections under short- and long-term loads:

$$\Delta_{max} = \Delta_{ST} + \Delta_{LT}\, K_{creep}$$

where:

Δ_{ST} = elastic deflection due to short-term and/or standard-term loads, without dead loads in combination

Δ_{LT} = instantaneous elastic deflection due to long-term loads

K_{creep} = creep adjustment factor; this is equal to 2.0 for dry service condition

Deflection under a specified uniformly distributed load, ω, acting perpendicular to the face of a single-span panel may be calculated as the sum of the deflections due to moment and shear effects using the effective bending stiffness, $(EI)_{eff}$, and the effective in-plane (planar) shear rigidity, $(GA)_{eff}$:

$$\Delta = \frac{5}{384}\frac{\omega L^4}{(EI)_{eff}} + \frac{1}{8}\frac{\omega L^2 \kappa}{(GA)_{eff}}$$

For a concentrated load, P, located in the middle of a single span CLT panel acting perpendicular to the panel, the deflection may be calculated as follows:

$$\Delta = \frac{1}{48}\frac{PL^3}{(EI)_{eff}} + \frac{1}{4}\frac{PL\kappa}{(GA)_{eff}}$$

where:

κ = form factor and is equal to 1.0

The shear form factor in CSA O86 Update 1 is based on the original work on the shear analogy method by Kreuzinger (1999) and related research reports. As the shear form factor is only applicable if the Timoshenko beam theory with modified shear correction factors by Schickhofer et al. (2009) is used for the structural design of CLT, the “form factor” is set to 1.0 in this Handbook, and also in the 2019 Edition of CSA O86 (CSA, 2019).

When the shear deformation component of the total deformation of a CLT panel under out-of-plane standard-term loading such as snow and live loads is significant (i.e., in short spans, short span cantilever, etc.) as determined by the designer, the shear deformation under these loads are required to be increased by 30% to account for the time-dependent effect associated with rolling shear.

On the other hand, for lumber and glulam products the L/180 limit controls the instantaneous deflection under total serviceability loads, and the L/360 controls the elastic deflection under long-term loads (when long-term loads exceed 50% of total serviceability loads); for CLT, a creep adjustment factor (K_{creep} = 2) is applied to the instantaneous elastic deflection due to long-term loads. The application of the creep adjustment factor satisfies the permanent deformation provision, therefore eliminating the need to meet the additional requirements of Clause 5.4.3 of the CSA standard O86 Update 1 for CLT.

6.4 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 O86-14 Update 1 specifies the same load duration factors, K_D, for fastenings, as those shown in Table 1. Service condition factors for fastenings, K_{SF}, are also tabulated in the CSA standard. It is important to note that service condition factors for fastenings are different than those for lumber or for glulam seasoned at a 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. 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 the CSA O86 standard. Additional information on connections with CLT is given in Chapter 5, *Connections in Cross-Laminated Timber Buildings*.

6.5 PRODUCT-SPECIFIC PARAMETERS THAT MAY AFFECT DURATION OF LOAD AND CREEP EFFECTS OF CLT

6.5.1 Effect of 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, 2013). The CSA O112.10 standard requires that creep tests be carried out under specific conditions: environment "A" (7 days at 20ºC and 95% RH), environment "B1" (7 days at 70ºC and ambient RH), and environment "B2" (2 hours at 180ºC), 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 at the bond line, which is considered insignificant relative to the creep that occurs in CLT products due to the orientation of crosswise laminations.

6.5.2 Effect of 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 their 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.

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). Ehrhart et al. (2015) recommended characteristic values of rolling shear strength (for Norway spruce $f_{R,k}$ = 1.4 N/mm^2) and shear modulus (for Norway spruce, $G_{R,mean}$ = 100 N/mm^2) for width-to-thickness ratios that are equal to or greater than 4:1. The authors also provided formulae for determining these properties for width-to-thickness ratios less than 4:1. Based on a width-to-thickness ratio of 2:1, they recommended characteristic values of rolling shear strength (for Norway spruce $f_{R,k}$ = 0.80 N/mm^2) and shear modulus (for Norway spruce, $G_{R,mean}$ = 65 N/mm^2) that are applicable to width-to-thickness ratios less than 4:1. These values can also be found in (Brandner et al., 2016) who also included a proposed framework for characteristic properties for strengths, moduli, and densities of CLT strength classes for adoption in Eurocode 5. The PRG 320 standard requires the width of the lamination to be not less than 3.5 times the lamination thickness if the lamination in the cross layers are not edge bonded, unless shear strength and creep are evaluated by testing in accordance with the standard and the principles of ASTM D6815 (2015).

6.5.3 Effect of 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 the 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 the 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 be verified by testing, when using cross laminations with release grooves.

6.5.4 Effect of Nails or Wooden Dowels in Non-Adhesively Bonded CLT Products

Mechanically fastened CLT is outside the scope of this 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 the University of British Columbia have found deflections that are four times larger for nailed CLT specimens compared to glued CLT specimens, for the same specimen thickness (Chen and Lam, 2008). The range of deflections obtained was due to different nailing schedules of the CLT layers. These products may be more suitable for wall applications; the load duration and creep factors recommended in this document are not applicable to non-adhesively bonded CLT products.

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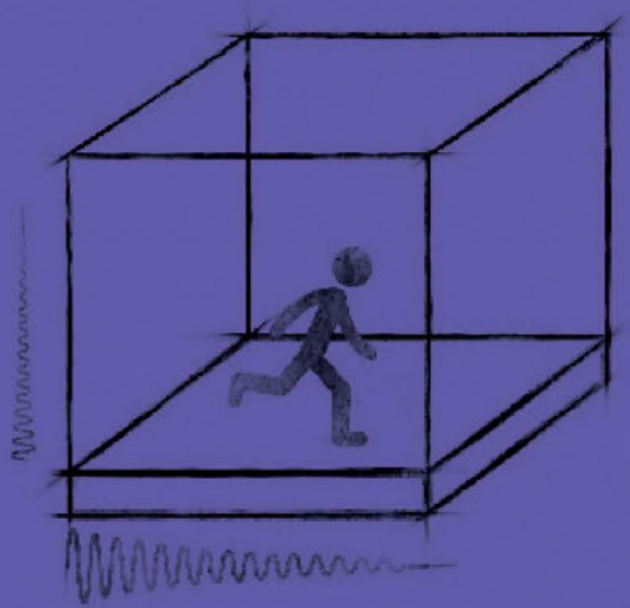

CHAPTER 7

Vibration performance of cross-laminated timber floors

Lead Authors
Lin Hu
Y.H. Chui
Samuel Cuerrier-Auclair

Special Collaborators
Julie Frappier
Bernhard Gafner

Reviewer
Erol Karacabeyli

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ABSTRACT

This Chapter addresses the vibration serviceability of CLT floors caused by normal human activities, as well as vibration serviceability of tall wood buildings under wind-induced excitation.

Studies at FPInnovations found that bare CLT floor systems differ from traditional lightweight wood joisted floors. Consequently, the existing vibration-controlled design methods for lightweight floors may not be applicable for CLT floors. A new design method is proposed which is based on the use of calculated 1 kN static deflection and fundamental natural frequency for bare CLT floors as the criterion parameters. A simple form to directly calculate the vibration-controlled spans from CLT stiffness and density is provided.

This Chapter is an update of the same chapter published in the first edition of the Canadian CLT Handbook, in 2011. In 2016, a vibration design method for CLT floors was accepted by the CSA Technical Committee on Engineering Design in Wood, and was subsequently published in the 2016 Update 1 of the CSA Standard O86-14 (CSA, 2016) as well as in the 2019 Edition of CSA Standard O86-19 (CSA, 2019). This method was largely based on the method presented in the 2011 edition of the CLT Handbook with a few modifications. The revised vibration design method for CLT floors in CSA O86 Standard has two key features:

1) The vibration-controlled span is directly calculated using CLT floor effective bending stiffness in the major strength direction and its mass without iteration.

2) An empirical approach to account for the effects of multiple-span, toppings and non-structural elements such as internal partition walls and finishes.

This updated Chapter was then extended to include a new design method for Timber Concrete Composite (TCC) floors based on recent research conducted by FPInnovations, and a more sophisticated stiffness requirement for floor supporting beams than in the previous edition.

For controlling tall building vibrations, this updated chapter provides preliminary guidelines based on the recent technical information and data collected by FPInnovations and others, allowing for proposed simple equations to calculate the first two transverse natural frequencies of wood buildings and recommendations for damping ratios of wood buildings.

To assist with the understanding of the proposed design methods, examples are provided through out the Chapter. Background studies and results are presented in Appendices.

7.1 INTRODUCTION

In 2016, a vibration-controlled design method for CLT floors was accepted by the CSA O86 Technical Committee on Engineering Design in Wood and was subsequently published in the CSA Standard O86-14 Updates 1 & 2 (CSA, 2016), as well as in the 2019 Edition of CSA Standard O86-19 (CSA, 2019). This method was largely based on the method presented in the 2011 edition of the CLT Handbook with few modifications. This updated Chapter is in line with the design method in CSA O86. The revised vibration-controlled design method for CLT floors in CSA O86 has two key new features:

1) The vibration-controlled span is directly calculated using the CLT floor's effective bending stiffness in the major strength direction and its mass, without iteration.

2) An empirical approach was used to account for the effects of multiple-span and non-structural elements such as toppings, ceilings, partition walls, and finishes.

A design method was also developed for Timber-Concrete-Composite (TCC) floors based on research conducted by FPInnovations. This design method for TCC floors is now included in this Chapter.

This Chapter also provides a more sophisticated stiffness requirement for floor supporting beams than that in the previous edition.

For controlling tall wood building vibrations, this Chapter provides preliminary guidelines based on the recent technical information and data collected by FPInnovations and others.

To make the document user-friendly, this Chapter only contains the design procedures, the scope of each method, and worked examples. It forms a self-explanatory document ready for users who wish to apply these methods to their projects. The explanations of the science behind the design methods are provided in Appendix I. The details of the development of the CLT floor design method, and the adoption in CSA O86 are provided in Appendix II. The details of the development of the TCC floor design method can be found in an FPInnovations report (Cuerrier-Auclair et al., 2018). The development of the floor supporting beam stiffness requirement can be found in an FPInnovations report (Hu, 2018).

It should be noted that the document presented here is a living document containing up-to-date knowledge that will evolve over time when more understanding and better information than what we have collected to date becomes available.

7.2 BACKGROUND

In 2010, when CLT was new to North America, and there were no CLT producers or buildings containing CLT elements, FPInnovations conducted laboratory studies on CLT floor vibration performance; these studies were conducted on bare CLT floors with variable support conditions, joint details, thicknesses and spans, with a wood topping and a dropped ceiling. The results were summarized in a report (Hu, 2013).

More than 40 years of research at FPInnovations has uncovered the science behind wood-based floor vibrations induced by human normal walking actions. The science and the solutions for controlling wood-based floor vibration, along with laboratory studies on CLT floors led to the development of a design method to control CLT floor vibration. The design method was described in Chapter 7: Vibration Performance of Cross-Laminated Timber Floors in the first Canadian edition (2011) of the CLT Handbook. The original method was based on a bare floor assumption.

Since then, a number of CLT buildings have been erected in Canada and the U.S.A. using Canadian CLT products. The original design method has been verified with the feedback from occupants, designers and CLT producers. Positive feedback has been received, as the proposed design method led to satisfactory CLT floors. However, there is also a demand to further simplify and expand the method to account for floors with non-structural components, different support conditions, and heavy toppings. Based on the feedback received, the original design method was modified as follows:

- The original formula was simplified to determine vibration-controlled spans for CLT floors by replacing the apparent bending stiffness in the major strength direction with the shear-free effective bending stiffness. Therefore, the updated formula does not involve iterations anymore, and it also does not impact significantly on the vibration-controlled spans calculated from the original method.
- CLT floors with toppings not bonded to the CLT structural floors have been included.
- Construction features that enhance vibration performance, including multi-span continuity and non-structural components (partition walls, finishes, and ceilings) are now accounted for. The accepted finishing materials may include wood flooring and ceramic tiles.

The CSA O86 Technical Committee has decided to include this revised design method in the CLT design guidance CSA Standard O86-14 Updates 1 & 2 (CSA, 2016; CWC, 2017). Meanwhile, the approach developed by FPInnovations for establishing a human acceptability criterion for vibrational serviceability of timber floors has attracted international interest and has been published in an ISO Technical Report, ISO/TR 21136 (ISO, 2017).

An approach similar to that used for the development of the CLT floor design method, i.e. the ISO/TR 21136 method, was applied to TCC floors for the development of a tentative vibration-controlled design method for TCC slab floors. TCC slab floors can be a supplement to CLT floors in long span and shallow depth floor applications. FPInnovations conducted field and laboratory study on TCC floors to develop the preliminary vibration-controlled design method for TCC slab floors. Details of the study were described in an FPInnovations report (Cuerrier-Auclair et al., 2018). The design method for TCC slab floors is included in this document.

To control floor vibration, attention also must be paid to the supporting system, because it can significantly affect the floor vibration response. The supporting system must have sufficient stiffness, so that it will not lower the natural frequencies or enlarge the deflections of the floor significantly. There is currently no sophisticated design criterion for the supporting beams. To address this shortcoming, FPInnovations conducted field and laboratory studies to develop such a requirement. This new requirement for supporting beam stiffness is provided in this document. Details concerning the development of this requirement can be found in an FPInnovations report (Hu, 2018).

Finally, the advent of tall wood buildings has raised questions about controlling lateral building vibrations induced by wind. The National Building Code of Canada (NBC) provides a general design provision for wind-induced vibration. Application of the NBC provision to wood buildings requires prior knowledge of the wood building's dynamic characteristics such as damping ratios and natural frequencies. A database on these dynamic characteristics has been assembled from the literature, including measurements on wood buildings conducted by FPInnovations and by others around the world. This data was used in the development of the preliminary design guide for wind-induced vibration control of tall wood buildings.

7.3 DESIGN METHODS TO CONTROL VIBRATION INDUCED BY NORMAL WALKING, FOR MASS TIMBER FLOORS

7.3.1 CLT Floors

7.3.1.1 Scope

The assumptions underpinning the proposed design method to control walking vibration in CLT floors are given below:

1. Vibration is induced by normal walking actions, not by rhythmic activities.

2. End supports are simple and effectively pinned.

Guidance is also given for multiple-span systems and floors with a topping that is not structurally bonded to the CLT panels, and CLT floors supported on beams.

7.3.1.2 Design Method

The vibration-controlled span for a CLT floor, with both ends rigidly supported and meeting at least the simple support requirement (e.g. on loadbearing walls, on rigid supporting beams, etc.), can be calculated using the following Equation [1]:

$$L \leq 0.11 \frac{\left(\frac{(EI)_{eff}}{10^6}\right)^{0.29}}{m^{0.12}} \quad [1]$$

where:

L = vibration-controlled span limit (m). It should be the clear span measured from face to face, of the two end supports;

m = linear mass of CLT for a 1-m wide panel (kg/m). See producer's specifications;

$(EI)_{eff}$ = effective bending stiffness in the major strength direction for a 1-m wide panel (N-mm^2). See producer specifications, product standard (ANSI/APA, 2018) or calculated in accordance with accepted mechanics method such as specified in CSA Standard O86-14 Updates 1 & 2 with CLT design guidance (CSA, 2016; CWC, 2017), or in Chapter 3 of this Handbook.

For multiple-span CLT floors with non-structural elements that are considered to provide an enhanced stiffening effect, including partition walls, finishes and ceilings, and with support conditions beyond simple support, the calculated vibration-controlled span may be increased by up to 20%, provided it is not greater than 8 m and that the floor does not have a concrete topping. Accepted finish materials may include wood flooring and ceramic tiles. All spans in multiple-span CLT floors shall not be greater than the vibration-controlled span, *L*.

It must be pointed out that partition walls stiffen wood floor systems, but they may not be permanent components and can be removed or added at any time. This is also true for the finishes. Therefore, caution should be exercised when taking advantage of these features to increase vibration-controlled spans for CLT floors.

For floors with a topping, Equation [1] can be used by assuming a bare floor construction for calculation purposes, i.e., stiffness and weight of topping are ignored, provided the area density of the topping is not greater than twice the bare CLT floor area density. If the area density of the topping is larger than twice the bare CLT floor area density, it is recommended that the calculated vibration-controlled span be reduced by up to 10%. Area density is defined as the mass per unit area of floor, in kg/m^2. Topping is defined as the material that is placed directly, or through a resilient layer on CLT structural floors without physical attachment to the CLT structural floor.

Application of Equation [1] to other mass timber slab floors needs to be verified with subjective evaluations.

7.3.1.3 Design Examples

Examples are given below to calculate the vibration-controlled spans of CLT floors with different construction details using Equation [1], as would be conducted by designers to perform a design check, or by producers to develop their CLT span tables.

Example 1:

This is an example of a single-span floor with open space. The floor is finished with carpet. There are no partition walls or topping on the floor. It is located in a one-storey single-family house. The proposed floor span is 5.0 m, the floor is constructed using 5-ply 175-mm thick CLT of APA stress grade E1. The density of the CLT panel is provided by the producer and the EI_{eff} is provided in the ANSI/APA standard (2018), as shown below:

Design properties of the CLT used:

- Thickness = 0.175 m
- Mass density = 515 kg/m^3
- $(EI)_{eff}$ = 4.166 x10^{12} N-mm^2/m (in major strength direction)

Substituting the above values into Equation [1], the vibration-controlled span for this floor using the 175-mm thick CLT can be calculated as follows:

$$L \leq 0.11 \frac{\left(\frac{4.166 \text{ x } 10^{12}}{10^6}\right)^{0.29}}{(1.0 \text{ x } 0.175 \text{ x } 515)^{0.12}} = 5.33 \text{ m}$$

The vibration-controlled span is 5.33 m, which is larger than the proposed floor span. Therefore, the 5.0-m span proposed by the designer is acceptable.

Example 2:

This floor uses the same CLT as in Example 1. However, this floor has different construction details. The floor uses wood flooring as a finish instead of carpet, with partition walls built on the floor and a dry-wall ceiling under the floor attached with resilient channels. It is a multi-span continuous floor located on the second level of a 4-storey building.

Taking advantage of all stiffness enhancements of this CLT floor from the wood finish, partitions, ceiling, and continuity, plus the enhanced support conditions (instead of simple support assumption), the designer has decided that the span can be increased by up to 20%, so the relaxed vibration-controlled span for this floor would be:

$$L_{relaxed} = 5.33 \text{ x} 1.2 = 6.40 \text{ m}$$

Example 3:

In addition to the construction details described above, if the example CLT floor has a 50-mm thick normal weight concrete topping with a radiant heating system, or a concrete topping floating on a resilient layer for enhanced sound insulation, then:

1) Use Equation [1] to calculate the vibration-controlled baseline span as in Example 1; the baseline span obtained will be 5.33 m.

2) Calculate the area density of the base CLT floor:

   Area density of the 175-mm thick CLT floor = (CLT density) x (CLT thickness)

   = 515 kg/m^3 x 0.175 m = 90.13 kg/m^2

3) Calculate the area density of the 50-mm normal weight concrete topping:

   Area density of the 50-mm normal weight concrete = (concrete density) x (concrete thickness)

   = 2300 kg/m^3 x 0.05 m = 115 kg/m^2

4) Calculate the ratio of the concrete area density to the CLT area density:

   Ratio of the concrete area density to the CLT area density = 115/90.13 =1.3 < 2.

Because the ratio of the concrete area density to the CLT area density is less than 2, the vibration-controlled span of the 175-mm thick CLT floor with the 50-mm thick concrete topping is 5.33 m, the same as the span of the bare CLT floor; therefore, there is no need to reduce the baseline span. It is worth noting that the span relaxation is not applied to CLT floors with toppings.

7.3.2 Timber-Concrete-Composite (TCC) Floors

In general, TCC floors are floors made of a reinforced concrete slab of at least 70 mm in thickness (TiComTec GmbH, 2011) and a bottom layer of thick timber panels (Figure 7A.3(b) in Appendix) or heavy timber beams. Shear connectors are used to connect these two layers to form a composite cross-section. In some TCC floors, the radiant-heating system is embedded in the concrete. To prevent potential heat damage of the wood in these radiant-heating TCC floors, a thermal insulator is usually placed between the timber and the reinforced concrete. If the shear connectors pass through the insulation layer, the connection stiffness is expected to be reduced and this should be taken into consideration when determining the bending stiffness of the composite cross-section.

The requirement for the maximum vibration-controlled span has been developed based on FPInnovations' database of laboratory studies on TCC floors. Details can be found in an FPInnovations report (Cuerrier-Auclair et al., 2018).

7.3.2.1 Scope

The assumptions underpinning the proposed design method to control walking vibration in TCC floors are given below:

1. Vibration is induced by normal walking actions, not by rhythmic activities.

2. End supports are simple and effectively pinned.

3. Normal weight concrete is used.

Moreover, the present document will only focus on TCC slab floors made of concrete and mass timber slab but not timber beams.

7.3.2.2 Preliminary Design Method

It is suggested that the vibration-controlled span of a TCC floor with both ends simply supported be calculated using the following Equation [2]:

$$L \leq 0.329 \frac{(EI)_{eff}^{0.264}}{m^{0.206}} \tag{2}$$

where:

L = the vibration-controlled span of a TCC slab floor (m); this should be the clear span measured from face to face, of the two end supports;

(EI)_{eff} = effective composite bending stiffness in the major strength direction of a 1-m wide strip of a TCC slab floor (N-m^2), calculated using Equation [3];

m = mass per unit length of a 1-m wide TCC slab (kg/m), i.e. the sum of the masses of the 1-m wide concrete and timber slabs.

The gamma-method (γ-method) developed by Möhler (1956) and used in Eurocode 5 was adopted to calculate the $(EI)_{eff}$ of a TCC section, as given in Equation [3] below:

$$(EI)_{eff} = (EI)_c + (EI)_t + \gamma_c (EA)_c a_c^2 + \gamma_t (EA)_t a_t^2 \tag{3}$$

where:

$(EI)_t$ and $(EA)_t$ = bending and axial stiffness of a 1-m wide mass timber panel in (N-m^2) and (N), respectively, obtained from the producer's specification or calculated according to CSA O86-14 Update 1 (CSA, 2016; CWC, 2017). For the estimation of the $(EA)_t$, only the longitudinal layer must be considered.

*(EI)*c and *(EA)*c = bending and axial stiffness of a 1-m wide concrete panel in (N-m^2) and (N), respectively, calculated from the equations below:

$$(EI)_c = \frac{E_c b_c h_{c,eff}^3}{12}$$

$$(EA)_c = E_c b_c h_{c,eff}$$

where:

E_c = modulus of elasticity for normal density concrete in Pa (N/m^2) with compressive strength between 20 and 40 MPa, calculated from the equation below according to clause 8.6.2.3 of the CSA A23.3-14 (CSA, 2014);

$E_c = 10^6 \times 4500\sqrt{f_c'}$ where f_c' = compressive strength (MPa)

b_c = width of a concrete section (m) = 1.0 m for the 1-m wide TCC;

$h_{c,eff}$ = effective depth of concrete in compression (m), calculated from the equation below:

$$h_{c,eff} = \sqrt{\alpha^2 + \alpha(h_t + 2h_c + 2t)} - \alpha \leq h_c$$

where:

$$\alpha = \frac{\gamma_t (EA)_t}{\gamma_c E_c b_c}$$

where:

$$\gamma_c = 1.0$$

$$\gamma_t = \frac{1}{1 + \frac{\pi^2 (EA)_t}{KL^2}}$$

where:

K = load-slip modulus per unit length in span direction (N/m/m).

h_t = thickness of mass timber panel (m);

h_c = thickness of concrete panel (m);

t = thickness of interlayer between the timber and the concrete, if such an interlayer is used (i.e. insulation material, acoustic material, or construction gap) (m).

a_c and a_t = distance between the centroid of the concrete section and the timber section to the neutral axis of the composite section, respectively, (m), calculated from the equations below:

$$a_c = \frac{\gamma_t (EA)_t r}{\gamma_c (EA)_c + \gamma_t (EA)_t}$$

$$a_t = \frac{\gamma_c (EA)_c r}{\gamma_c (EA)_c + \gamma_t (EA)_t}$$

where $r = \frac{h_t}{2} + t + h_c - \frac{h_{c,eff}}{2}$

The proposed requirement was well matched to the test observations of TCC floor systems obtained in laboratory and has been validated with one floor located in a building. Additional details about the development of the criterion equation may be found in an FPInnovations report (Cuerrier-Auclair et al., 2018).

The shear stiffness of the timber-concrete connection per unit length in the span direction, *K,* can be obtained from the shear connector producer. Alternatively, it can be measured using the test method described in (Cuerrier-Auclair et al., 2018).

Typical values of the load-slip modulus per connector, reported by Cuerrier-Auclair et al. (2018), are around 34.2 kN/mm for truss plate, and around 21.2 kN/mm for a pair of self-tapping screws inserted at a 45° angle to the face of the CLT panel.

7.3.2.3 Design Examples

Example 4:

A TCC slab floor is made of a 5-ply 175-mm thick CLT slab with an APA E1 stress grade (ANSI/APA, 2018) and a 100-mm thick normal density concrete slab, and has a span of 8.6 m. The CLT and the concrete are connected together by shear connectors made of a pair of self-tapping screws (ASSY 3.0) inserted at a 45° angle to the face of the CLT panel and at 300 mm o.c. in the span direction, with 5 rows in a 1-m wide panel.

Floor details and material properties are as follows:

1 100-mm thick normal weight concrete:

1.1 Concrete density = 2300 kg/m^3

1.2 Concrete compressive strength = 30 MPa

Calculated concrete MOE (E_c) = 10^6 x 4500 x $(30)^{0.5}$ = 24.6x10^9 Pa

2 5-ply 175-mm thick CLT:

2.1 *(EI)*$_t$ = 4.166x10^6 N-m^2 for a 1-m wide CLT panel (ANSI/APA 2018)

2.2 *(EA)*$_t$ = 1.23x10^9 N for a 1-m wide CLT panel (considering only the stiffness of the longitudinal layer, $11.7 \times 10^9 \times 3 \times 0.035 \times 1 = 1.23 \times 10^9$ N)

2.3 Wood density = 515 kg/m^3 (producer's specification)

3 A pair of self-tapping screws (ASSY 3.0) inserted at a 45° angle to the face of the CLT panel and at 300 mm o.c. in the span direction with 5 rows in a 1-m wide panel:

3.1.1 Number of connector rows in a 1-m wide panel = 5

3.1.2 Connector spacing = 0.3 m

3.1.3 Measured load-slip modulus of a single connector = 21.2x10^6 N/m

3.1.4 Calculated *K* = 5 x 21.2 x10^6 / 0.3 = 353 x10^6 N/m/m

4 Design span, *L* = 8.6 m

5 Calculated $\gamma_t = \frac{1}{1+\frac{\pi^2}{L^2}\frac{(EA)_t}{K}} = \frac{1}{1+\frac{\pi^2}{8.6^2}\frac{1.23\times10^9}{353\times10^6}} = 0.683$

6 Calculated $\alpha = \frac{\gamma_t (EA)_t}{\gamma_c E_c b_c} = \frac{0.683\times1.23\times10^9}{1\times24.6\times10^9\times1} = 0.034$ m

7 Calculated $h_{c,eff} = \sqrt{\alpha^2 + \alpha(h_t + 2h_c + 2t)} - \alpha =$ $\sqrt{0.034^2 + 0.034(0.175 + 2\times0.1 + 2\times0)} - 0.034 = 0.084$ m

8 Calculated $(EI)_c = E_c \frac{b_c h_{c,eff}^3}{12} = 24.6\times10^9 \frac{1\times0.084^3}{12} = 1.215\times10^6$ N-m^2

9 Calculated $(EA)_c = E_c b_c h_{c,eff} = 24.6\times10^9 \times 1 \times 0.084 = 2.066\times10^9$ N

10 Calculated $t_{eff} = h_c - h_{c,eff} + t = 0.1 - 0.084 + 0 = 0.016$ mm

11 Calculated $a_t = \frac{\gamma_c (EA)_c (h_t + h_{c,eff} + 2t_{eff})}{2(\gamma_c (EA)_c + \gamma_t (EA)_t)} = \frac{1\times2.066\times10^9(0.175+0.084+2\times0.016)}{2(1\times2.066\times10^9+0.683\times1.23\times10^9)} = 0.103$ mm

12 Calculated $a_c = \frac{\gamma_t (EA)_t (h_t + h_{c,eff} + 2t_{eff})}{2(\gamma_c (EA)_c + \gamma_t (EA)_t)} = \frac{0.683\times1.23\times10^9(0.175+0.084+2\times0.016)}{2(1\times2.066\times10^9+0.683\times1.23\times10^9)} = 0.042$ mm

13 Calculated $(EI)_{eff} = (EI)_t + (EI)_c + \gamma_t (EA)_t a_t^2 + \gamma_c (EA)_c a_c^2 = 4.166\times10^6 + 1.215\times10^6 + 0.683\times1.23\times10^9\times0.103^2 + 1\times2.066\times10^9\times0.042^2 = 18.02\times10^6$ N-m^2 for a 1-m wide TCC panel using Equation [3]

14 Calculated linear mass density $m = (\rho_c h_c + \rho_t h_t) \times 1.0 = (2300 \times 0.100 + 515 \times 0.175) \times 1.0 =$ 320 kg/m for a 1-m wide TCC panel

15 Calculated vibration-controlled span limit of the floor design $L \leq 0.329 \frac{(EI)_{eff}^{0.264}}{m_L^{0.206}} = 0.329 \frac{\left(18.02 \times 10^6\right)^{0.264}}{320^{0.206}} =$ 8.2 m using Equation [2] with the linear mass in kg/m and the $(EI)_{eff}$ in N-m^2.

Since the design span, 8.6 m, is longer than the vibration-controlled span limit, 8.2 m, the designer has to redesign the floor. There are several options:

- increase the thickness of the concrete slab,
- increase the thickness of the CLT slab,
- use a stiffer connector and/or reduce the spacing of the connectors in both directions along the floor span and across the floor width, or
- increase the distance between the concrete and the CLT slabs, e.g. add an insulation layer.

After the redesign, the designer will have to repeat the design check procedure. The calculation procedure can be implemented in an Excel spreadsheet.

Example 5:

The CLT slab in Example 4 is changed to a 7-ply 197-mm (35L-19T-35L-19T-35L-19T-35L) slab. The design procedure is repeated, and the following results are obtained:

1 Concrete slab = same as item No. 1 in example 4

2 197-mm thick CLT:

2.1 $(EI)_t$ = 6.171x10^6 N-mm^2 for a 1-m wide CLT panel (calculated using equation in CSA O86-14 Update 1 (CSA, 2016))

2.2 $(EA)_t$ = 1.638x10^9 N for a 1-m wide CLT panel (considering only the longitudinal layer)

2.3 Wood density = 515 kg/m^3 (producer's specification)

3 Same shear connectors as item No. 3 in example 4

4 Design span, L = 8.6 m

5 Calculated $\gamma_t = \frac{1}{1+\frac{\pi^2}{L^2}\frac{(EA)_t}{K}} = \frac{1}{1+\frac{\pi^2}{8.6^2}\frac{1.638 \times 10^9}{353 \times 10^6}} = 0.618$

6 Calculated $\alpha = \frac{\gamma_t (EA)_t}{\gamma_c E_c b_c} = \frac{0.618 \times 1.638 \times 10^9}{1 \times 24.6 \times 10^9 \times 1} = 0.041$ m

7 Calculated $h_{c,eff} = \sqrt{\alpha^2 + \alpha(h_t + 2h_c + 2t)} - \alpha = \sqrt{0.041^2 + 0.041(0.197 + 2 \times 0.1 + 2 \times 0)} - 0.041 = 0.093$ m

8 Calculated $(EI)_c = E_c \frac{b_c h_{c,eff}^3}{12} = 24.6 \times 10^9 \frac{1 \times 0.093^3}{12} = 1.655 \times 10^6$ N-m^2

9 Calculated $(EA)_c = E_c b_c h_{c,eff} = 24.6 \times 10^9 \times 1 \times 0.093 = 2.291 \times 10^9$ N

10 Calculated $t_{eff} = h_c - h_{c,eff} + t = 0.1 - 0.093 + 0 = 0.007$ mm

11 Calculated $a_t = \frac{\gamma_c (EA)_c \left(h_t + h_{c,eff} + 2t_{eff}\right)}{2(\gamma_c (EA)_c + \gamma_t (EA)_t)} = \frac{1 \times 2.291 \times 10^9 (0.197 + 0.093 + 2 \times 0.007)}{2(1 \times 2.291 \times 10^9 + 0.618 \times 1.638 \times 10^9)} = 0.105$ m

12 Calculated $a_c = \frac{\gamma_t (EA)_t \left(h_t + h_{c,eff} + 2t_{eff}\right)}{2(\gamma_c (EA)_c + \gamma_t (EA)_t)} = \frac{0.618 \times 1.638 \times 10^9 (0.197 + 0.093 + 2 \times 0.007)}{2(1 \times 2.291 \times 10^9 + 0.618 \times 1.638 \times 10^9)} = 0.047$m

13 Calculated $(EI)_{eff} = (EI)_t + (EI)_c + \gamma_t (EA)_t a_t^2 + \gamma_c (EA)_c a_c^2 = 6.171 \times 10^6 + 1.655 \times 10^6 + 0.618 \times 1.638 \times 10^9 \times 0.105^2 + 1 \times 2.291 \times 10^9 \times 0.047^2 = 24.031 \times 10^6$ N-m^2 for a 1-m wide TCC panel using Equation [3]

14 Calculated linear mass density $m = (\rho_c h_c + \rho_t h_t) \times 1.0 = (2300 \times 0.100 + 515 \times 0.197) \times 1.0 =$ 331 kg/m for a 1-m wide TCC panel

15 Calculated vibration-controlled span limit of the floor design $L \leq 0.329 \frac{(EI)_{eff}^{0.264}}{m_L^{0.206}} = 0.329 \frac{\left(24.031 \times 10^6\right)^{0.264}}{331^{0.206}} =$ 8.8 m using Equation [2].

Therefore, increasing the thickness of the CLT slab to 197 mm is acceptable.

7.3.2.4 Recommendations

The preliminary design method was developed based on limited field and laboratory testing of TCC floors. In the study conducted by FPInnovations (Cuerrier-Auclair et al., 2018), only one TCC floor located in a building was available for testing. In the future, the proposed design equation will be refined when additional data become available.

7.3.3 Requirement for Support Conditions

The support conditions for any floor should be 'rigid' (i.e., negligible flexibility), to fulfill the assumption of simple support. This condition must be met, since the natural frequency of the floor will be reduced and the vibration magnitude amplified if the support deviates from this assumption, leading to poorer performance than is predicted by Equations [1] and [2]. It should be noted that if the support does not have adequate stiffness, the floor vibration performance will be affected by the support flexibility, irrespective of the stiffness and mass properties of the CLT or TCC floors.

In platform construction, where the floors rest on a solid supporting wall below, the support condition approaches that of a non-flexible support, which is the underlying assumption for all vibration-controlled design methods proposed to date. Deviation from the simple support condition can occur in several ways, if the supporting walls are not properly built.

In post-and-beam construction, where the floors rest on supporting beams, the supporting beams exhibit a defined stiffness that influences the natural frequencies and other structural responses of the floor system. In this case, the underlying assumption of simple support for Equations [1] and [2] is violated. Accordingly, the supporting beams should be designed to ensure that they do not act as flexible supports.

This supporting beam stiffness requirement has been developed based on FPInnovations' database of field supporting beams and laboratory studies on supporting beams. The details of the development of the requirement can be found in an FPInnovations report (Hu, 2018). The design stiffness requirement is shown in Equation [4] below.

7.3.3.1 Scope

The assumption underpinning the proposed requirement for the supporting beam stiffness is that the ends of the supporting beams should be at least supported on load bearing walls or columns. The supporting beams should be made of wood, engineered wood, composite wood, or other wood-based material.

7.3.3.2 Supporting Beam Stiffness Requirement

The supporting beam stiffness, $(EI)_{beam}$, should meet the requirement below:

$$(EI)_{beam} \geq F_{span} 132.17\, l_{beam}^{6.55} \qquad [4]$$

where:

(EI)$_{beam}$ = supporting beam apparent bending stiffness (N-m^2) provided by the beam producer, or calculated using the following equation:

= MOE x b x h^3/12

where:

b = beam width (m);

h = beam depth (m);

MOE = modulus of elasticity (N/m^2); for wood, see CSA O86 design standard, for other materials, see the appropriate standard.

l_{beam} = clear span of supporting beam (m)

F_{span} = 1.0 for simple span beam and ≈ 0.7 for a multi-span continuous beam

The proposed requirement is well matched to the test observations of appropriate floor systems found in post-and-beam wood buildings, especially in new mid-rise and tall wood buildings (Hu, 2018).

7.3.3.3 Design Examples

To demonstrate the application of the proposed stiffness criterion for floor supporting beams, three design examples are given below.

Example 6:

The designer proposes to use a glulam beam 137-mm wide and 362-mm deep. The floor beam is supported on two columns with a clear single span of 5.3 m. The design check for the stiffness of the supporting beam can be performed using Equation [4] as shown below:

Step-1: calculate the required stiffness for the supporting beam using the criterion given in Equation [4]:

$(EI)_{beam} \geq F_{span} \times 132.17 \times (\text{beam span})^{6.55} = 1.0 \times 132.17 \times 5.3^{6.55} = 7.3\text{x}10^6$ (N-m^2)

Step-2: according to the producer's specification, the specified EI for the beam is 6.7x10^6 (N-m^2).

Thus, the selected glulam beam does not have the required stiffness, and the supporting beam has to be redesigned. A 137-mm by 406-mm glulam beam from the same producer has an EI of 9.5 x10^6 (N-m^2), which meets the required stiffness of 7.3x10^6 (N-m^2). Another option is a 184-mm by 363-mm glulam beam having an EI of 9.02 x10^6 (N-m^2). Both are acceptable.

Example 7:

From Example 6, if the supporting beam is a multi-span continuous beam with a longest span of 5.3 m, then the required stiffness for the supporting beam will be:

$(EI)_{beam} \geq F_{span}$ x 132.17 x (beam span l)$^{6.55}$ = 0.7 x 132.17 x 5.3$^{6.55}$ = 5.1 x10^{6} (N-m^{2}).

Therefore, the designed 137-mm by 362-mm continuous supporting beam with an EI of 6.7x10^{6} (N-m^{2}) meets the stiffness requirement.

Example 8:

The designer proposes to use a glulam beam 215 mm by 484 mm, made of Douglas Fir-Larch grade 24f-EX. The beam is supported by two columns with a clear span of 6.4 m (measured from face to face of the columns). The design check for the stiffness of the supporting beam is performed using Equation [4] following the steps below:

Step-1: calculate the required stiffness for the supporting beam using the criterion given in Equation [4]:

$(EI)_{beam} \geq F_{span}$ x 132.17 x (beam span)$^{6.55}$ = 1.0 x 132.17 x 6.4$^{6.55}$ = 2.5 x10^{7} (N-m^{2}).

The design value EI can be calculated using the CSA O86 design value for the glulam species and grade, along with the beam width and depth:

Step-2: calculate the design value of the bending stiffness of the supporting beam:

EI = MOE x width x depth^{3} ÷ 12 = 1.28x10^{10} x 0.215 x 0.484^{3} ÷ 12 = 2.6 x10^{7} (N-m^{2}).

The selected supporting beam meets the supporting beam stiffness criterion. The design is acceptable.

7.3.4 Best Practice Tips

Following are three best practice tips to ensure a satisfactory vibration performance for all wood-based floors:

- The first is to design the floor and its supports by using a proper design method such as the method described in this document. There is no simple and cost-effective method to fix an over-spanned floor on-site, once the building is occupied. For example, a 10% over-span will require a 40% increase in CLT stiffness to compensate for it. It is even more difficult to fix a poor support than a poorly designed floor.
- Secondly, an on-site quality control of the floor construction is important to ensure that the support condition conforms to the design assumption.
- Finally, it is valuable to have builders, developers, architects, designers, contractors and/or product manufacturers conduct a subjective evaluation before the building is occupied. If the subjective evaluation results are not positive, then chances are that occupants will experience similar issues regarding the floor vibration performance.

7.4 PRELIMINARY DESIGN GUIDE TO CONTROL VIBRATION INDUCED BY WIND FOR TALL WOOD BUILDINGS

The National Building Code of Canada (NBC) (NRC, 2015) provides provisions for controlling wind-induced vibration in buildings. The following is a brief summary of those provisions.

7.4.1 Type of Buildings that Are Required by the NBC to Undergo a Wind Vibration-Controlled Design Check

The NBC (NRC, 2015) requires that a vibration-controlled design check be performed if a building is dynamically sensitive according to NBC classification, as shown below:

a) its lowest natural frequency is less than 1 Hz and greater than 0.25 Hz,

b) its height is greater than 60 m, or

c) its height is greater than 4 times its minimum effective width.

As mentioned in the last versions of the NBC, lightweight buildings such as wood buildings may be prone to larger amplitude vibration than heavy buildings of the same height. For tall wood buildings, it is worth making the effort to perform a wind vibration design check.

7.4.2 Design Criteria Recommended by the NBC

The NBC does not provide design criteria for controlling wind-induced vibration of buildings; rather, the code provides a review of the criteria used in North America and an ISO criterion. Below are the findings listed in the NBC:

- in North America in the period covering 1975-2000, many of the tall buildings were designed for a peak one-in-ten-year acceleration in the range of 1.5% to 2.5% of g (acceleration due to gravity – $9.81 m/s^2$). The lower end of this range was generally applied to residential buildings and the upper end to office towers. The performance of buildings evaluated based on these criteria appears to have been generally satisfactory;

- the ISO criterion can be expressed as a peak acceleration not exceeding $0.928f^{-0.412}$ once every 5 years, where f is the lowest natural frequency in Hz.

7.4.3 Design Equations Recommended by the NBC

7.4.3.1 Fundamental Periods (T) of Buildings

The NBC recommends the following empirical formulae to determine the fundamental periods of buildings as a function of either building height or number of storeys:

$T = 0.085h^{3/4}$	for steel moment frames	[5]
$T = 0.075h^{3/4}$	for reinforced concrete moment frames	[6]
$T = 0.1N$	for other moment frames	[7]
$T = 0.025h$	for braced frames	[8]
$T = 0.05h^{3/4}$	for shear walls and other structures	[9]

where *T*, *h*, and *N* represent the fundamental period (in seconds), the building height (in meters) and the number of storeys, respectively. Note that the natural frequency = *1/T*.

7.4.3.2 Peak Accelerations of Building Vibrations Induced by Wind

Equations [10] and [11] are the NBC formulae used to determine the peak accelerations of building vibration induced by wind, for the design check using the above criteria.

Across-wind direction: $$a_W = f_{nW}^2 g_p \sqrt{Wd}\left(\frac{a_r}{\rho_B g \sqrt{\beta_W}}\right) \quad [10]$$

Along-wind direction: $$a_D = 4\pi^2 f_{nD}^2 g_p \sqrt{\frac{KsF}{C_{eH}\beta_D}\frac{\Delta}{C_g}} \quad [11]$$

where:

W , d = across-wind direction effective width and along-wind direction effective depth, respectively (m);

$a_{W,}$ a_D = peak acceleration in across-wind and along-wind directions, respectively (m/s^2);

$a_r = 78.5 \times 10^{-3}\left[V_H/\left(f_{nW}\sqrt{wd}\right)\right]^{3.3}$ (N/m^3);

ρ_B = average density of the building (kg/m^3);

β_W, β_D = fraction of critical damping in across-wind and along-wind directions, respectively;

f_{nW} , f_{nD} = fundamental natural frequencies in across-wind and along-wind directions, respectively (Hz);

Δ = maximum wind-induced lateral deflection at the top of the building in along-wind direction (m);

g = acceleration due to gravity = 9.81 (m/s^2);

g_p = statistical peak factor for the loading effect;

K = a factor related to the surface roughness coefficient of the terrain;

s = size reduction factor;

F = gust energy ratio at the natural frequency of the structure;

C_{eH} = exposure factor at the top of the building;

C_g = gust effect factor.

7.4.4 Application of the NBC Provision to Tall Wood Buildings

The NBC provides design examples for wind vibration design checking. Application of the design check procedure to tall wood buildings is needed to estimate the tall wood building natural frequencies and the design values of damping ratios for these buildings. FPInnovations has conducted numerous field measurements on newly built mid-rise and tall wood buildings to determine their natural frequencies and damping ratios, and has also collected data from the literature. The database contains the measured frequencies and damping ratios of more than 35 wood buildings and structures across the world. The database was used to verify the NBC frequency Equations 5 to 9; it was found that the estimated frequencies of the wood buildings obtained from NBC Equation 9 for "shear walls and other structures" were well correlated to the measured fundamental natural frequencies, as showed in Figure 1. The field tall wood building study led to the recommendations listed below.

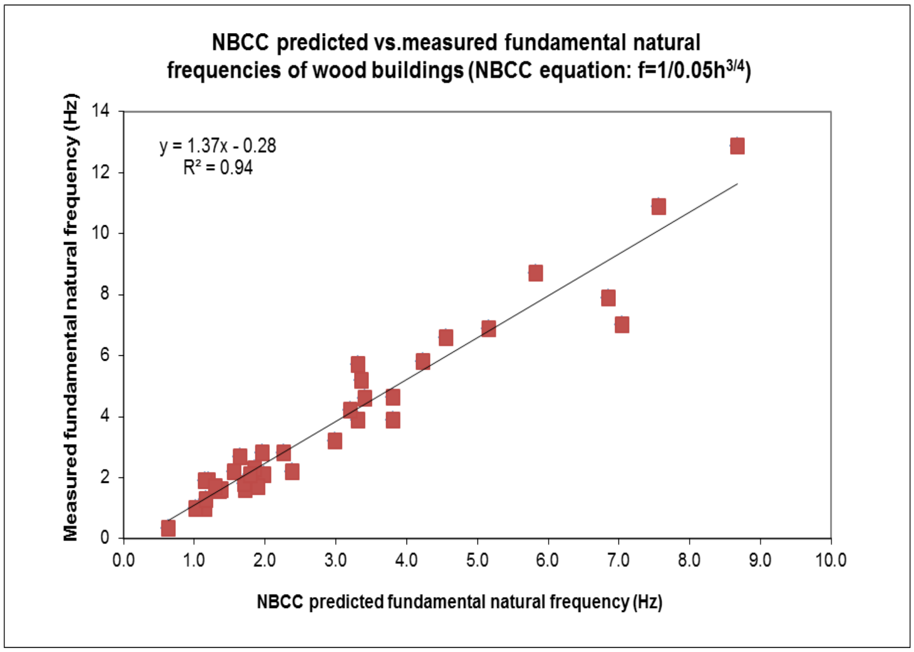

Figure 1 Measured vs. estimated wood building fundamental natural frequencies using an NBC equation

7.4.4.1 Recommendations for Estimation of the First Two Transverse Natural Frequencies of Wood Buildings

In general, the natural frequencies of a building can be estimated using any validated finite element (FEM) software package with proper assumptions and models for the connections. It is also possible to estimate the natural frequencies of wood buildings using the modified NBC Equation [12] shown below:

$$f_1 = \frac{1}{0.035h^{0.8}} \qquad [12]$$

where:

*f*1 = first transverse vibration frequency of the wood building (Hz);

h = building height (m).

The second transverse vibration frequency of a wood building can be simply estimated from the first transverse vibration frequency, the geometry of the building, and mechanical properties (if the building has a rectangular shape in elevation, i.e. the building looks like a rectangular beam). Modelling the building as a cantilever beam, the second transverse natural frequency of the building can then be approximately estimated using Equation [14].

$$f_2 = \frac{D_L}{D_S}\sqrt{\frac{MOE_L}{MOE_S}}\, f_1 \qquad [13]$$

where:

f_2 = second transverse vibration frequency of the wood building (Hz);

D_L = longer dimension of the building cross-section (m);

D_S = shorter dimension of the building cross-section (m);

MOE_L = equivalent elastic modulus of the building in the longer axis of the building cross-section (N/m^2);

MOE_S = equivalent elastic modulus of the building in the shorter axis of the building cross-section (N/m^2).

Example 9:

This example shows how to estimate the first two transverse natural frequencies of an 18-storey hybrid wood-concrete building in Vancouver, and to compare them with the measured frequencies. The building information is as follows:

1. 18-Storeys and a rectangular shape
2. Height = 53 m
3. Longer dimension, D_L = 56 m
4. Shorter dimension, D_S = 11.5 m
5. Building designed in a way that we can approximately assume $MOE_L = MOE_S$

Using Equation [12], the first transverse vibration frequency is calculated as:

f_1 = $1/(0.035 \times 53^{0.8})$ = 1.2 Hz (measured f_1 of the completed building is 1.0 Hz).

Using Equation [13], the second transverse vibration frequency estimated from the measured f_1 is:

f_2 = 1.0 x 56/11.5 = 4.8 Hz (measured f_2 of the completed building is 4.0 Hz).

The discrepancy is thought to be due to the approximation that $MOE_L = MOE_S$.

During the design check, only the estimated value of f_1 was available and the second transverse vibration frequency had to be calculated from the estimated f_1 of 1.2 Hz; therefore, the calculated f_2 = 1.2 x 56/11.5 = 5.8 Hz (measured f_2 of the completed building is 4.0 Hz).

It must be noted that, up to now, no finite element model has been found to reliably estimate wood building natural frequencies (periods) for design checking. Even if these two simple equations are approximate and for buildings with a rectangular shape and with an almost uniformly distributed stiffness and mass, it was observed that these equations gave better estimations of the natural frequencies of wood buildings, especially for the first frequencies, than the finite element models. Therefore, they could provide some degree of guidance for designers to refine their finite element models, assumptions or designs, during design check.

7.4.4.2 Recommendation for Damping Ratios of Wood Buildings

It is recommended to use a damping ratio of 2% for wood buildings without finish, and 3% for wood buildings with finish.

7.5 FINAL REMARKS

This Chapter has been written based on up-to-date technical information and knowledge gathered from researchers and engineers. It attempts to address major issues related to vibration-controlled design of mass timber floors and tall wood buildings. However, the topic is complex and vast. Therefore, limitations in this Chapter are unavoidable. For example:

1. For CLT floors with a concrete topping, the proposed approach to limit the vibration-controlled span is empirical in nature, as opposed to mechanics-based.

2. For TCC floors, the field test data was limited. More field TCC floor data is needed to validate and improve the design method to limit the vibration-controlled span.

3. The preliminary guide for tall wood building vibration control does not address all issues in the NBC provision, such as the design criterion and the calculation equations for the peak accelerations. In an attempt to further address these issues, FPInnovations is monitoring a 13-storey and an 8-storey Glulam/CLT building to measure their acceleration time history responses, as well as the wind forces and directions. It is hoped that more tall wood buildings in Canada or elsewhere will be monitored, so that more comprehensive guidelines can be developed.

In summary, the Chapter presented here is a living document that comprises the state-of-the art knowledge. The document will evolve over time when additional information, knowledge, and data become available.

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APPENDIX A

Science Behind Wood-Based Floor Vibration Control

This Appendix provides background information on the design methods described in this document to control walking vibration of CLT and TCC floors, i.e. the science behind the control of floor vibration induced by normal walking.

To control vibration induced by normal walking on wood-based floors in general, we need to understand:

1. What causes the annoying vibration?
2. How does a floor respond to normal walking?
3. What are the special features of wood-based floors in general (including CLT and TCC floors) that affect the response?
4. What are the critical relevant design parameters affecting the walking vibration of wood-based floors in general?
5. How do humans perceive floor vibration?
6. What should be the strategy for controlling vibration induced by normal walking on wood-based floors in general?

This Section provides some answers to these questions.

7A.1 CAUSE OF FLOOR VIBRATIONS INDUCED BY NORMAL WALKING – CHARACTERISTICS OF FOOTSTEP FORCE GENERATED BY NORMAL WALKING

When a person is walking on a floor, the heel drop of each footstep creates an impact on the floor. The impulse of the heel drop impact force vibrates the floor. The time history of a heel impact force measured by Lenzen and Murray (1969) is shown in Figure 7A.1.

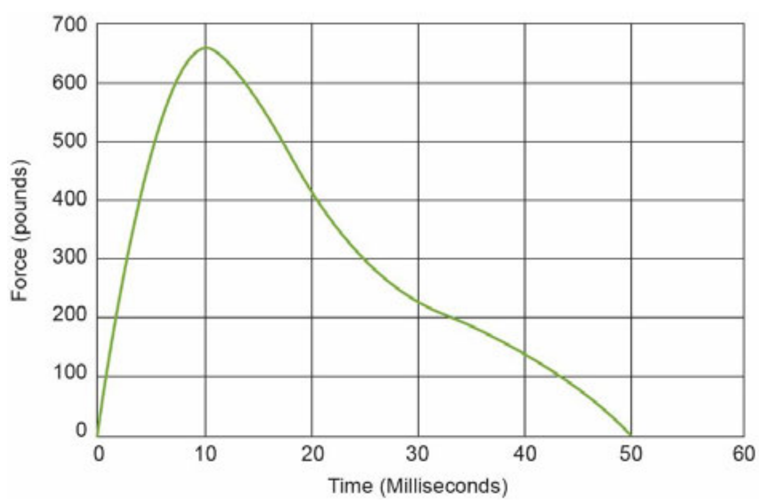


Figure 7A.1 A forcing function based on an average of five heel drop forces on a concrete surface measured by Lenzen and Murray (1969)

Considerable research has been conducted on measuring the dynamic forces produced by a series of footsteps, by a person or a group of people walking on a floor (Rainer and Pernica, 1986; Ohlsson, 1991; Ebrahimpour et al., 1994; Kerr and Bishop, 2001). It was found that the dynamic forces generated were composed of harmonic wave trains of the walking rate, at about 2 Hz, and that the significant low frequency contributions were generally contained within the first three to four harmonics (Figure 7A.2).

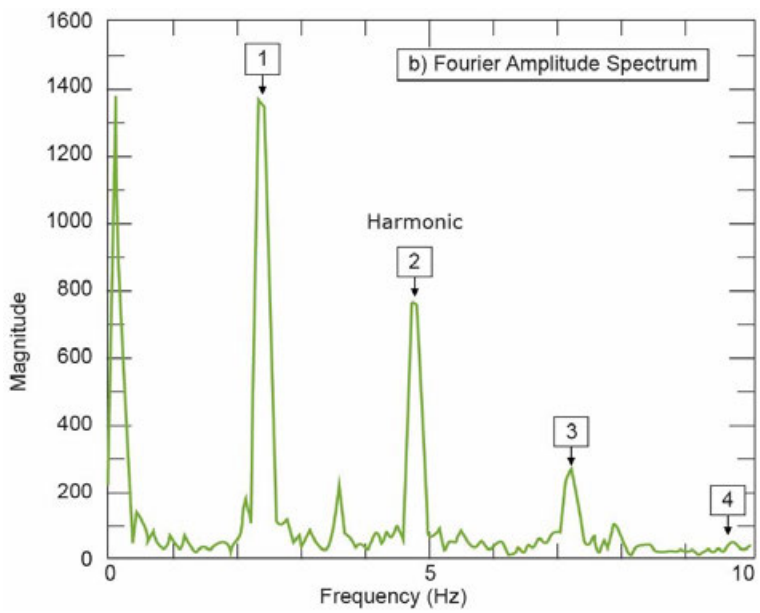


Figure 7A.2 Fourier transform spectrum of the loading time history of a person's normal walking action measured by Rainer and Pernica (1986)

Ohlsson (1991) summarized the findings described previously as follows:

The footstep force generated by walking comprises two components:

1) A short duration impact force induced by the heel of each footstep on the floor surface, as illustrated in Figure 7A.1. The duration of the heel impact varies from about 30 ms to 100 ms, depending on the conditions and the materials of the two contact surfaces (the floor and the shoes worn by the person walking), and on the weight and gait of the person.

2) The walking rate, a continuous series of footsteps consisting of a wave train of harmonics, at multiples of about 2 Hz.

7A.2 WOOD-BASED FLOORS AND THEIR DYNAMIC CHARACTERISTICS

There is a broad spectrum of wood-based floors in construction, including light frame joisted floors (Figure 7A.3), mass timber slab floors such as CLT floors (Figure 7A.4 (a)), TCC floors (Figure 7A.4 (b)), etc. Regardless of the type of wood-based floors used in construction, it was found that they all have similar dynamic characteristics. A common feature is that, in general, good performance wood-based floors have a fundamental natural frequency above 8 Hz, but it may be lower for heavy floors or floors with spans longer than 8 m.

However, based on studies of laboratory and field wood-based floors, it was found that CLT floors had some unique features compared with conventional light weight joisted floors:

1) In general, CLT floors do not have joists, and are slab floors.

2) CLT floors are made of CLT panels jointed together using various jointing details. Thus, in the across-width direction of CLT floors, the stiffness is controlled by the joints. This joint stiffness is relatively low, in comparison with the bending stiffness in the minor strength direction of the individual CLT panels. Therefore, the vibration behavior of CLT floors mainly exhibits a one-way action, e.g. is controlled by the CLT stiffness in its major strength direction.

3) In comparison with light frame joisted floors with the same span and performance, a CLT floor is heavier, and has a lower natural frequency and damping ratio.

4) CLT panels can be produced to a very long length. Therefore, multi-span continuous floor systems comprising long CLT panels are common.

TCC slab floors are similar to CLT slab floors without joists and have lower damping than joisted floors, but they are heavier than CLT slab floors. A TCC floor is usually shallower than a CLT slab floor having the same span. In contrast with CLT floors, TCC slab floors are often constructed as a two-way system.

Due to these differences, the design methods for lightweight joisted floors are not applicable to massive timber slab floors such as CLT or TCC floors.

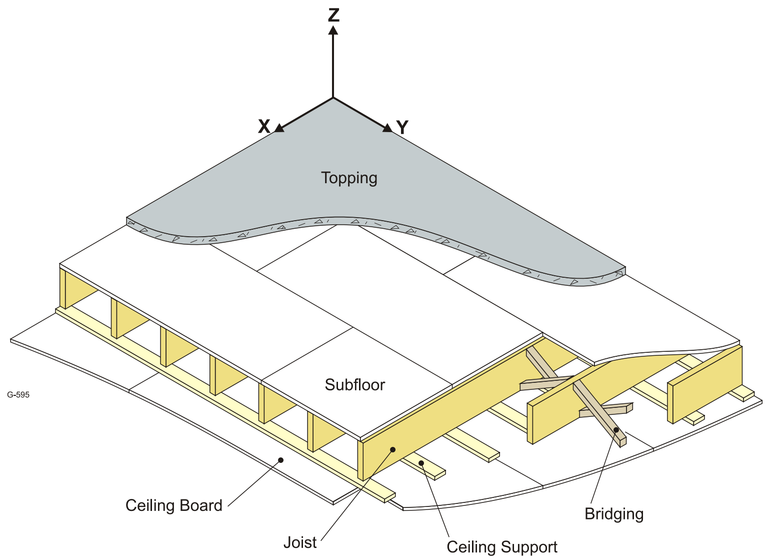
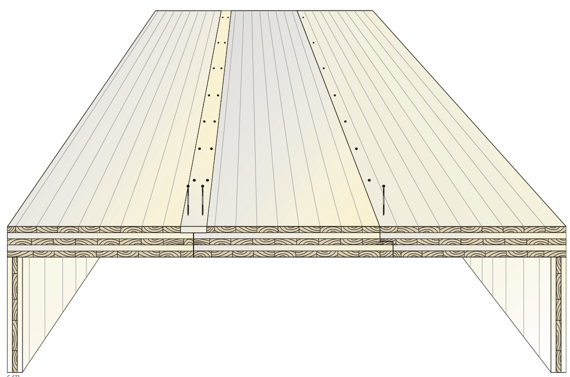
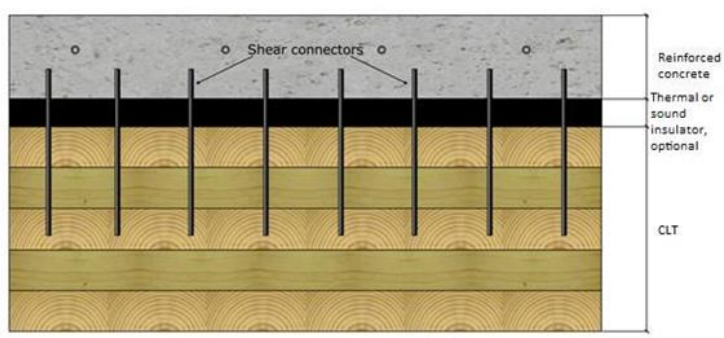


Figure 7A.3 Conventional light frame wood-based floor built with joists and subfloor without showing the finish and the resilient layer under the topping



(a)



(b)

Figure 7A.4 Typical CLT and TCC slab floor constructions with (a) cross-section of a bare CLT slab floor with the edges supported, showing the joints between two CLT panels; and (b) cross-section of a bare TCC slab floor showing the shear connectors and the thermal insulator, using CLT as an example, without being limiting to it

7A.3 RESPONSE OF WOOD-BASED FLOORS TO THE FOOTSTEP FORCE GENERATED BY NORMAL WALKING

The way a floor responds to the footstep excitation described above depends on the floor's inherent properties such as its mass, its stiffness, and its capacity to dissipate the excitation energy (i.e., damping of the floor system). Understanding the nature of the footstep force leads to the conclusion that the two components in the walking excitation can initiate two types of vibrations, depending on the inherent properties of the floor. The two types of vibrations are transient vibration and resonance.

If the fundamental natural frequency of a floor is above 8 Hz and is above the footstep frequency and its predominant harmonics, then the vibration induced by the footstep forces is most likely dominated by a transient response caused by the individual heel impact force from each footstep. The transient vibration decays quickly and takes place at multiples of the footstep frequency. The peak values of a transient vibration are mainly governed by the stiffness and mass of the system.

On the other hand, if the floor's fundamental natural frequency is below 8 Hz and is in the range of the footstep frequency and its predominant harmonics, then the floor most likely will resonate with one of the harmonics, and the resonance will be constantly maintained by the action of the walking excitation. The magnitude of the resonance is significantly affected by the damping ratio of the floor system. Furthermore, if the floor's fundamental natural frequency is around 2 Hz, which is close to the frequency of footsteps, the magnitude of the resonance will be high because, as shown in Figure 7A.2, most of the energy in the walking excitation is concentrated at the walking frequency.

As found in FPInnovations' laboratory and field studies, the fundamental natural frequency of wood-based floors, including CLT and TCC floors that are heavier than light frame joisted floors having the same performance, is usually above 8 Hz for good performance. Therefore, it can be concluded that the response of wood-based floors including CLT and TCC floors to footstep forces is dominated by transient vibration.

7A.4 CRITICAL DESIGN PARAMETERS FOR CONTROLLING WALKING VIBRATION OF WOOD-BASED FLOORS

Based on the discussion above, it can be concluded that the critical design parameters involved in the walking vibration control of wood-based floors are the floor mass and stiffness properties.

7A.5 HUMAN PERCEPTION OF FLOOR VIBRATIONS

Numerous efforts have been made to identify the factors affecting human perception of floor vibration, through subjective evaluation of laboratory and field floors along with the measurement of response parameters (e.g., Wright and Green, 1959; Ohlsson, 1980; Onysko, 1985; Smith and Chui, 1988; Foschi et al., 1994; Dolan et al., 1999; Hu, 2000; Toratti and Talja, 2006; Homb, 2008; Hamm et al., 2010; etc.).

In general, it was found that humans are more tolerant of short duration vibration (e.g., transient vibration) than the longer lasting resonance. Vibration performance parameters such as floor static deflection, natural frequency, peak velocity, peak and root-mean-square (rms) acceleration have been correlated to human perception of floor vibration. The combination of fundamental natural frequency and static point load deflection correlated well with human perception of vibrations, for a broad range of wood frame floors (Hu 2000). Other combinations of fundamental natural frequency with vibration magnitude indicators such as peak velocity, peak acceleration and root-mean-square (rms) acceleration also yielded good correlations with human acceptability (Hu, 2000).

7A.6 STRATEGY FOR CONTROLLING WOOD-BASED FLOOR VIBRATIONS

The above discussion clearly shows that vibration performance parameters such as fundamental natural frequency, static deflection, velocity, and acceleration are potential predictors of human perception of floor vibrations. These response parameters can be predicted from inherent properties of the floors such as stiffness and mass of the floor, and damping. As noticed before, the fundamental natural frequency is a function of the floor stiffness and mass, the static deflection depends on the floor stiffness, and the velocity and acceleration responses are dependent upon stiffness, mass, dynamic excitation, and possibly damping. Furthermore, the above discussion also shows that the response of wood-based floors to walking excitation is most likely transient vibration, which is mainly determined by the floor stiffness and mass. So, the strategy for controlling transient vibration of wood-based floors is to control the combination of the floor stiffness and mass.

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APPENDIX B

Development of a Vibration-Controlled Design Method for CLT Floors in a CLT Design Guidance of CSA O86-14 Updates 1 & 2 (CSA, 2016)

7B.1 APPLICATION OF ISO/TR 21136 PROCEDURE (ISO, 2017)

In 2010, there were no CLT buildings or CLT producers in Canada, so it was impossible to conduct field surveys on CLT floors to develop a CLT floor vibration-controlled criterion. To develop a performance criterion typically requires the evaluation of a large number of field floors covering all construction details, which was not a reality in 2010, or even now. But, the ISO/TR 21136 (ISO, 2017) provides a general form for a human acceptability criterion of vibration, as shown below:

$$\frac{f}{y^{x1}} \geq C \qquad [B.1]$$

where:

f = *frequency,*

y = *vibration response indicator that can be: deflection, velocity, acceleration,*

x_1 and C = constants.

Using this general form of a human acceptability criterion (Equation [B.1]) and selecting the frequency (f) and a 1-kN static deflection (d) as the two design parameters *f* and *y* in Equation [B.1], the development of the vibration-controlled design criterion for CLT floors became possible; by using this approach, only the two unknowns in the criterion equation, i.e. x_1 and *C,* needed to be determined. To determine these two unknowns experimentally requires building a minimum of two marginal performance CLT floors with known values of the two design parameters, "f' and "d". These values can be approximately calculated using simple equations.

7B.2 CALCULATION OF DESIGN PARAMETERS I.E. THE DEFLECTION (D) AND FREQUENCY (F) OF CLT FLOORS

For simplicity, it was decided to use the simple beam Equations [B.2] and [B.3] below to approximately calculate the two design parameters, i.e. the fundamental natural frequency (f) and the 1-kN static deflection (d) for CLT floors:

$$f = \frac{3.142}{2L^2}\sqrt{\frac{(EI)_{app}}{\rho A}} \qquad [B.2]$$

where:

f = fundamental natural frequency of a 1-m CLT panel simply supported (Hz);

L = CLT floor vibration-controlled span (m);

$(EI)_{app}$ = apparent stiffness in the major strength direction for a 1-m wide panel (N-m^2);

ρ = mass density of the CLT panel (kg/m^3);

A = area of cross-section of a 1-m wide CLT panel = thickness x 1.0 m width (m^2).

$$d = \frac{1000\, pL^3}{48\,(EI)_{app}} \quad \text{[B.3]}$$

where:

d = static deflection at mid-span of the 1-m wide simply supported CLT panel under a 1-kN load (mm);

P = 1000 (N).

Further simplification led to an approximation for $(EI)_{app}$ by assuming:

$(EI)_{app} = 0.9(EI)_{eff}$

where:

$(EI)_{eff}$ = the effective bending stiffness in the major strength direction for a 1-m wide panel (N-m^2).

It must be noted that the equations are not derived to determine the exact deflection and frequency of a CLT floor. They are two auxiliary parameters linking acceptability to the floor stiffness and mass. The potential inaccuracy in the calculation equations will be accounted for in the design criterion derived from these two calculated parameters for the two marginal CLT floors.

7B.3 FPINNOVATIONS' LABORATORY FULL-SCALE CLT FLOOR STUDY

A total of 20 configurations of full-scale CLT floors made of CLT panels of three thicknesses, i.e. 140 mm, 185 mm and 230 mm, were tested and subjectively evaluated in FPInnovations' laboratory. The spans ranged from 4.5 m to 8 m with variable joint details, support conditions, toppings, and ceilings (Hu, 2013). For each size of CLT panel, the first-floor system was over-spanned to ensure poor vibration performance. Then, the floor span was gradually reduced. For each span, the floor performance was subjectively evaluated by 20 evaluators, for each floor. As the span was reduced, the floor performance gradually improved from unacceptable, to marginal, and finally to acceptable.

7B.4 DERIVATION OF THE DESIGN CRITERION USING LABORATORY CLT FLOOR DATA

As specified in Equation [B.1], the general form for the human acceptability criterion was expressed as:

$$\frac{f}{y^{x1}} \geq C$$

The CLT floor design criterion was derived by using the calculated 1-kN static deflections and fundamental natural frequencies of two CLT floors with marginal performance.

For these two marginal floors, the acceptability was found to be on the borderline of human acceptability. Therefore, equation $f/y^{x1} \geq C$ becomes:

$$\frac{f}{y^{x1}} = C \qquad [B.4]$$

where:

y = the calculated 1-kN static deflection of the marginal CLT floor (mm) using Equation [B.3];

f = the calculated fundamental natural frequency of the marginal CLT floor (Hz) using Equation [B.2];

C and x_1 = the two constants to be determined.

For each marginal CLT floor, there is one equation with two unknown constants. The calculated "*f*" values for the two marginal CLT floors were 9.91 Hz and 11.38 Hz. The calculated *"y"* values for the two marginal CLT floors were 0.68 mm and 0.83 mm, respectively. Substituting the calculated "*f*" and "*y*" values into the general form for the marginal floors (Equation [B.4]), two equations with the two unknown constants "*x1*" and "*C*" were obtained, as shown in Equations [B.5] and [B.6] below:

$$\frac{9.91}{0.68^{x1}} = C \qquad [B.5]$$

$$\frac{11.38}{0.83^{x1}} = C \qquad [B.6]$$

"*x1*" and "*C*" were determined by solving these two equations simultaneously (resulting in values of 0.7 and 13.0, respectively). Therefore, the design criterion (human acceptability criterion using calculated values for *"f"* and *"d"*) for the CLT floors was expressed as shown below:

$$\frac{f}{d^{0.7}} \geq 13.0 \;\text{ or }\; d \leq \frac{f^{1.43}}{39.0} \qquad [B.7]$$

Figure 7B.1 illustrates the derivation of the design criterion for CLT floors using the data obtained for the two marginal laboratory CLT floors and the verification using other data.

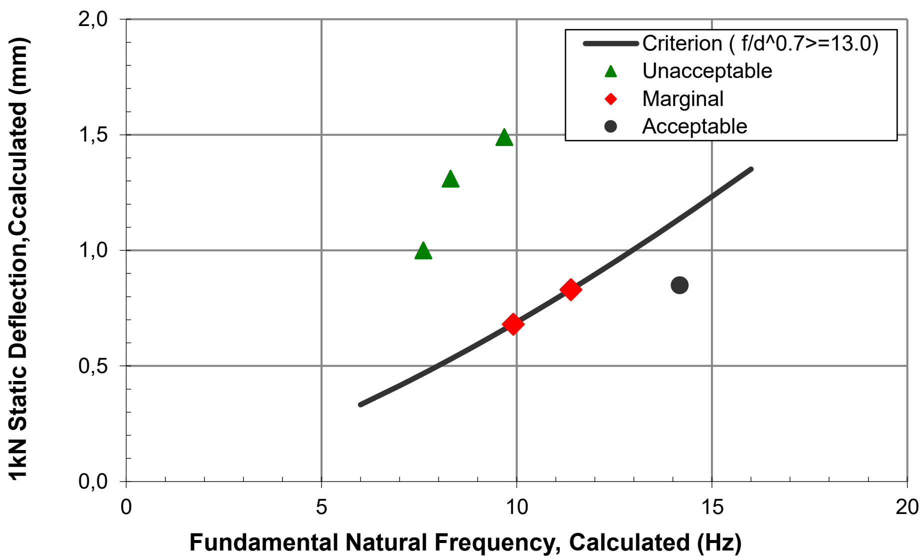

Figure 7B.1 Derivation of the design criterion for CLT floors using data from the two marginal floors and verifications using other data for CLT floors

7B.5 DERIVATION OF THE VIBRATION-CONTROLLED SPAN LIMITS FOR CLT FLOORS

Merging the design criterion, (Equation [B.7]) and Equations [B.5] and [B.6] to calculate the frequency and deflection resulted in a simple design method. Inserting Equations [B.5] and [B.6] into the design criterion (Equation [B.7]), along with the assumption that $(EI)_{app} = 0.9(EI)_{eff}$ led to an equation that can be used to calculate the vibration controlled-span directly, which was presented in Section 7.3.1.2 of this document as Equation [1] and is quoted below:

$$L \leq 0.11 \frac{\left(\frac{(EI)_{eff}}{10^6}\right)^{0.29}}{m^{0.12}} \quad [1]$$

where:

L = vibration-controlled span limit (m);

m = linear mass of CLT for a 1-m wide panel (kg/m);

$(EI)_{eff}$ = effective bending stiffness in the major strength direction for 1-m wide panel (N-mm²).

7B.6 IMPACT STUDY

Since the publication of the CLT floor design method in 2011, production of CLT panels and erection of buildings containing CLT systems have increased in Canada. CLT producers have conducted an impact study by comparing field CLT floor spans with the vibration-controlled spans calculated using the design method described in this document (Eq. 1), along with feedback on the vibration performance of these field CLT floors. Table 1 presents this comparison.

Table 7B.1 Comparison of field CLT floor spans with the spans calculated from Equation (Eq.1) for vibration-controlled CLT floor spans

Floor ID	Longest Clear Span in the Multi-Span Field CLT Floor (m)	Span Determined Using the Vibration-Controlled Span Equation (m)	Vibration Performance Feedback
1	2.8	3.8	Satisfactory
2	2.8	3.8	Satisfactory
3	4.4	4.3	Satisfactory
4	4.0	3.8	Satisfactory
5	4.5	5.3	Satisfactory
6	1.8	3.1	Satisfactory
7	2.4	5.3	Satisfactory
8	3.2	4.3	Satisfactory
9	3.7	3.8	Satisfactory
10	4.5	5.6	Satisfactory
11	3.8	3.8	Satisfactory
12	9.3	6.6	Unsatisfactory
13	6.3	5.3	Satisfactory
14	6.7	5.7	Satisfactory
15	6.0	5.3	Satisfactory
16	5.5	4.3	Satisfactory

The spans of the majority of the field floors were designed to be more conservative than the vibration-controlled spans calculated using design Equation [1] in Section 7.3.1.2, except for floors no. 3 and no. 12 to 16. In comparison with the spans calculated with the base equation of the vibration-controlled span, Equation [1], floor no. 12, which was a single span floor with no finishes at the time of evaluation, was over-spanned by 41% and was bouncy; the other four floors were satisfactory even if they were over-spanned by 28% to 13%. A closer examination of the construction details of these four floors revealed that they consisted of multi-span continuous CLT panels, had partitions, and that the end supports were better than a simple support.

7B.7 RELAXATION OF THE VIBRATION-CONTROLLED SPAN LIMIT FOR CLT FLOORS

The impact study results and the data provided in Table 1, along with the knowledge and experience of the positive effects on the floor vibration performance of wood flooring finish, multi-span floor continuity, partitions, and supports beyond simple supports, revealed that these construction details improve floor vibration. Therefore, working closely with the CLT industry and the CSA O86 committee, it was agreed to increase the vibration-controlled spans calculated with the base equation, i.e. Equation [1], by up to 20% to account for these stiffness enhancement features, for floors without a topping and with spans of less than 8 m.

7B.8 COMPARISON OF THE VIBRATION-CONTROLLED SPANS DETERMINED WITH VARIOUS METHODS

The vibration-controlled spans calculated using base Equation [1], were compared with the spans calculated by other methods. Tables 2 and 3 present these comparisons.

Table 7B.2 Vibration-controlled CLT floor spans determined using the base Equation (Eq. 1) in the CSA O86 design method vs. spans determined using the CLTdesigner software

CLT Thickness	Base Equation, Eq.1, in the CSA O86 Method	CLTdesigner Software's Proposed Span for 1% Damping and No-Topping Floors (Schickhofer and Thiel, 2010)
(mm)	(m)	(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

The comparison provided by Schickhofer and Thiel (2010) showed a good match between the spans determined by the two methods.

The calculation of the vibration-controlled spans in Table 3 below assumed that the CLT panels were manufactured as per the ANSI/APA PRG 320 CLT standard (APA, 2018). The assumptions used in the calculations using the Eurocode 5 (CEN, 2004) and the Hamm et al (2010) methods were a 1% damping and 10-m wide CLT floors (Ramzi, 2016). In addition, the deflection limit for Eurocode 5 was assumed to be 1.5 mm, and the constant "b" for the peak velocity limit was assumed to be 100, which represents the normal performance. The equation used to calculate the 1-kN static deflection in Eurocode 5 (CEN, 2004) was adapted from Natterer et al. (2011).

In comparison with the spans calculated from the base equation in the CSA O86 method, the Hamm et al. method was more conservative for shorter span floors, and more liberal for longer span floors. This diverges from the human perception of vibration described in the ISO 2631-2 (ISO, 1989) standard. Assuming a 1.5-mm deflection and a "b" value of 100 (velocity limits), the Eurocode 5 method resulted in more liberal spans. A detailed discussion of the pros and cons of the Eurocode 5 and the Hamm et al methods is provided in Section 7A.2 of this document.

Table 7B.3 Comparison of vibration-controlled spans of CLT floors determined by various methods

CLT Grade	CLT Thickness (mm)	Base Equation in the CSAO86 Method	Eurocode 5 Method (Floor Width=10 m)	Hamm et al Method Lower Demand (Floor Width=10m)	Hamm et al Method Higher Demand (Floor Width=10m)
E1	105	3.80	5.25	3.21	2.32
	175	5.29	6.48	*7.48	5.78
	245	6.62	7.47	*8.63	7.47
E2	105	3.66	5.10	3.10	2.22
	175	5.10	6.28	*7.25	5.59
	245	6.39	7.24	*8.36	7.24
E3	105	3.44	4.58	2.66	1.96
	175	4.79	5.95	*6.74	4.88
	245	5.99	6.86	*7.91	6.86
V1	105	3.74	5.18	3.18	2.30
	175	5.20	6.38	*7.37	5.73
	245	6.51	7.36	*8.50	7.36
V2	105	3.58	4.99	2.97	2.15
	175	4.98	6.15	*7.10	5.35
	245	6.24	7.09	*8.19	7.09

* The spans were determined based on the Hamm et al. 6-Hz criterion and met the deflection criterion. However, it is impossible for floors having a frequency between 4.5 Hz and 8 Hz to meet the acceleration criterion. Therefore, the spans did not fully meet the Hamm et al. criteria.

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CHAPTER 8

Fire performance of cross-laminated timber assemblies

Lead Authors
Christian Dagenais
Lindsay Ranger
Noureddine Bénichou

Reviewers
Marc Alam
Andrew Harmsworth
Joseph Su

SP-532E (Ch.8)

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ABSTRACT

Since the first edition of the CLT Handbook in 2011, cross-laminated timber (CLT) has been used in numerous applications ranging from single-family dwellings to mid- and high-rise wood buildings. CLT has been shown to be a promising wood-based structural component and has great potential to provide cost-effective building solutions for residential, commercial, and institutional buildings as well as large industrial facilities. Design provisions for CLT has recently been implemented in the 2019 Edition of Canadian Design Standard for Engineering Design in Wood, and a code proposal has been submitted to Codes Canada for recognition of a method for determining fire-resistance ratings for mass timber elements, including CLT, in an upcoming edition of NBCC. This Chapter 8 provides the up-to-date information related to fire performance attributes of CLT elements conforming to the bi-national product manufacturing standard ANSI/APA PRG 320.

CLT elements are used in building systems in a similar manner to concrete slabs and solid wall elements as well as those from heavy timber construction by limiting concealed spaces due to the use of mass timber elements, thereby reducing the risk of concealed space fires. Moreover, CLT construction typically uses CLT panels for floor and load-bearing walls, which allow inherent fire-rated compartmentalization, therefore further reducing the risk of fire spread beyond its point of origin (compartment of origin).

In an attempt to provide the scientific and technical information of CLT fire performance attributes for building code implementation, extensive fire testing has been conducted in North America on CLT elements, from a “component level” to a “system level”. On a “component level”, several fire-resistance tests and a number of surface flame spread and fire stopping tests have been conducted on CLT elements. The results have shown that CLT elements, with or without gypsum board protection, can achieve significant fire resistance, beyond 3 hours in some cases. Surface flame spread tests confirm that the risk of ignition of mass timber elements is greatly reduced compared to traditional interior wood finish products. Tests have also shown that fire stops approved for concrete construction are suitable for CLT elements, so long as adequate detailing is provided. The informative calculation method from Annex B of the Update No.1 of CSA O86 is detailed in this revised Chapter 8. A refined stepped charring model, as initially developed in the 2014 revision of this Chapter 8 and validated by test data, is also being discussed.

Lastly, a discussion on the use of CLT as vertical exit stair shafts as an alternative to traditional noncombustible construction is presented, in addition to an overview on how to incorporate CLT in a performance-based fire design. Fire safety during construction is also addressed.

8.1 INTRODUCTION

Cross-laminated timber (CLT) has been used in numerous applications ranging from single-family dwellings to mid-rise and tall wood buildings. As more research becomes available, particularly in relation to the fire performance of CLT, the number of larger and taller CLT projects being approved has grown, which has resulted in the construction of several high-profile CLT buildings in Canada. Tallwood House at Brock Commons at the University of British Columbia is one of the tallest hybrid (wood and concrete) buildings in the world. Another example is the 13-storey Origine project located in Québec City, which received approval as a result of the fire tests presented herein. CLT is a promising wood-based structural component and has great potential to provide cost-effective building solutions for residential, commercial, and institutional buildings, as well as for large industrial facilities. Chapter 8 provides the most up-to-date information related to fire performance attributes of CLT elements conforming to the bi-national product manufacturing standard ANSI/APA PRG 320, *"Standard for Performance-Rated Cross-Laminated Timber"* (1).

CLT is manufactured in a manner similar to glued-laminated timber elements, following ANSI/APA PRG 320, which provides requirements and test methods for qualification and quality assurance, for performance-rated CLT. A number of CLT manufacturers are already accredited to meet this standard for design and manufacture of CLT elements in Canada and in the United States (2, 3, 4, 5). Further discussion on the manufacturing process and quality assurance can be found in Chapter 2 of this Canadian CLT Handbook.

CLT elements not manufactured according to ANSI/APA PRG 320 with respect to lumber grades, species, densities and/or structural adhesives may not perform similarly to those detailed in this Chapter, and therefore are beyond the scope and applicability of this Chapter.

Acceptance of CLT construction into the Canadian regulatory environment necessitates compliance with the fire-related provisions of the NBCC (6, 7), among other regulations. Part 3 of Division B of the NBCC provides prescriptive fire safety provisions in order to meet these objectives, based on a building's major occupancy group, its height and area, as well as the presence of automatic fire sprinklers. Examples of prescriptive fire safety strategies include limitations on the use of combustible materials for structural and interior finishes, fire-resistance ratings of separating and loadbearing elements, limitations on the surface flammability characteristics of interior finishes, as well as provisions allowing safe means of egress for building occupants. All of these attributes are required in every building design and structural system, whether the building is of a combustible or noncombustible construction. Chapter 8 addresses some of the common code-mandated fire performance requirements.

Classification of a building according to its "type of construction", as defined in the NBCC, is one of the key elements in identifying the prescriptive limitations on the height and allowable floor areas of a building. CLT has been used in a variety of buildings across Canada and it is gradually becoming recognized as an accepted building material; however, it is not specifically addressed in the prescriptive language of the 2015 NBCC. With the publication of its Update No.1 in 2016, the CSA O86-14 standard "*Engineering Design in Wood*" (8) now specifically includes structural design provisions for CLT. However, Update No. 1 has not yet been referenced in the NBCC. It is expected that the 2020 NBCC will reference the 2019 edition of CSA O86, including the structural design provisions for CLT.

CSA O86-14 also includes the new informative Annex B, which deals with fire resistance of large cross-section wood elements. It is noted that this is an informative annex and therefore its use by designers needs to be formally accepted by the Authorities Having Jurisdiction (AHJs) as an alternative solution. A code proposal has been submitted to Codes Canada for recognition of Annex B as an acceptable method for determining fire-resistance ratings for mass timber elements, including CLT, in an upcoming edition of NBCC.

Encapsulated mass timber construction (EMTC), as proposed for inclusion in the 2020 Edition of the NBCC, is defined as a new type of construction in which a degree of fire safety is attained by the use of encapsulated mass timber elements (that includes CLT), with an encapsulation rating and minimum dimensions for the structural timber members and other building assemblies. If accepted, this will facilitate the use of CLT elements in residential and commercial buildings up to 12 storeys. The proposed provisions have been adopted in British Columbia.

CLT and heavy timber elements both achieve an inherent degree of fire safety from their larger dimensions, in comparison to lightweight lumber, due to their ability to char at a slow, predictable rate, and the avoidance of concealed spaces under floor and roof elements. This Chapter provides the basis not only for establishing the fire resistance of CLT elements, but also demonstrates how other fire safety-related attributes can be achieved by using CLT.

8.2 BACKGROUND

Mass timber products are generally known to perform well under fire conditions due, to a slow rate of charring, which generates a thick layer of low-density insulating char and thereby protects the timber below from elevated heat effects. Charring is a material-specific property attributed to timber; understanding this behaviour is fundamental in estimating the reduced thickness of full-strength timber when exposed to fire, which designers can use to calculate a member's residual strength for a given fire exposure.

Several fire tests have been performed by CLT manufacturers on a proprietary basis. There is also a range of full-scale standard fire-resistance tests performed with CLT assemblies under various structural loading that are publicly available in the literature. An adapted methodology predicting fire-resistance performance for CLT assemblies has been developed in Europe and is currently being used on a proprietary basis by many European CLT manufacturers (9, 10, 11, 12, 13). The European model essentially follows the principles prescribed in Eurocode 5: part 1-2 (EN1995-1-2) (14) applicable to timber components and is based on an extensive campaign of finite element numerical simulations as well as a number of fire tests. While some guidance is provided with respect to the separating function of CLT assemblies, most work is aimed at evaluating their loadbearing function, based on a given charring rate. As of 2019, this new method had yet to be implemented in the European regulatory environment, but it possibly will be included in the next edition of EN1995-1-2.

According to the European work, it was found that various factors may affect the performance of CLT in fire to some degree, including—but not limited to—wood species, the type of adhesive used, the thickness of the panel (number of laminates), the thickness of the laminates, the type of fire exposure, the panel-to-panel joint configuration, and the protection methods used. The same outcomes were found from the results of the North American fire tests summarized in this Chapter.

A fire-resistance design method for CLT was first published in Canada in 2011 in the first edition of the CLT Handbook; the method was largely based on the European approach and was similar to methods used in Canada that are applicable to other mass timber elements such as solid wood and glued-laminated timber (15). Similar to the European approach, the 2011 method evaluated the loadbearing function of CLT assemblies; it was developed with the limited Canadian data available at that time.

In 2014, an alternative approach was introduced in the revised edition of this Chapter, based on new test results, which provided provisions to evaluate the separating function of CLT assemblies (integrity); this approach included refinements to the structural and charring models with respect to the loadbearing function of CLT assemblies.

This 2019 edition of Chapter 8 details the method for evaluating the separating function, and the calculation method found in Annex B of CSA O86-14 (R2019), for evaluation of the loadbearing function of CLT elements exposed to a standard fire.

It is noted that the models listed above were developed with CLT elements conforming to ANSI/APA PRG 320, up to its 2017 edition. This bi-national manufacturing standard has been revised in early 2018 and new mandatory elevated temperature performance requirements for adhesives have been implemented. These changes will have a positive impact on the charring behavior of CLT elements. The intent of these new performance requirements is to limit or prevent localized heat delamination (i.e., fall-off) from occurring, thus improving the fire resistance of a given thickness of CLT, and will likely allow for a revised calculation method for determining fire-resistance ratings using a lower constant charring rate throughout (vs. using an increased effective charring rate or a stepped charring model), depending on the degree of conservatism that is chosen to be maintained. Further details are provided in Subsection 8.5.2.1 of this Chapter.

This updated Chapter 8 also includes new guidance on the use of encapsulation methods, fire safety during construction, detailing of service penetrations, considerations for vertical exit shafts, and support for the development of performance-based designs.

In order to facilitate the acceptance of proposed code provisions for the design of CLT panels with regard to fire resistance in Canada and the U.S., a number of research projects were launched at FPInnovations during the past years, in close collaboration with National Research Council Canada and other industry members such as the Canadian Wood Council and the American Wood Council. The main objectives of these projects were to:

1) Determine the fire resistance of CLT panels through full-scale testing
2) Develop mechanics-based calculation procedures for fire resistance of CLT assemblies
3) Develop a small-scale test protocol for evaluating CLT adhesive's fire performance.

The information collected served as the basis for developing and validating design procedures published in the fire chapter of the 2013 U.S. edition and the revised 2014 Canadian edition of the CLT Handbooks (16, 17), as well as for the methodology in Annex B of the 2016 Update No.1 of CSA O86-14. Those methodologies provide a mechanics-based calculation method that has been suitably adapted to the current structural design methodologies for ambient conditions found in the National Design Specification for Wood Construction (NDS) (18) and CSA O86, applicable to large timber elements. The methods were found to predict average fire-resistance times for CLT wall and floor assemblies that closely track actual fire-resistance times for tested assemblies.

8.3 FIRE SAFETY IN BUILDINGS

From its first edition, published in 1941, and up to and including the 1995 edition, the National Building Code of Canada (NBCC) had historically been published as a prescriptive code. With the publication of the 2005 edition of the NBCC as an objective-based code, the prescriptive provisions within the NBCC became the deemed-to-satisfy solutions to the Code objectives and represent the minimum performance levels to be met.

In the NBCC, fire safety provisions are based, in principle, on the NFPA 550 Fire Safety Concepts Tree (19, 20), where fire impact management and ignition prevention are the two primary concepts (Figure 1). Ignition prevention may be addressed by following the National Fire Code of Canada (NFCC) (21), while fire impact management can be addressed by the prescriptive provisions in Part 3 of Division B of the NBCC, entitled "*Fire Protection, Occupant Safety and Accessibility*".

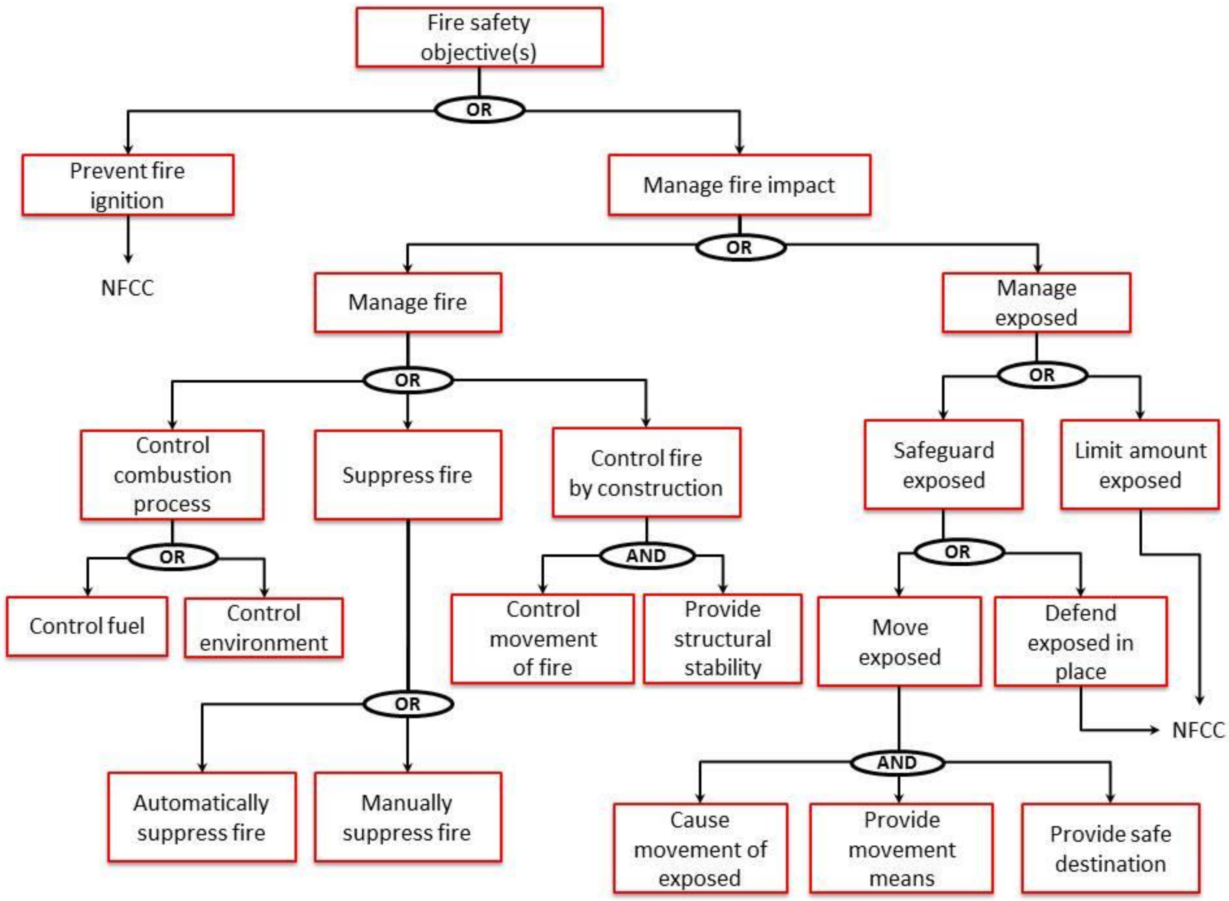


Figure 1 NFPA Fire Concepts Tree adapted to the Canadian regulatory environment

Figure 1 demonstrates the many factors that need to be considered for fire safety in buildings and how these factors are inter-related. In this figure, an *"and"* connection means that all branches need to be satisfied to meet a specific fire safety objective, while an *"or"* connection means that any one branch will achieve the fire safety objective it is connected to. For example, using either an automatic or a manual fire suppression system will satisfy the *"Suppress fire"* objective. On the other hand, *"Control fire by construction"* will be achieved only if the construction is designed in such a way that it can control the movement of fire and can provide the required structural stability (which relates to the fundamentals of the fire-resistance concept as detailed in Section 8.5 of this Chapter).

8.3.1 Objectives

The NBCC provides technical provisions for the design and construction of new buildings, as well as for the alteration, change of use, and demolition of existing buildings. The intent of the NBCC is to establish requirements addressing the following four objectives, which are fully described in Division A of the NBCC:

1. Safety (OS)
2. Health (OH)
3. Accessibility for persons with disabilities (OA)
4. Fire and structural protection of buildings (OP)

The Objectives describe, in very broad and qualitative terms, the overall goals that the NBCC's requirements are intended to achieve. They also describe undesirable situations and their consequences, which the NBCC aims to avoid. The NBCC recognizes it cannot entirely prevent or eliminate all undesirable events or risks. Therefore, its objectives are to *"limit the probability"* of *"unacceptable risk."* It is therefore assumed, within the NBCC, that an undesirable situation may occur and means shall be provided to limit its consequences.

8.3.2 Fire Performance Attributes of CLT

CLT elements have the potential to provide excellent fire resistance, comparable to that of other building materials, including noncombustible materials. This is due to the inherent nature of thick timber members to char slowly at a predictable rate, allowing mass timber systems to maintain significant structural capacity for extended durations, when exposed to fire.

As with any combustible material, CLT may contribute to the growth of a compartment fire. Frangi et al. (22) were among the first to study the impact of additional fixed fuel load from CLT panels on fire growth. They evaluated a three-storey CLT building constructed with 85-mm thick CLT wall panels and 142-mm thick CLT floor slabs exposed to a natural, full-scale fire. In this particular experiment, walls were protected with a face layer of 12.7-mm fire-rated gypsum board (directly exposed to fire) and a base layer of 12.7-mm standard gypsum board, while the ceilings were protected with 25.4-mm mineral wool insulation and a layer of 12.7-mm fire-rated gypsum board (Figure 2a). In an attempt to replicate similar fire severity, such as that encountered in typical residential dwellings, a design fire load of 790 MW/m^2 was used and burned for slightly over 1 hour. It is reported that flashover occurred after about 40 minutes, due to the initial low levels of ventilation provided. The fire severity started to decline after 55 minutes and was extinguished, as planned, after one hour. Furthermore, the measured charred depth on the gypsum-protected CLT compartment elements was very low, ranging from approximately 5 to 10 mm. No elevated temperatures were measured and no smoke was observed in the room above the fire room. From this full-scale design fire test, it may be concluded that CLT buildings can effectively be designed to limit fire spread beyond the point of fire origin.

As the desire to construct with CLT has grown, in particular for taller and larger buildings in North America, so has the demand for a better understanding of how CLT may contribute to a compartment fire. This also includes understanding the impact of leaving some CLT surfaces exposed, i.e., without gypsum board or other protection.

Studies conducted at Carleton University in Ottawa, Ontario showed an increase in fire growth in fully exposed CLT room fires (Figure 2b), leading to faster flashover conditions when compared to those from CLT rooms lined with gypsum board (23). Where CLT was protected by two layers of gypsum board, the fire self-extinguished when all combustibles were consumed and the CLT provided no noticeable contribution to fire growth, duration, or intensity.

The findings of a real-scale fire test of a representative 8.2-m x 6.4-m (27-ft. x 21-ft.) residential suite (i.e. apartment) conducted at the National Research Council Canada (NRCC) laboratory in Ottawa indicated that using 2 layers of 12.7-mm Type X gypsum board delayed the effect of the fire on the CLT structural elements (Figure 2c). The fire separations on the floor of the fire origin remained intact, limiting fire spread for more than 2 hours (24).

Research has also been conducted in an attempt to understand the contribution of exposed mass timber to fire growth. Two calculation methods have been proposed by Barber (25) to assess this contribution, based on the amount of exposed mass timber from ceilings or walls within a compartment and its resulting emitted energy. This phenomenon is discussed in more detail in Section 8.10 of this Chapter. Recent large-scale testing has also been completed to further establish the safe amount of exposed mass timber in a compartment, and further develop and refine calculation methods to estimate how these surfaces contribute to a fire.

a) 3-storey CLT fire test in Tsukuba, Japan (22)

b) CLT room fire test at Carleton University in Ottawa, Canada (23)

c) CLT apartment fire test at NRCC in Ottawa, Canada

Figure 2 Real-scale CLT fire tests

A series of five tests were conducted by the USDA Forest Service's Forest Product Laboratory (USDA FPL) in cooperation with the American Wood Council, the Bureau of Alcohol, Tobacco, Firearms, and Explosives (ATF), and the Forest Service's State and Private Forestry in Beltsville, Maryland (26). The tests involved two-storey 175-mm thick 5-ply CLT structures with compartments measuring approximately 9.2 m x 9.2 m (30 ft. x 30 ft.). Two ventilation openings were provided for the full 9-ft. height, without the inclusion of a lintel. The compartments were furnished with a fuel load density in accordance with Bwalya et al. (27), ranging from 412 MJ/m^2 in the living room to 807 MJ/m^2 in the kitchen. The main variables studied in these tests were the size and location of exposed mass timber surfaces. The following is a summary of the details of the tests:

Test 1: Full encapsulation - entire CLT surfaces covered with 2 layers of 15.9-mm Type X gypsum board

Test 2: Partial encapsulation - 30% exposed CLT on center portion of ceiling in living room and bedroom

Test 3: Exposed CLT on walls in bedroom and living room

Test 4: Exposed CLT throughout - automatic sprinkler activation

Test 5: Exposed CLT throughout - automatic sprinkler activation delayed 20 minutes.

The fires in Tests 1 to 3 (ranging from full encapsulation to having some walls exposed) all displayed similar heat release rate profiles, and the fire was found to decay naturally in all three. In Tests 4 and 5 using sprinklers, the sprinklers were effective at lowering temperatures within a few minutes and preventing flashover from occurring. The published results (26) were presented to the International Code Council (ICC) to support the acceptance of code change proposals to include tall wood buildings in the U.S. ICC codes.

a) Overview of CLT compartment

b) Flashover during Test 1

Figure 3 CLT fire tests to support acceptance of code changes for tall wood buildings (26)

The National Fire Protection Association (NFPA) Fire Protection Research Foundation (FPRF) also commissioned a series of tests to quantify the contribution of exposed CLT surfaces to a compartment fire. A series of six tests was completed by NRCC at the National Institute of Standards and Technology (NIST) National Fire Research Laboratory in Gaithersburg, Maryland (28). The 5-ply 175-mm CLT compartments were either fully or partially encapsulated, and the size of the window opening was varied (either small or large). The main parameters of each of the tests were:

Test 1: Baseline test, all CLT surfaces with 3 layers of 15.9-mm Type X gypsum board, small opening

Test 2: Same as test 1, but with a larger opening

Test 3: One wall with exposed CLT, other walls with 2 layers of 15.9-mm Type X gypsum board, ceiling with 3 layers of 15.9-mm Type X gypsum board, large opening

Test 4: Ceiling with exposed CLT, all walls with 3 layers of 15.9-mm Type X gypsum board, small opening

Test 5: One wall with exposed CLT, other walls and ceiling with 3 layers of 15.9-mm Type X gypsum board, small opening

Test 6: One wall and ceiling with exposed CLT, other walls with 3 layers of 15.9-mm Type X gypsum board, small opening.

These tests demonstrated that the encapsulation was effective at delaying and preventing involvement of the CLT and limited or eliminated their contribution to the fire. When the amount of exposed mass timber increased, the contribution to the fire also increased. The smaller window opening resulted in more burning within the compartment and more severe fire conditions.

These research projects highlight fire hazards associated with construction using CLT in situations where no active fire protection is provided, and where the fire burns over extended periods without intervention or response from fire services. However, as required in many buildings by the NBCC, including all mid-rise and tall buildings, automatic sprinklers would provide active protection against fire growth, as they would be activated before significant fire growth and fire involvement of exposed CLT panels would occur. The effectiveness of sprinklers was demonstrated through Tests 4 and 5 of the USDA FPL.

It is noted that the tests conducted by Carleton University, USDA FPL and NRCC used CLT elements conforming to the applicable edition of ANSI/APA PRG 320, at that time. CLT elements manufactured to the updated ANSI/APA PRG 320-2018 can be expected to have better performance, as discussed later. CLT elements not manufactured according to ANSI/APA PRG 320 may perform differently.

8.3.3 CLT and Fire Provisions of Building Codes

CLT elements are used in building systems in a manner similar to concrete slabs and solid wall elements, as well as floor and roof elements in heavy timber construction, to limit combustible concealed spaces, thereby reducing the risk of concealed space fires.

Moreover, CLT construction typically uses CLT panels for floor and loadbearing walls, which can provide fire-rated compartmentalization, thereby further reducing the risk of fire spread beyond its point of origin (compartment of origin).

The various types of construction defined within the NBCC are discussed in detail in Section 8.4 of this Chapter. Section 8.4 will also highlight areas where CLT components may be used in compliance with the NBCC in the future, once the 2019 edition of CSA O86 that includes the CLT design provisions is referenced in the NBCC (likely the 2020 edition of the NBCC). Until that time, use of CLT can be achieved via an "alternative solution" in accordance with Clause 1.2.1.1.(1)(b) of Division A of the NBCC. (Note: In Québec, CLT can be used as described in Section 8.4 in compliance with the Québec Construction Code as a result of the acceptance of CSA O86-14 Update 1-2016 by the Régie du bâtiment du Québec as an "acceptable solution").

8.4 TYPES OF CONSTRUCTION AND OCCUPANCY CLASSIFICATION

Building systems in the NBCC are broadly classified into two categories solely based on a single material chemical property: combustibility. Whether structural and other building materials pass or fail the ULC S114 (29) test for noncombustibility determines whether they may be used in either one of the two types of construction recognized in the NBCC, that is: 1) noncombustible construction, and 2) combustible construction. In general, noncombustible structural materials may be used in either type of construction, while combustible structural materials are to be used only in combustible construction, except where specifically permitted for use in noncombustible construction.

The NBCC defines "noncombustible construction" as "that type of construction in which a degree of fire safety is attained through the use of noncombustible materials for structural members and other building assemblies". The intent in requiring noncombustible structural materials is "to limit the probability that combustible construction materials within a storey of a building will be involved in a fire, which could lead to the spread of the fire within the storey during the time required to ensure occupant safety and for emergency responders to perform their duties, which in turn could result in harming people and damaging the building" (30). However, the NBCC does not explicitly address the actual expected performance/behavior of noncombustible structural materials when exposed to fire conditions. The NBCC also provides a long list of exceptions to the requirement to use only noncombustible materials in buildings required to be of noncombustible construction, including many wood products, which are found in Subsection 3.1.5. of Part 3 of Division B of the NBCC.

Combustible construction is simply any type of construction that does not meet the requirements for noncombustible construction. It includes conventional, lightweight wood-frame and mass timber construction systems such as CLT, including the code-defined "heavy timber construction". The NBCC defines heavy timber construction as "that type of combustible construction in which a degree of fire safety is attained by placing limitations on the minimum sizes of structural wood members and on the thickness and composition of wood floors and roofs and by the reduction of concealed spaces under floors and roofs". It is a historic, prescriptive construction type in which solid-sawn timber or glued-laminated timber may be used (Articles 3.1.4.6. and 3.1.4.7. of Part 3 of Division B of the NBCC). As a subset of combustible construction, the code-defined heavy timber construction type is permitted to be used anywhere combustible construction is permitted to be used and is not required to have a fire-resistance rating of more than 45 minutes. Moreover, the NBCC recognizes the enhanced level of fire safety in buildings made of heavy timber construction, by allowing its use in several applications in lieu of noncombustible construction.

The 2020 NBCC is expected to recognize the advantages of mass timber construction and effectively introduce a new construction type, called "Encapsulated Mass Timber Construction", which would include protected CLT construction.

8.4.1 Building Size Relative to Occupancy

Most fire safety provisions set forth in the NBCC are based on the building's major occupancy, the principal occupancy for which a building, or part thereof, is used or intended to be used. This includes subsidiary occupancies that form an integral part of the principal occupancy. The classification of buildings by major occupancy type may be found in Subsection 3.1.2 of Part 3 of Division B of the NBCC. This building classification is the starting point for other fire safety provisions, namely allowances for combustible construction, height and area limits, and fire-resistance ratings of assemblies.

Building sizes are regulated in Subsection 3.2.2 of Part 3 of Division B of the NBCC and are dependent on the major occupancy type. Typically, stricter fire safety provisions are imposed when a building becomes larger and taller, and the allowance for combustible construction becomes limited. Greater building areas are allowed when an automatic sprinkler system is installed.

8.4.2 Use of CLT in Combustible Construction

A CLT element or panel is a prefabricated solid engineered wood product made from at least three (3) orthogonally-bonded layers of finger-jointed, solid-sawn, visually-graded or mechanically-graded lumber or structural composite lumber (SCL) (as defined per ANSI/APA PRG 320).

Part 4 of Division B of the NBCC requires that buildings and their structural members made of wood conform to CSA O86. Since the 2016 Update No.1 of CSA O86-14 has not yet been referenced in the NBCC or any of the provincial building codes except in Québec, CLT is not a building material that is recognized as an "acceptable solution". The exception to this is in Québec, where the Régie du bâtiment du Québec (RBQ) recognized Update No. 1 as an acceptable solution in April 2017.

However, now that CLT structural design equations have been implemented in the wood design standard CSA O86, a designer has all the information needed to develop a structural design at ambient temperature conditions using CLT as an alternative solution within a building permitted to be of combustible construction in those provinces other than Québec. In Québec, CLT already is generally accepted for use in buildings allowed to be of combustible construction.

Once an edition of the CSA O86 that includes CLT is referenced in a specific building code (most likely the 2020 NBCC) or accepted by an authority-having-jurisdiction (e.g. the RBQ), CLT may be used in any form of combustible construction as long as it has been designed to provide the requisite fire-resistance rating. The rating required is generally 45 minutes or 1 hour; in some cases (e.g., for some major occupancy separations), it can be up to 2 hours. CLT assemblies have been tested to CAN/ULC S101 and have been demonstrated 45 minutes, 1 hour, and much greater fire-resistance ratings, as detailed in Subsection 8.5 of this Chapter.

8.4.3 Use of CLT in Heavy Timber Construction

As defined in the NBCC, "heavy timber construction" is a special type of combustible construction allowed to be used where combustible construction is permitted and is not required to have a fire-resistance rating greater than 45 minutes. It is also generally built from a post-and-beam structural system, where the loadbearing elements need to conform to the minimum dimensions indicated in Table 3.1.4.7. of Part 3 of Division B of the NBCC, as well as other provisions in Article 3.1.4.7.

Similarly, to plank floors and roofs in heavy timber construction, CLT is most likely to be used in thick and massive floor, roof, and wall panels. Provided that CLT elements are of sufficient dimensions to provide a 45-minute fire-resistance rating, CLT may reasonably be accepted as an alternative solution equivalent to heavy timber construction. It is possible that in the future, CLT panels may be added to the NBCC's prescriptive provisions for the heavy timber construction type, which would then make it an acceptable solution.

The 2015 *International Building Code* (IBC), the model building code widely used in the United States, implemented new prescriptive provisions to allow the use of CLT in Type IV construction, also known as Heavy Timber Construction, for exterior walls, interior walls, floors, and roofs. Specific minimum dimensions are provided for CLT walls, floors, and roofs; specific provisions are also provided to allow CLT use in exterior walls, where noncombustible materials are traditionally required in Type IV construction.

8.4.4 Use of CLT in Noncombustible Construction

In general, noncombustible construction, as defined in the NBCC, requires structural elements to be of noncombustible materials. Subsection 3.1.5. of Part 3 of Division B currently does not include the use of CLT as an acceptable solution for structural elements in noncombustible construction.

However, in Article 3.2.2.16., the roofs (as well as their supporting structure) of buildings up to two-storeys are permitted to be of heavy timber construction, regardless of building area or type of construction required, provided the building is entirely protected by automatic sprinklers. Therefore, there is potential for CLT to be used as roof elements and some wall elements in low-rise and large buildings required to be of noncombustible construction, as an alternative solution to the permission allowing roofs and supports of heavy timber construction.

Moreover, a design professional wanting to use CLT in a project beyond the current prescriptive code limitations found in Division B of the NBCC for structural elements (that would otherwise be required to be of noncombustible materials), is allowed to develop an alternative solution in accordance with Clause 1.2.1.1.(1)(b) of Division A of the NBCC. Such an alternative solution may range from the development of a simple equivalency to undertaking a full performance-based design. Developing an alternative solution requires demonstration that the proposed solution achieves at least the minimum level of performance provided by the Division B acceptable solution it is proposing to replace, in the areas defined by the objectives and functional statements attributed to the applicable acceptable solution. In this regard, a technical guide and technical reports for designing taller and larger wood buildings have recently been published; these provide various approaches and discuss concepts that may be useful in developing a design using mass timber structural elements as an alternative solution, in buildings otherwise required to be of noncombustible construction (31, 32, 33, 34). Further information related to performance-based design may be found in Section 8.10 of this Chapter.

8.4.5 Encapsulated Mass Timber Construction

With the desire to build taller and larger buildings using mass timber construction and the restrictions provided by the current types of construction recognized in the NBCC, a number of prescriptive code change proposals to Part 3 of Division B of the NBCC were submitted to Codes Canada by the Canadian Wood Council (CWC) over the last few years. These were proposed to facilitate both existing and new proposed uses of mass timber elements, including CLT. Many seem likely to be included in the next edition of the NBCC, once the Code’s review, revision, consultation, and approval processes are complete.

Among the proposed changes that have progressed in the national code process is the creation of a new type of construction called “encapsulated mass timber construction” (EMTC). This proposed construction type may be considered the logical evolution of the existing heavy timber construction type in the NBCC, resulting from the development of new and innovative structural engineered wood products, and increased knowledge and skills from designers (structural, fire, seismic, etc.) and manufacturers, as well as changes in complementary building regulations and standards. Mass timber construction applying the encapsulation concept has been used in most of the real-scale compartment fire tests referenced in Subsection 8.3.2 of this Chapter.

It is noted that the Code language provided in this Subsection is for information purposes only and is currently being reviewed by Codes Canada and several stakeholders, including public review. Actual code language and prescriptive provisions, when/if approved, may be different than those presented herein.

Encapsulated mass timber construction (EMTC), as currently expressed in the proposed Code changes, is defined as a new type of construction in which a degree of fire safety is attained by the use of encapsulated mass timber elements, with an encapsulation rating and minimum dimensions for the structural timber members and other building assemblies. Many of the requirements applicable to noncombustible construction are also to be applied to EMTC, and there are limitations placed on combustible concealed spaces in the EMTC proposed change package. For these and other reasons, in the NBCC proposed changes, EMTC is considered a construction type separate from the combustible construction type, whereas the heavy timber construction type is considered a special type (subset) of combustible construction. Large cross-section timber products, such as solid-sawn timber, structural glued-laminated timber (glulam), structural composite lumber (SCL), cross-laminated timber (CLT), and nail-laminated timber (NLT) are envisioned to be used within this new type of construction.

“Encapsulation rating” refers to the time that a material or assembly of materials will delay the ignition and combustion of structural timber elements when exposed to fire under specified test conditions and performance criteria. Based on fire tests, research, and analysis, a minimum encapsulation rating of 50 minutes was chosen as the requirement to be applied to EMTC; this is consistent with the encapsulation performance afforded by two layers of 13-mm (½”) Type X gypsum board directly fastened to mass timber elements, when exposed to the CAN/ULC S101 standard fire (35). Encapsulation materials such as gypsum board, gypsum concrete, noncombustible materials, materials conforming to 3.1.5.1.(2) to (4) of Division B of the NBCC, or any combination of the materials listed previously are permitted to be used, provided they offer the minimum 50-minute encapsulation rating. A new CAN/ULC S146 standard test method has been developed by NRCC, FPInnovations, and CWC (36) to standardize the assessment (including instrumentation during testing) of encapsulation materials and assemblies of materials.

Moreover, provisions have been proposed to permit some exposed mass timber surfaces within a fire compartment and/or suite to suit architectural needs. Based on recent results of CLT compartment fire tests conducted by NRCC (37), some amount of exposed surface and specific wall/ceiling configurations have been developed in an attempt to limit the contribution of these exposed surfaces to fire growth and intensity.

It has been proposed that buildings for residential use (Group C) with a building area of up to 6000 m^2, and business and personal services (Group D) occupancies with a building area of up to 7200 m^2 and up to twelve-storeys in height, be permitted to be constructed using the EMTC construction type. The proposal also includes permission for several mixed-use occupancy types, including mercantile (Group E) and Group A Division 2 (assembly) occupancies such as restaurants, to be located on the lower storeys of buildings using the EMTC type of construction.

As discussed earlier in this Chapter, Code provisions for heavy timber construction include minimum size requirements for a variety of elements. This is a result of the desire to retain expected fire resistance performance, when creating prescriptive code language to permit the use of historic post-and-beam construction (sometimes called “mill construction”). In a similar way, it was found necessary to prescribe minimum dimensions for mass timber structural elements in EMTC, to provide assurance that they continue to exhibit the expected fire performance characteristics of mass timber, rather than those of small-dimensioned wood elements typically used in wood-frame construction, including reduced ignition propensity and reduced average rate of fuel contribution, for a period of time. Table 1 summarizes the proposed prescriptive minimum dimensions for EMTC. It is noted that larger dimensions may be required to satisfy fire-resistance rating requirements.

Table 1 Proposed minimum dimensions for EMTC

Structural Wood Elements	Minimum Thickness (mm)	Minimum Width x Depth (mm x mm)
Walls for fire separation or exterior walls	96	-
Walls that require a fire-resistance rating, but are not for fire separation	192	-
Floors and roofs	96	-
Beams, columns, and arches (2- or 3-sided fire exposure)	-	192 x 192
Beams, columns, and arches (4-sided fire exposure)	-	224 x 224

While the above proposals for EMTC and its use to permit the design and construction of tall wood buildings as an acceptable solution are still being reviewed at the National level, some provinces took the initiative to move forward on their own. In 2015, the Régie du bâtiment du Québec (RBQ) published a design guide for allowing buildings up to twelve storeys made of mass timber construction (thirteen storeys in total when over a one-storey reinforced concrete podium) (38). The design provisions prescribed in the RBQ guide were largely based on the design concepts used in the Origine project located in Québec City. The guide provides prescriptive design provisions and additional information related to fire safety, structural design, and fire safety during construction, as well as administrative directives. When a designer fully follows and respects these design provisions, there is no need to request approval of an alternative solution from the RBQ, as the guide is intended to be a “provincially-approved” alternative solution.

In 2017, the Ontario Ministry of Natural Resources and Forestry (OMNRF) released a tall wood building reference guide (39). The OMNRF guide is intended not only to help applicants in the design of tall wood buildings as alternative solutions achieving the level of performance required by Ontario’s Building Code, but also to assist authorities-having-jurisdiction (AHJs) in Ontario in understanding some of the key aspects that should be addressed in such an alternative solution approach. The guide provides design information related to fire safety and structural design. As a result of the different regulatory environment in Ontario compared to Québec, in contrast to the use of the RBQ guide, a designer following the OMNRF guide is still required to request an approval of their alternative solution from the AHJ. It also does not provide prescriptive solutions related to designing such buildings.

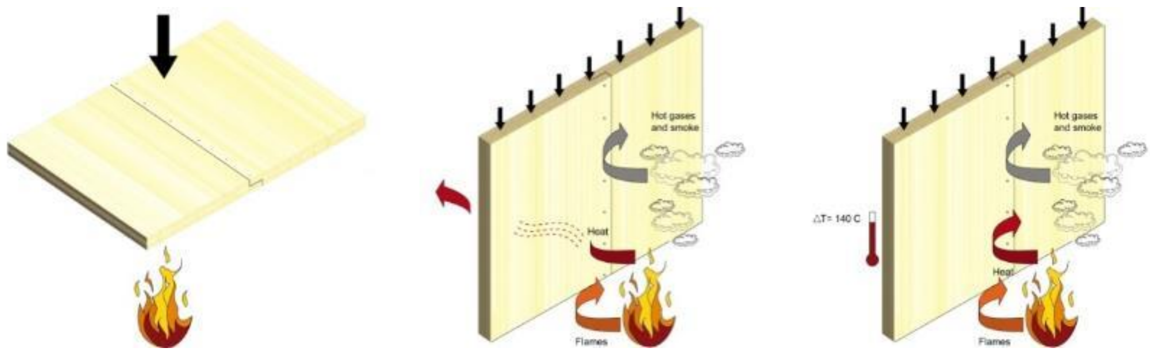
British Columbia has implemented an approach using ‘Site Specific Regulations’ to facilitate new mass timber buildings; this approach was used to obtain approval for tall wood buildings (e.g. above 6 storeys) based on demonstration of equivalent performance or following the proposed Code changes for EMTC construction. More recently, the Province of British Columbia adopted the proposed provisions for the 2020 Edition of the NBC for 12-storey mass timber buildings.

8.5 FIRE RESISTANCE OF CLT ELEMENTS

Building regulations require that key building assemblies exhibit sufficient fire resistance to allow time for occupants to escape and to minimize property losses, as well as for emergency responders to carry out their duties. The intent is to limit the possibility of structural collapse and to subdivide a building into fire-rated compartments. A fire compartment, as defined in the NBCC, means an enclosed space in a building that is separated from all other parts of the building by enclosing construction providing a fire separation having a required fire-resistance rating. The objective of the compartmentalization concept is to limit fire spread beyond its point of origin by using boundary elements (e.g., walls, ceilings, floors, partitions, etc.) having a fire-resistance rating no less than the minimum ratings prescribed by the NBCC. Fire-resistance ratings are usually assigned in whole numbers of hours (e.g., 1 hour and 2 hours) or parts of hours (e.g., ½ hour or 30 minutes, and ¾ of an hour or 45 minutes).

The main aspects of fire performance of building assemblies are assessed by conducting fire-resistance tests in accordance with CAN/ULC S101 (35). Fire resistance is defined as the period of time that a building element, component, or assembly maintains the ability to perform its separating function (i.e., confining a fire by preventing or retarding the passage of excessive heat, hot gases, or flames), continues to perform a given loadbearing function, or both, when exposed to fire under the specified conditions of the test and performance criteria. More specifically, for most assemblies a standard fire-resistance test entails three performance criteria (Figure 3). The time at which the assembly can no longer satisfy any one of these three criteria defines the assembly's fire resistance.

1. **Structural resistance**: the assembly must support the applied load for the duration of the test (relates to the loadbearing function).
2. **Integrity**: the assembly must prevent the passage of flame or gases hot enough to ignite a cotton pad (relates to the separating function).
3. **Insulation**: the assembly must prevent the rise in temperature of the unexposed surface from being greater than 180°C at any location, or an average of 140°C measured at a number of locations, above the initial temperature (relates to the separating function).



a) Structural resistance b) Integrity c) Insulation

Figure 4 Fire-resistance criteria per CAN/ULC S101

It should be noted that structural elements that are not required to perform a separating function (e.g., beams, columns, and walls that are required to have a fire-resistance rating but are not required to be a fire separation) need only satisfy the structural resistance criterion.

When designing buildings with CLT elements, it is necessary to use assemblies that comply with the prescribed fire-resistance ratings. In some instances, such as for non-loadbearing partition wall assemblies, only the separating function is required in defining the fire resistance (e.g., the assembly must meet only the insulation and integrity criteria). In the case of loadbearing walls and all floor/roof assemblies, the assembly must provide both the loadbearing function (structural resistance) as well as the separating function for no less than the duration of the fire-resistance rating required in the NBCC, when the assembly is also required to be a fire separation.

In this regard, in this Chapter, the discussion of the determination of the fire resistance of CLT assemblies has therefore been split into requirements based on loadbearing function and requirements based on separating function.

8.5.1 Test Method – CAN/ULC S101

The NBCC requires that the fire-resistance rating of a building assembly be assessed by subjecting a specimen of the assembly to a standard fire-resistance test, CAN/ULC S101, which is a test method that provides a fire severity and performance criteria that are similar to those set forth in other international fire-resistance standards, such as ASTM E119 (40) and ISO 834 (41). They require structural and separating elements, including wall (Figure 5) or floor (Figure 6) assemblies, to be exposed to a post-flashover fire specified by a time-temperature curve (Figure 7). CAN/ULC S101 is a standard test method that evaluates the fire performance of a system of materials under a specified fire scenario for comparison with other systems, and is not related to the combustibility of materials, which is typically assessed in accordance with CAN/ULC S114 "*Standard Method of Test for Determination of Noncombustibility in Building Materials*" (29).

a) Unprotected CLT before test

b) Unexposed surface during test

c) Protected CLT after test

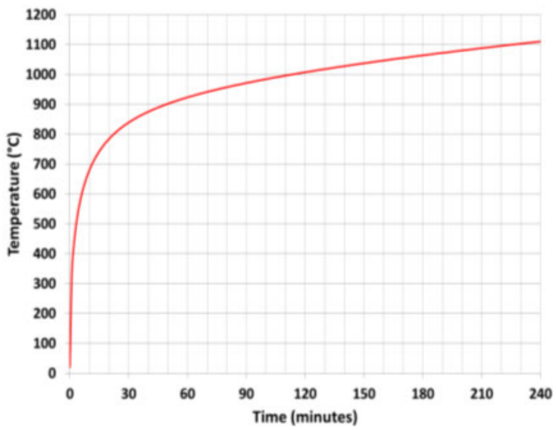
Figure 5 CLT fire-resistance wall tests conducted at NRCC in Ottawa, ON

a) Unprotected CLT before test

b) Unexposed surface during test

c) Protected CLT after test

Figure 6 CLT fire-resistance floor tests conducted at NRCC in Ottawa, ON



Time (min)	Temperature (°C)
5	538
10	704
30	843
60	927
120	1010
240	1093

Figure 7 Standard time-temperature curve from CAN/ULC S101

For loadbearing assemblies, the test standard requires the assembly to be structurally loaded during fire exposure. In order to satisfy the structural criterion, the test specimen needs to sustain the applied load, called "*superimposed load*" as per CAN/ULC S101, throughout the fire test period. Such loading requirements are applicable to all structural elements, including timber, steel, and concrete assemblies. The superimposed load may represent a "*full*" specified load condition or a "*restricted*" load use condition. According to CAN/ULC S101, the full specified load condition is satisfied when the test specimen is subjected to the specified gravity loads that produce a factored load effect as close as practical to the factored resistance of the test specimen, determined in accordance with the appropriate limit-states design standard, such as CSA O86 Engineering Design in Wood (8). A test conducted under the maximum load ensures that the fire-resistance rating obtained is appropriate for use in any equal or lesser loading conditions (assuming they satisfy the loadbearing requirements for ambient conditions/normal design), as well as in assemblies with the same construction configuration that are larger or smaller in area than the specific area size that is tested. This provides a degree of conservatism in using the standard test for all load cases in the field, since it is not cost effective to test all possible load use conditions.

Moreover, the limited short test span for floors (typically around 4.7 m) and test height and length for walls (typically around 3 m and 3.7 m, respectively), and the maximum loading capacity from the fire laboratories steel-framed test apparatus, make it very difficult for CLT assemblies to be evaluated under full loading conditions, particularly for wall elements. In fact, most North American fire-resistance test facilities do not have the capacity to load many CLT assemblies to their full loading conditions. Moreover, it is doubtful that many CLT assemblies (floors, in particular) will be structurally loaded anywhere near their ultimate capacity in practice (i.e. in the field); quite often, they may be carrying loads much lower than their design capacity due to serviceability limits (deflection or vibration). As a result, fire-resistance-rating calculation methods developed based on standardized testing results that actually take into consideration the effect of load/capacity ratio on the fire-resistance rating (i.e., increased fire-resistance rating at lower load-to-capacity ratios), and are validated to a variety of load-to-capacity ratios, can provide a more optimized and economical fire-resistance-rated design. As such, a rational fire-resistance calculation methodology, such as the ones typically developed for mass timber products (including CLT), which are based on first principles such as charring rate, effective reduced cross-section, and load ratio, and are calibrated to the results seen in standard fire-resistance testing, is more suitable to ensure an efficient and economical CLT building design.

8.5.2 CSA O86 Annex B Mechanics-Based Methodology

The design methodology for determining the structural performance aspect of fire-resistance ratings for CLT elements presented in this Chapter is a mechanics-based design method, based on limit states design calculation procedures. It calculates the capacity of exposed CLT elements using basic wood engineering mechanics for fire-resistance calculations. It is good practice to use the methods presented for the calculation of fire-resistance ratings of up to 3 hours only, limited by the currently available test data, even though there is some indication that the methodology may provide increasingly-conservative values for higher fire-resistance ratings. In addition, the proposed calculation methodology is valid only for CLT manufactured with lumber boards and adhesives meeting the requirements set forth in ANSI/APA PRG 320, as of its 2017 edition.

The actual mechanical and physical properties of the specific product utilized are used as parameters within the method, and the remaining capacity of the element is directly calculated for a given period of time of exposure (in this case, to the standard fire-resistance test fire exposure). The section properties are computed assuming an applicable charring rate for a given time of fire exposure. Reductions in the strength and stiffness of wood directly adjacent to the char layer are addressed by a zero-strength layer for uniformity and consistency with a calculation methodology developed by Dagenais & Osborne (42) applicable to timber, glued-laminated timber, and structural composite lumber of large cross-sections, as well as with other methodologies used around the world. The typical resistance factor used in limit-states design is set to unity and the duration of load is considered short-term. Lastly, the element's specified strength properties are adjusted to the average strength value (i.e., mean value), based on existing accepted statistical procedures, such as ASTM D2915 (43), used to evaluate allowable properties for structural lumber.

8.5.2.1 Structural Adhesives

A constant one-dimensional charring rate is commonly assumed for solid-sawn and glued laminated softwood members that are fully exposed to fire on one side only (i.e., unprotected) throughout the time of a standard fire exposure. This is with the assumption, based on testing and past experience, that glued structural products behave similarly to solid (non-glued) timber when exposed to fire and that the adhesive used does not exhibit delamination characteristics and/or becomes a limiting design factor.

An adhesive used in a glued structural product, such as CLT, should maintain its bond between wood members when exposed to high temperature, in order to prevent release at the bond line.

Some of the wood layers that are lost as a result of such a release may still have considerable residual strength; such a bond-line release should be accounted for in the methodology used to assign a fire-resistance rating to such an element type. Also, the adhesive should maintain sufficient strength when subjected to temperatures associated with charring of the wood (generally taken as 300°C), in order to ensure that charred layers do not fall off, as this would result in a sudden exposure of the next lamination that has not yet fully heated and initiated pyrolysis, otherwise, such an effect should be accounted for in the methodology used to assign a fire-resistance rating to such an element type. This is, in part, because char performs an important insulating function and helps protect the remaining wood cross-section. If the adhesive is not capable of preventing fall off or separation of layers or elements, then consideration should be given to the impact this has on the charring rate and the associated effect on the fire-resistance rating of the assembly. Theoretical considerations concerning adhesives and the CLT manufacturing process can be found in Chapter 2 of this CLT Handbook.

ANSI/APA PRG 320 (up to and including the 2017 edition) requires that, when the CLT is intended for use in Canada, adhesives used in the manufacture of the CLT meet the requirements of CSA O112.10 *Evaluation of Adhesives for Structural Wood Products (Limited Moisture Exposure)* (44), and Sections 2.1.3. and 3.3. (ASTM D7247 S*tandard Test Method for Evaluating the Shear Strength of Adhesive Bonds in Laminated Wood Products at Elevated Temperature*) of AITC 405 *Standard for Adhesives for use in Structural Glued Laminated Timber* (45). In addition, adhesives shall be evaluated for heat performance in accordance with Section 6.1.3.4 of the U.S. Product Standard PS1 on *Structural Plywood* (46).

The CLT panels used for developing the fire-resistance calculation methodology presented herein were manufactured with a structural polyurethane (PUR) adhesive conforming to ANSI/APA PRG 320 (up to its 2012 edition), for use in both the U.S. and Canada. During full-scale fire research on CLT walls and floors (47, 48, 49, 50), localized pieces of the charred layers were observed to fall off when the temperature at the CLT lamination interface (glue line) was between 115 to 250ºC, indicating that the adhesive released. Such behavior resulted in an increased “effective” charring rate of the second and subsequent laminations, which was also found to be more pronounced for thinner laminations, as shown in Figures 8 and 9. It also suggested that, if it was desirable in certain applications (for example, in some tall wood buildings) to ensure no release of the adhesive occurred prior to reaching a temperature of 300ºC or more, additional evaluation of the adhesive used in the CLT for heat durability would be needed.

Moreover, while it was not a mandatory requirement until the 2017 edition of ANSI/APA PRG 320, if the heat performance test as described in the product standard DOC PS1 (46) determines that a structural adhesive exhibits heat delamination that could potentially impact the charring behavior of a product when compared to traditional (unglued) lumber, ANSI/APA PRG 320 recommends that appropriate design adjustments be made. However, the product standard does not provide any guidance or specific recommendations on the types of design adjustments that could be made.

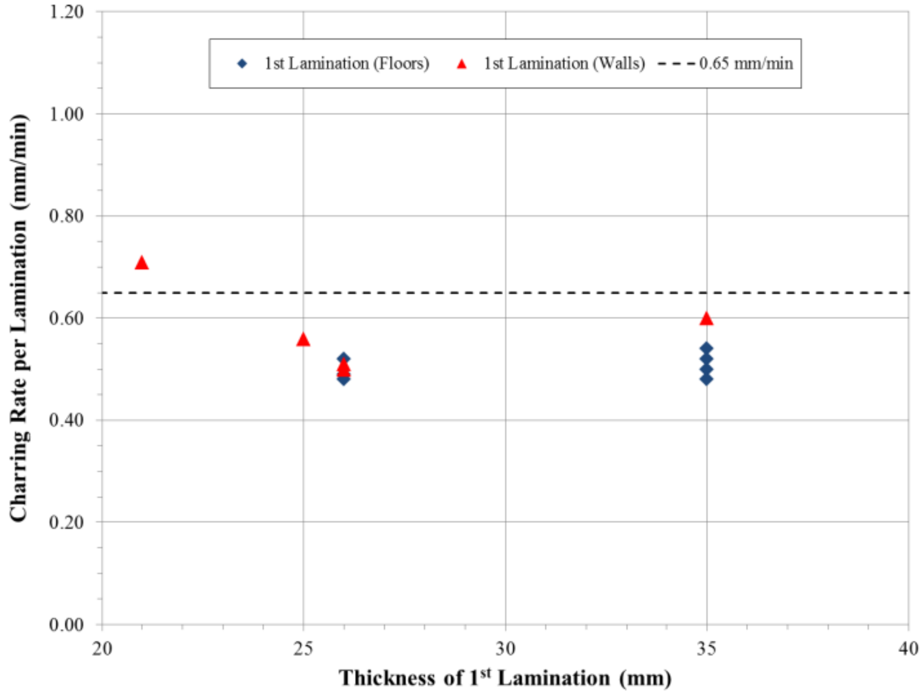


Figure 8 **Charring rate of first lamination - Test results from C. Dagenais (50)**

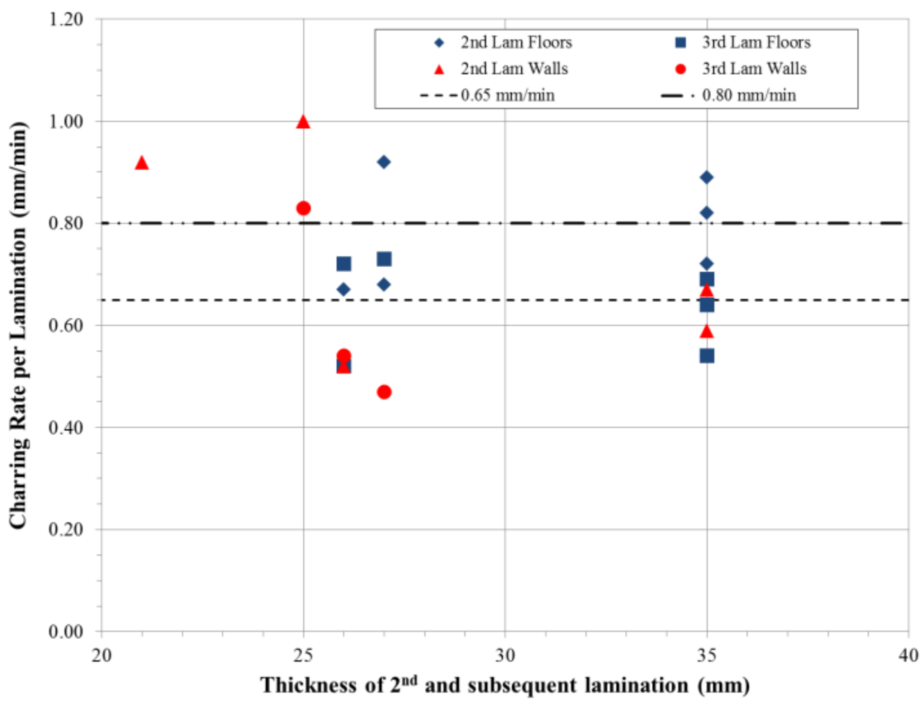


Figure 9 **Charring rate of second and subsequent laminations - Test results from C. Dagenais (50)**

To account for the observed heat delamination characteristics of current adhesives in CLT, two fire-resistance calculation methods were developed in Canada over the last few years. It is noted that such specific behavior is not unique to North American CLT products, as it was also observed and reported in European and Japanese fire-resistance tests, which also resulted in the development of "increased" (or "stepped") charring models (9, 10, 11, 12, 51, 52, 53, 54). The two methods developed in Canada are:

1) **Annex B from the 2016 Update No. 1 of CSA O86-14**: Annex B uses a constant "effective" charring rate for implicitly considering the effect of heat delamination of any type of CLT thickness and configuration. This approach was calibrated using standard fire-resistance test results of Canadian CLT assemblies (55). The methodology provides conservative predictions of structural failure times when compared to experimental data. This model is now incorporated into the informative Annex B calculation methodology for large wood elements in the 2016 Update No.1 of CSA O86-14, and a code change request has been submitted to the National Code to recognize the Annex B methodology as part of an acceptable solution process for developing fire-resistance ratings for several types of mass timber elements, including CLT, in the 2020 edition of the NBCC. The method is further explained in Subsection 8.5.2.2 of this Chapter, along with comparisons of its fire-resistance rating predictions for code compliance (Subsection 8.5.7).

2) **Chapter 8 of the CLT Handbook – 2014 Revised Version**: This method uses a "stepped" charring model that explicitly accounts for the effect of lamination thickness of any CLT configuration, as observed and reported by Dagenais (50). This stepped model is described in the 2014 version of Chapter 8 of the Canadian CLT Handbook (17). This model is very similar to the stepped charring model currently mandated in the 2015 *National Design Specifications for Wood Construction* (18). This approach was also calibrated using Canadian fire-resistance tests of CLT assemblies and provides a prediction of the char front at any given time, while also allowing for conservative predictions of structural failure times when compared to experimental data. Designers may use this approach for determining the char front location in an accurate manner.

As noted earlier, the 2018 edition of ANSI/APA PRG 320, which is likely to be the edition of the standard that will be referenced in the 2019 edition of CSA O86, contains additional fire-related performance requirements for adhesives, in order to eliminate the delamination effect.

On this matter, as mentioned in Section 8.2 of this Chapter, a number of research projects were launched at FPInnovations (56, 57, 58) to develop a small-scale test protocol for evaluating the fire performance of CLT adhesives, in an attempt to eliminate the heat delamination effect. It was argued that ASTM D7247, as well as its temperature target of 220°C typically stipulated for evaluating adhesives at elevated temperatures for glulam and I-joists, may not properly reflect the actual behavior of a CLT element, due to its orthogonal configuration. The findings of this research allowed for the addition of a qualification small-scale flame test method that is now mandatory in the 2018 edition of ANSI/APA PRG 320.

In addition, research performed at the Southwest Research Institute (59) resulted in the development of a large-scale test method to evaluate adhesives to be used in CLT products in Canada and the USA, using a CLT floor-ceiling assembly tested using a specified fire exposure for 240 minutes. The test methodology has been incorporated as an additional mandatory requirement in the 2018 edition of ANSI/APA PRG 320.

Adhesives fulfilling these new performance requirements are expected to exhibit "non-delaminating" properties. As a result, it may be possible to use a lower constant one-dimensional charring rate throughout the fire duration in the calculation methodology, to determine fire-resistance ratings for CLT assemblies rather than using the "increased" or "stepped" charring model. A series of full-scale fire resistance tests with CLT made with different adhesives meeting these new requirements were recently conducted by FPInnovations (60) to evaluate this possibility. It was found that a constant linear charring rate could be used, and changes will be proposed for a future edition of Annex B of CSA O86, to reflect the improved performance.

8.5.2.2 Charring Rate and Char Depth

When designing in Canada using the informative Annex B of the 2016 Update No.1 of CSA O86-14, a preliminary verification of the char depth (x_c, in mm) as a function of the entire duration (t, in minutes) of the fire-resistance required is needed for determining the applicable design charring rate for structural fire-resistance design. A one-dimensional charring rate of 0.65 mm/min is to be used, as per Equation [1], if the char depth is not expected to pass the first bond line between the exposed lamination and the subsequent lamination (i.e., first exposed lamination is not fully charred) within the time of the fire-resistance-rating period required, as illustrated in Figure 10a). Otherwise, an increased "effective" charring rate of 0.80 mm/min, as per Equation [2], is to be used when the char front is expected to be located beyond the first bond line (first lamination entirely charred), as illustrated in Figure 10b). As an example, presuming that the first lamination is 21 mm in thickness and the required fire-resistance is 45 minutes, the char depth would exceed the first bond line (45 min x 0.65 mm/min = 29.25 mm > 21 mm); therefore a charring rate of 0.80 mm/min (Equation [2]) would be used when designing for fire-resistance.

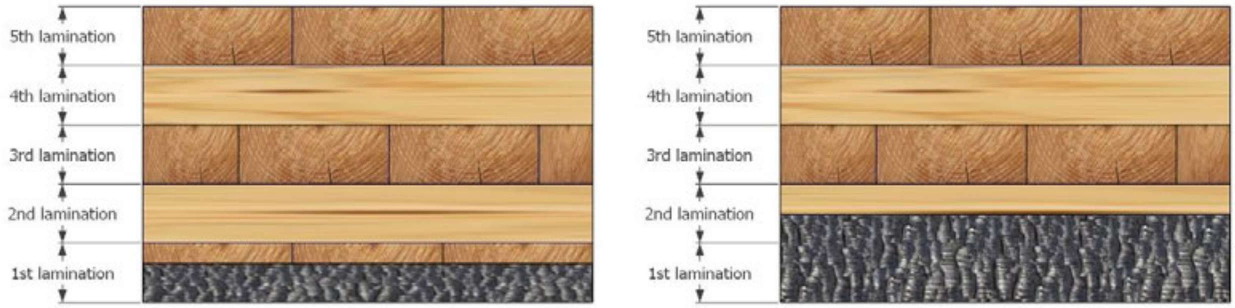
Due to the lack of test data and charring data on laminates thinner than 21 mm, it is recommended to limit the application of this methodology to CLT made with laminations of at least 19 mm in thickness (e.g. nominal 1" x 6" lumber boards), unless a CLT product that is in compliance with the 2018 edition of ANSI/APA PRG 320 is used.

$$x_{c,0} = \beta_0 \cdot t \quad [1]$$

where β_0 = 0.65 mm/min when $x_{c,0} \leq 1^{st}\ lamination$

$$x_{c,n} = \beta_n \cdot t \quad [2]$$

where β_n = 0.80 mm/min when $x_{c,n} > 1^{st}\ lamination$



a) β_0 = 0.65 mm/min b) β_n = 0.80 mm/min

Figure 10 Design charring rate based on expected char depth

After the char depth is calculated, an additional thickness is subtracted to account for the loss of strength in the heated zone beneath the char front (i.e., a zero-strength layer, x_t). If the exposure time is less than 20 minutes, then the zero-strength layer is to vary linearly from zero at time zero to a depth of 7 mm at 20 minutes (Equation [3]). The same practice is used in EN1995-1-2 for the heated zone for glulam and heavy timber. The justification for the 7-mm thickness is also provided by Schaffer (61). The effective char depth ($x_{c,eff}$) may then be evaluated as per Equation [4].

$$x_t = \begin{cases} \left(\frac{t}{20}\right) \times 7 & \rightarrow t < 20\ min \\ 7 & \rightarrow t \geq 20\ min \end{cases} \quad [3]$$

$x_{c,eff} = x_{c,0} + x_t$ or $x_{c,eff} = x_{c,n} + x_t$, whichever is applicable [4]

As such, considering the above one-dimensional charring rates and the zero-strength layer, as well as the fact that the applicable design charring rate is influenced only by the thickness of the first lamination exposed to fire (i.e. 0.65 vs. 0.80 mm/min) and the fire-resistance rating desired, Table 2 provides resulting effective char depths based on the thickness of the first lamination.

Table 2 Effective char depth for CLT design per Annex B of CSA O86-14

Standard Fire Exposure	Effective Char Depth, $x_{c,eff}$ (mm)					
	Thickness of first lamination (fire-exposed lamination) (mm)					
	19 (¾”)	21 (⅞”)	25 (1”)	32 (1¼”)	35 (1⅜”)	38 (1½”)
30 minutes	31	27				
45 minutes	43			36		
1 hour	55					
1.5 hours	79					
2 hours	103					

As mentioned previously, it is anticipated that when designing for CLT manufactured with adhesives that do not exhibit heat delamination characteristics at temperatures below the char front (i.e., would char at a similar rate as solid wood), it may be allowable to use a slower constant charring rate, e.g. 0.65 mm/min, to calculate the effective char depth without the need to verify whether the first bond line is exceeded or not. Full-scale fire-resistance tests were conducted by Oregon State University (62) to evaluate the charring behavior of a number of structural adhesives. A CLT panel manufactured with Douglas-Fir laminates and a melamine-formaldehyde (MF) adhesive (traditionally known not to exhibit heat delamination) was evaluated for 2 hours of standard fire exposure. The resulting global charring rate, based on thermocouple measurement at the second bond line, was found between 0.61 and 0.64 mm/min. When using the increased “effective” charring rate of 0.80 mm/min, the predicted failure time under these test conditions would be 106 minutes (thus, would be quite conservative as it would not predict a 2-hour fire-resistance rating), while the predicted time to failure would increase to 129 minutes if 0.65 mm/min was used throughout (thus predicting a 2-hour fire-resistance rating, which is closer to the test result).

Another example could be the calculation of the effective char depth of a CLT assembly made of 35-mm thick laminations exposed to a standard fire for 2 hours. Using a constant charring rate of 0.65 mm/min throughout, the effective char depth calculated would be reduced to 85 mm versus 103 mm, when using a charring rate of 0.80 mm/min. At 85 mm, the third ply, in the major strength direction, has a residual thickness of 20 mm versus only 2 mm remaining when subtracting the 103 mm. One can observe that such a difference in the effective char depth would impact the resulting loadbearing capacity, as the char depth may now fall within a longitudinal rather than a transverse layer and/or allow for a greater remaining thickness of the lamination in the major strength direction.

As mentioned in the previous Section, a new series of fire-resistance tests on CLT made with adhesives meeting the new performance requirements of the 2018 edition of ANSI/APA PRG 320 has been conducted, to further evaluate the possibility of using a lower constant charring rate when calculating the fire-resistance rating of CLT assemblies using Annex B of CSA O86.

8.5.2.3 Approximation of Member Strength and Capacity

Parametric analyses are typically used to establish 5th percentile tolerance limits with a 75% confidence level. Strength properties such as bending, tension, compression, and shear are derived with this parametric analysis (deriving a 5th percentile), while modulus of elasticity (E) for serviceability is derived from the mean values (i.e., average value). ASTM D2915 "*Standard Practice for Sampling and Data Analysis for Structural Wood and Wood-Based Products*" (43) provides a methodology for estimating the parametric tolerance limit (PTL or p^{th} percentile) for a normal statistical distribution, as a function of test data average values and coefficients of variation.

The average design strength of CLT laminates may be approximated by multiplying design values (F_b, F_t, F_c and E_{05}) by the appropriate strength adjustment factor (K_{fi}) shown in Table 3 and the other adjustment factors shown in Table 4. Additional information regarding the strength adjustment factors may be found in Dagenais & Osborne (42).

Table 3 Strength adjustment factor (K_{fi}) for CLT structural fire-resistance design

CLT Stress Grade	Lumber Type in Major Strength Axis	K_{fi}
E-grade	Machine stress rated lumber (MSR)	1.25
V-grade	Visually graded sawn lumber	1.50

Table 4 Applicable adjustment factors for CLT fire-resistance design

	Strength Adjustment Factor	Load Duration Factor	System Factor	Service Condition Factor	Treatment Factor	Curvature Factor	Size Factor [(1)]	Column Slenderness Factor [(2)]	Beam Lateral Stability [(2)]	Notch Factor
Bending	K_{fi}	K_D	K_H	K_S	-	-	-	-	-	-
Compression	K_{fi}	K_D	K_H	K_S	-	-	-	K_C	-	-
Tension	K_{fi}	K_D	K_H	K_S	-	-	-	-	-	-
Shear	K_{fi}	K_D	K_H	K_S	-	-	-	-	-	-
MOE (E_{05}) (Buckling)	*(3)*	K_D	K_H	K_S	-	-	-	-	-	-
MOE (E) (Serviceability)	-	K_D	K_H	K_S	-	-	-	-	-	-

(1) Factor to be based on initial cross-section dimensions.
(2) Factor to be based on reduced cross-section dimensions.
(3) Average value of E_{05} shall be taken as the tabulated value of E, rather than using K_{fi}.
System, service condition and treatment factors for CLT are to be taken as unity.

8.5.3 Fire Resistance of CLT – Structural Requirement

The procedure set forth in CAN/ULC S101 is applicable to floor and roof assemblies and requires fire exposure to the underside of the specimen being tested. When wall assemblies are evaluated, the specimen is exposed to fire from one side only. This structural requirement is essential in limiting the risk of structural failure or collapse of physical elements due to the effects of a fire.

Calculation of the structural fire-resistance failure time of CLT floor or wall assemblies is outlined in the following four (4) steps. The time at which the CLT assembly can no longer support the applied load defines its structural fire-resistance (t_{Struc}). Once the CLT assembly capacity has been determined using the effective section properties from Subsection 8.5.2.2 and the member strength approximations from Table 3 of this Chapter, CLT elements can be designed using the design provisions found in Clause 8 "*Cross-Laminated Timber (CLT)*" of the 2016 Update No.1 of CSA O86-14. All member strength and cross-sectional properties should be adjusted prior to the interaction calculations. The char layer may be assumed to have zero strength and stiffness. Calculations are typically made for a unit width of CLT panel (e.g., 1 meter).

Figure 11 shows a CLT element exposed to fire and some of the nomenclature used in calculating its fire resistance. Initially, CLT elements are typically of a symmetrical configuration (i.e., a balanced layup). However, as the char depth progresses through the CLT, its cross-section reduces and becomes asymmetrical (i.e., an unbalanced layup where the resulting neutral axis is shifting away from the fire-exposed side). Special considerations should be given to asymmetrical CLT elements, namely thicker CLT walls elements where the applied axial load (typically assumed to be initially concentric) may become eccentric and thereby may induce second order effects (i.e. combined axial compression and bending). In such situations, verifying only the axial compression resistance may not be sufficient. Engineering judgment is required to determine if applicable eccentricities due to charring for fire-resistance design need to be taken into consideration in calculating the fire resistance of CLT walls (see Section 8.5.7).

Figure 11 Nomenclature used in calculating fire resistance of CLT exposed to fire

Step 1: Calculation of the effective char depth

The effective char depth ($x_{c,eff}$) can be determined by multiplying the appropriate charring rate (β_o *or* β_n) by the duration of fire exposure (t) (i.e. the required fire-resistance rating, such as those prescribed in Part 3 of Division B of NBCC), with the additional inclusion of the zero-strength layer (x_t), as per Equation [4]. Alternatively, the effective char depth for common laminate thicknesses can be taken directly from Table 2.

Step 2: Determination of the effective reduced cross-section

The effective reduced cross-section depth remaining for design under fire conditions (h_{fire}) may be calculated using Equation [5].

$$h_{fire} = h - x_{c,eff} \qquad [5]$$

where:

h_{fire} = effective reduced cross-section depth, mm

h = thickness of the CLT panel (typically its initial thickness), mm.

According to Annex B of the 2016 Update No. 1 of CSA O86-14 and as explained in the 2017 Wood Design Manual (63), only the plies that run in the direction of the applied stress (i.e. major strength direction for most applications, other than 2-way bending) should be considered in calculating the section properties and member resistance. This assumption was made in an effort to simplify the calculations and is deemed insignificant in terms of resulting fire resistance. However, one can observe through calculations that neglecting the effect of the cross-plies (i.e. minor strength direction) when determining the section properties will not result in the same stiffness and resistance values (e.g. $(f_bS)_{eff,f,0}$ and $EI_{eff,f,0}$) than those published in Table A4 of ANSI/APA PRG 320.

It is recommended to consider the cross-plies when calculating the CLT bending moment, in accordance with Clause 8.4.3.2 of the 2016 Update No.1 of CSA O86-14 (for EI_{eff} and GA_{eff}) and ANSI/APA PRG 320, but to neglect them when computing the axial compressive resistance per Clause 8.4.5.3 (for I_{eff} and A_{eff}). Considering the cross-plies will allow the determination of section properties consistent with those published in ANSI/APA PRG 320. Engineering judgment is required to select appropriate first principles when calculating the section properties.

Should h_{fire} fall within a cross-ply (i.e., between plies that are parallel to the applied stress), h_{fire} is automatically reduced to the distance from the unexposed face to the edge of the nearest inner ply located in the major strength direction (Figure 12).

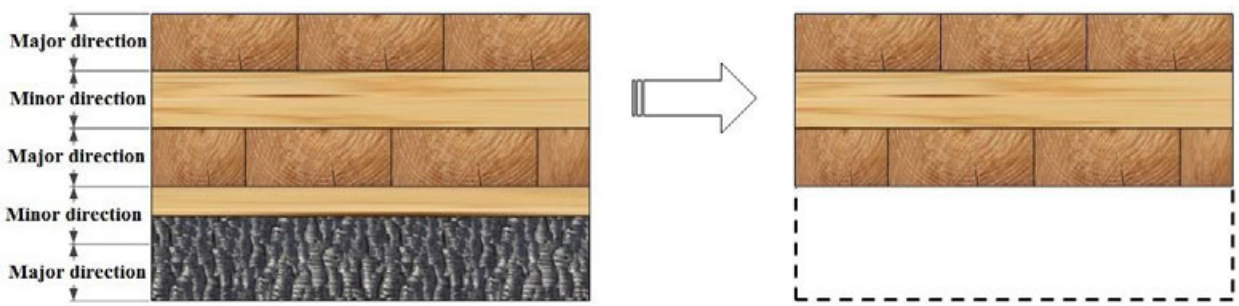


Figure 12 Reduced cross-section due to charring

Step 3: Finding the location of the neutral axis and section properties of the effective reduced cross-section

Equation [6] is the general form that is typically used to calculate the location of the neutral axis ($\bar{y}$), for elements subjected to out-of-plane bending. The effective bending stiffness and effective in-plane (planar) shear rigidity of the reduced cross-section are determined using Clause 8.4.3.2 of the 2016 Update No.1 of CSA O86-14, accounting for all layers.

$$\bar{y} = \frac{\sum_i \tilde{y}_i h_i E_i}{\sum_i h_i E_i} \tag{6}$$

$$(EI)_{eff,y} = \sum_{i=1}^{n} E_i b_y \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i b_y h_i z_i^2 \tag{7}$$

$$(GA)_{eff,zy} = \frac{\left(h - \frac{h_1}{2} - \frac{h_n}{2}\right)^2}{\left[\left(\frac{h_1}{2G_1 b_y}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i b_y}\right) + \left(\frac{h_n}{2G_n b_y}\right)\right]} \tag{8}$$

where:

b_y = width of the CLT panel for the major strength axis, typically 1 m or 1,000 mm

E_i = modulus of elasticity of laminations in the *i*-th layer, MPa

= E, for laminations in the longitudinal layers, MPa

= E_{90}, for laminations in the transverse layers, MPa

G_i = shear modulus of laminations in the *i*-th layer, MPa

= G, for laminations in the longitudinal layers, MPa

= G_{90}, for laminations in the transverse layers, MPa

h = thickness of the CLT panel (taken as h_{fi} for fire design), mm

n = number of layers in the CLT panel

h_i = thickness of laminations in the *i*-th layer, mm

$\bar{y}$ = distance from the unexposed surface of the CLT panel to the neutral axis (mm)

$\tilde{y}_i$ = distance from the unexposed surface of the CLT panel to the centroid of ply *i* (mm)

z_i = distance between the center point of the *i*-th layer and the neutral axis, mm

Estimations for the transverse modulus of elasticity, shear modulus, and rolling shear modulus may be found in the 2016 Update No.1 of CSA O86-14 (footnotes (3), (4) and (5) of Table 8.2.4).

Step 4: Calculation of the structural resistance

Using the effective reduced cross-section determined in Step 3, the member capacity may be calculated by multiplying the adjusted stress design values. The design values are determined through accepted engineering design procedures related to fire design of wood members.

The calculation of the factored bending moment and the compressive resistance parallel to grain have been split into Steps 4a and 4b respectively, due to the different interactions used.

Step 4a: Calculation of the factored bending moment resistance

The factored bending moment resistance of a CLT element is calculated using Clause 8.4.3 of the 2016 Update No.1 of CSA O86-14. The effective section modulus of the reduced cross-section in the major and minor strength axes ($S_{eff,y}$ *and* $S_{eff,x}$) is calculated as per Equation [9]. In fire design, the cross-section is most likely to become asymmetric. As such, the neutral axis may no longer be located at mid-depth (*h*/2) and so this term must be changed to the actual distance (*c*) between the extreme tension fibre and the resulting neutral axis (e.g. tension fibre is at the exposed/bottom side for a floor element).

$$S_{eff,y} = \frac{(EI)_{eff,y}}{E} \cdot \frac{1}{c} \quad \text{and} \quad S_{eff,x} = \frac{(EI)_{eff,x}}{E} \cdot \frac{1}{c} \qquad [9]$$

The factored bending moment resistance in the major and minor strength axes ($M_{r,y}$ *and* $M_{r,x}$) of a CLT element is calculated based on the adjusted specified strength in bending of the wood and the effective section modulus of the reduced cross-section, as shown in Equations [10] and [11].

$$M_{r,y} = \phi F_b S_{eff,y} K_{rb,y} \qquad [10]$$

$$M_{r,x} = \phi F_b S_{eff,x} K_{rb,x} \qquad [11]$$

where:

ϕ = resistance factor, taken as 1.0 for fire design

F_b = f_b (K_D K_H K_{Sb} K_T K_{fi})

f_b = specified strength of laminations in the longitudinal or transverse layer, MPa

K_D = load duration factor, taken as 1.15 (short term)

K_H = system factor, taken as 1.0 for fire design

K_{Sb} = service condition factor for bending, taken as 1.0 for fire design

K_T = treatment factor, taken as 1.0 for fire design

K_{fi} = strength adjustment factor based on CLT stress grade (Table 3)

$K_{rb,y}$ = adjustment factor for bending moment resistance of CLT panels

= 0.85, for the major strength direction

$K_{rb,x}$ = adjustment factor for bending moment resistance of CLT panels

= 1.0, for the minor strength direction

Provided that the structural resistance of the effective reduced cross-section is greater than the effects induced by the specified design loads, the fire resistance of the CLT element will be equal to or greater than that of the fire-resistance time used in determining the reduced cross-section. Serviceability limit states, such as deflections, are usually not as much of a concern as ultimate limit states (i.e., strengths) during fire design considerations.

A sample calculation of the bending moment resistance of a CLT floor assembly is given in Subsection 8.5.9 of this Chapter.

Should a performance-based fire safety design approach be used in which the specific fire scenario has design fire(s) with time-temperature relations other than those specified in the standard CAN/ULC S101 fire-resistance test, additional analysis may be required. For example, a thermal and mechanical analysis may be needed in order to determine an appropriate charring rate and zero-strength layer.

Moreover, in such a design scenario and based on the judgment of the fire protection engineer, it may be appropriate to use the load factors suggested in paragraph 25 of Structural Commentary A of the *User's Guide – NBC 2010 Structural Commentary (Part 4 of Division B)*, along with appropriate resistance (ϕ) and adjustment factors in accordance with CSA O86. It is not recommended to use strength adjustment factors and the reduced load combination specified in paragraph A-25 of the Structural Commentary when conducting performance-based fire design.

Step 4b: Calculation of the factored compressive resistance parallel to grain

The factored axial compressive resistance parallel to the grain of a CLT assembly is calculated using Clause 8.4.5 of the 2016 Update No.1 of CSA O86-14, where only the layers oriented parallel to the axial force are considered and assumed to carry the load. The slenderness ratio (C_c) of a CLT element of constant rectangular cross-section shall not exceed 43, as per Equation [12]. This upper limit assigned to normal design should be maintained for fire-resistance design.

$$C_c = \frac{L_e}{\sqrt{12}r_{eff}} \leq 43 \qquad [12]$$

where:

$r_{eff} = \sqrt{\frac{I_{eff}}{A_{eff}}}$

I_{eff} = effective out-of-plane moment of inertia of the CLT panel accounting only for the layers with laminations oriented parallel to the axial load, mm^4

A_{eff} = effective cross-sectional area of the CLT panel accounting only for the layers with laminations oriented parallel to the axial load, mm^2

The factored compressive resistance under axial load (P_r) of a CLT element is calculated based on the adjusted specified strength in compression of the wood and the effective area of the reduced cross-section, as shown in Equation [13].

$$P_r = \phi F_c A_{eff} K_{Zc} K_C \qquad [13]$$

where:

ϕ = resistance factor, taken as 1.0 for fire design

F_c = f_c (K_D K_H K_{Sc} K_T K_{fi})

f_c = specified strength of laminations in the layer parallel to the applied axial load, MPa

K_D = load duration factor, taken as 1.15 (short term)

K_H = system factor, taken as 1.0 for fire design

K_{Sc} = service condition factor for compression parallel to grain, taken as 1.0 for fire design

K_T = treatment factor, taken as 1.0 for fire design

K_{fi} = strength adjustment factor based on CLT stress grade (Table 3)

K_{Zc} = size factor for compression for CLT element, based on initial cross-section

= $6.3\left(\sqrt{12} r_{eff} L\right)^{-0.13} \leq 1.3$

K_C = slenderness factor for CLT compression members, based on reduced cross-section

= $\left[1.0 + \frac{F_c K_{Zc} C_c^3}{35 E_{05} (K_{SE} K_T)}\right]^{-1}$

where: E_{05} = E of the laminations in the longitudinal layers (fire design only), MPa

As shown by test data and reported by many researchers (47, 51, 64), a CLT wall element exposed to fire from one side will experience a reduction in its loadbearing capacity, mainly due to the following four factors:

1) Reduction in the effective cross-area (A_{eff}) supporting the applied axial load
2) Reduction of the strength and stiffness of the actual CLT wall element, due to wood thermal degradation
3) Increase in the effective slenderness ratio (C_c)
4) Increase of the effective eccentricity of the applied axial load, due to charring of the CLT element (typically the applied load is assumed to be concentric before charring due to fire).

As the CLT wall element chars, it may be subjected to second-order effects (i.e., P-Δ effects), due to the transient shift of the neutral axis away of the fire-exposed surface (Figure 13). While the informative Annex B of the 2016 Update No.1 of CSA O86-14 does not explicitly address this behavioural interaction for fire-resistance design, its mandatory Clause 8.4.5.4.1 stipulates that bending moments due to eccentrically applied axial loads shall be taken into account using the combined bending and axial compressive load interaction, as per the general form shown in Equation [14]. However, it is not clear whether such verification is only applicable for axial loads being initially eccentric or also applicable to axial loads becoming eccentric (as with the case of the reduced cross-section of a CLT wall exposed to fire). It should be noted that the U.S. fire-resistance design methodology provided in Technical Report 10 (65) recognizes such structural behaviour by explicitly stipulating that the effects of combined axial compression and bending loads should be considered.

Considering the effect of combined loads when determining the structural fire-resistance of wood components is not new to CLT walls. It was also observed and reported by Bénichou et al. (66) when studying and modelling the fire-resistance of wood-frame stud walls. With respect to CLT walls, such effect may be of particular importance with thicker elements where a significant amount of char is expected. A 3-ply CLT is most likely to fail due to its axial capacity, while 5-ply and thicker elements may be subjected to the combined effect of bending and axial compression. Engineering judgment is required to determine if applicable eccentricities due to charring for fire-resistance design need to be taken into consideration in calculating the fire resistance of CLT wall elements (see Section 8.5.7). It is also noted that previous editions of the Canadian CLT Handbook provided an equation for combined bending and axial compression similar to that for glulam column elements (i.e. with the squared value of the axial term). With the collaboration of FPInnovations and the Canadian Wood Council, NRCC has conducted a number of tests in support of the CSA O86 Technical Committee in 2015 (67). The test data analysis suggested that it is not conservative to use the squared axial term for CLT wall elements; this is now reflected in the 2016 Update No.1 of CSA O86 CLT design provisions (Clause 8.4.6).

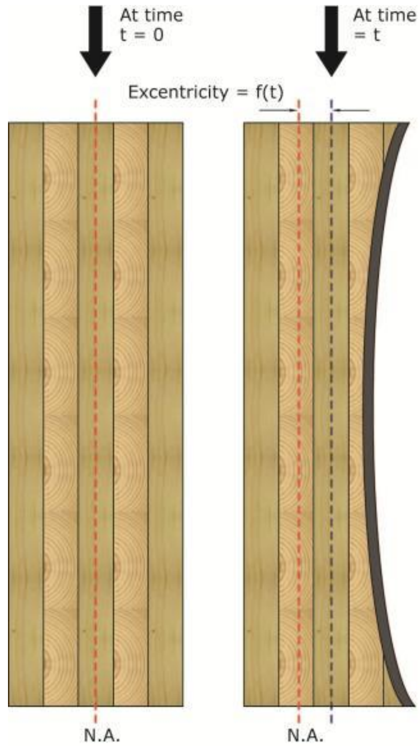


Figure 13 **CLT wall element subjected to combined bending and axial compression**

$$\frac{P_f}{P_r} + \frac{M_f}{M_r}\left[\frac{1}{1-\frac{P_f}{P_{E,v}}}\right] \leq 1 \qquad [14]$$

where:

P_f = specified compressive axial load for fire design

P_r = factored compressive resistance under axial load for fire design (see step 4b)

M_f = specified bending moment for fire design

M_r = factored bending moment resistance (see step 4a)

$P_{E,v}$ = Euler buckling load in the plan of the applied bending moment adjusted for shear deformation, N

$= \frac{P_E}{1+\frac{\kappa P_E}{(GA)_{eff}}}$

where:

P_E = Euler buckling load in the plane of the applied bending moment in accordance with Clause 7.5.12 of CSA O86-14, where E_{05} and I_{eff} are determined accounting only for the layers with laminations oriented parallel to the axial load, N

$$= \frac{\pi^2 E_{05} K_{SE} K_T I_{eff}}{L_e^2}$$

E_{05} = E of the laminations in the longitudinal layers (fire design only), MPa

κ = form factor, taken as 1.0 for rectangular cross-sections

GA_{eff} = effective in-plane (planar) shear rigidity of CLT element accounting for all layers (see step 3), N

Equation [15] is a reformatting of Equation [13] for explicitly considering potential eccentricities and applied out-of-plane bending moments.

$$\frac{P_f}{P_r} + \frac{1}{M_r}\left[M_f + \frac{P_f \Delta}{1 - \frac{P_f}{P_{E,v}}}\right] \leq 1 \qquad [15]$$

where:

M_f = maximum out-of-plane induced bending moment in fire design, N·mm

Δ = deflection due to axial load eccentricity, mm

$$= \frac{P_f(\Delta_f + e_0 + \Delta_0)L_e^2}{16(EI)_{eff,y}}$$

where:

Δ_f = deflection due to out-of-plane loading (bending), mm

e_0 = eccentricity of the axial load resulting from the shift of the neutral axis, mm

Δ_0 = initial wall imperfections at mid-height of the panel, usually taken as $L/500 + h/6$, where L is the panel height and h is the panel initial depth, mm

A sample calculation of a CLT wall assembly subjected to combined bending and axial loading is given in Section 8.5.10 of this Chapter.

Given that CLT wall elements exposed to fire from one side are subjected to second order effects, it is recommended to evaluate the interaction for combined bending and axial compression (Equation [15]). Engineering judgment is required to determine if applicable eccentricities due to charring for fire-resistance design need to be taken into consideration in calculating the fire resistance of CLT wall elements (see Subsection 8.5.7).

8.5.4 Fire Resistance of CLT – Integrity Requirement

As mentioned in Section 8.5 of this Chapter, integrity is one of the two (2) requirements of the separating function of building assemblies. The time at which the assemblies can no longer prevent the passage of flame or gases hot enough to ignite a cotton pad, when exposed to a standard fire, defines the integrity fire resistance (t_{Int}). For CLT this failure usually occurs at the panel-to-panel joint. This requirement is essential in limiting the risk of fire spread to compartments beyond the compartment of fire origin.

During a CLT compartment fire test conducted at Carleton University (Ottawa, Ont.), Medina (68) observed flame-through between CLT panel-to-panel joints. The wall elements were connected together using a half-lapped joint detail, and it had been initially planned to seal the joint using a fire-resistance rated caulking, as in the previous compartment fire tests. Unfortunately, the sealant was not applied, and hot gases flowed through the joint, resulting in large flaming outside the compartment (Figure 14). This demonstrates the utmost importance of sealing construction joints and having proper detailing and field installation.

a) Smoke leakage and charring

b) Flame-through

Figure 14 Flame-through in a half-lapped joint during a CLT compartment fire test (68)

Panel-to-panel joint performance depends on joint configuration and connection details (refer to Chapter 5 of this CLT Handbook), where an integrity failure may occur when the connection detail can no longer withstand the applied load in either shear or withdrawal. For instance, when using wood screws to connect CLT panels together, a minimum penetration of no less than 5 times the wood screw diameter is required for single shear connections.

As the exposed face chars over a period of time, the thickness that would provide an adequate lateral or withdrawal capacity is reduced. One can argue that as long as the char depth has not reached the internal spline or the location of the lapped joint, for example, a panel-to-panel joint could be deemed adequate to fulfil its integrity function. A number of intermediate- and full-scale fire-resistance tests have been conducted to verify such an assumption (69) and the results were used to develop panel-to-panel joint factors for use in a simplified model, as shown in Figure 15. The joint factors and simplified model were inspired by the European methodology for timber assemblies specified in EN1995-1-2. All joint details were sealed with a fire-stop sealant approved for fire-resistance-rated joints in wood and concrete constructions. The simplified model, shown in Equation [16], assumes that fasteners used for connecting the CLT panels have a minimum penetration depth of at least $5d_F$ into the main member (d_F is the fastener shank diameter), in accordance with the provisions in CSA O86.

A coupled thermal-structural model has also been proposed by Dagenais (69), allowing for optimization of CLT panel-to-panel joint details by moving the joint interface across the CLT thickness until the best balance is achieved between fire integrity performance and lateral capacity against in-plane loading (e.g. wind and seismic forces). Fastener yield modes can also be predicted as a function of time (fire duration). For example, moving the lapped joint upward at 30% or 40% of the thickness of a 5-ply CLT (175-mm) floor element would increase the predicted failure time to 147 and 120 minutes, respectively (vs. 94 minutes if lapped joint is located at 50% - mid-depth). Shifting the lapped joint interface at these two locations also does not influence the lateral resistance of the self-tapping screws and does not affect the volume of wood to be machined/grooved on the edges of CLT panels.

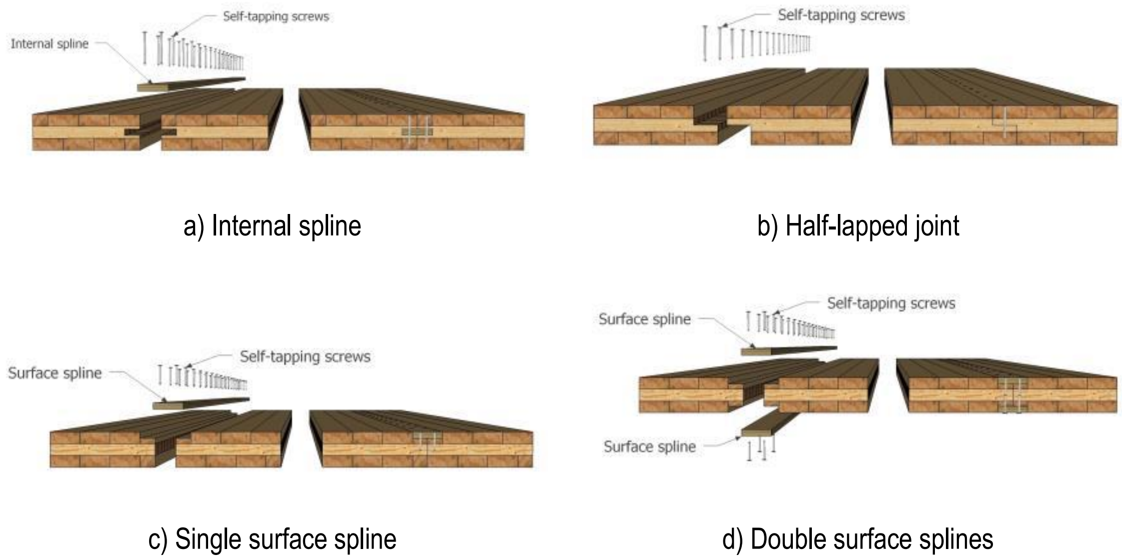


Figure 15 CLT panel-to-panel joint details

$$t_{Int} = K_j \cdot \frac{h}{\beta_0} \tag{16}$$

where:

K_j = panel-to-panel joint factor

= 0.30 for internal spline

= 0.35 for half-lapped

= 0.60 for single surface spline (fire-exposed side opposite to the spline)

= 0.60 for double surface spline

h = initial CLT thickness, mm

β_0 = one-dimensional char rate for panel-to-panel joint integrity calculation

= 0.65 mm/min

When the integrity requirement cannot be fulfilled by the CLT panels alone, additional floor coverings or wall sheathings can be used to increase the integrity failure time. For example, the thickness of wood floor coverings can be added to the CLT assembly thickness (h) when using Equation [16]. If gypsum board is used, the assigned time listed in Subsection 8.5.6 of this Chapter can be added to the unprotected CLT assembly integrity failure time, provided it is used on the fire-exposed side. Moreover, Equation [16] considers the influence of CLT panel-to-panel joints not backed by other means on the unexposed side. As such, when unexposed surface protection (e.g. structural elements, panels, flooring, or a concrete topping) are added, the integrity criteria can be assumed to be satisfied, as these additional membranes will prevent flame penetration through the assembly, then allowing the joint coefficient to be set to unity (K_j = 1.0).

It was reported by Dagenais (69) that both surface spline details provided better fire integrity performance than the other two joint configurations. Notwithstanding, compared to surface spline details, the half-lapped joint is easier to manufacture at the plant and to assemble at the job site, as it requires significantly less field inspection during construction. The lower fire performance of the internal spline joint detail highlights the importance of proper field installation when a spline is used to effectively provide the anticipated level of fire integrity performance. Inadequate field installation may result in potential gaps at the butt joints between the splines (or splines not being installed at all at some locations). However, potential gaps between butt joints could be minimized if tongue-and-groove or scarf joints are used between splines.

A symmetrical joint detail is preferable for CLT wall applications, as fire-resistance is to be assessed from a fire occurring from either side, thus necessitating the use of a double surface spline. The single surface spline joint detail should be limited to floor applications.

As noted previously, CLT panel-to-panel joints should be sealed using a fire-resistance-rated sealant or caulking to prevent smoke leakage, as reported and supported by fire test data (68, 69, 70, 71). The use of adequate sealant or caulking is also required when using the simplified model from Equation [16], as it was developed using adequately-sealed CLT panel-to-panel joints.

8.5.5 Fire Resistance of CLT – Insulation Requirement

As mentioned in Section 8.5 of this Chapter, insulation is the other requirement of the separating function of building assemblies. The time at which the CLT assembly can no longer prevent the temperature on the unexposed surface from rising 180°C above the initial temperature at any location, or an average of 140°C measured at a number of locations, defines the insulating fire resistance (t_{Ins}). This requirement is intended to limit the risk of fire spread to compartments beyond the compartment of fire origin, as well as to allow safe egress within the space located on the side of the assembly away from the fire (unexposed side).

8.5.5.1 Theoretical Temperature Profiles for CLT Assemblies

Heat transfer occurs from regions of high temperature to regions of cooler temperature within solids (e.g., from the fire room of origin to adjacent compartments through a wall or floor assembly). This mode of heat transfer in solid materials is conduction, which is also related to the material's thermal conductivity (k), represented by the three dimensional (3-D) transient heat transfer partial differential equation shown in Equation [17].

$$\frac{\partial}{\partial x}\left[k_x \frac{\partial T}{\partial x}\right] + \frac{\partial}{\partial y}\left[k_y \frac{\partial T}{\partial y}\right] + \frac{\partial}{\partial z}\left[k_z \frac{\partial T}{\partial z}\right] + \dot{Q} = \rho c \frac{\partial T}{\partial t} \quad [17]$$

where

T = temperature, K

$k_{x,y,z}$ = thermal conductivities in x, y, z directions, W/m·K

$\dot{Q}$ = the internally generated heat due to the chemical reaction, W/m³

ρ = the density, kg/m³

c = the specific heat, J/kg·K

t = the time, s

Heat transfer through a material that exhibits charring behaviour is slightly more complicated than that of other materials such as steel or concrete. The internally generated heat due to the chemical reaction consists of two parts: 1) the pyrolysis of the wood expressed by an Arrhenius function and 2) the heat absorption per unit volume due to evaporation of water. More information with regards to the rate of heating, pyrolysis of the wood, and evaporation of water may be found in the SFPE Handbook of Fire Protection Engineering (72).

Materials with a high thermal conductivity (such as steel) are usually considered to be good thermal conductors, while those having a low thermal conductivity (such as wood) are considered to be good thermal insulators. As such, the transient or steady-state heat transfer by conduction through CLT is low when compared with other materials having higher thermal conductivity.

Charring of wood is a complex process that can be quite challenging to model. Defining thermal properties for every stage of pyrolysis can also be onerous. Thus, commercially available finite element software packages are normally used for solving the differential equations. Such temperature predictions may be useful for determining the rate of wood charring when conducting a performance-based fire design. It should be noted that current thermal properties are calibrated for fire tests with standard fire exposure (CAN/ULC S101, for example) and should theoretically not be used for thermal analyses for other fire exposures, unless verified and validated.

8.5.5.2 Experimental Temperature Profile Data for CLT Assemblies

Although the use of finite element analysis may not be available to most building designers, experimental temperature profile data is available for solid wood slabs. In one such generic temperature profile discussed by Janssens & White (73), the temperature at a certain distance from the char front can be described by Equation [18], when the member behaves as a semi-infinite solid. Moreover, the authors reported that temperature profiles in wood do not appear to be very sensitive to moisture content.

$$T = T_i + \left(T_p - T_i\right) \cdot \left(1 - \frac{x}{a}\right)^2 \qquad [18]$$

where

T = the temperature, °C

T_i = the initial temperature (room temperature, usually assumed to be 20°C)

T_p = the char front temperature, 300°C

x = the distance from the char front, mm

a = the thermal penetration depth, mm

Based on data for eight species (74), the best fit values for the thermal penetration depth (*a*) were 34 mm for spruce with a dry density of 425 kg/m³, and 33 mm for southern pine with a dry density of 510 kg/m³. In the previous edition of Eurocode 5 (EN1995-1-2:1994), the thermal penetration depth was assigned a value of 40 mm.

Annex B of CSA O86 assumes a thermal penetration depth of 35 mm. As a result, no temperature rise is assumed to occur on the unexposed side, while the CLT thickness remains greater than 35 mm. The remaining CLT thickness required to keep the average unexposed temperature increase below 140°C (or a temperature of about 160°C at a single point) indicated by Equation [18] is 12 mm. At this thickness, the slab will no longer behave as a semi-infinite solid and the unexposed face will likely not be at ambient temperature.

A series of full-scale wall and floor fire-resistance experiments in accordance with the standard time-temperature curve of CAN/ULC S101 were conducted, to allow a comparison between the fire resistance measured during a standard fire-resistance test and that calculated using an alternative analytical method (e.g. Annex B of CSA O86) (47). Figure 16 shows the experimental temperature profile data obtained from this series of tests compared to the profile obtained when using Equation [18]. This illustrates that insulation failure is unlikely to be a concern for CLT assemblies prior to integrity or structural failure, since the temperatures on the unexposed side of the char front rapidly decrease to ambient temperatures.

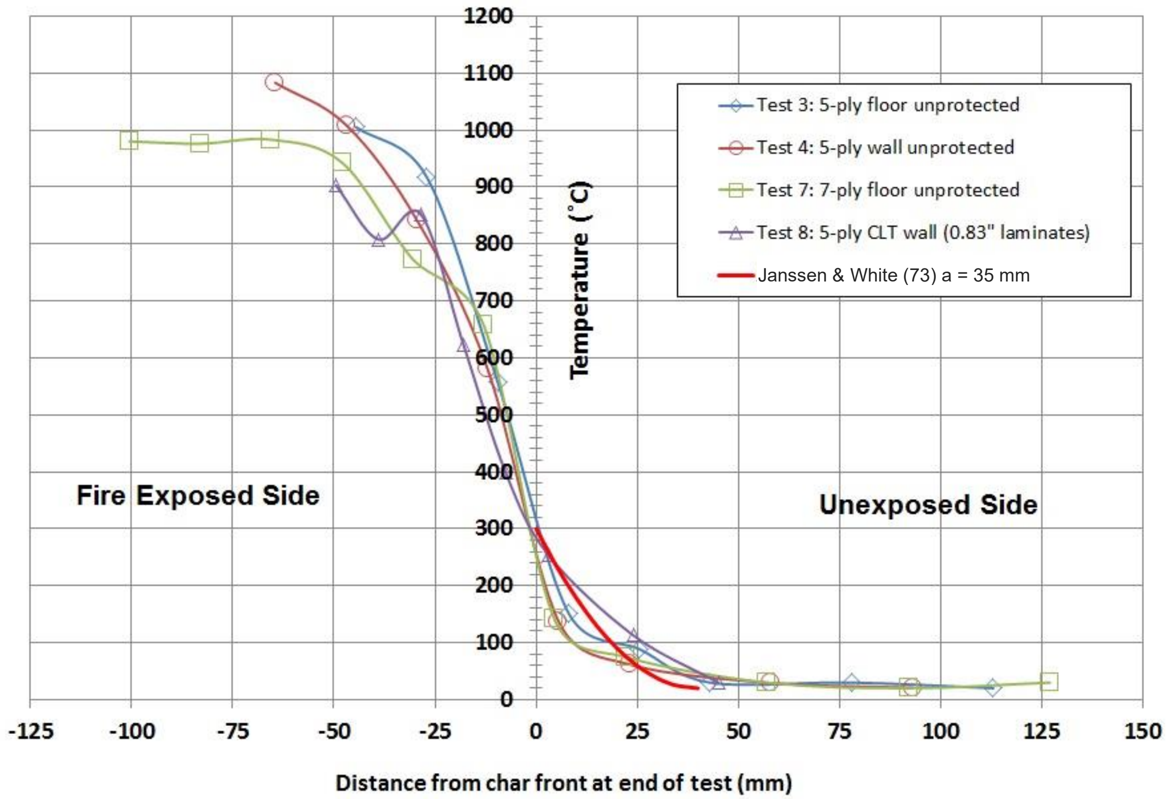


Figure 16 Experimental temperature profiles from (47) and Equation [18]

The results presented in Table 5 demonstrate that the insulation requirement is easily met, because the temperature rise was very minimal on the unexposed surface. This is true even for a temperature difference across a CLT wall of 1020°C, where the effective reduced CLT thickness (remaining thickness) was as thin as 49 mm.

Table 5 Maximum temperature rises at unexposed surface (47)

CLT Stress Grade and Thickness		Failure Time (min)	Effective Reduced Thickness (mm)	Average Temperature			Temperature Rise on Unexposed Surface
				Furnace	Unexposed Surface	Initial Condition	
Wall	E2 – 114 mm (3-ply)	106	97	992°C	24°C	23°C	1°C
	E1 – 175 mm (5-ply)	113	92	1015°C	21°C	21°C	0°C
	V2 – 105 mm (5-ply)	57	49	1050°C	30°C	21°C	9°C
Floor	E2 – 114 mm (3-ply)	77 (1)	105	971°C	22°C (2)	23°C (2)	-1°C (2)
	E1 – 175 mm (5-ply)	96	105	982°C	20°C	20°C	0°C
	V2 – 105 mm (3-ply)	86	56	973°C	60°C	22°C	38°C
	V2 – 175 mm (5-ply)	124	89	1006°C	27°C	23°C	4°C
	V2 – 245 mm (7-ply)	178	105	1049°C	30°C	20°C	10°C

(1) Test was stopped due to equipment safety concerns. Failure was not reached.

(2) Temperature at 77 minutes (see note (1)). Temperature reached 23.5°C after 106 minutes while removing the specimen.

8.5.6 Use of Protective Membranes to Increase Fire Resistance

The mechanics-based fire design procedure, as discussed herein, predicts average fire-resistance times for CLT wall and floor assemblies that closely track actual fire-resistance times for tested assemblies. Full-scale fire-resistance wall and floor tests have been conducted on CLT either protected and unprotected with gypsum board. These tests have shown that the fire resistance of CLT elements may be increased when protective membranes are used. These membranes also contribute to delaying the onset of charring of the wood element underneath.

The calculation methods in the previous Subsections are based on an unprotected CLT panel fully exposed to a standard fire. Experiments by the U.S. Forest Products Laboratory on tension members (75) and by FPInnovations on CLT assemblies protected with Type X gypsum boards (47) indicate that the structural failure time of protected assemblies may be calculated in the same manner as the failure time of an unprotected assembly, in accordance with this Chapter, but with the inclusion of an additional time to account for protection measures. In accordance with Annex B of CSA O86, when gypsum board is directly applied to the fire-exposed side, the following times may be added to the structural failure time of an unprotected CLT element:

a) 15 minutes for one layer of 13-mm (½-in.) Type X gypsum board

b) 30 minutes for one layer of 16-mm (⅝-in.) Type X gypsum board

c) 60 minutes for two layers of 13-mm (½-in.) Type X gypsum board.

Recent fire research has shown that three layers of at least 16-mm (⅝-in.) Type X gypsum board can contribute at least 120 minutes of additional time to the structural fire resistance (76).

The Type X gypsum board protective membranes should be attached directly to the mass timber elements using fasteners with a minimum penetration of 25 mm (1 in.) into the CLT and spaced at 305 mm (12 in.) on-center along the perimeter and throughout. Screws should be kept at least 38 mm (1½ in.) from the edges of the boards. All exposed joints and fastener heads should be covered with tape and coated with joint compound. When using two layers of thermal protective membranes, the joints shall be staggered between the base layer and face layer. Adding steel or wood furring strips providing a gap between the protective membranes and the CLT will not reduce the additional structural failure time as long as the fastener penetration and spacing requirements are met.

Various parameters that affect the ability of encapsulation materials to delay the onset of charring of CLT have been studied in intermediate-scale tests (77). Increasing screw length and decreasing screw spacing beyond what is specified above does not enhance performance. For example, when a 16-mm (⅝-in.) Type X gypsum board is applied to CLT over furring strips (creating a 13-mm (½-in.) air gap), it was found that the encapsulation time was increased by 9%.

Rock fiber insulation using screws and washers was shown to be very effective at improving encapsulation time, partially due to its ability to stay in place for long fire exposure times (in excess of 2 hrs). Using 51-mm (2-in.) and 76-mm (3-in.) rock fiber insulation directly applied using screws and washers can provide 40 minutes and 50 minutes of encapsulation time, respectively. When 51-mm (2-in.) of rock fiber insulation is further protected by gypsum board, with the inclusion of a 51-mm (2-in.) air gap in-between (the gypsum board hung from resilient channels attached to acoustic hangers), encapsulation time reached over 70 minutes.

As reported in Subsection 8.4.5, Code change proposals are currently under review in regard to the concept of encapsulated mass timber construction (EMTC). This would include CLT panels with gypsum board protection and some other materials.

8.5.7 Comparison - Calculation Method vs. Test Data

This Subsection presents an analysis of a wide array of fire-resistance test results for CLT elements that were either fully exposed to a standard fire, or initially protected by Type X gypsum board or another assembly of materials. The CLT panels were manufactured in accordance with the product standard ANSI/APA PRG 320, with respect to lumber species groups and adhesives deemed to comply with the requirements applicable at that time (up to its 2017 edition). Additional information on the specimens, instrumentation, and test configurations can be found in reports from FPInnovations (47, 49, 50, 78), the National Research Council of Canada (79), Canadian CLT manufacturers (48, 80), the Canadian Wood Council (81, 82), and the American Wood Council (83).

The test results for a total of eleven (11) CLT walls and thirteen (13) CLT floors manufactured from different ANSI/APA PRG-320 certified mills across North America are presented in Table 6. The failure times observed in the test data is also compared to those obtained when using the 2016 Update No.1 of CSA O86-14 fire-resistance design methodology (Figure 17).

In Table 6, the assigned fire-resistance values take into consideration the additional time needed to account for the use of gypsum board protection, per Subsection 8.5.6. It is noted that the insulation requirement is not listed, since temperatures that would indicate an insulation failure were never reached in any of the tests, as shown in Table 5; therefore, only the structural (loadbearing) and integrity failure (flame through) times are indicated. The time to structural failure shown in columns 5 and 6 in Table 6 are calculated per the methodology detailed herein. It can be observed from Figure 17 that most of the predicted structural failure times are on the conservative side (i.e. lower than actual test failure times). It can also be observed from the values in Table 6 that the CLT wall loading ratio directly influences the differences in the predicted times between the axial compression resistance and the combined bending and axial compression resistance; i.e., for CLT walls subjected to low load ratios, smaller differences are obtained between the two predicted times (axial vs. combined). Engineering judgment is required to determine if applicable eccentricities due to charring for fire-resistance design need to be taken into consideration in calculating the fire resistance of CLT wall elements.

Table 6 Comparison between fire test data and CSA O86-14 calculation method

	CLT Stress Grade and Thickness	Load Ratio[1]	Type X Gypsum Board (on fire side)	Test Failure Time (mode) (min.)	CSA O86-14 – Annex B (min.) Axial[2] or Bending[3]	CSA O86-14 – Annex B (min.) Combined Effects[4]
Wall	E1 – 105 mm (3-ply)	68%	-	32 (R)	30	22
Wall	V2 – 105 mm (3-ply)	44%	1 x 16 mm	76 (R)	66	62
Wall	E1 – 105 mm (3-ply)	73%	1 x 16 mm	66 (R)	62	53
Wall	E2 – 114 mm (3-ply)	70%	2 x 13 mm	106 (R)	94	84
Wall	V2 – 105 mm (5-ply)	31%	-	57 (R)	58	53
Wall	E1 – 175 mm (5-ply)	34%	-	113 (E)	110	98
Wall	E1 – 175 mm (5-ply)	13%	1 x 16 mm	185 (R)	148	145
Wall	E1 – 175 mm (5-ply)	46%	2 x 16 mm	219 (R)	165	148
Wall	V1 – 175 mm (7-ply)	15%	-	132 (R)	138	130
Wall	E1 – 184 mm (7-ply)	18%	-	170 (R)	118	112
Wall	E1 – 209 mm (7-ply)	22%	-	> 170[5]	165	153
Floor	E2 – 114 mm (3-ply)	35%	2 x 13 mm	> 77[6]	101	
Floor	V2 – 105 mm (3-ply)	75%	1 x 16 mm	86 (R)	61	
Floor	E1 – 175 mm (5-ply)	60%	-	96 (E)	98	
Floor	V2 – 175 mm (5-ply)	100%	1 x 16 mm	124 (E)	117	
Floor	E1 – 175 mm (5-ply)	50%	Refer to (78)	128 (R)	133	
Floor	E1 – 175 mm (5-ply)	25%	Refer to (79)	128 (R)	113	
Floor	E1 – 175 mm (5-ply)	17%	-	150 (R)	116	
Floor	E1 – 175 mm (5-ply)	10%	Refer to (49)	214 (R)	198	
Floor	E1 – 185 mm (7-ply)	27%	-	136 (R)	142	
Floor	E1 – 184 mm (7-ply)	21%	-	> 136[5]	116	
Floor	E1 – 209 mm (7-ply)	24%	-	177 (R)	164	
Floor	E1 – 221 mm (7-ply)	18%	-	> 177[5]	174	
Floor	V2 – 245 mm (7-ply)	100%	-	178 (R)	101	

[1] Load ratio calculated using limit states design (i.e. factored load effect ÷ factored resistance (P_r or M_r), per Clause 8 of the 2016 Update No.1 of CSA O86-14. Self-weight based on 550 kg/m^3.

[2] Factored compressive resistance as per Equation [13].

[3] Bending moment resistance as per Equation [10].

[4] Resistance to combined bending and axial compression as per Equation [15].

[5] Specimens tested simultaneously. Test was stopped when one specimen failed. No failure was reached for this specimen.

[6] Test was stopped due to equipment safety concerns. No failure was reached for this specimen.

(R) = structural failure (E) = integrity failure

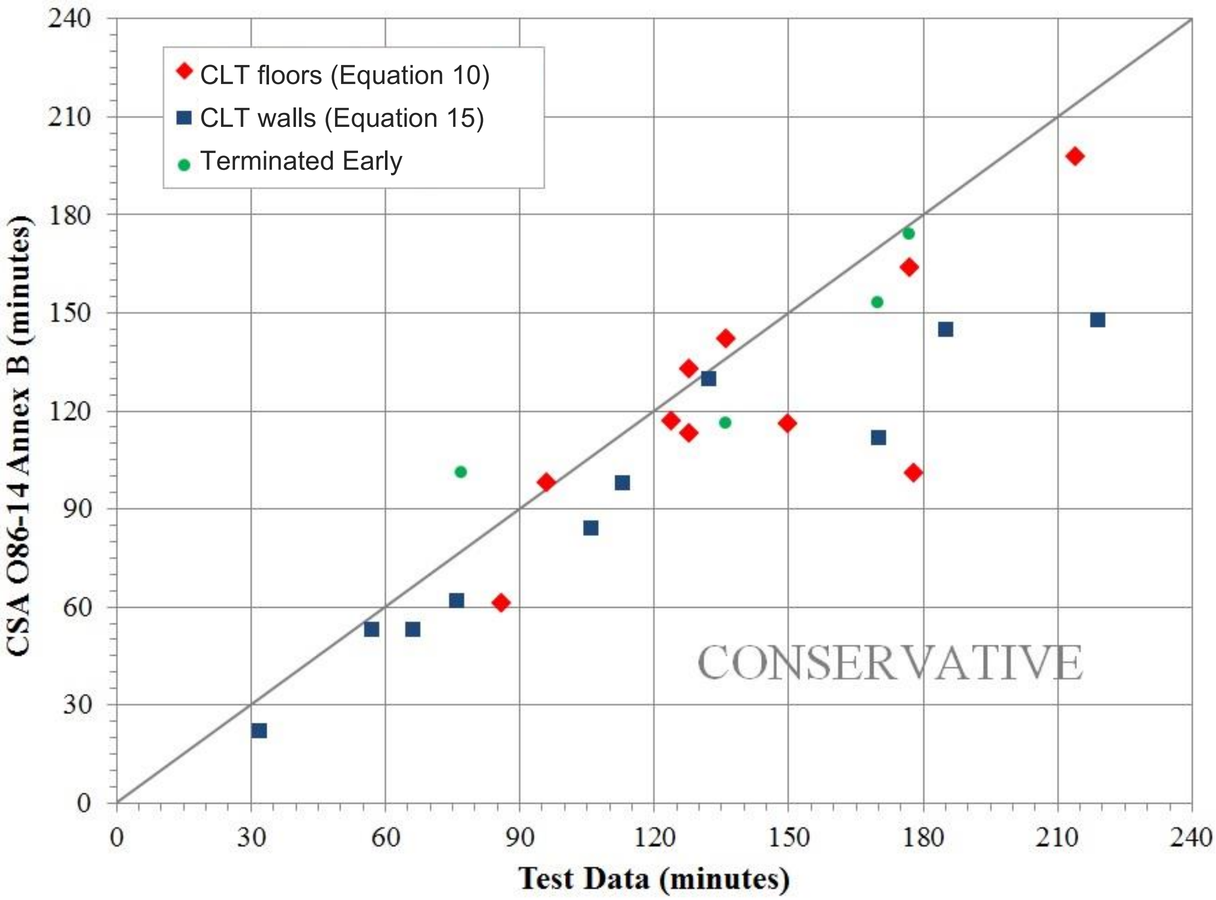


Figure 17 Test data vs. predictions using CSA O86-14 for CLT elements

8.5.8 Timber Concrete Composite Floor Systems

In an attempt to develop and validate a calculation method to predict the time of structural fire-resistance of timber-concrete composite (TCC) floors, FPInnovations (49, 84) tested three different TCC floors exposed to the CAN/ULC S101 standard fire. The first series of tests were conducted on floors consisting of 5-ply (175-mm) E1 stress grade CLT and 89-mm concrete (30 MPa), interconnected with self-tapping screws driven in at 45° angles into the CLT (Figure 18). The CLT-concrete composite floor was tested simultaneously and side-by-side with a TCC floor consisting of a series of screw-laminated 2”x8” “joists” (SLT) (38 x 184 mm, on edge) using conventional truss plates as shear connectors. The second series of tests were conducted on floors consisting of 133- x 406-mm (5¼” x 16”, on flat) laminated veneer lumber (LVL), using lag screws as shear connectors. All of these floors were fully exposed to the standard fire from underneath (i.e. timber components were exposed to fire) and subjected to a 2.4 kPa live load, in addition to their self-weight. Construction details may be found in Osborne (48) and Ranger et al. (84).

a) Self-tapping screws at 45° angle b) CLT-concrete interface (inside furnace frame)

Figure 18 CLT-concrete composite floor assembly

The results from these fire-resistance tests, shear connectors tests (stiffness and resistance) and FEM modeling allowed for the development of an accurate design method, which is expected to be published soon by FPInnovations' "*Technical Design Guide for use in Canada*" (85). Comparisons between actual test results and the predicted failure times based on the proposed design method are shown in Table 7. The predicted structural fire-resistance failure times are calculated using laboratory shear connector test results (stiffness and resistance). The shear connector properties were proportionally reduced linearly as a function of its remaining penetration depth, once it was exposed to fire. The predicted times are conservative for the CLT-concrete and LVL-concrete composite floors. For the SLT-concrete floor, the predicted failure time seems realistic, but it is uncertain whether the estimation is conservative, or not, since the test was stopped after 214 minutes, the time at which the CLT-concrete floor failed.

Table 7 Fire-resistance of TCC floors – Test data vs. design method

	TCC Floor Assembly		
	CLT-concrete	**SLT-concrete**	**LVL-concrete**
Shear connector	Self-tapping screws	Truss plates	Lag bolts
Test failure time (min)	214	>214[(1)]	191
Predicted failure time (min)	198[(2)]	247	165

[(1)] Test stopped when the CLT-concrete floor failed. No failure was reached for the SLT-concrete floor.
[(2)] Calculated using Annex B of CSA O86-14 and neglecting cross-plies.

In summary, when a TCC floor is made with a wood-based slab, the wood component protects the concrete from the thermal effects of a fire underneath (refer to Subsection 8.5.5 with respect to thermal insulation of CLT). As such, the resistance of a TCC floor can be calculated by simply determining the reduced cross-section of the wood component and the shear connector properties, as a function of its remaining penetration depth.

8.5.9 Floor Design Example

The CLT floor design example below follows the steps listed in Section 8.5.3 of this Chapter for determining whether the fire resistance of an exposed 5-ply CLT floor assembly meets the hypothetically required fire-resistance rating of 1 hour. The floor assembly has the following specifications:

- 5-ply CLT floor panel made from 35-mm thick laminates (total thickness of 175 mm)
- V2 CLT grade as per ANSI/PRG 320
- Floor span = 6.10 m
- Major strength direction plies
    - $f_{b,0}$ = 11.8 MPa
    - E_0 = 9 500 MPa
- Minor strength direction plies
    - $f_{b,90}$ = 7.0 MPa
    - E_{90} = 9 000 MPa
- Adhesive in accordance with ANSI/PRG 320 (2012) requirements
- Floor panels are connected using a single surface spline joint, as per Figure 15c).
- Applied loads of 2.40 kPa (live) + 2.25 kPa (assembly self-weight)
- Induced bending moment represents a load ratio of 81% (applied moment and resistance moment calculated at normal conditions).

<u>**Calculation of the Loadbearing Function after 1 Hour of Standard Fire Exposure:**</u>

Step 1: Calculation of the effective char depth

Given that the 1st bond line is expected to be exceeded (60 min x 0.65 mm/min = 39 mm > 35 mm), a char rate of 0.80 mm/min is appropriate. The effective char depth may also be directly obtained from Table 2 and is equal to 55 mm for a CLT made of 35-mm thick laminates exposed to a standard fire for 1 hour.

$$x_{c,eff} = x_c + x_t = \left(0.80\frac{mm}{min} \cdot 60\ min\right) + 7\ mm = 55\ mm$$

Step 2: Determination of the effective reduced cross-section

The effective reduced cross-section is then calculated using Equation [5].

$$h_{fire} = 175 - 55 = 120\ mm \rightarrow 105\ mm$$

In this floor design example, h_{fire} falls within a ply in the minor strength direction (i.e., within the second ply from the exposed side); therefore only the 3rd and 5th plies (from the exposed side) are to be considered, providing a total effective reduced thickness of 105 mm, which essentially consists of a full 3-ply V2 CLT floor panel. Thus, one could have taken the stiffness and unfactored resistance values directly from Table A4 of the 2018 Edition of ANSI/APA PRG 320, without further calculating the section properties of the reduced cross-section (e.g. $(f_bS)_{eff,y}$ = 18 x 10^6 N·mm/m and $(EI)_{eff,y}$ = 884 x 10^9 N·mm²/m).

Step 3: Find location of neutral axis and section properties of the effective reduced cross-section

Equations [6] and [7] are used to determine the neutral axis and the moment of inertia of the effective reduced cross-section. Calculations are made for a unit width of CLT panel (1000 mm). The third ply centroid is located at 87.5 mm from the unexposed side (i.e. top of floor panel).

$$\bar{y} = \frac{\sum_i \tilde{y}_i h_i E_i}{\sum_i h_i E_i} = \frac{\left(\frac{35}{2} \cdot 35 \cdot 9500\right) + (87.5 \cdot 35 \cdot 9500)}{(35 \cdot 9500) + (35 \cdot 9500)} = 52.5\ mm$$

$$(EI)_{eff,y} = \sum_{i=1}^{n} E_i b_y \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i b_y h_i z_i^2$$

$$= \left(9500 \cdot \frac{1000 \cdot 35^3}{12}\right) + \left(9500 \cdot \frac{1000 \cdot 35^3}{12}\right)$$

$$+ \left(9500 \cdot 1000 \cdot 35 \cdot \left(52.5 - \frac{35}{2}\right)^2\right)$$

$$+ (9500 \cdot 1000 \cdot 35 \cdot (87.5 - 52.5)^2) = 882.5\ \times\ 10^9\ N \cdot mm^2/m$$

It is noted that ANSI/APA PRG 320 assigns an effective bending stiffness ($(EI)_{eff,y}$) of 884 x 10^9 N mm²/m. The difference between this worked example and the published value in ANSI/APA PRG 320 is due to the fact that the product standard considers the cross-plies when determining the section properties, while Annex B of CSA O86 suggests otherwise.

Step 4a: Calculation of the factored bending moment resistance

Using the effective reduced cross-section determined in Step 2 and ignoring any contribution to the strength provided by the cross-plies (i.e., minor strength direction), the factored bending moment resistance of the CLT floor assembly may be determined following the basic procedure described, with Equations [10] and [11].

$$S_{eff,y} = \frac{(EI)_{eff,y}}{E} \cdot \frac{1}{c} = \frac{882.5 \times 10^9 \, N \cdot \frac{mm^2}{m}}{9500 \frac{N}{mm^2}} \cdot \frac{1}{52.5mm}$$
$$= 1.77 \times 10^6 mm^3/m$$

$$M_{r,y} = \phi F_b S_{eff,y} K_{rb,y} = \phi f_b \left(K_D K_H K_{Sb} K_T K_{fi}\right) S_{eff,y} K_{rb,y}$$
$$= 1 \cdot 11.8 \cdot (1.15 \cdot 1 \cdot 1 \cdot 1 \cdot 1.50) \cdot 1.77 \times 10^6 \cdot 0.85 = 30.6 \, \frac{kN \cdot m}{m}$$

It is noted that ANSI/APA PRG 320 assigns an unfactored bending resistances ($(f_b S)_{eff,y}$) of 18 x 10^6 N·mm/m for a 3-ply V2 CLT panel made with 35-mm thick laminates. In this worked example, a value of 17.7 x 10^6 N·mm/m is obtained (e.g. $(f_b S)_{eff,y} = f_b \cdot S_{eff,y} \cdot K_{rb,y}$ = 11.8 MPa · 1.77 x 10^6 mm³/m · 0.85). As with the effective bending stiffness, the difference between this worked example and the published value in ANSI/APA PRG 320 is due to the fact that the product standard considers the cross-plies when determining the section properties, while Annex B of CSA O86 suggests otherwise.

The applied specified loads are as follows:

$$w_{Total} = L + D = 2.4 + 2.25 = 4.65 \, kPa$$

The induced bending moment in the fire-resistance design is then equal to:

$$M_f = \frac{w_{Total} \cdot Span^2}{8} = \frac{4.65 \, kPa \cdot (6.1 \, m)^2}{8} = 21.6 \, \frac{kN \cdot m}{m} \, \left(< M_{r,y}\right)$$

The factored bending moment resistance after 1 hour of standard fire exposure is calculated as 30.6 kN·m/m, compared to the induced bending moment of 21.6 kN·m/m; this represents a load ratio of 71% (fire conditions). Therefore, the CLT floor assembly meets the required 1-hour fire resistance under these loads, span, and CLT grade and configurations.

Calculation of the Separating Function after 1 Hour of Standard Fire Exposure:

The separating function of the CLT floor assembly is determined by using Equation [16] as follows:

$$t_{Int} = K_j \cdot \frac{h}{\beta_0} = 0.60 \cdot \frac{175\ mm}{0.65\ mm/min} = 161\ min\ (> 1\ hour)$$

Assuming that an unexposed surface protection is added on top of the CLT panels, the joint coefficient (K_j) would be taken as unity, and the integrity fire resistance would be 269 minutes. In some situations, the calculated failure mode may change from integrity to structural, or vice-versa.

From the calculated fire resistance of this particular CLT floor assembly (loadbearing and separating functions), the CLT panels could then be left exposed from underneath (ceiling), provided they also meet other fire-related provisions from Part 3 of Division B of the NBCC (e.g., flame spread rating).

8.5.10 Wall Design Example

The following CLT wall design example follows the steps listed in Subsection 8.5.3 of this Chapter for determining whether the fire resistance of a 5-ply CLT wall assembly meets the hypothetically required fire-resistance rating of 2 hours. The wall assembly has the following specifications:

- 5-ply CLT wall panel made from 35-mm thick laminates (total thickness of 175 mm)
- E1 CLT grade as per ANSI/PRG 320
- Wall height = 3.66 m (assuming K_e = 1.00)
- Major strength direction plies
  - $f_{b,0}$ = 28.2 MPa
  - $f_{c,0}$ = 19.3 MPa
  - E_0 = 11 700 MPa
  - G_0 = 731 MPa
- Minor strength direction plies
  - $f_{b,90}$ = 7.0 MPa
  - E_{90} = 9 000 MPa
  - G_{90} = 56.2 MPa
- Adhesive in accordance with ANSI/PRG 320 (2012) requirements
- Wall panels are connected using a half-lapped joint, as per Figure 15
- Wall panels are protected by one layer of 16-mm Type X gypsum board on both sides

- Applied concentric load of 265 kN/m (live) + 50 kN/m (assembly self-weight)
- Induced axial load represents a load ratio of 36% of the factored axial compression capacity (normal conditions).

Calculation of the Loadbearing Function after 2 Hours of Standard Fire Exposure:

Since the protective membrane provides 30 minutes before the onset of charring of the CLT panels (refer to Subsection 8.5.6), the structural fire-resistance calculation is made for a fire exposure of only 90 minutes (1.5 hours).

Step 1: Calculation of the effective char depth

Given that the 1st bond line is expected to be exceeded (90 min x 0.65 mm/min = 58.5 mm > 35 mm), a char rate of 0.80 mm/min is appropriate. The effective char depth may also be directly obtained from Table 2 and is equal to 79 mm for a CLT made of 35-mm thick laminates exposed to a standard fire for 1.5 hours.

$$x_{c,eff} = x_c + x_t = \left(0.80\frac{mm}{min} \cdot 90\ min\right) + 7\ mm = 79\ mm$$

Step 2: Determination of the effective reduced cross-section

The effective reduced cross-section is then calculated using Equation [5].

$$h_{fire} = 175 - 79 = 96\ mm$$

In this wall design example, h_{fire} falls within a ply of the major strength direction (i.e., within the third ply from the exposed side); therefore, only a portion of the exposed ply (26 mm) and the complete first unexposed ply are used to calculate the residual strength of the CLT in this example. The tabulated values in Table A4 of the 2018 edition of ANSI/APA PRG 320 cannot be used in this example, as the residual cross-section no longer reflects a complete and balanced 3-ply configuration, as per the product standard assumptions.

Step 3: Find location of neutral axis and section properties of the effective reduced cross-section

Equations [6] and [7] are used to determine the neutral axis and the moment of inertia of the effective reduced cross-section. Calculations are made for a unit width of CLT panel (1000 mm). The 3rd ply centroid is located at 83.0 mm from the unexposed side of the wall panel.

$$\bar{y} = \frac{\sum_i \tilde{y}_i h_i E_i}{\sum_i h_i E_i} = \frac{\left(\frac{35}{2} \cdot 35 \cdot 11700\right) + (83.0 \cdot 26 \cdot 11700)}{(35 \cdot 11700) + (26 \cdot 11700)} = 45.4\,mm$$

$$(EI)_{eff,y} = \sum_{i=1}^{n} E_i b_y \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i b_y h_i z_i^2$$

$$= \left(11700 \cdot \frac{1000 \cdot 35^3}{12}\right) + \left(11700 \cdot \frac{1000 \cdot 26^3}{12}\right)$$

$$+ \left(11700 \cdot 1000 \cdot 35 \cdot \left(45.4 - \frac{35}{2}\right)^2\right)$$

$$+ (11700 \cdot 1000 \cdot 26 \cdot (83.0 - 45.4)^2)$$

$$= 807.8 \times 10^9\ N \cdot mm^2/m$$

$$I_{eff,y} = \frac{(EI)_{eff,y}}{E_0} = \frac{807.8 \times 10^9\ N \cdot mm^2/m}{11700\ N/mm^2/m} = 69.0 \times 10^6\ mm^4/m$$

$$(GA)_{eff,zy} = \frac{\left(h - \frac{h_1}{2} - \frac{h_n}{2}\right)^2}{\left[\left(\frac{h_1}{2G_1 b_y}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i b_y}\right) + \left(\frac{h_n}{2G_n b_y}\right)\right]}$$

$$= \frac{\left(96 - 0 - \frac{35}{2}\right)^2}{\left[0 + 0 + \left(\frac{26}{731 \cdot 1000}\right) + \left(\frac{35}{56.2 \cdot 1000}\right) + \left(\frac{35}{2 \cdot 731 \cdot 1000}\right)\right]}$$

$$= 9.03 \times 10^6\ N/m$$

$$A_{eff,y} = \sum_i h_i = (1000 \cdot 35) + (1000 \cdot 26) = 61000 \frac{mm^2}{m}$$

$$r_{eff} = \sqrt{\frac{I_{eff}}{A_{eff}}} = \sqrt{\frac{69.0 \times 10^6}{61000}} = 33.6\,mm$$

$$C_c = \frac{L_e}{\sqrt{12} r_{eff}} = \frac{3660\,mm}{\sqrt{12} \cdot 33.6\,mm} = 31.5 (< 43)$$

Step 4b: Calculation of factored compressive resistance parallel to grain

Using the effective reduced cross-section determined in Step 2 and ignoring any contribution from the cross-plies (i.e., minor strength direction), the factored compressive resistance parallel to the grain of the CLT wall assembly may be determined using Equation [13].

$$K_{Zc} = 6.3\left(\sqrt{12} r_{eff} L\right)^{-0.13} \leq 1.3 \text{ (based on initial cross-section)}$$

where

$$(EI)_{eff,y} = 4166 \times 10^9 N \cdot mm^2/m \text{ (from Table A4 of ANSI/APA PRG 320)}$$

$$I_{eff,y} = \frac{(EI)_{eff,y}}{E_0} = \frac{4166 \times 10^9\ N \cdot mm^2/m}{11700\ N/mm^2/m} = 356.1 \times 10^6\ mm^4/m$$

$$A_{eff,y} = \sum_i h_i = 3 \cdot (1000 \cdot 35) = 105000 \frac{mm^2}{m}\ (longitudinal\ layers)$$

$$r_{eff} = \sqrt{\frac{I_{eff}}{A_{eff}}} = \sqrt{\frac{356.1 \times 10^6}{105000}} = 58.2\,mm$$

$$C_c = \frac{L_e}{\sqrt{12}r_{eff}} = \frac{3660\ mm}{\sqrt{12} \cdot 58.2\ mm} = 18.1(< 43)$$

$$K_{Zc} = 6.3\left(\sqrt{12} \cdot 58.2 \cdot 3660\right)^{-0.13} = 1.09$$

$$K_C = \left[1.0 + \frac{F_c K_{Zc} C_c^3}{35 E_{05}(K_{SE} K_T)}\right]^{-1} \text{(fire design, using reduced cross-section)}$$

$$= \left[1.0 + \frac{f_c\left(K_D K_H K_{Sc} K_T K_{fi}\right) K_{Zc} C_c^3}{35 E_{05}(K_{SE} K_T)}\right]^{-1}$$

$$= \left[1.0 + \frac{19.3 \cdot (1.15 \cdot 1 \cdot 1 \cdot 1 \cdot 1.25) \cdot 1.09 \cdot 31.5^3}{35 \cdot 11700 \cdot (1 \cdot 1)}\right]^{-1} = 0.30$$

$$P_r = \phi F_c A_{eff} K_{Zc} K_C = 1 \cdot 27.7 \cdot 61000 \cdot 1.09 \cdot 0.30 = 552\ kN/m$$

The applied specified axial loads are as follows:

$$P_{Total} = L + D = 265\ + 50 = 315\ kN/m\ (\leq P_r = 552)$$

The factored axial compression resistance after 90 minutes of standard fire exposure is calculated as 552 kN/m, compared to the applied axial load of 315 kN/m, and represents a load ratio of 57% (fire conditions).

As mentioned in Subsection 8.5.3 of this Chapter, a CLT wall assembly may be subjected to second-order effects (i.e., P-Δ effects) due to the charring of the fire exposed surface (Figure 13). Engineering judgment is required to determine if applicable eccentricities due to charring for fire-resistance design need to be taken into consideration in calculating the fire resistance of CLT wall elements (see Subsection 8.5.7). Should one decide to evaluate the effect of combined bending and axial loading, Equation [14] or [15] is to be used.

$$S_{eff,y} = \frac{(EI)_{eff,y}}{E} \cdot \frac{1}{c} = \frac{807.8\ \times\ 10^9\ N \cdot \frac{mm^2}{m}}{11700 \frac{N}{mm^2}} \cdot \frac{1}{45.4\ mm}$$
$$= 1.521 \times 10^6 mm^3/m$$

(Note: the tension side of a CLT wall subjected to combined bending and axial loading is on the opposite side exposed to fire vs. a floor where it is the fire-exposed side).

$$M_{r,y} = \phi F_b S_{eff,y} K_{rb,y} = \phi f_b \left(K_D K_H K_{Sb} K_T K_{fi}\right) S_{eff,y} K_{rb,y}$$

$$= 1 \cdot 28.2 \cdot (1.15 \cdot 1 \cdot 1 \cdot 1 \cdot 1.25) \cdot 1.521 \times 10^6 \cdot 0.85 = 52.4\ \frac{kN \cdot m}{m}$$

$\Delta_f = 0\ mm$ (M_f = 0 N·mm)

$$e_0 = \frac{h}{2} - \bar{y} = \frac{175}{2} - 45.4 = 42.1\ mm$$

$$\Delta_0 = \frac{L}{500} + \frac{h}{6} = \frac{3660}{500} + \frac{175}{6} = 36.5\ mm$$

$$\Delta = \frac{P_f\left(\Delta_f + e + \Delta_0\right)L_e^2}{16(EI)_{eff,y}} = \frac{315 \cdot (0 + 42.1 + 36.5) \cdot 3660^2}{16 \cdot 807.8\ \times\ 10^9} = 25.7\ mm$$

$$M_{f,\Delta} = P_f \Delta = 315 \cdot \frac{25.7}{1000} = 8.1\ kN \cdot m/m$$

$$P_E = \frac{\pi^2 E_{05} K_{SE} K_T I_{eff}}{L_e^2} = \frac{\pi^2 \cdot 11700 \cdot 1 \cdot 1 \cdot 69.0 \times\ 10^6}{3660^2} = 595\ kN/m$$

$$P_{E,v} = \frac{P_E}{1 + \frac{\kappa P_E}{(GA)_{eff}}} = \frac{595}{1 + \frac{1.0 \cdot 595}{9.03\ \times 10^3}} = 558.2\ kN/m$$

$$\frac{P_f}{P_r} + \frac{1}{M_r}\left[M_f + \frac{P_f \Delta}{1 - \frac{P_f}{P_{E,v}}}\right] \leq 1$$

$$\frac{315}{552} + \frac{1}{52.4}\left[0 + \frac{315 \times 0.0257}{1 - \frac{315}{558.2}}\right] = 0.93\ (\leq 1)$$

The induced combined bending moment and axial compression represents a ratio of 93% of the reduced cross-section capacity after 90 minutes of standard fire exposure; therefore, the CLT wall assembly meets the required 2-hour fire resistance under these loads, wall height, CLT grade, and configurations, as well as with a 16-mm Type X gypsum board protective membrane on both sides.

Calculation of the Separating Function after 2 Hours of Standard Fire Exposure:

As mentioned in Subsection 8.5.4 of this Chapter, when the unexposed side of a CLT panel-to-panel joint is backed by other means, such as a Type X gypsum board, to prevent flame penetration, the joint coefficient (K_j) may be considered to be unity. Moreover, one layer of 16-mm Type X gypsum board directly attached to a CLT element delays the onset of charring by 30 minutes. Therefore, the separating function of the CLT wall assembly may be determined using Equation [16], as follows:

$$t_{Int} = \left(K_j \cdot \frac{h}{\beta_0}\right) + 30\ min = 1.0 \cdot \frac{175\ mm}{0.65\ mm/min} + 30\ min = 269\ min\ (> 2\ hours)$$

8.6 CONNECTIONS

As described in Chapter 5 of this CLT Handbook, there exist a wide variety of fasteners and many different types of joint details that may be used to establish roof-to-wall, wall-to-floor, and inter-storey connections in CLT assemblies. This is also true for connecting 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 panel-to-panel connections in floors (Figure 15) and floor-to-wall assemblies, traditional dowel-type fasteners, such as wood screws, nails, lag screws, rivets, bolts, and dowels may also be effectively used in connecting panel elements.

Connections in post-and-beam timber construction, including those built with CLT, play an essential role in providing strength, stiffness, stability, ductility, and structural fire resistance. Moreover, connections using metallic fasteners, such as bolts, dowels, and steel plates or brackets, are widely used to assemble mass timber components or CLT panels, and to provide an adequate load path for gravity and/or lateral loads; consequently, these connections require the designer's attention to ensure that they maintain their strength when exposed to fire and do not unnecessarily transfer heat into wood elements. It is noted that the NBCC does not require connections that are required solely for lateral loads to be protected.

Performance of timber connections exposed to fire may be quite complex due to the influence of numerous parameters, such as fastener type, geometry of the connection, different failure modes, as well as differences in the thermal conductivity properties of steel, wood, and char layer components. As such, most building codes, including the NBCC, do not provide a specific fire design methodology for determining the fire performance of timber connections.

Due to the high thermal conductivity of steel, metallic fasteners and plates directly exposed to fire may heat up and conduct heat into wood members. The wood components may then experience charring on the exposed surface and around the fastener. As a result, the capacity of a metallic connection is decreased by the strength reduction of the steel fastener at elevated temperatures, and the charring of the wood members (11, 86, 87, 88, 89, 90, 91, 92, 93); therefore, where a fire-resistance rating is required by the NBCC, connections and fasteners are required to be protected from fire exposure by wood, gypsum board, or other protection approved for the required rating.

However, some connections are not vulnerable to the damaging impact of fire. For example, a CLT wall-to-floor connection used to resist wind or seismic load in a platform-frame construction, (examples shown in Figure 19), will not be significantly impacted by fire. Nevertheless, connections used to resist gravity loads in a balloon-frame construction, (shown in Figure 20), may require some special considerations to increase their resistance to fire exposure from underneath.

a) Steel plates used to transfer lateral loads

b) Steel brackets used to transfer lateral loads

Figure 19 Examples of connections seen in CLT platform construction (do not require protection)

a) CLT floors supported from underneath by steel angles (subsequently protected by two layers of Type X gypsum board)

b) CLT floor supported from underneath by a glulam ledger adequately sized to allow for char

Figure 20 Examples of connections seen in CLT balloon construction (may require protection)

To improve aesthetics, designers often prefer to conceal connection systems. Hidden metal plates similar to those shown in Figure 21 may be used, but they require machining to produce grooves in the CLT panel to conceal the metal plates.

When the connections are used in fire-retardant- or preservative-treated wood, recommendations with regards to the types of metal fasteners need to be obtained from the chemical manufacturer, since some treatments cause corrosion of certain metals.

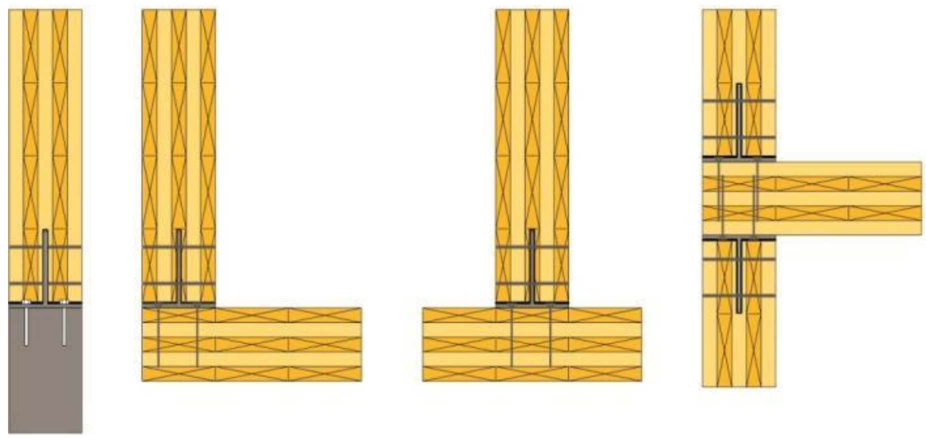


Figure 21 Concealed metal plates

Many proprietary products for CLT connections are now available in North America, some of which originate from Europe. Any connection method that is being used in Canada should have its fire resistance properly evaluated in accordance with the Canadian standard fire test methods. It is recommended to have a qualified fire protection engineer review fire test results and product information from manufacturers that present results from different test methods. As was previously discussed, it is important for any metallic connection to have adequate protection. The latter may be provided by wood cover or other means, such as gypsum board.

It is advisable to review the recommendations in Chapter 5 of this CLT Handbook with respect to proper detailing of connections in CLT assemblies.

8.7 INTERIOR FINISH

The spread of flames over solid materials is a fundamental behaviour influencing fire dynamics and growth within a compartment; therefore, many provisions in the NBCC limit the use of combustible interior finishes, such as interior wall, ceiling, and floor finishes. The concept of Flame Spread Rating (FSR) is a secondary fire protection measure used to limit the rate of fire growth and/or fire spread, as shown in Figure 1.

The rate of fire growth will depend on the time it will take a flame to spread from the point of origin (i.e., ignition) to involve an increasingly large area of combustible material (94). Factors influencing a material's thermal response include, among others, its thermal conductivity, density, heat capacity, thickness, and blackbody surface reflectivity. An increase in the values of these properties usually corresponds to a decrease in the rate of flame spread (95). Previous studies and results presented by White et al. (96) suggest that there is a relation between the time to reach flashover conditions in an ISO 9705 room/corner fire test (97) and the ASTM E84 (98) flame spread indices of materials, where longer times to flashover conditions were observed in rooms lined with materials exhibiting low flame spread indices.

Once a fire is ignited, the rate at which it grows has a significant impact on the life safety of occupants and the time available to evacuate. This is directly influenced by the surface flammability of building contents and materials; consequently, Part 3 of Division B of the NBCC limits the allowable Flame Spread Rating (FSR) and Smoke Development Class (SDC) of interior finishes based on the location, building occupancy, and availability of an automatic fire suppression system. These provisions are intended to limit the spread of fire and products of combustion through a building, in a manner that allows safe egress of the occupants and limits the damage to the building in which the fire originated.

8.7.1 Test Method – CAN/ULC S102

In Canada, the FSR of a material, assembly or structural member is determined on the basis of no less than three standard fire tests conducted in conformance with CAN/ULC S102, "*Standard Method of Test for Surface Burning Characteristics of Building Materials and Assemblies*" (99). Some construction materials may be assigned an FSR in generic terms, such as in the case of gypsum board and most softwood lumber species, based on historical data, which are specified in Appendix D-3 of the NBCC. Results of FSR testing on proprietary materials are usually available from accredited fire testing laboratories.

The primary purpose of the tests is to determine the comparative burning characteristics of the material or assembly by evaluating the flame spread over its surface when exposed to a test fire. The test method attempts to establish a basis on which surface burning characteristics of different materials or assemblies may be compared, without specific considerations of all the end-use parameters that might affect these characteristics. Flame spread rate and smoke density are recorded as dimensionless values in this test, and these two measurements are not necessarily related. The test method is only a means of evaluating the response of materials, products, or assemblies to a particular fire exposure under controlled laboratory conditions and may not reflect the relative surface burning characteristics of tested materials under all building fire conditions.

The CAN/ULC S102 standard test method, a variant of the "Steiner Tunnel" test, exposes a 7.3-m x 508-mm (nominal 24-ft. long x 20-in. wide) specimen to a controlled air flow of 1.2 ± 0.025 m/s and a flaming fire exposure of 90 kW (Figure 22). In a successful calibration of the test parameters using an 18-mm thick red oak flooring sample conditioned to 7% moisture content, the flame reaches the end of the tunnel and the vent-end thermocouple registers a temperature of 527°C in 5.5 ± 0.25 minutes. Such a calibration assigns a benchmark FSR of 100 to the red oak specimen.

Figure 22 Flame spread testing apparatus (at Intertek in Coquitlam, BC)

8.7.2 Flame Spread Rating of CLT

In order to evaluate the surface burning characteristics of fully exposed mass timber assemblies, flame spread tests on CLT assemblies have been conducted in accordance with the CAN/ULC S102 test method (Figure 23). Test results show low FSR when compared to those of common combustible interior finish materials, listed in Appendix D-3 of the NBCC. The flame spread values for the 3-ply CLT specimens are listed in Table 8.

a) CLT before ULC S102 test

b) CLT during the ULC S102 test

Figure 23 CLT specimen (fully exposed) in a CAN/ULC S102 test

Table 8 **Flame spread test results for 3-ply CLT specimens (100, 101)**

CLT Assembly	Flame Spread Rating	Smoke Developed Classification
SPF – E1 Stress grade (min. 105 mm)	35	40
SPF – V2 Stress grade (min. 99 mm)	40	30

The use of materials that exhibit FSR that are lower than typical combustible interior finish materials would result in a reduced *"risk"* of ignition, fire growth, and a potentially longer time to flashover conditions, depending on the configuration of the room of fire origin. In such cases, this reduced risk would make it possible to achieve the NBCC objectives and functional statements [F02 – OS1.2, OP1.2] when developing an alternative solution.

8.7.3 Fire Retardants

Wood products may be treated with fire retardants to improve their fire performance, for example, by delaying time to ignition, reducing heat release rate, and lowering FSR. Such fire retardant treatments (FRT) may also reduce the SDC of FRT wood and wood-based products. While FRT enhances the flame spread performance of wood and wood-based products, such treatments do not make them noncombustible materials.

There are two types of FRT: 1) surface coatings and 2) pressure-impregnated chemicals. There are also two objectives for treating wood products with fire retardant chemicals. One objective is to take advantage of some provisions in the NBCC, where fire retardant-treated wood (FRTW) may be used in lieu of noncombustible construction. The other objective is to meet requirements set forth in Division B of the NBCC for a specified FSR. The term "fire retardant-treated wood" in the NBCC is limited to wood that has been pressure-impregnated in conformance with CSA O80 (102) series and that exhibits an FSR of no more than 25 when tested per CAN/ULC S102.

CLT components treated to meet FRTW specifications are not expected to be available in the near future. The wood industry currently does not recommend the use of fire-retardant treatments with glulam. This is likely due to the potential effects of proprietary treatments on the mechanical properties and performance of the adhesives.

Should CLT components be subjected to pressure-impregnated fire retardant treatments, the effects on mechanical properties will need to be addressed in the design. The tabulated specified strength values published in CSA O86 and ANSI/APA PRG 320 are for untreated members. Reference-specified strength values for CLT pressure-treated with fire retardant chemicals, should be obtained from the manufacturer providing the treatment.

In addition to pressure-impregnated treatments, fire retardant surface treatments may also be used to address interior finish requirements that are more restrictive than the flame ratings for untreated wood. Surface treatments, including clear intumescent coatings, allow designers to use unprotected CLT (e.g., without gypsum board or other cladding), while achieving the more restrictive finish rating requirements. While Division B of the NBCC permits the use of coatings to address the finish rating requirements, field application of these coatings and questions of durability in certain applications may create difficulties in its acceptance in new construction by the authorities having jurisdiction. Flame spread tests conforming to CAN/ULC S102 on a 3-ply (105-mm) CLT initially protected by an intumescent coating demonstrated that an FSR of 25 may be achieved (100). However, as surface treatment products are proprietary in nature, this low FSR may not be achievable with all treatment products available on the market, even if they claim to achieve such a low rating when tested on thin wood planks or structural panels (e.g., OSB). It is strongly recommended that proper test data be obtained from the manufacturers before specifying that such products comply with NBCC provisions.

It should be noted that pressure-impregnated fire retardant treatments are marketed to reduce the FSR and provide lower flammability performance. Such fire retardant treatments do not have an appreciable effect on the charring rate, which is an important parameter in assessing fire resistance, i.e. they are not used to improve fire resistance.

In an attempt to evaluate the effect of surface treatments (e.g., intumescent coatings) on fire resistance, full-scale fire resistance tests were conducted on 3-ply (105-mm) CLT wall assemblies. Surprisingly, the treated CLT assembly failed earlier than the untreated CLT wall assembly (48). The difference was not very significant, but one explanation for such a variance may be that by the time the intumescent coating had degraded and no longer provided its thermal insulation, the furnace temperature (i.e., heat flux emitted to the CLT surface) was significantly greater. At that point, the uncharred wood ignited and burned faster than usual (at a rate much higher than 0.65 mm/min), thereby reducing the effective cross-section more quickly. Further research is required to properly evaluate the effect of surface coatings on charring rate.

8.7.4 Use of Other Membrane Products to Address Interior Finish Requirements

The most common method for addressing FSR and SDC interior finish requirements is the installation of gypsum board. According to Table D-3.1.1.A of the NBCC, gypsum board has an FSR of 25 or less. For situations where there is no fire resistance rating requirement, the gypsum board can be regular or non-fire-rated gypsum board. When used to address fire resistance requirements, the gypsum board will need to be fire-rated as either Type X or Type C. Likewise, lower FSR interior finish requirements may also be addressed by decorative hardwood plywood panels, particleboard, or medium-density fiberboard panel products that have been treated with fire retardant chemicals to achieve an FSR of no more than 25.

8.7.5 Automatic Fire Sprinklers

Automatic fire sprinklers are an important fire safety feature in any building. For certain buildings and occupancies, the NBCC will require the installation of an approved automatic sprinkler system. As previously discussed, the inclusion of such an approved system in a building may provide benefits in terms of allowable heights and areas, as well as in terms of lower fire resistance requirements for building elements. In the case of interior finishes, higher surface flammability (i.e. FSR) is permitted when automatic sprinklers are provided. The applicable standards for automatic sprinkler systems are NFPA 13, 13R, and 13D (103, 104, 105).

8.8 PENETRATIONS AND CONSTRUCTION JOINTS

8.8.1 Fire-Stopping Building Elements and Assemblies

Fire stopping is a performance attribute required by all types of construction in the NBCC. Service penetrations and gaps in fire separations are inevitable in construction. Firestop systems are needed to ensure the integrity of a fire compartment by maintaining the fire resistance rating of the floor and/or wall assemblies that they penetrate, or at construction joints. A firestop system consists of a material, component, and means of support used to fill gaps between fire separations, between fire separations and other assemblies, or used around items that wholly or partially penetrate a fire separation. Furthermore, smoke-tight joints must be provided where fire separations abut on or intersect a floor, a wall, or a roof. Subsections 3.1.8 and 3.1.9 of Division B of the NBCC detail the specific provisions for enclosures and penetrations in fire-rated separations. It requires that firestops, or firestop systems be evaluated in accordance with CAN/ULC S115, "*Standard Method of Fire Tests of Firestop Systems*" (106), or be cast-in-place. Penetrations through concrete assemblies can be cast-in-place, which ensures there are no gaps between the penetrating item and the assembly it penetrates. CAN/ULC S115 test results are specific to the firestop material, type of penetration (for example plastic or metallic pipe), and the assembly construction tested.

A firestop needs to provide an F- or FT-rating that is no less than the fire protection rating required for the closures within that fire separation; these ratings are given in Table 3.1.8.4 of Division B of the NBCC. A 1½-hr rating is required for closures in a 2-hr fire resistance-rated assembly. An F-rating is given to a firestop system that is capable of staying in place during the test and prevent the spread of fire through the penetration (or flaming of any element on the unexposed side), when exposed to a standard fire for a given time. A T-rating is assigned to a firestop system that is capable of preventing a 180°C increase in temperature on the unexposed surface of the firestop system, similar to insulation failure criteria in fire resistance testing. T-ratings are generally reserved for firewalls and separations between buildings.

8.8.1.1 Through and Partial Penetrations

Through-penetration firestops are used to seal any opening around a penetrating item (such as cables, cable trays, conduits, ducts, and pipes), which passes entirely through an assembly. They ensure the integrity of a fire compartment by maintaining the fire resistance rating of the floor or wall assembly that they penetrate. Examples of through penetrations are given in Figure 24. Prefabrication of CLT elements has advanced to a degree where panels can arrive on-site with all openings pre-cut. This includes not only windows and doors, but also any locations for large and small service penetrations, such as for refuse chutes or plumbing. This level of prefabrication requires a high degree of accuracy, which has been made possible in part by the development of Building Information Modeling (BIM). An entire building can now be modeled using multiple dimensions (3D, scheduling, and costs) prior to the start of construction. This helps to eliminate discontinuities on site between trade drawings (such as mechanical, electrical, and plumbing), resulting in faster construction with fewer delays. Penetration locations can be predetermined and are therefore known before CLT panels are manufactured. Once structural elements are installed on-site, tradespeople can arrive and install their services with ease.

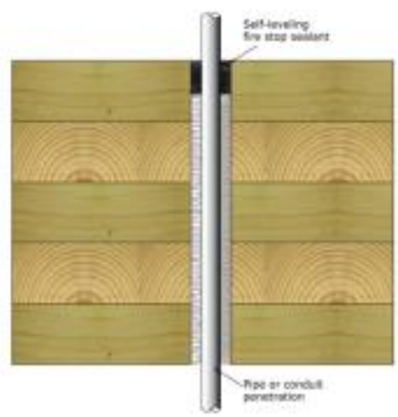


Figure 24 Examples of through penetrations in CLT assemblies

Not all penetrations need to pass entirely through an assembly. In the case of CLT, this can include elements that are embedded within the CLT from the surface, such as electrical conduits, as shown in Figure 25 below.

Figure 25 Examples of partial penetration in CLT assemblies

8.8.1.2 Joint Firestop Systems

Fire stopping is necessary at construction joints and connections where gaps can allow hot gases to pass through in the event of a fire (Figure 14). To ensure the integrity of a fire compartment, construction joints and connections need to be fire stopped to provide a seal along the continuous linear opening between fire resistance-rated assemblies.

Subsection 3.1.8 of Division B of the NBCC requires that joints be smoke-tight where fire separations terminate (where it abuts on or intersects a floor, a roof slab, or a roof deck). The continuity of a fire separation where it abuts against another fire separation, a floor, a ceiling, or an exterior wall assembly is typically maintained by filling all openings at the juncture of the assemblies with a material that will ensure the integrity of the fire separation at that location.

8.8.1.3 Closures

Any door that is used to close an opening through a fire separation is considered a closure that shall be installed in conformance with NFPA 80 "*Fire Doors and Other Opening Protectives*" (107) and shall be tested according to CAN/ULC S104 "*Standard Method for Fire Tests of Door Assemblies*" (108). Careful consideration of fire stopping details is needed around these closures to limit heat transmission and charring of the CLT.

8.8.2 Firestop Systems in CLT Construction

A breadth of firestop systems that meet various F- and T-ratings are available for concrete assemblies. Concrete is similar to CLT in that it is also a solid mass and typically does not have void cavities. Because of this, many concrete slab firestop systems are translatable and suitable for timber slab penetrations (109), provided that suitable allowance is made for char from heat transfer through any metal component. So long as the materials are comparable (such as both are continuous slabs), and the firestop material itself is not located within the depth of the potential char layer during fire exposure, it can be demonstrated that fire test results for firestop systems in concrete assemblies are also applicable to CLT. Based on research conducted in Europe, firestop systems currently used in reinforced concrete may be successfully used in solid wood construction; many of these designs relied on lining openings with gypsum board; however some tests have indicated that this may not be necessary in all cases (109). Additional information may also be found in Teibinger and Matzinger (110).

Some firestop test results for wood-frame assemblies are also applicable to CLT, due to similarities between these assemblies (110). Firestop systems used for penetrations that pass through wood stud assemblies can also be effective for CLT since, in simple terms, the CLT is a large wood element. In this case, the wood substrate material is the same, which ensures a good bond or interaction between the firestop and the wood.

Due to the proprietary nature of most firestop systems, it is recommended that a qualified fire protection engineer undertake or oversee the design and use of firestop systems in CLT construction. Typically, manufacturers of firestop systems publish technical documentation on their website. Additional information, such as construction details and appropriate ratings, may be found on websites of accredited agencies such as Intertek Testing Services or Underwriters Laboratories. Engineering judgments can be developed by firestop manufacturers on a project-by-project basis. There is currently no listed firestop system available for service penetrations and construction joints in solid-wood wall and floor assemblies in North America, which can make it challenging for architects and engineers to gain acceptance of their designs.

However, limited testing has been conducted to evaluate the use of different firestops in CLT construction. FPInnovations has evaluated the performance of partial and through penetrations, as well as construction joints in accordance with CAN/ULC S115. Testing has also been conducted in the U.S. by the American Wood Council following ASTM E814 "*Standard Test Method for Fire Tests of Penetration Firestop Systems*" (111), which includes exposure to ASTM E2226 "*Standard Practice for Application of Hose Stream*" (112).

It was reported that the risk of fire spreading beyond its room of origin is mainly due to inadequate joint design and improper installation of firestops in service penetrations (70). Proper detailing, installation, and field inspection are therefore very important, to ensure that the firestop is installed and will perform as intended/designed.

8.8.2.1 Through Penetrations

Firestops for through penetrations in CLT can achieve a 1½-hr F-rating, as would be required through a 2-hr fire resistance-rated assembly (71). These penetrations and firestops should be installed to limit the probably of ignition or charring of the CLT adjacent to the penetration.

It is common practice in concrete construction to leave through-penetrating items resting directly on the concrete opening. However, it is essential that through penetrations in mass timber construction do not directly interact or touch the timber components and be properly insulated; this limits heat transfer from the penetrating item to the surrounding timber, to prevent premature ignition, charring, and potential flame penetration. This detail is shown in Figure 26 below. It is also important that any gap(s) within the firestop joint filler be eliminated with an appropriate thermal insulating material such as rock or mineral wool, to prevent convective heat transfer from the through-penetrating item to the surrounding timber component.

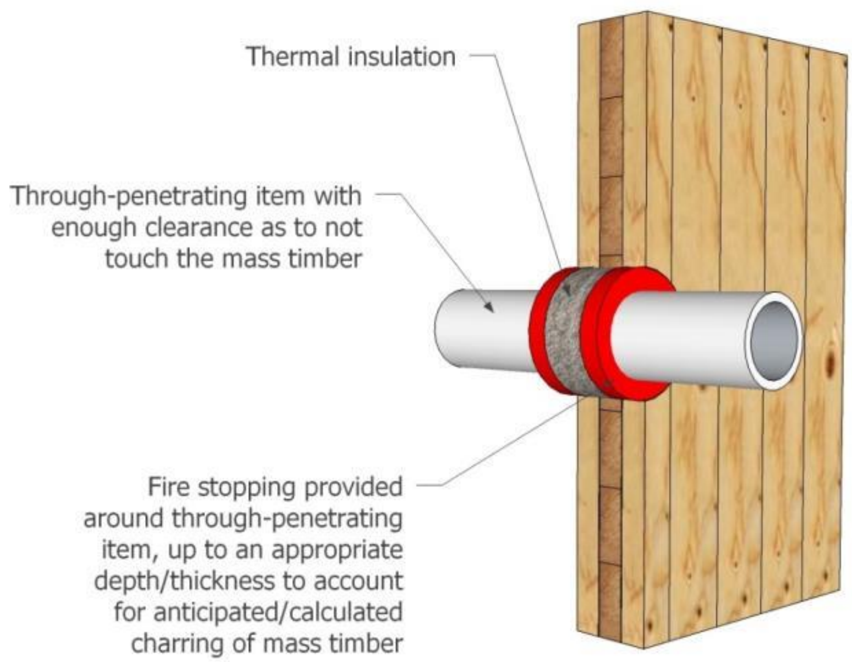


Figure 26 Fire stopping of through penetrations

8.8.2.1.1 Metallic Penetrations

Several firestops for use with metallic penetrations and previously approved for use in concrete, have been tested with CLT with satisfactory results. Cast iron, steel, or electrical metallic tubing (EMT) can be fire-stopped by placing mineral wool inside the CLT opening, and applying joint filler on either side to a depth of 25 mm (1”) within the annulus (71). Larger diameter holes, penetrating items being positioned offset from the center, and closer spacing were also evaluated. Figure 27 shows a series of penetrations before and after a CAN/ULC S115 test.

a) Joint filler over mineral wool

b) Joint filler over steel sleeves and mineral wood

c) Before fire testing

d) After fire testing

Figure 27 Through penetrations in CLT assemblies

A series of tests were conducted to develop generic firestop details for metallic penetrations through CLT. These tests involved exposure to fire for 2 hours; this was followed by a hose stream test (113). This fire exposure was longer than needed (1 hour), given that penetrations need only achieve a 1½-hr F-rating for use in a 2-hr fire resistance- rated assembly. The results indicated that for 100-mm (4”) copper pipe penetrations, a 25-mm (1”) annular space should be filled with mineral wool up to 13 mm (½”) from the CLT surface, and the remaining 13 mm (½”) filled with intumescent caulking. The caulking creates an air tight seal between the metal penetration and the CLT. Within a 25-mm (1”) annular space, an offset as low as 13 mm (½”) is acceptable. Also, a 100-mm (4”) spacing for copper pipe penetrations is acceptable, where there is 50 mm (2”) of CLT and 25 mm (1”) of annular spacing on either side. Activated intumescent caulking was not enough to provide backing support to prevent mineral wool from being forced out during a hose stream test after a 2-hour exposure. These tests also indicated that a tolerance of +/- 12mm (½”) performs in an acceptable manner, when the thickness of mineral wool insulation varies from 12 to 37mm (½” to 1½”).

Because copper penetrations expose CLT to higher temperatures than cast iron or EMT, the same design considerations as for 100-mm (4") copper penetrations can be conservatively applied to 150-mm (6") and smaller cast iron penetrations, and 50-mm (2") and smaller EMT penetrations.

8.8.2.1.2 Plastic Penetrations

Some generic details for plastic penetrations through CLT have been developed to achieve a 1½-hr F-rating (114). Research suggests that two bands of one or two layers of intumescent wrap are adequate to fire stop nominal 38-mm (1½") or smaller PVC or PEX pipe. The inclusion of a 30-gauge steel sleeve with these smaller pipes provided good results. For larger pipes (with a 50-mm (2") nominal diameter or greater) a 30-gauge steel sleeve around one or two bands of two layers of intumescent wrap was sufficient. These firestops require intumescent caulking to seal the top of the opening between the pipe and the CLT, for smoke control.

In the USA, three CLT through penetrations designed for use with concrete achieved a 2-hr F- and T-rating, when tested in accordance with ASTM E814 and ASTM E2226 (115). This included two PVC pipes and a fiber optic cable with no steel sleeve penetrating a 5-ply CLT, with two layers of 16-mm (5/8") Type X gypsum board.

8.8.2.2 Joints

Several commercially-available fire-rated joint fillers, caulking, and sealing tapes approved for concrete assemblies have been shown to achieve 1½- or 2hr FT-ratings with mass timber elements (71). Joint firestops perform much better when they are not directly exposed to fire, so when possible they should be placed away from locations where charring is expected to occur (i.e., on the unexposed side of an assembly), as shown in Figures 28 and 29.

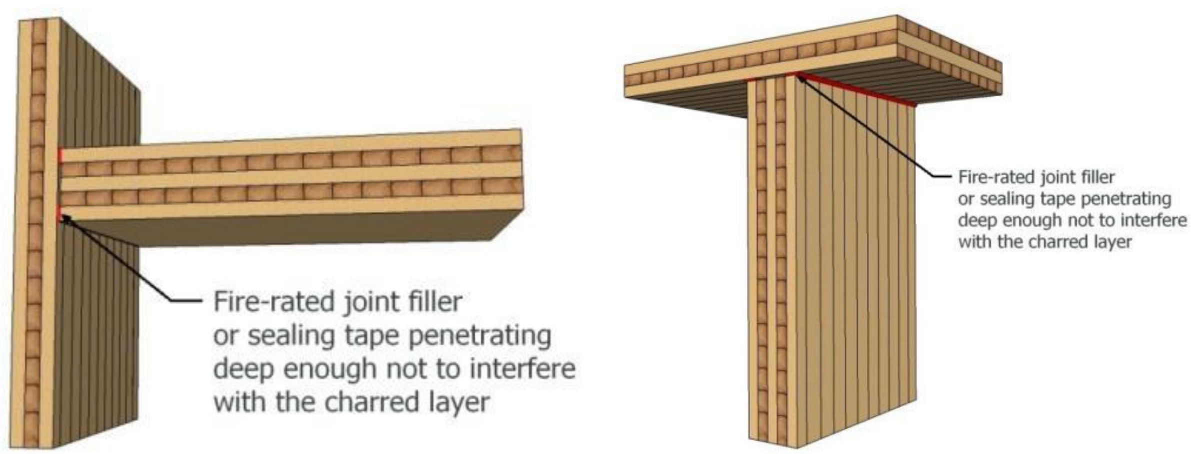


Figure 28 Examples of firestop systems evaluated for CLT joint assemblies

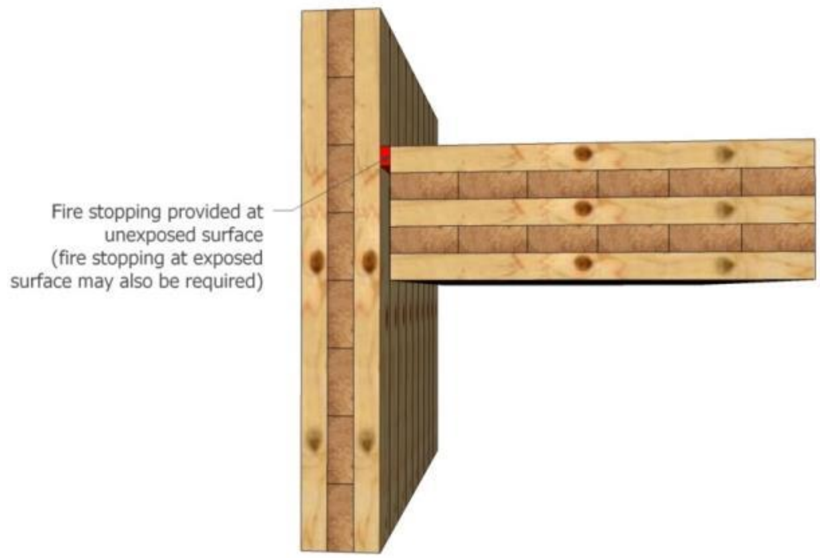


Figure 29 Example of joint firestop at unexposed surface of CLT joint assemblies

8.8.2.3 Closures

Preliminary testing has been conducted on double steel egress doors in a unprotected 5-ply CLT wall and a 3-ply wall with two layers of 16-mm (5/8") Type X gypsum board (Figure 30) (116, 117). The results suggest that a 1½-hr closure penetration can be installed in a 2-hr rated CLT wall provided the inside edges of the CLT, including the threshold, are well protected. However, further testing is warranted.

a) 5-ply CLT wall before test (unexposed side) b) 3-ply CLT wall after test (exposed side)

Figure 30 Double steel egress door in CLT wall

To prevent inconsistent charring adjacent to the door and in locations away from the door, the edges around the opening should be protected from hot gases and heat. This can be accomplished by filling the void space behind the door frame with rock wool insulation. Before the door frame is installed, a fire-resistance membrane (such as 16-mm Type X gypsum board) should be applied to the top and sides of the opening, and a layer of a noncombustible member (e.g., 11-mm cement board) that is robust and can withstand repeated physical abuse under the threshold. Continuity between the gypsum board and cement board needs to be maintained; this can be accomplished by applying a bead of firestop caulking across the width of the CLT, at CLT-to-CLT corners, at gypsum board-CLT interfaces (before installing the gypsum board or cement board), at the ends of each cap, and between butt joints of the boards. To prevent the passage of smoke and hot gases through the interface, a bead of caulking should be applied to the edges of the door frame and threshold that meet the CLT wall.

The following screw details will likely improve the fire performance of closures. Screws should not extend into the char region, as this may transfer heat into the CLT and promote premature charring. The diameter and penetration should be sufficient to provide out-of-plane resistance after fire exposure. The diameter of the screws should be chosen to provide the vertical lateral load carrying capacity that the anchor straps are intended to carry.

Door frames with anchor designs can be attached with pairs of 64-mm No.8 wood screws. When screwed through 16-mm gypsum board, the screw will penetrate the CLT about 50 mm.

8.9 VERTICAL SHAFTS

CLT has been used to construct elevator shafts and exit stairs in numerous buildings around the world and here in Canada. Elevator shafts, which fall under the category of vertical service spaces in the NBCC, have different considerations than interior stair shafts, which are considered exits (per Article 3.4.1.4. of Division B of the NBCC). It is important to understand the differences between the two, in order to know how the building code requirements apply. Exits are needed during evacuations, to ensure safe egress of occupants in the event of a fire, and for the fire services to stage their operations and gain access to the fire floor. For this reason, it is essential that these exit shafts maintain their integrity and tenability in these situations. Neither exit stair shafts nor elevator shaft walls are required to be constructed as firewalls.

Maintaining the integrity of exit stairways is essential to ensure safe egress. All joints and connections need to be appropriately sealed and fire stopped to prevent any smoke leakage. This is particularly important in the case of scissor stairs, to stop any smoke or hot gases from moving between the two stairways. Effectively sealing CLT can be difficult; it is recommended that a layer of gypsum board be used on the interior of scissor stairs, or to pressurize the shaft.

While some provinces have required exit stair shafts for midrise construction to be of noncombustible construction, this is not required in the NBCC 2015. Further, in November of 2014, a full-scale fire test of a CLT stair shaft adjacent to a residential suite demonstrated that CLT stair shafts can remain tenable and provide a fire resistance in excess of 2 hours (see Subsection 8.9.6).

It is relevant to note that statistics on fires in multi-unit residential buildings indicate that fires are typically confined to the compartment of fire origin as well as the floor of fire origin, and that the majority of casualties and injuries are also within that same space. Thus, it is unlikely that a fire will move into a stair or elevator shaft, or that injuries will occur in an exit shaft or elevator shaft that is remote from the fire (118).

8.9.1 Compatibility

For mass timber structures, it is advantageous to use a continuous building material throughout, to avoid connection or compatibility issues such as differential shrinkage between materials or differential movement when subjected to lateral and/or gravity loads. Examples of buildings that experienced differential movement issues may be found in Karacabeyli and Lum (31). FPInnovations has prepared a report related to considerations for wood vertical shafts in mid-rise and tall buildings, which discussed these issues in more detail (119). Differential shrinkage can be a particular problem for exits because of the potential disparity at door frames, which would create a tripping hazard and impede egress efforts. Shrinkage can be an issue for elevator construction that requires highly precise tolerances; guidance should be sought from elevator manufacturers as to whether wood construction can meet their specifications (39).

Traditionally, elevator shafts in tall buildings of noncombustible construction are built using a continuous reinforced concrete core, which acts as an enclosure and is able to carry gravity and lateral loads, or are enclosures made from cold-formed steel partitions. These, can also be used in tall buildings using combustible construction, or in combination with other wood materials. A hybrid elevator shaft can use CLT for two facing walls and concrete blocks for the opposite facing walls, where elevator rails are attached. This is likely not an economical solution and is wrought with compatibility and tolerance issues.

A CLT elevator shaft can perform similarly to one made with a concrete core. The use of concrete cores slows construction time and can contribute to an increase in the period of time where a wood building is left unprotected, which ultimately will increase the risk of a fire during construction. Ensuring an efficient construction schedule can reduce insurance premiums, and reduce potential exposure to the environment (such as humidity and rain).

8.9.2 Flame Spread Rating

Flame spread ratings (FSR) for interior finishes become more stringent as building height increases, to limit their contribution to fire growth and spread.

Table 3.1.13.7 in Division B of the NBCC provides details on the maximum FSR and Smoke Developed Classification (SDC) based on the location of the element and the type of element (wall, ceiling, or floor). The FSR for both exit stairways and vertical service spaces is limited to no more than 25, and the SDC is limited to 50 in high buildings (per NBCC definition), regardless of whether the building has a sprinkler system.

Elevator and stair shafts that are not protected by automatic sprinklers are required to have an FSR no greater than 25. Given the need for stable surfaces for mounting of components, an effective method for elevator shafts is the use of a layer of fire-retardant-treated plywood. For stair shafts, a layer of gypsum board is effective. Surface-applied coatings may not provide the durability required for the life of the building.

8.9.3 Fire Separation

Article 3.4.4.1 in Division B of the NBCC stipulates that every exit must be separated from the remainder of the building, through the use of fire separations. The fire separations must have a fire-resistance rating at least equivalent to the surrounding floor assemblies and be no less than 45 minutes. These separations help prevent the fire from spreading into the stair shaft. These ratings are generally 1 hour in mid-rise buildings, and 2 hours in tall buildings.

Similarly, vertical service spaces for elevator hoistways must be separated from adjacent storeys with a fire separation having an FRR conforming to Table 3.5.3.1. For tall buildings, where assemblies are required to have a 2-hr FRR, the vertical service space must have an FRR of 1.5 hr. These provisions aim to limit the spread of the fire into the service space; this prevents the fire from moving between storeys, and also protects any person in an elevator car. It is straightforward to design a CLT assembly to achieve a 2-hr fire resistance rating, especially with the inclusion of Type X gypsum board, as has been detailed in the previous Section on fire resistance.

CLT can be manufactured in longer lengths, extending through multiple storeys; this helps ensure the continuity of the fire separation for a shaft, by reducing potential paths for smoke leakage. It is imperative that joints be appropriately fire stopped and sealed, at each floor level.

8.9.4 Sprinklers

Exit stairs in mid-rise buildings of combustible construction where the shafts have exposed timber must have sprinklers installed to meet NFPA 13 (103). This is more stringent than what is required for shafts lined with noncombustible material such as gypsum board. NFPA 13 includes provisions specific to shafts with combustible surfaces (8.15.2.2), which require that sprinklers be installed in vertical shafts at each alternate floor level, and that stairway sprinklers be installed beneath all floor landings and at the top of the stair shaft. Article 8.15.5.4 requires a sprinkler at the top of an elevator hoistway. However, this does not preclude the requirement for the elevator shaft to achieve an FSR no greater than 25.

8.9.5 Elevators

In tall buildings, elevators are being increasingly used for evacuation and for fire service operations, because they have the advantage of moving people quickly in time sensitive situations. Article 3.2.6.5 in Division B of the NBCC provides provisions for elevator use by firefighters in high buildings.

The occurrence of a fire originating within an elevator shaft is low, due to lack of public access in the space. There is the potential for garbage or elevator grease in the pit to be an ignition source, as well as a risk of fire from the machine room, although these are typically placed at the top of the shaft (120). Routine maintenance of the elevator pit helps to reduce this risk.

8.9.6 Demonstration Fire

With the financial support of the Ministère des Forêts, de la Faune et des Parcs du Québec, a demonstration fire was conducted to exhibit how a CLT exit shaft could withstand the effects of a severe fire in an adjacent apartment unit (121). The design of the setup was based on the 13-storey Origin building in Québec City, using 5-ply CLT walls and floors. An additional stud wall with rock fibre insulation was built between the shaft and the unit, to enhance sound isolation. The CLT surfaces in the interior of the shaft were left exposed. A high fuel load fire was permitted to burn for 2 hours, before being extinguished. Figure 31 shows the structure shortly after the fire was started. No impact was observed in the CLT shaft throughout the duration of the fire: there was no evidence of temperature rise and no apparent smoke leakage. After the fire, some gypsum board was still in place on the common shaft/apartment wall, which was removed. There was limited localized light charring of the exposed CLT surface and the remainder of the surface was unaffected (i.e. it was uncharred). This suggests there was little to no impact on the structural resistance of the CLT shaft itself.

This research confirmed that exit stairs and/or elevators shafts are capable of maintaining their structural integrity and tenable conditions (in terms of smoke and temperature criteria). This is critical for the safety of occupants during evacuation, as well as for allowing fire services to safely access the fire to carry out their operations.

Figure 31 Full-scale demonstration of CLT shaft fire performance

8.10 PERFORMANCE-BASED DESIGN

Currently in Canada, performance-based design (PBD) can be used in the development of Alternative Solutions in accordance with the NBCC, to gain approval for designs that do not directly adhere to the prescriptive provisions found in Division B of the code. PBD focuses on identifying and achieving a given safety level, as opposed to following prescriptive design provisions. This method of design allows for much greater flexibility and fosters innovation, while providing an opportunity to potentially use technologies that are not yet recognized by the NBCC. This is particularly attractive for larger and taller CLT buildings, which are not presently permitted by the prescriptive provisions of the NBCC; it is also attractive for potentially allowing more exposed wood surfaces in a building. A PBD approach for tall and large wood buildings provides guidance on the design process (34).

It is essential that all project stakeholders be involved from the early stages in the project, to ensure effective communication during design, approval, and construction, throughout the performance-based design process. Having any concerns from Authorities Having Jurisdiction and/or the Fire Service known upfront will result in a robust design that will be better suited to gain approval.

In British Columbia, performance-based design may also be the basis for a Site Specific Regulation, as permitted by the new Building Act.

8.10.1 Design Process

For a performance-based design (PBD) to be successful, it needs to clearly demonstrate, with supportive documentation, how the necessary safety level will be met. A thorough engineering analysis of the design needs to be undertaken.

The first task is to establish what the necessary level of safety is and how it can be defined by quantitative performance criteria. Once these are known, the design can be developed and evaluated, based on these criteria. This process should be iterative until satisfactory results are achieved, as outlined in ISO/DIS 23932-1 (122) and shown below in Figure 32.

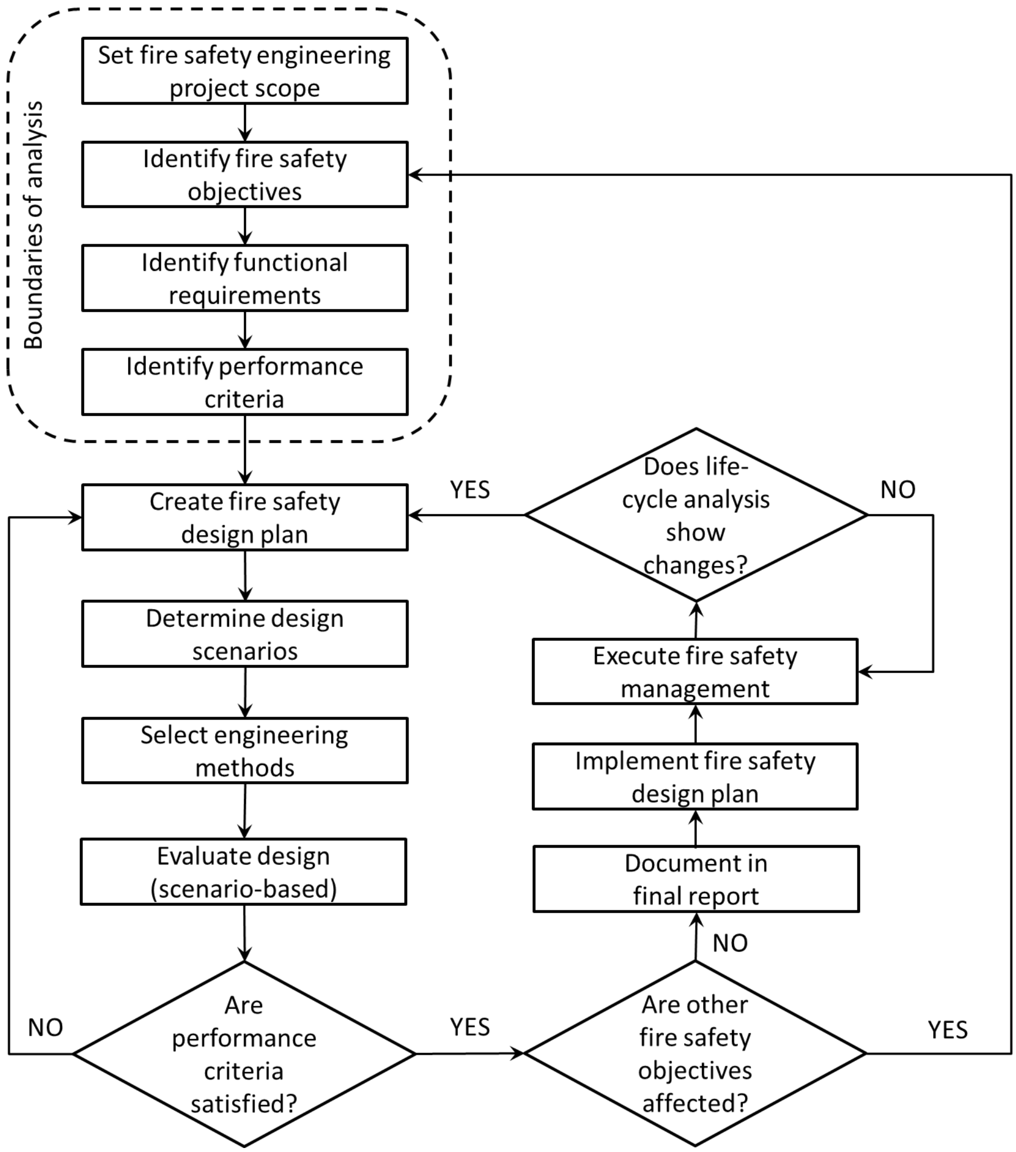


Figure 32 Performance-based fire safety design process, as presented in (122)

8.10.2 Performance Criteria and Verification Methods

A set of criteria that will be used to evaluate the design needs to be determined. In order to gain approval in Canada, the design should address the objectives outlined in the NBCC, in relation to Safety (OS), Health (OH), Accessibility (OA), Fire and Structural Protection of Buildings (OP), and Environment (OE). These include limiting the risks associated with fire spread beyond the point of origin, fire contribution and severity, structural collapse, fire spread to adjacent structures, and risk to occupants and fire service personnel.

An analysis of the NBCC in comparison to other performance-based codes around the world identified that the intent of many of the objectives are consistent, such as preventing fires from impacting beyond their point of origin and collapse of physical elements due to fire (32). These objectives can form the basis for the selection of performance criteria, such as for structural fire performance of limiting fire spread. The NBCC, however, does not provide specific details on how the safety of occupants needs to be maintained, but many other building codes do set specific tenability criteria in terms of exposure to heat or toxicity levels. Tenability criteria for people are available, as are methods to evaluate them (123).

Once a design has been established, it needs to be assessed based on the selected performance criteria. The verification process involves selecting several fire scenarios to evaluate the performance of the building and determine the resulting fire dynamic profiles. This includes calculating factors such as flashover, smoke layer height, heat fluxes, and tenability conditions (e.g., CO levels and Fractional Effective Dose). The fire scenarios are selected to represent potential situations that could occur and encompass a range in terms of size and severity of design fires; other factors to consider include occupancy at the time of the event and location of the fire. NFPA 5000 "*Building Construction and Safety Code*" (124) the characteristics of at least eight fire scenarios that should be considered in a PBD.

Tall CLT buildings will be required to be fully sprinklered and may involve a high degree of encapsulation or, alternatively, will require an enhanced reliability for the sprinkler system installation. Structural elements in these buildings should likely be robust enough to withstand a full-burn out scenario (125), which will ensure the safety of the structure and occupants in scenarios where sprinklers have failed or fire fighters are unable to readily intervene (25). This can be achieved by CLT compartments when encapsulation works in conjunction with the inherent fire resistance of the CLT.

While encapsulated mass timber construction can provide the required fire performance, it is likely that a certain amount of exposed wood surfaces will be desired by designers. In an attempt to assess the fire contribution from a given amount of exposed mass timber from ceilings or walls within a compartment, Barber (25) has developed two methods. The first is an iteration calculation to determine an average charring rate based on heat fluxes of a design fire, and the second relies on advanced computational fluid dynamics (CFD) modeling. Extensive computer modeling research and validation is supporting the acceptance of these models by qualified AHJs in some jurisdictions. Barber suggests limitations for exposed CLT based on a methodology that has previously been applied for exposed timber structures, and tongue and grove floors (25); these include:

1. Only one timber surface should be exposed in a compartment, to prevent re-radiation between exposed surfaces
2. Loadbearing elements should be protected with gypsum board, which is expected to stay in place for the duration of a fire
3. Exposed CLT should be limited, so that it will not impact the FRR of the compartment

This work also provides guidance on an alternative calculation methodology to determine the FRR for connections in glulam beams and columns, for situations where a FRR of up to 2 hrs might be required. This is based on the rate of char and the depth of the heated zone, to determine the depth of timber cover needed to ensure the timber retains 80% of its embedment strength.

8.11 FIRE SAFETY DURING CONSTRUCTION

The construction period is the time when wood buildings are most susceptible to fire risk. This is because most safety systems, including both active and passive, are not yet in place. It is important to be aware of the risks, so that appropriate measures can be considered during the design stage and appropriately implemented prior to construction, to ensure the safety of everyone on site. The fire service should be consulted during design, so that they are aware of the risks during construction and are familiar with the site and building details, which will improve their ability to address a fire should they be called to the site.

Many organizations and jurisdictions have begun developing their own guidelines in relation to fire safety practices during construction of wood buildings. The Canadian Wood Council has published many useful resources on this topic (126, 127, 128, 129). The Construction Fire Safety Coalition, under the supervision of the American Wood Council, has developed several manuals, online courses, and a comprehensive website to help builders address fire safety during construction (130, 131, 132). FPInnovations recently documented the fire safety plans that were in place during construction of the UBC Brock Commons and Arbora buildings (133). This report highlighted common fire safety measures that should be in place (such as the abundant presence of fire extinguishers, as shown in Figure 33), but also the need for fire safety plans to be project specific, as different designs and construction methods require different considerations.

Figure 33 Importance of access to fire extinguishers on site during construction (133)

Construction fires are generally started by hot work or heaters on site, or careless fire safety practices (such as improper discarding of cigarettes), but the majority are incendiary (arson). Removing some of these dangers can be straight forward, such as eliminating hot work and enacting strict no smoking policies on site. An example of the dangers of hot work near CLT is shown in Figure 34, where cutting a steel plate resulted in charring of the nearby CLT surface. Appropriate awareness and education of fire risk should be mandatory for anyone on site. To address incendiary fires, 24-hour security should be provided.

Figure 34 Dangers of hot work near CLT, charred CLT slab (133)

CLT construction has several inherent benefits to improve fire safety on site. Risk of ignition is highest while wood elements are exposed and have not been covered with gypsum board. During construction, loose combustible debris or piles of discarded combustibles represent locations where fires could be initiated. For certain construction designs, where compartment walls are installed later, such as the use of glulam columns and CLT floor plates, the lack of walls presents an opportunity for fire to spread more readily. For buildings using CLT for interior wall construction, this hazard is reduced (but not eliminated), since walls are erected simultaneously with the structure and can act as a barrier to fire spread.

The speed at which CLT structures are erected improves their fire safety, compared to traditional construction timelines. The installation of interior finishing limits the ignition potential of surfaces (due to the application of gypsum board). As construction progresses, active fire safety systems can be installed (such as sprinklers), which are effective at limiting fire growth and fire spread. One strategy that has been used for tall CLT buildings included limiting the number of storeys having exposed CLT, before construction could continue. This meant that interior finishing began on lower storeys as upper levels were added. It is recommended that no more than 4 storeys be left unexposed at any time.

The use of prefabricated elements greatly streamlines the installation process, removing many unnecessary steps. This helps eliminate waste on site, as elements arrive ready for installation, and cuts do not need to be made. This limits the generation of saw dust piles which can be potential locations of ignition. Building Information Modeling (BIM) is a very effective tool that has been successfully used in the construction of the Brock Commons building in B.C., where construction was completed rapidly.

Fire safety during construction must be a priority considered during design. It is of utmost importance to ensure that everyone on site is appropriately educated on the fire risks, that measures are in place to quickly alert personnel in the event of a fire, and that adequate means for evacuation are available to ensure occupant safety.

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Erol Karacabeyli, M.A.Sc., P.Eng., FPInnovations
Sylvain Gagnon, P.Eng., FPInnovations

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CHAPTER 9

Acoustic performance of cross-laminated timber assemblies

Lead Author
Lin Hu

Co-Author
Sylvain Gagnon

Reviewers
Ineke Van Zeeland
Jean-Philippe Migneron
Anes Omeranovic

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ABSTRACT

Noise control, e.g. mitigation of unwanted sound, is an important serviceability consideration for the design of buildings. There is a need for a noise control procedure that would guide practitioners and contractors to fulfill their design goals. This was the motivation behind Chapter 9, *Acoustics Performance of CLT,* of the 2011 Edition of the Canadian CLT Handbook.

In 2011, when this Chapter was first written, the National Building Code of Canada (NBC) was in the process of replacing Airborne Sound Transmission Class (STC) by Apparent Sound Transmission Class (ASTC). Same as STC, ASTC is a single number rating of the apparent airborne sound insulation performance of the combined wall and floor/ceiling assemblies in buildings, as perceived by the occupants. The apparent airborne sound insulation accounts for direct transmission through the demising element, as well as flanking transmission. In 2015, ASTC was finally implemented in the NBC (NRC, 2015) as a measure for airborne insulation performance.

Also in 2011, there were no CLT products or buildings in Canada; therefore, the first edition of this Chapter was based on European experience, designs, and materials. Canadian CLT panels are now being produced and are available in the market, and significant research efforts have been devoted to study sound insulation performance of CLT wall and floor/ceiling assemblies, and to develop solutions for the assemblies to meet code requirements and consumer expectations. A number of CLT buildings were built using Canadian products and solutions, and apparent sound insulation performance tests have been conducted on these buildings. Feedback on their sound insulation performance has also been monitored. These studies have resulted in significant advancements in knowledge and have provided solutions for CLT building sound insulation in Canada, and this has led to updating this Chapter. Readers will find a variety of design examples of CLT walls and floor/ceiling assemblies in this updated Chapter. These examples meet a broad range of requirements for sound insulation ranging from minimum code requirements to occupant high demands.

This updated Chapter compiled the latest knowledge, data, and experience for noise control of CLT buildings. In writing this Chapter, it proved necessary to adopt the trade names of finish, membrane, topping, underlayment, and sound absorptive materials in the descriptions of the design examples given, due to the lack of assessment and classification of these products in the current standards. Thus, these trade names have been included for the only purpose of providing accurate details of reasonably good and functional assemblies, without any intention to promote specific products or manufacturers. As a living document, the Chapter will evolve in the future with the development of such generic standards and criteria.

9.1 SCOPE

This Chapter addresses sound insulation for cross-laminated-timber (CLT) separating walls, and floor/ceiling assemblies between adjacent spaces, such as between dwelling units, and between dwelling units and adjacent public areas such as halls, corridors, stairs, elevator or service areas in buildings. This Chapter does not include sound insulation of exterior CLT walls and roofs and, due to lack of information and data, just slightly touches on the topic of sound insulation of stepped story CLT buildings.

The noise control measures described in this Chapter are relevant for all types of CLT construction; the main difference between noise transmission in low and mid- to high-rise CLT buildings is due to the different wall designs. The walls in mid- to high-rise buildings are expected to carry higher axial and lateral loads. In contrast to the design of mid- to high-rise wood walls, the design of mid- to high-rise wood floors does not differ from those of low-rise buildings.

The goal of this Chapter is not only to provide solutions for noise control, but also to provide a road map for controlling noise, by showing the readers how to use a systematic and logical approach to control noise transmission in buildings. A systematic approach includes four steps: 1) understanding the fundamentals of building acoustics, 2) knowing the principles of noise control, 3) understanding the effects of various construction details on sound insulation performance, and 4) developing strategies to address noise control in buildings. The examples of noise control design solutions presented in this Chapter were selected from various sources to ensure they meet or exceed the code requirements for sound insulation. The design solutions are ready to be applied to new CLT building projects, and also illustrate the effects of various details on sound insulation. A summary of the general effects of various design parameters and construction details on sound insulation of CLT walls and floors is also provided. By following the road map and examples provided in this Chapter, new and innovative design solutions may be derived.

9.2 FUNDAMENTALS

9.2.1 Noise and its Source

Basically, noise is commonly defined as “unwanted sound”. To control noise transmission through walls and floors between units, and between units and adjacent public areas in multi-family buildings, one needs to know what sound is and where it comes from. Sound has been defined as a physical disturbance in an elastic medium (i.e., in a gas, liquid, or solid) that is capable of being detected by the human ear. The medium in which the sound or pressure waves travel must have mass and elasticity. Thus, sound waves will not travel through a vacuum (Harris, 1957; 1991).

Sound waves in air are caused by variations in pressure above and below the static value of atmospheric pressure (Harris, 1959 and 1991). These pressure variations originate in many ways, for example: 1) by a pulsating airstream, such as that produced by fan blades as they rotate, or by a loudspeaker, 2) by supersonic flight of an aircraft, which creates shock waves, 3) by the vibration of a surface, such as a wall or a floor, 4) by talking or by a musical instrument.

A pressure wave propagating through air is referred to as airborne sound, and the pressure wave propagating through a solid structure is structure borne sound. Figure 1 illustrates the pressure wave propagating through air or a solid structure (Kappagantu, 2010). It should be pointed out that, for the sake of simplicity, Figure 1 only illustrates a simple harmonic sound wave. The simple harmonic sound wave can be generated by most musical instruments that produce several simple harmonics simultaneously. However, sound produced by machines or structures do not behave as simple harmonic sound waves, but are random in time and are commonly referred to as noise (Crocker, 2007).

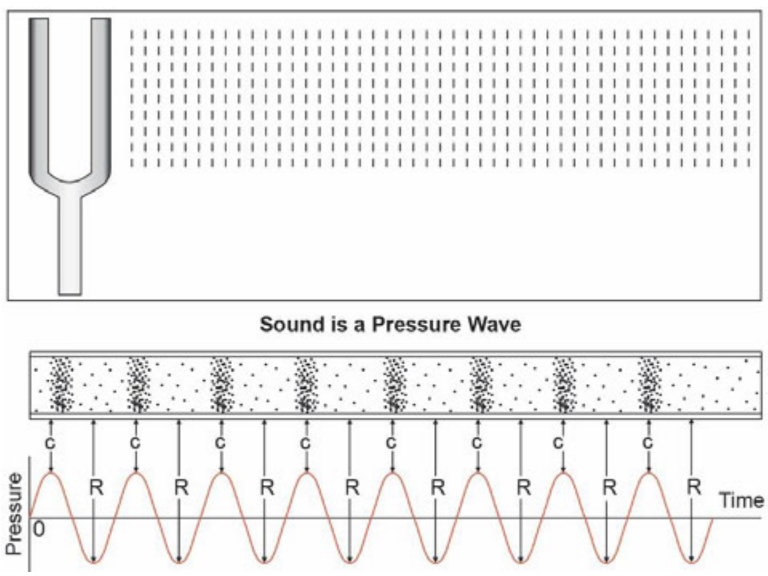


Figure 1 Simplified illustration of sound (pressure wave) propagation through air/solid (Kappagantu, 2010). In this figure, C signifies regions of compression and R signifies regions of rarefaction of the air molecules. Furthermore, the "0" pressure line in the graph represents the atmospheric pressure level.

9.2.2 Quantification and Measurement of Sound

Sound has various attributes and can be described by various quantities. Sound has level or magnitude. According to Fourier's theory, sound waves can be characterized by a sum of infinite periodic waves. Sound also has a frequency content. Frequency is defined as the reciprocal of the period. In addition, sound can vary in level and frequency as a function of time. With respect to the level or magnitude of sound, the most commonly used metric that can be directly measured is sound pressure. Other quantities can then be derived from sound pressure, which is defined as force per area. Sound pressure is the pressure variation above and below the atmospheric pressure (Crocker, 2007); it is a fluctuating quantity measured in Pascal units (Pa). Sound pressure is influenced by the energy produced by the sound source, the environment, and the distance between the source and the receiver (Pope, 2003). It is usually characterized by its Root Mean Square (RMS) or Peak values, with mean pressure disregarded (Pope, 2003).

To convey an understanding of sound pressure, Pope (2003) gave some examples of the sound produced by various sources and their pressure (Table 1).

Table 1 Sound generated by various sources and their pressure (Pope, 2003)

Sound from	RMS pressure (Pa)
Music club (loud)	~ 10
Heavy traffic at 10 m	~ 1
Busy office	~ 10^{-1}
Normal speech at 1 m	~ 10^{-2}

Sound pressure is related to atmospheric pressure at the point where the sound pressure is measured. In contrast, sound is better quantified using absolute values independent of the atmospheric pressure, for comparison of sound performance in various atmospheres. Therefore, sound pressure level (SPL) is used. SPL is the power ratio of sound pressure to a reference sound pressure that is the sound pressure at the threshold of hearing at 1 kHz. The unit of SPL is the decibel (dB). The dB unit is read on a logarithmic scale.

Sound pressure levels can be measured with a microphone. The signals are recorded and processed by a device such as Sound Level Meter (SLM) or a Spectrum Analyzer. The device is equipped with software of data acquisition and process to transform the sound level versus time signal into a frequency domain for the spectrum analysis. It produces the frequency distribution of the sound pressure level of the signal. The spectrum analysis is important for understanding the behavior of sound performance of an object and for developing noise control measures. Figure 2 shows a one-third octave band center frequency spectrum of a sound pressure level measured under a wood floor/ceiling assembly in the low unit of a building, using a running ISO tapping machine on the floor of the upper unit. Figure 3 shows the impact sound test using ISO tapping machine. Figure 2 shows how the measured sound pressure level fluctuated with frequency.

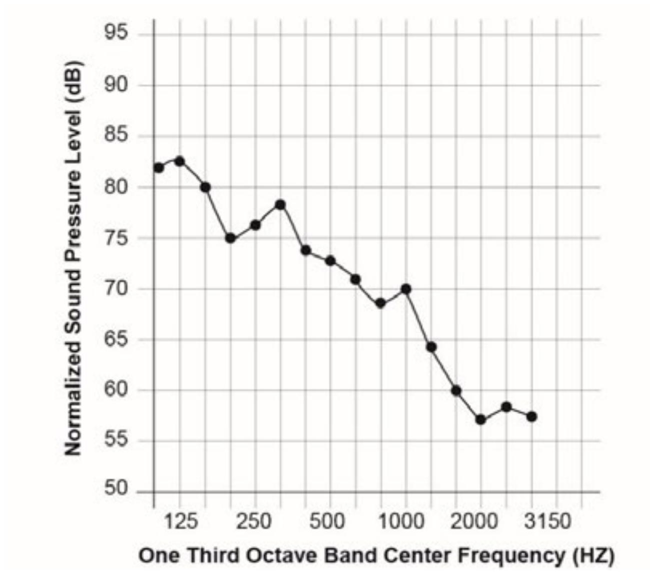


Figure 2 A typical 1/3 octave band center frequency spectrum of sound pressure level measured below a wood floor/ceiling assembly in conformance with ASTM E-1007 method

(a) ISO tapping machine on a wood floor

(b) Microphone under the floor in the low room

(c) Zoom of the ISO tapping machine

Figure 3 Impact insulation test on a floor, using an ISO tapping machine

9.2.3 Range of Human Hearing

According to Crocker (2007), humans can hear sound in the frequency range between 15/16 Hz and 15/16 kHz. However, we do not hear all sounds equally, which means that our hearing sensitivity is non-linear. The sensitivity of human hearing is frequency- and sound pressure level-dependent. Humans are most sensitive to sounds at about 4000 Hz, and less sensitive to sounds below 200 Hz or above 10000 Hz. Below 200 Hz, humans cannot hear sound well, unless the sound pressure level is high enough (Crocker, 2007). Figure 4 from White (1975) shows the average threshold curve for young adults with "normal" hearing. Note that the threshold of hearing is markedly dependent on frequency.

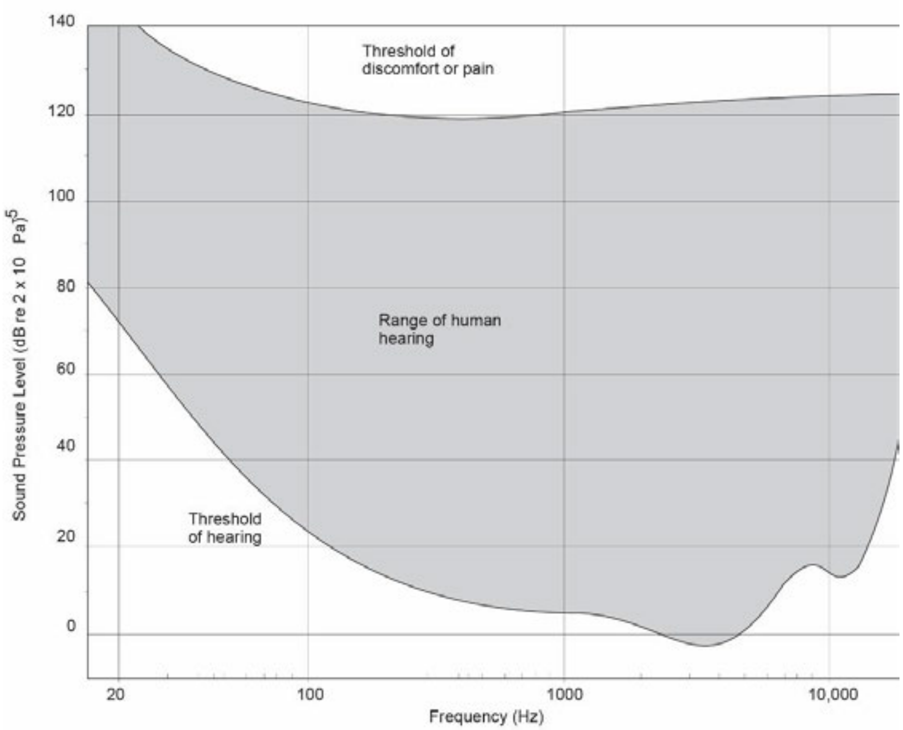


Figure 4 Range of human hearing (White, 1975).

9.2.4 Human Perception of Sound

Human perception of sound is both objective and subjective, and several factors affect it:

- level and frequency spectrum, sharpness, masking effects;
- variations such as fluctuation, roughness, modulations, transients;
- context, such as day vs. night, music vs. machine, etc.; and
- individual preferences.

Pope (2003) has described how humans perceive change in sound levels (Table 2). This knowledge provides very useful guidance for developing cost effective sound insulation solutions or to improve existing sound insulation strategies. The table below shows that a change (reduction or increase) in sound levels of less than 3 dB will most likely not be perceived by a listener, but that a change of 3 dB or greater will most likely is perceived by most people.

Table 2 Perceptible change due to the change in sound levels (dB) (Pope, 2003)

Change in sound levels (dB)	Change in perceived loudness
3	Just perceptible
6	Noticeable difference
10	Twice as loud, or reduced to half of the loudness
15	Large change
20	Four times as loud, or reduced to one quarter of the loudness

9.3 REVIEW OF THE REQUIREMENTS FOR NOISE CONTROL IN THE 2015 NBC

The 2015 National Building Code of Canada (NBC) requires protection from noise. Below are the 2015 NBC (NRC, 2015) requirements for building noise control.

9.3.1 Requirements in the 2015 NBC

For airborne noise, the 2015 NBC requires that a dwelling unit shall be separated from every other space in a building in which noise may be generated, by construction providing an apparent sound transmission class rating (ASTC) of no less than 47. It also requires that construction separating a dwelling unit from an elevator hoist way or a refuse chute shall have an STC rating of no less than 55.

For demonstrating compliance with the ASTC requirements, the 2015 NBC provides three separate paths in its acceptable solutions in Division B of the Code:

1. *In situ* field measurement using the ASTM E336 procedure (and the ASTM E413 calculation procedure), which can only be applied to completed buildings;
2. a prescriptive “deemed-to-comply” procedure, using the existing list of STC-rated assemblies in the Part 9 Fire and Sound Resistance Tables for light wood frame walls and floors (Tables 9.10.3.1.-A and -B) combined with certain joint configurations and other required details also provided in Part 9; and,
3. a design procedure that is based on the ISO 15712 calculation methodology, described in the NRC guide RR-331 for calculating airborne sound transmission in buildings (Hoeller et al., September 2017) (http://doi.org/10.4224/23002279).

The 2015 NBC does not set a requirement for impact noise (structure borne noise) protection, but the 2015 NBC recommends that bare floors tested without a carpet should achieve an impact insulation class (IIC) of 55 (NRC, 2015).

9.3.2 Sound Transmission Class (STC) and Its Measurement

STC is a single number rating of the airborne sound insulation of a building element, e.g., a wall or a floor/ceiling assembly. The greater the STC is, the better the airborne sound insulation of the building element will be. STC is determined from the sound transmission loss through the wall or floor/ceiling assembly in ideal laboratory acoustical chamber. Transmission Loss (TL) is the ratio of transmitted power to incident power in dB at 1/3 octave band center frequency. It is the measure of sound attenuation through the wall or floor/ceiling assembly. The greater the TL value, the less sound is transmitted through the building element, and thus, the better the sound insulation performance will be. Like sound pressure level (shown in Figure 2), TL varies with frequency.

ASTM E90 standard specifies the procedures that should be used to conduct laboratory measurements of airborne sound transmission loss of building partitions and elements. While the test is conducted according to ASTM E90, ASTM E413 standard provides the numerical procedure for determining the STC from the measured sound transmission loss data, and the sound insulation classification. According to ASTM E413, STC ratings correlate in a general way with subjective impressions of sound transmission for speech, radio, television, and similar sources of noise in offices and buildings.

9.3.3 Impact Sound Insulation Class (IIC) and Its Measurement

IIC is a single number rating for the impact sound insulating performance of a floor/ceiling assembly. The greater the IIC value, the greater the impact sound insulation will be. IIC is determined by measuring sound pressure levels in dB in the room under the floor/celling assembly being tested, while an impact is applied to the floor. The measurement is performed in an ideal laboratory acoustical chamber. The sound pressure level is the measure of the impact sound attenuation through the floor/ceiling assembly. The greater the impact sound level value, the more impact sound is transmitted through the floor/ceiling assembly, and the poorer the impact insulation of assembly will be. As shown in Figure 2, sound pressure level also varies with frequency.

ASTM E492 standard specifies the test method for laboratory measurement of impact sound transmission through floor/ceiling assemblies using a tapping machine, and ASTM E989 standard provides the classification for determination of IIC from the sound pressure level measured in the receiving room, produced by the tapping machine on the floor/ceiling assembly being tested.

However, in buildings, there are various flanking paths for sound to go around the separating wall or floor/ceiling assembly and not just directly through it. Due to the flanking sound transmission, the sound insulation performance of the walls and floor/ceiling assemblies perceived by the occupants is usually lower than the performance target values of STC and IIC.

9.3.4 Flanking Sound Transmission

Flanking sound transmission is sound transmission along paths other than the direct path through the common wall or floor/ceiling assembly (NRC:IRC, 2002).

Figure 5 illustrates an example of flanking and direct paths in a two-storey building; D and F standard for direct and flanking path, respectively.

Typical flanking sound transmission paths for all types of floors, walls and ceilings can include:

- Above and through the ceiling (plenum) spaces;
- Through floor and floor joist space of the wood frame floors;
- Through windows and doors;
- Through fixtures and electrical outlets, light switches, telephone outlets, and recessed lighting fixtures;
- Shared building components, such as continuous topping, continuous floor joists and decks of the wood frame floors, continuous concrete slab or mass timber slab floors, and continuous walls;
- Perimeter joints at wall and floor, through wall and ceiling junctions;
- Through plumbing chases and joints between the walls and floor slab above, or at the exterior wall junctions; and
- Around the edges of partitions, through the adjacent walls.

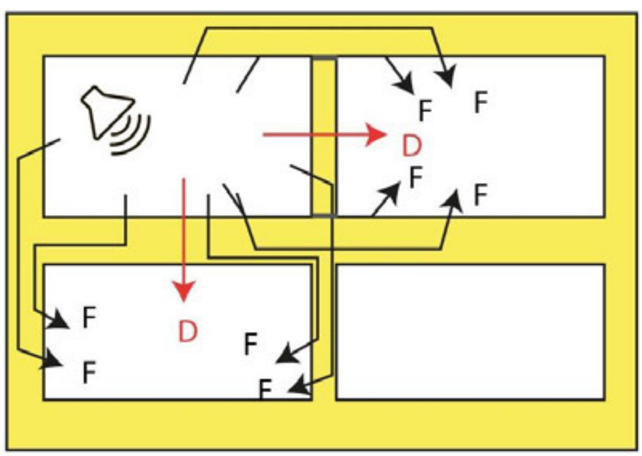


Figure 5 An illustration of direct and flanking sound transmission paths indicated by D and F, respectively

The National Research Council has developed a predictive software tool, soundPATHS, to estimate the apparent airborne sound insulation (ASTC rating) for many types of building construction, including wood frame construction and CLT construction, taking into account flanking sound transmission through the wall and floor/ceiling junctions. It also provides STC rating information, including for assemblies of both those types of wood construction. It can be accessed at https://www.nrc-cnrc.gc.ca/eng/solutions/advisory/soundpaths/index.html. The NRC website also provides the guide to calculating airborne sound transmission in buildings (Hoeller et al*.,* September 2017) (http://doi.org/10.4224/23002279); the data currently available for CLT assemblies needed for the calculation method can be found in a report entitled "RR-335: Apparent Sound Insulation in Cross-Laminated Timber Buildings" (Hoeller et al., July 2017) (http://doi.org/10.4224/23002009).

However, in buildings, there are other flanking paths than the wall and floor/ceiling junctions as described above. Currently, no established method exists to predict the effects of all these additional flanking paths on sound insulation in any type of building construction, including those wood buildings of CLT or wood frame construction. However, it is expected that sound insulation design that is done in compliance with the code and constructed with attention to workmanship will exhibit conformance to the Code requirements.

9.3.5 Apparent Sound Transmission Class (ASTC) and Its Measurement

ASTC is a single number rating of the apparent airborne sound insulation performance of the wall and floor/ceiling assemblies in buildings, as perceived by the occupants. The apparent airborne sound insulation accounts for direct transmission through the wall or floor/ceiling assembly, as well as flanking transmission. For completed buildings, the ASTC rating can be determined in accordance with ASTM E413, from the measured data of apparent transmission loss (ATL) according to ASTM E336.

9.3.6 Apparent Impact Sound Insulation Class (AIIC) and Its Measurement

AIIC is a single number rating of the apparent impact sound insulation performance of the floor/ceiling assemblies in buildings, as perceived by the occupants. The apparent impact sound insulation accounts for direct transmission through the floor/ceiling assembly, as well as flanking transmission. The AIIC is determined in accordance with ASTM E989, from the measured data of absorption normalized impact sound pressure level (ANISPL) according to ASTM E1007.

9.4 BEYOND THE CODE REQUIREMENTS FOR OCCUPANTS' SATISFACTION

It is worth noting that the NBC provides only the minimum requirements for building noise control. The minimum requirements may not meet the various expectations of occupants, and a designer/builder/developer may wish to provide better noise control than the minimum prescribed by the Code. As well, because the amount of flanking sound transmission can also depend on the quality of workmanship in following acoustic design details, it can be prudent to develop designs that are expected to achieve higher ASTC values than the minimum Code requirements. The following sections provide some additional and helpful approaches to avoid claims by dissatisfied occupants.

9.4.1 Subjective Evaluation by Developers, Designers, and Engineers

It is advisable that builders, developers, architects, designers, contractors and/or product manufacturers conduct an informal subjective evaluation of the building sound insulation performance once the building is completed and before the occupants move in. This will allow them to obtain quick and easy feedback regarding the sound insulation performance of the completed building. If the design professionals are not satisfied, it is highly probable that the occupants, after moving in, will not be satisfied either, so the problems should be remedied immediately.

Below is the informal subjective evaluation protocol developed at FPInnovations that have been used in our laboratory studies and field investigations.

The informal subjective evaluation protocol is intended for use by builders, developers, architects, designers, contractors and/or product manufacturers. It is preferable to conduct the informal subjective evaluation when the ambient noise level is low, such as in the evening. Combining the measured ASTC and AIIC with the subjective evaluation ratings allows the establishment of a correlation between the perception and the ratings.

Procedure for informal subjective evaluation of airborne sound insulation of a separating wall:

Step 1: Ask the evaluator to sit quietly in the receiving room.

Step 2: Turn on music at normal volume in a room adjacent to the receiving room, on the other side of the separating wall, or ask at least two people to talk in a normal voice.

Step 3: The evaluator can then determine if the airborne sound insulation in the receiving room is acceptable.

Procedure for informal subjective evaluation of airborne sound insulation of a floor:

Step 1: Ask the evaluator to sit quietly in the receiving room (i.e., the low room, the room below the floor to be evaluated).

Step 2: Turn on music at normal volume in the source room (i.e., the upper room, the room above the evaluator), or ask at least two people to talk in a normal voice.

Step 3: Switch the source and receiving rooms: i.e., move the evaluator to the upper room and ask him to sit quietly, and move the music equipment or people who are to talk to the low room (i.e., now the upper room is the receiving room and the low room becomes the source room).

Step 4: Turn on music at normal volume, or ask the people to talk in a normal voice.

Step 5: The evaluator can then determine if the airborne sound insulation in the receiving room is acceptable in both cases.

Procedure for informal subjective evaluation of impact sound insulation of a floor:

Step 1: Ask the evaluator to sit quietly in the receiving room (i.e., the low room).

Step 2: In the source room (i.e. the upper room), ask a person to walk at a normal pace on the floor, first with shoes — preferably high-heeled shoes — and then bare foot.

Step 3: The evaluator can then determine if the impact sound insulation in the receiving room is acceptable.

9.4.2 Correlation between ASTC/AIIC and Human Perception – Our Experience

Based on subjective evaluation of many wall and floor/ceiling assemblies in FPInnovations and other’s mock-ups, and of buildings in the field, our research team has agreed on the following observations:

- When a wall or a floor/ceiling assembly has an ASTC/AIIC rating of less than 50, one can clearly hear the normal activities of the neighbor.
- When a wall or a floor/ceiling assembly has an ASTC/AIIC rating between 50 and 60, one can hear the normal activities of the neighbor, but muted to some degree.
- When a wall or a floor/ceiling assembly has an ASTC/AIIC rating larger than 60, one will not hear the normal activities of the neighbor, except for wood floors without a floating heavy topping and with carpet only. It is possible for wood floor/ceiling assemblies with a carpet and without a floating heavy topping to have an AIIC rating of 60 or above, but one may still hear the low frequency footstep noise.

However, the above observations are based on subjective evaluations; individuals can conduct their own subjective evaluation and correlate them with the field-measured sound insulation performance of walls and floor in completed buildings.

9.5 STRATEGY FOR CONTROLLING NOISE TRANSMISSION IN BUILDINGS: THREE-LINE DEFENSE APPROACH

With an understanding of the mechanisms of airborne sound and impact sound transmission, one can logically derive a strategy for controlling noise transmission in buildings, by forming three defense lines:

- First, one must develop methods to reduce the noise from the source that will transmit through a wall or a floor/ceiling assembly.
- Second, one needs to develop methods to reduce the vibration of the wall or floor/ceiling assembly caused by the noise source.
- Third, one needs methods to prevent the vibration of the wall or the floor/ceiling assembly from transmitting to the adjacent unit or to the unit below.

Therefore, for controlling airborne noise transmission through wood walls or floor/ceiling assemblies, the first line of defense is to have wall or floor finishes with a low porosity surface, which will reflect or radiate the noise back to the source room. The second line of defense is to make the wall and floor sheathing materials (e.g., gypsum board, wood sheathing), the structural components of walls or floors, and the floor topping have a sufficient mass so that the vibration amplitude of the walls or the floors will be reduced; that, in turn, will reduce the noise level. Decoupling the gypsum board wall or ceiling sheathing from the structural components of the walls and/or floors is the final, third defense line, to minimize transmission of wall or floor vibrations to or from the gypsum board, thus minimizing sound transmission to the adjacent room.

Similarly, for controlling impact noise transmission through wood floor assemblies, the first defense line is to select a finish that has the highest capacity to dissipate impact force into heat, therefore significantly reducing the impact force applied to the floor structure and the floor vibration amplitude. The second defense line is for the floating topping and the structural components of the floor assembly to have sufficient mass, to further reduce the floor vibration amplitude, and thus the impact noise level. The third defense line is to decouple the gypsum board ceiling from the floor structure.

It is also very important to fill wall and floor/ceiling cavities with good sound absorptive materials to reduce cavity air resonance for walls and floor/ceiling assemblies, even when the gypsum board is decoupled from the wall or floor structure. When the gypsum board is rigidly attached to the floor or wall structure, then the absorptive materials in the cavities are less effective.

9.6 SOUND INSULATION PERFORMANCE OF BARE CLT FLOORS AND WALLS

Table 3 provides the measured STC and IIC values of some bare CLT walls and floors, in various laboratories (Gagnon and Kouyoumji, 2011) and (AcoustiTECH, 2018), and the ASTC and AIIC values measured by FPInnovations in the field for bare CLT floors and walls in completed CLT buildings (Hu, 2014). Since the effects of flanking sound paths are included in the field test results, the measured ASTC and AIIC values are dependent on the particular design of the buildings tested (e.g., connection types between walls and other walls, and between walls and floors; size, arrangement and any protection of penetrations through assemblies; size, shape and arrangement of rooms). Therefore, the ASTC and AIIC values presented in Table 3 provide only an indication of the sound insulating performance of bare CLT floors and walls measured in the field in other buildings.

Table 3 clearly reveals that the sound insulation ratings of the bare CLT walls and floors cannot meet the minimum code requirements and satisfy occupants, and that design solutions are needed. The next sections provide a road map for the development of solutions, as well as examples of workable solutions.

Table 3 Sound insulation ratings of bare CLT floors and walls

Number of plies	Thickness (mm)	Area mass (kg/m^2)	Assembly type	STC	IIC	Source(2)
3	95-115	47.5-57.5(1)	Wall	32 -34	N.A.	Gagnon and Kouyoumji (2011)
5	131	65.5(1)	Floor	N.A.	23	AcoustiTECH (2018)
5	135	67.5(1)	Floor	39	23	Gagnon and Kouyoumji (2011)
5	146	73(1)	Floor	38-39	24-26	Gagnon and Kouyoumji (2011)
	FPInnovations' field measurements of bare CLT walls and floors					
Number of plies	**Thickness (mm)**	**Area mass(1) (kg/m^2)**	**Assembly type**	**ASTC**	**AIIC**	**Source**
3	105	52.5	Wall	28	N.A.	Hu (2014)
7	175	87.5	Floor	34	22	
7	208	104	Floor	N.A	25-30 depending on room details	

Note:

(1) The area mass was estimated by assuming that the density of the elements is 500 kg/m^3. The intention of providing the estimated area density here is merely to show the effect of the area mass of the CLT elements on the sound insulation. Knowledge of CLT area mass is very useful in the sound insulation designs of CLT walls and floors.

(2) The source documents provide additional information about the design configurations for which the STC/ASTC and IIC/AIIC values were determined.

9.7 EFFECTS OF MASS (THICKNESS) AND CONSTRUCTION DETAILS ON SOUND INSULATION PERFORMANCE OF CLT WALLS AND FLOOR/CEILING ASSEMBLIES

Tables 4 and 5 summarize the up-to-date knowledge and findings obtained from studies conducted in Canada, mainly at NRC (Sabourin, 2015; Schoenwald et al*.,* 2014) and FPInnovations (Hu, 2014), on CLT wall and floor/ceiling assemblies with various details. Its purpose is to simply demonstrate the effects of CLT mass (thickness), as well as construction details, on sound insulation performance of CLT wall and floor/ceiling assemblies. More details can be found in their reports.

Table 4 Summary of the effects of area mass (thickness) and construction details on sound insulation of CLT walls

Factor	Effect
CLT area mass (thickness)	Doubling mass of bare CLT wall increased STC by about 5 points
Gypsum board on one surface	Depending on attachment, i.e. the coupling degree, the effect can range from “no change”, “important”, “significant”, to “very significant”
Gypsum board on both wall surfaces	Depending on the coupling degree, the effect can range from “reducing sound insulation”, to “very significant”
Decoupling gypsum board from CLT wall structure	Very significant
Use of double-leaf wall	Very significant

Note:
Coupling Degree - 1 = decoupling, e.g. there is no attachment between gypsum board and CLT wall;
Coupling Degree - 2 = flexible coupling, e.g. gypsum board is attached to CLT wall through resilient channels (RC);
Coupling Degree - 3 = semi-flexible coupling, e.g. gypsum board is attached to CLT wall through furring at minimum 600 mm o.c.;
Coupling Degree - 4 = semi-rigid coupling e.g. gypsum board is attached to CLT wall through furring at maximum 400 mm o.c.;
Coupling Degree - 5 = rigid coupling, e.g. gypsum board is directly attached to CLT wall.
“Not significant” means less than 3 points change in STC;
“Important” means around 3 points change in STC;
“Significant” means around 6 points change in STC;
“Very significant” means more than 6 points change in STC.

Caution should be taken when selecting resilient channels (RCs) to attach gypsum board, because the quality of the RCs varies from product to product, and the variation in RCs affects their effectiveness in improving sound insulation. More data is needed to quantify the effects of various RC products on wall sound insulation.

Table 5 Summary of the effects of area mass (thickness) and construction details on sound insulation of CLT floor/ceiling assemblies

Factors	Effect on airborne sound insulation	Effect on impact sound insulation
CLT area mass (thickness) increased from 175 mm to 245 mm	Important	Important
Have wood flooring floating on membrane	Not significant	Important
Increase in topping mass	Important	Important
Have underlayment to float topping(1)	Very significant	Very significant
Number of layers of gypsum board (1 to 2 layers) in dropped ceiling	Not significant	Not significant
Decoupling gypsum board from CLT	Very significant	Very significant
Method of attachment of gypsum board ceiling (e.g. via wood furring versus dropped ceiling using metal grillage)	Very significant	Very significant
Cavity thickness increase from 100 mm to 200 mm	Not significant	Not significant

Note:
(1) It is recommended to select an underlayment with dynamic stiffness less than 10 MN/m^3, and loss factor between 0.1-0.3, where:
dynamic stiffness = ratio of dynamic force to dynamic displacement (ISO, 1989).
loss factor = measure of the damping

"Not significant" means less than 3 points change in STC or IIC;
"Important" means around 3 points change in STC or IIC;
"Significant" means around 6 points change in STC or IIC;
"Very significant" means more than 6 points change in STC or IIC.

9.8 NOISE CONTROL THROUGH DESIGN

9.8.1 Considerations for Noise Control Design of CLT Buildings

Using the strategy for controlling noise transmission in buildings described in Section 5, one can design to control noise by specifying products with low porosity surface, a highly impact force absorptive finish, and sufficient mass. In addition, decoupling and discontinuing building components are basic principles for building noise control design. Specifically, the main general factors affecting airborne sound insulation of any wood walls and floor/ceiling assemblies, as well as the impact sound insulation of wood floor/ceiling assemblies are summarized below (NRC, 2002):

- Porosity of the materials, especially the finish materials: the lower the porosity, the better the airborne sound insulation; FPInnovations has found that having absorbing materials with low porosity film on their surface in wall cavities improved the wood wall ASTC significantly (Hu, 2014).
- Total weight per unit area: the greater the weight, the better the sound insulation, especially for low frequency sound;
- Multi-layers with single air space between the layers, such as wall and floor cavities: the larger the airspace, the better the sound insulation; avoid smaller cavities that are less than 12.7 mm thick;
- Sound absorption: sound absorbing material in the airspace or the cavity between layers helps improve sound insulation for assemblies with non-rigidly connected faces; for assemblies with rigidly connected faces, using absorbing materials in the cavity does not improve sound insulation noticeably;
- Contacts between layers: the softer the contacts, the better the sound insulation; contacts include the attachment between gypsum board and CLT, as well as the contact between finish and topping or CLT, and between topping and CLT. Using resilient materials at these contacts is necessary.
- Continuity of CLT elements or topping between two adjacent units: continuities form flanking paths, so they should be avoided.

In general, the stiffness of the wall or the floor has some contradictory effect on sound insulation of wood walls or floor/ceiling assemblies. For “heavy” monolithic assemblies (such as CLT, concrete, etc.), the stiffer the assembly, the better the sound insulation. But this cannot be generalized to other types of assemblies.

However, the general design details that are effective in limiting sound transmission in wood floor systems are: a) breaking of direct structural transmission of sound by separating the CLT floor panels and topping between occupancy areas, b) providing a relatively high floor mass or topping, and c) providing soft materials for floor covering or between the structural assemblies, to attenuate the sound.

Furthermore, in general, to ensure acceptable sound insulation performance, the concept should be to a) contain the sound in one room or occupancy area by planning traffic patterns and penetrations to avoid direct transmission to the adjoining occupant area; and b) provide high mass materials that will either absorb or attenuate the sound in between the occupancy areas.

Designers should be aware that simply addressing the wall and floor construction details might not be sufficient. Openings are very effective at transmitting sound. For example, a well-designed wall might not transmit much sound, but if there are openings such as doors into a common hallway, or penetrations to allow plumbing, electrical, ventilation, etc. to pass from one room or floor to the next, the sound barrier will be rendered ineffective. For the latter, the solutions needed to meet the code requirements to maintain fire separations between suites (e.g., fire stops) may be sufficient to address this concern. However, this must be verified on-site prior completion. The NRC report NRCC-49677, entitled *Best practice guide on fire stops and fire blocks and their impact on sound transmission*, provides additional information regarding possible impacts of fire protection measures on sound insulation. Penetrations and access patterns should be considered and additional methods for isolating these locations should be employed.

9.8.2 Systems Approach for CLT Building Noise Control Design – Need for a Trade-Off

When selecting a solution, a “trade-off and systems approach” is required. The systems approach for sound insulation takes into account material and labor costs, ease of installation, and impact on other performance aspects, such as those related to deformation, fire, thermal insulation and structural integrity.

For example, the best acoustical design requires decoupling and discontinuity of floor and wall components, and of building components. However, this decoupling and discontinuing reduce the stiffness of the floors and walls, and therefore can affect the structural integrity of the entire building.

Another example is the selection of the resilient layer for a floating floor and for a topping to achieve a good impact sound insulation. Manufacturers should be consulted about the compressive resistance of their products to avoid excessive deformation of the sandwich made of finish, membrane, topping, and underlayment. Excessive deformation can lead to a series of problems such as cracks in a concrete topping and ceramic tiles, excessive movement felt by the occupants who walk on the floor, and discomfort for the occupants.

Air quality may also need to be considered when selecting the underlayment and absorbing materials.

The cost effectiveness can be improved by understanding human perception of noise, as described in Section 9.4 and Table 2. Table 2 demonstrates that a change (reduction or increase) in sound level of less than 3 dB will most likely not be perceived by a listener. However, a change of 3 dB or greater will most likely be perceived by most people. Therefore, the design effort and the cost should not emphasize one- or two-point improvements in sound insulation ratings.

9.9 NOISE CONTROL THROUGH INSTALLATION

To optimize the efficiency of the designed sound insulation solutions, a quality-controlled installation and inspection protocol should be implemented, in order to eliminate avoidable flanking paths during construction. Meanwhile, other requirements for installations should also be respected.

9.9.1 Eliminating Avoidable Flanking Paths

There are two types of flanking sound transmission: sound leaking through openings and vibration transfer between coupled surfaces or through continuous structural elements. The basics of flanking control are to seal gaps and openings, decouple surfaces and components, and discontinue structural elements, if this does not affect structural safety, fire safety or serviceability. However, compromise will sometimes be necessary. Table 6 provides a flanking path checklist along with suggested treatments. The list does not exhaust all flanking paths. It includes the most obvious and crucial flanking paths that must be controlled or eliminated, based on current knowledge. If the flanking paths can be controlled, then the design solutions should meet the design goal. It should be noted that this checklist may not be complete. As more knowledge is gained on flanking, this checklist will be updated.

It is important to follow the installation guides of the various products used for floors and walls, such as finish, membrane, dry or wet topping products, underlayment, absorption materials, resilient channels, acoustic hangers, gypsum board, etc. These guides specify the details for flanking control. For example, the length of screws to attach gypsum board to RCs should follow the RC installation guide so that the screws only attach the gypsum board to the RCs, and do not attach the gypsum board to CLT to avoid from forming the rigid connections. There are many such details given in the product specifications and installation guides that should be followed. Another example is that underlayment producers require that the underlayment not be connected to the CLT floor structure using any mechanical fasteners or rigid adhesives, except acoustical glue, because the connectors can form flanking paths.

Table 6 Flanking path checklist and treatment

Flanking Path	Treatment
Leaks around edges of partitions (ASTM E336).	Seal leaks with tape, gaskets, or caulking compound (ASTM E336). Plan the traffic patterns such that doors do not open onto common areas where sound can be easily transmitted around the dividing wall, floor etc.
Cracks at wall/floor junctions.	Caulk joint between gypsum board and floor (NRC, 2002).
Debris between floor and wall sill plates.	Clean floor and caulk sill plate (NRC, 2002).
Leaks through electrical outlets.	Avoid back-to-back outlets by offsetting them 16" (400 mm) or by at least one stud space from side to side (NRC, 2002).
If gypsum board is rigidly attached to wood wall or floor structural elements, the wall and floor could contribute to flanking (NRC, 2002).	Attach gypsum board on resilient channels (NRC, 2002).
If gypsum board is not properly installed on resilient channels, e.g. using long fasteners that push the gypsum board to the wall or floor structural elements, the fasteners may form flanking paths.	Make sure to attach the gypsum board to the resilient channels only, and not to the wall or floor structural elements.
Joint between the flooring or topping perimeter and the surrounding walls, especially if the flooring or topping is floating or not rigidly attached to the subfloor.	Leave a gap around the entire perimeter of the flooring or topping assembly and the walls. Fill it with resilient perimeter isolation board or backer rod and seal the joint with acoustical caulking.
Continuous CLT or topping between two adjacent units.	Discontinue CLT as much as possible. Add floating topping and floating flooring if the continuity is unavoidable; the floating topping or flooring should not continue from one unit to adjacent units.

9.9.2 Respect Other Installation Requirements for Noise Control: Another Trade-Off

Besides following the installation guides of the various products for flanking control, it is also important to respect other requirements for installation of other floor and wall non-structural components, such as finish, membranes, dry or wet topping products, underlayment, absorption materials, RCs, acoustic hangers, gypsum boards, etc.

For example, wood flooring installation requires a vapour barrier for moisture control, and a minimum flooring thickness to control excessive deformation. Ceramic tile installation guides provide details to prevent tile cracking. Membrane and underlayment installation guides provide details to prevent excessive deformation of the flooring or the topping. Gypsum board installation limits maximum spacing of RCs, so that the deformation in gypsum board meets the maximum deformation limit. For flanking control, the wider the RC spacing, the less the flanking sound will be transmitted. But on the other hand, the wider the RC spacing, the larger the deformation of the gypsum board will be.

The aging characteristic of the floor underlayment, membrane and finish is also important to understand. For example, a steel truss floor with a concrete deck and a gypsum board ceiling with new carpet had an AIIC rating of 81, as measured in 1994 in a room in a new building built that year. The on-site impact sound insulation test was repeated on the same floor in the same room in 2012. It was found that the measured AIIC had decreased to 68 after 18 years of service, due to the carpet wearing out, which greatly affected the impact sound insulation performance (FPInnovations, 2012). Recycled rubber and plastic foam underlayment may also age, as well as most glued products. The aging could reduce the sound insulation performance after several years of service.

9.10 EXAMPLES OF WORKABLE AND COST-EFFECTIVE SOLUTIONS

This Section provides practical and cost-effective design examples of CLT walls and floor/ceiling assemblies; the airborne and impact sound insulation ratings were measured in various laboratories and field in Canada. The CLT wall and floor/ceiling assemblies used CLT and other materials available in North American markets. They were built using common North American construction practices.

9.10.1 Solutions to Achieve STC ≥ 50 and IIC ≥ 55 with CLT Floor/Ceiling and Wall Assemblies

NRC tested many CLT walls and floor/ceiling assemblies with various CLT thicknesses and construction details in its laboratory chambers for STC and IIC. The 78-mm thick and 175-mm thick CLT were used for the wall assemblies. The wall assemblies included single-leaf and double-leaf walls with various degrees of couplings between gypsum board and the CLT. The floor/ceiling assemblies were made of the 131-mm, 175-mm and 245-mm thick CLT. The floor/ceiling assemblies were constructed with various toppings floating on various resilient layers, and with apparent or a dropped ceiling. The measured STC and IIC ratings of the CLT assemblies can be found in NRC reports (Sabourin, 2015; Schoenwald et al., 2014).

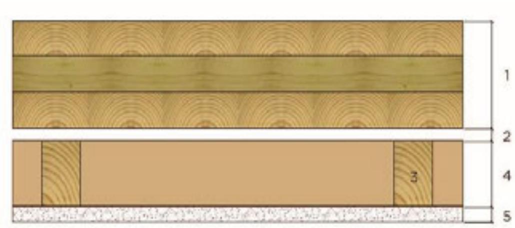
AcoustiTECH (2018) published its measured IIC ratings of 131-mm thick CLT floor assemblies with apparent ceiling, and various finishes and toppings floating on various resilient layers.

Pliteq Engineering Company (2016) published its STC and IIC ratings measured on 175-mm thick CLT floors with a 100-mm thick concrete topping and various Pliteq's rubber Mat, GenieMat™ underlayment and finish with apparent ceiling, and on the CLT floor with various dropped ceiling details using Pliteq's clips, GenieClip™. More details can be found in the company's web: http://pliteq.com.

9.10.2 Solutions to Achieve ASTC ≥ 45 with CLT Walls Tested in CLT Buildings

Tables 7 and 8 provide a description of CLT wall assemblies with their ASTC ratings measured in the field by FPInnovations (Hu, 2014; Ramzi, 2015 and 2017; Omeranovic, 2015; Hu and Cuerrier-Auclair, 2018a-c), in CLT buildings.

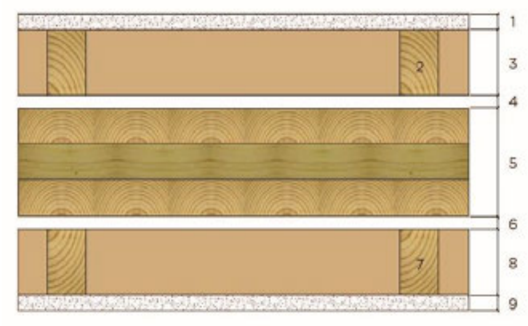
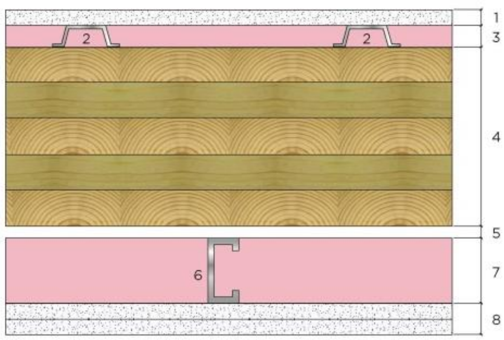
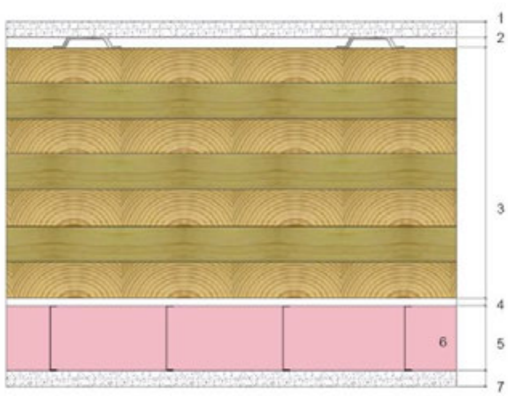
Table 7 Solutions to achieve ASTC ≥ 45 with field CLT walls tested in CLT buildings

Top view of the wall cross-section	Wall details, from one side to the other side	ASTC
	1. 15.9-mm thick Type X gypsum board 2. Type RC-1 (one leg) 25-gauge resilient channels at 600 mm O.C.(1) 3. 184-mm thick CLT 4. Type RC-1 (one leg) 25-gauge resilient channels at 600 mm O.C.(1) 5. 15.9-mm thick Type X gypsum board	46
	1. 15.9-mm thick Type X gypsum board 2. Type RC-1 (one leg) 25-gauge resilient channels at 600 mm O.C.(1) 3. 78-mm thick CLT 4. 25-mm thick air gap filled with rock fibre insulation (Roxul-AFB®) 5. 78-mm thick CLT 6. Type RC-1 (one leg) 25-gauge resilient channels at 600 mm O.C.(1) 7. 15.9-mm thick Type X gypsum board	47
	1. 3-layer 105-mm thick CLT 2. 12.7-mm thick air gap 3. 38-mm x 64-mm wood studs at 400 mm O.C. 4. 64-mm thick rock fibre insulation (Roxul-AFB®) in the wall cavity 5. 15.9-mm thick Type X gypsum board	47

(Continued in Table 8)

Note:
(1) Caution should be taken when selecting RCs to attach gypsum board; quality of RCs varies from product to product, and the variation in RCs affects their effectiveness in improving sound insulation. More data is needed to quantify the effects of various RC products on wall sound insulation.

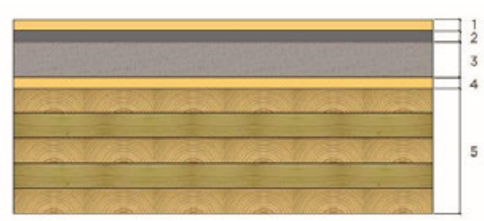
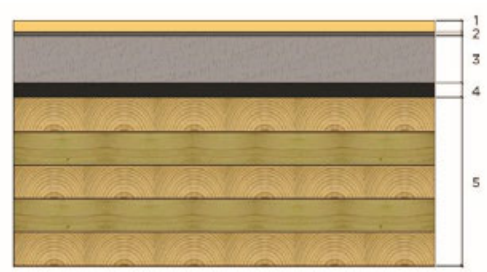
Table 8 **Solutions to achieve ASTC ≥ 45 with field CLT walls tested in CLT buildings (Continuing from Table 7)**

Top view of the wall cross-section	Wall details, from one side to the other side	ASTC
	1. 15.9-mm thick Type X gypsum board 2. 38-mm x 64-mm wood studs at 400 mm O.C. 3. 64-mm thick rock fibre insulation (Roxul-AFB®) in wall cavity 4. 12.7-mm thick air gap 5. 3-ply 105-mm thick CLT 6. 12.7-mm thick air gap 7. 38-mm x 64-mm wood studs at 400 mm O.C. 8. 64-mm thick rock fibre insulation (Roxul - AFB®) in wall cavity 9. 15.9-mm thick Type X gypsum board	54
	1. 15.9-mm thick Type X gypsum board 2. 22-mm deep steel channel 3. 22-mm thick glass fibre insulation in cavity 4. 175-mm thick CLT 5. 12.7-mm thick air gap 6. 64-mm deep steel studs (25-gauge) at 400 mm O.C. 7. 64-mm thick glass fibre insulation in wall cavity 8. 2x15.9-mm thick Type X gypsum board	58
	1. 2x15.9-mm thick Type X gypsum board 2. 22-mm deep W-14 hat steel channels at 400 mm O.C. 3. 245-mm thick CLT 4. 19-mm thick air gap 5. 90–mm deep light gauge steel studs at 400 mm O.C. 6. 90-mm thick glass fibre insulation in cavity 7. 2x15.9-mm thick Type X gypsum board	65

9.10.3 Solutions to Achieve AIIC ≥ 45 and ASTC ≥ 45 with CLT Floor/Ceiling Assemblies Tested in Buildings

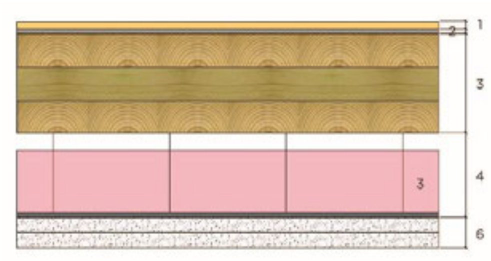
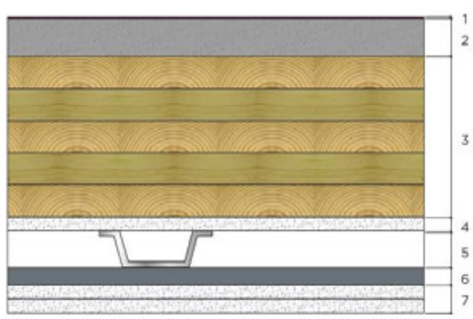
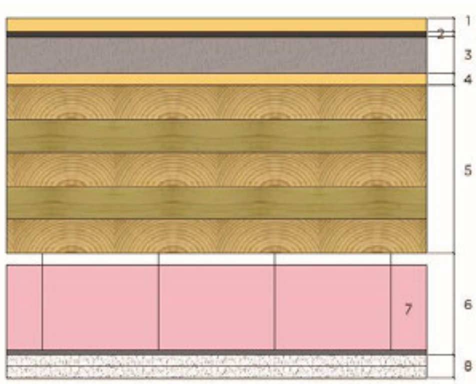
This Section provides design examples for CLT floor/ceiling assemblies. Their sound insulation performance was measured in actual buildings by FPInnovations (Hu, 2014; Ramzi, 2015 and 2017; Omeranovic, 2015; Hu and Cuerrier-Auclair, 2018a-c).

Table 9 Solutions to achieve ASTC ≥ 45 and AIIC ≥ 45 with field CLT floors with apparent ceiling tested in buildings

Side view of the floor cross-section	Floor details, from top to bottom	AIIC	ASTC
	1. 12-mm thick wood flooring 2. Resilient layer 3. 38-mm thick lightweight concrete (Ecomix) 4. 12.7-mm thick wood fibreboard (BP board) 5. 131-mm thick CLT	44	45
	1. 8-mm thick wood flooring 2. 5.5-mm thick felt (ThermaSonHD™) 3. 100-mm thick CLT 4. 38-mm x 38-mm wood furring at 400 mm O.C. 5. Sand 6. 100-mm thick CLT	46	51
	1. 12-mm thick wood flooring 2. 3.5-mm thick rubber mat (InsonoFloor) 3. 50-mm thick lightweight concrete (Ecomix) 4. 15-mm thick rubber mat (InsonoMat) 5. 175-mm thick CLT	50	53

(Continued in Table 10)

Table 10 **Solutions to achieve ASTC ≥ 45 and AIIC ≥ 45 with field CLT floor/ceiling assemblies tested in buildings (Continuing from Table 9)**

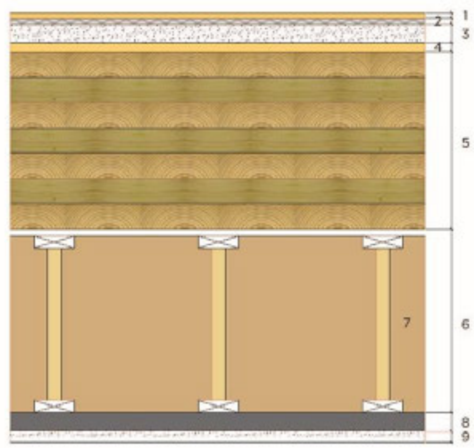
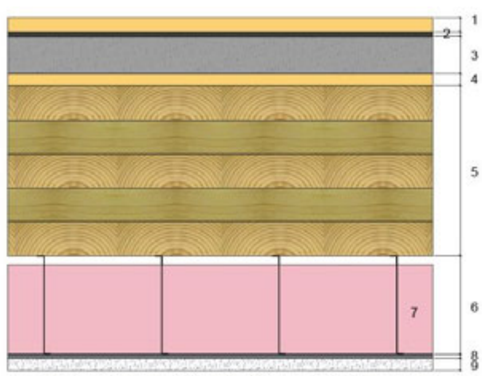
Side view of the floor cross-section	Floor details, from top to bottom	AIIC	ASTC
	1. 4-mm thick floating vinyl board (Armstrong Luxe Plank) 2. 1.5-mm thick soundproofing membrane (Shnier Quietblock™ non-adhered) 3. 105-mm thick CLT 4. Dropped ceiling on metal grillage (“drywall grid system”) 250 mm below CLT surface 5. 65-mm thick glass fibre 6. 2x15.9-mm thick Type X gypsum board	50	54
	1. 3-mm thick carpet (Flex-Aire® Modular Tandus carpet with 4-mm thick Flex-Aire® on back), mass of 3.5kg/m^2 2. 40-mm thick normal-weight concrete 3. 169-mm thick CLT 4. 15.9-mm thick Type X gypsum board 5. 38-mm deep steel hat track 6. 19-mm deep resilient channels at 400 mm O.C.(1) 7. 2x15.9-mm thick Type X gypsum board	53	55
	1. 15-mm thick engineered wood flooring 2. 2.4-mm thick resilient layer (AcoustiTECH VP) 3. 38-mm thick normal-weight concrete 4. 12.7-mm thick wood fibreboard (BP board) connected to CLT 5. 175-mm thick CLT 6. Dropped ceiling on metal grillage (“drywall grid system”) 100 mm below CLT surface 7. 89-mm thick glass fibre insulation 8. 2x12.7-mm thick Type X gypsum board	57	55

(Continued in Table 11)

Note:

(1) Caution should be taken when selecting RCs to attach gypsum board; quality of RCs varies from product to product, and the variation in RCs affects their effectiveness in improving sound insulation. More data is needed to quantify the effects of various RC products on wall sound insulation.

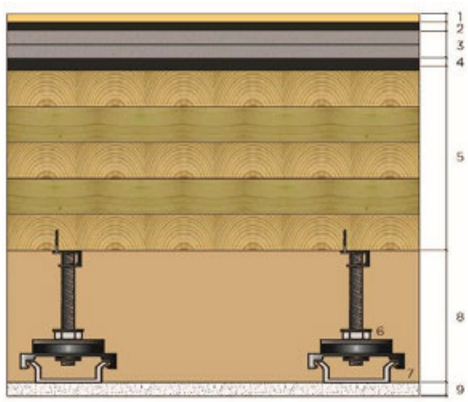
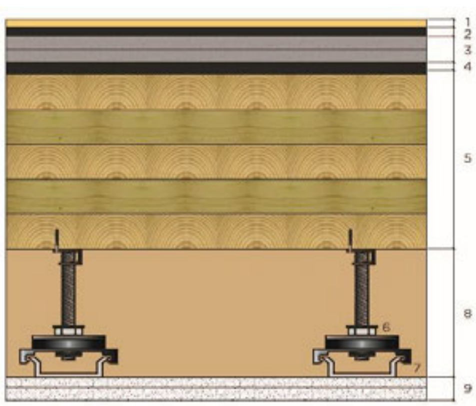
Table 11 Solutions to achieve ASTC ≥ 45 and AIIC ≥ 45 with field CLT floor/ceiling assemblies tested in buildings (Continuing from Table 10)

Side view of the floor cross-section	Floor details, from top to bottom	AIIC	ASTC
	1. Ceramic tile 2. Ceramic glue 3. 20-mm thick Fermacell (gypsum panels) 4. 10-mm thick rock fibre insulation (Roxul-AFB®) 5. 208-mm thick CLT 6. 200-mm deep I-joists at 600 mm O.C., not attached to CLT 7. 200-mm thick rock fibre insulation (Roxul-AFB®) 8. 19-mm x 64-mm wood furring at 400 mm O.C. 9. 15.9-mm thick Type X gypsum board	58	55
	Same as above, except for replacing layer-1 with 12-mm thick laminated flooring and layer-2 with vapour barrier	59	56
	1. 12-mm thick wood floating flooring 2. Acoustic membrane 3. 38-mm thick normal-weight concrete 4. 12.7-mm thick wood fibreboard (BP board) 5. 175-mm thick CLT 6. 90-mm deep Z shape steel channels at 600 mm O.C. 7. Glass fibre insulation in cavity 8. 22-mm deep W-14 hat steel channels at 400 mm O.C. 9. 15.9-mm thick Type X gypsum board	54	58

9.10.4 Solutions to Achieve AIIC ≥ 45 and ASTC ≥ 45 with CLT Floor/Ceiling Assemblies Tested in an FPInnovations' Mock-Up

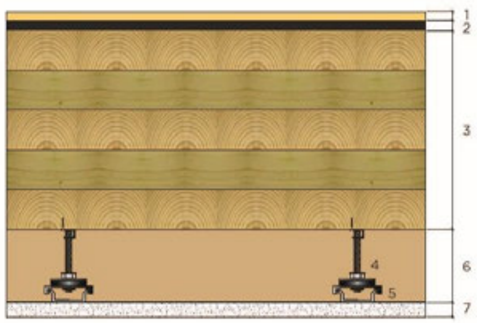
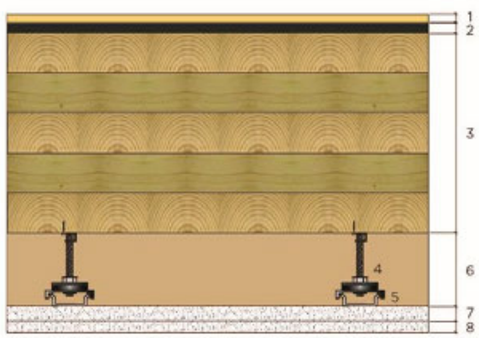
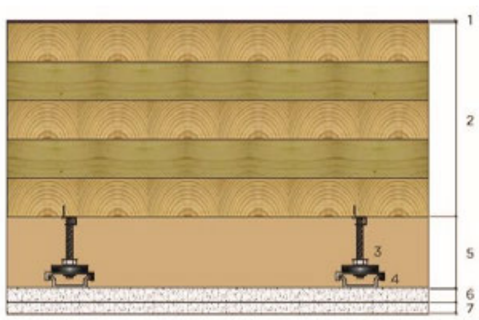
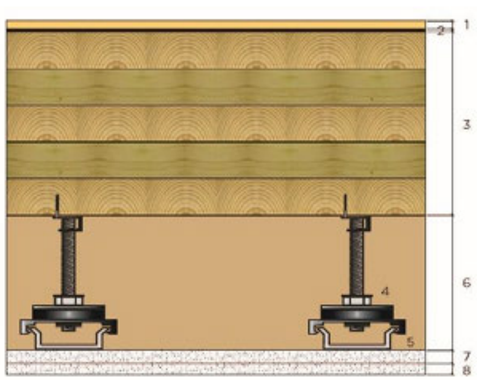
This Section provides design examples for CLT floor/ceiling assemblies. Their sound insulation performance was measured in FPInnovations' mock-up of a two-story wood building, by FPInnovations (Hu, 2014; Ramzi, 2017).

Table 12 Solutions to achieve AIIC ≥ 45 and ASTC ≥ 45 with CLT floor/ceiling assemblies tested in FPInnovations' mock-up

Side view of the floor cross-section	Description of the CLT floor specimen, from top to bottom	AIIC	ASTC
	1. 10-mm thick laminated flooring 2. 3.5-mm thick rubber membrane (InsonoFloor) 3. 2x16-mm thick Fiberock® 4. 15-mm thick rubber mat (InsonoMat) 5. 175-mm thick CLT	46	47
	1. 10-mm thick laminated flooring 2. 3.5-mm thick rubber membrane (InsonoFloor) 3. 2x16-mm thick Fiberock® 4. 15 mm thick rubber mat (InsonoMat) 5. 175-mm thick CLT 6. 200-mm high sound isolation clips (RSIC-1ADM® Multi-Clip) 7. Metal hat channels 8. Rock fibre insulation (Roxul-AFB®) 9. 15.9-mm thick Type X gypsum board	61	59
	1. 10-mm thick laminated flooring 2. 3.5-mm thick rubber membrane (InsonoFloor) 3. 2x16-mm thick Fiberock® 4. 15-mm thick rubber mat (InsonoMat) 5. 175-mm thick CLT 6. 200-mm high sound isolation clips (RSIC-1ADM® Multi-Clip) 7. Metal hat channels 8. Rock fibre insulation (Roxul-AFB®) 9. 15.9-mm thick and 12.7-mm thick Type X gypsum board for base and face layer, respectively	60	58

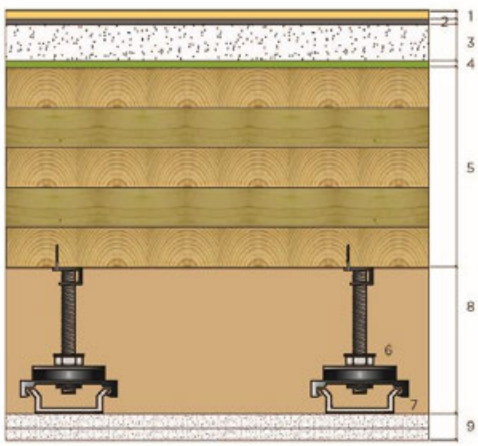
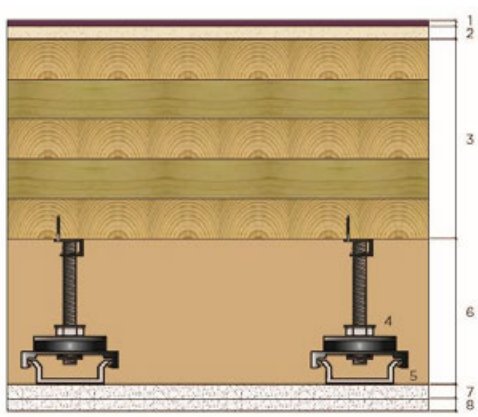
(Continued in Table 13)

Table 13 Solutions to achieve AIIC ≥ 45 and ASTC ≥ 45 with CLT floor/ceiling assemblies tested in FPInnovations' mock-up (Continuing from Table 12)

Side view of the floor cross-section	Description of the CLT floor specimen, from top to bottom	AIIC	ASTC
	1. 10-mm thick laminated flooring 2. 3.5-mm thick rubber membrane (InsonoFloor) 3. 175-mm thick CLT 4. 100-mm high sound isolation clips (RSIC-1ADM® Multi-Clip) 5. Metal hat channels 6. Rock fibre insulation (Roxul-AFB®) 7. 15.9-mm thick Type X gypsum board	52	53
	1. 10-mm thick laminated flooring 2. 3.5-mm thick rubber membrane (InsonoFloor) 3. 175-mm thick CLT 4. 100-mm high sound isolation clips (RSIC-1ADM® Multi-Clip) 5. Metal hat channels 6. Rock fibre insulation (Roxul-AFB®) 7. 15.9-mm thick Type X gypsum board 8. 12.7-mm thick Type X gypsum board	53	53
	1. 3-mm thick Flex-Aire® Modular Tandus carpet with 4-mm thick Flex-Aire® on back 2. 175-mm thick CLT 3. 100-mm high sound isolation clips (RSIC-1ADM® Multi-Clip) 4. Metal hat channels 5. Rock fibre insulation (Roxul-AFB®) 6. 15.9-mm thick Type X gypsum board 7. 12.7-mm thick Type X gypsum board	55	54
	1. 10-mm thick laminated flooring 2. 3.5-mm thick rubber membrane (InsonoFloor) 3. 175-mm thick CLT 4. 200-mm high sound isolation clips (RSIC-1ADM® Multi-Clip) 5. Metal hat channels 6. Rock fibre insulation (Roxul-AFB®) 7. 15.9-mm thick Type X gypsum board 8. 12.7-mm thick Type X gypsum board	56	54

(Continued in Table 14)

Table 14 Solutions to achieve AIIC ≥ 45 and ASTC ≥ 45 with CLT floor/ceiling assemblies tested in FPInnovations' mock-up (Continuing from Table 13)

Side view of the floor cross-section	Description of the CLT floor specimen, from top to bottom	AIIC	ASTC
	1. 10-mm thick laminated flooring 2. 5.5-mm thick felt (ThermaSonHD™) 3. 38-mm thick normal weight concrete 4. 12.7-mm thick wood fibreboard (BP) 5. 175-mm thick CLT 6. 100-mm high sound isolation clips (RSIC-1ADM® Multi-Clip) 7. Metal hat channels 8. Rock fibre insulation (Roxul-AFB®) 9. 15.9-mm thick and 12.7-mm thick Type X gypsum board for base and face layer, respectively	57	60
	1. 10-mm thick laminated flooring 2. 3.5-mm thick rubber membrane (InsonoFloor) 3. 25-mm thick gypsum board (PanoMag®) 4. 8 mm thick plastic underlayment (Insul-R®) 5. 175-mm thick CLT 6. 100-mm high sound isolation clips (RSIC-1ADM® Multi-Clip) 7. Metal hat channels 8. Rock fibre insulation (Roxul-AFB®) 9. 15.9-mm thick and 12.7-mm thick Type X gypsum board for base and face layer, respectively	60	58
	1. 3-mm thick Flex-Aire® Modular Tandus carpet with 4-mm Flex-Aire® on back 2. 14-mm thick sand (GreenBero) 3. 175-mm thick CLT 4. 100-mm high sound isolation clips (RSIC-1ADM® Multi-Clip) 5. Metal hat channels 6. Rock fibre insulation (Roxul-AFB®) 7. 15.9-mm thick Type X gypsum board 8. 12.7-mm thick Type X gypsum board	61	56

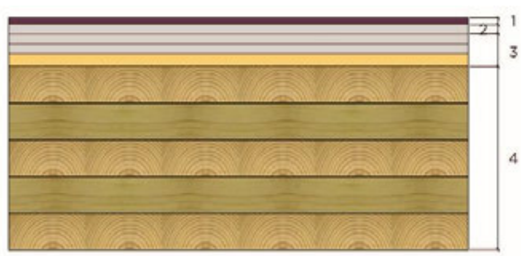
(Continued in Table 15)

Table 15 Solutions to achieve AIIC ≥ 45 and ASTC ≥ 45 with CLT floor/ceiling assemblies tested in FPInnovations' mock-up (Continuing from Table 14)

Side view of the floor cross-section	Description of the CLT floor specimen, from top to bottom	AIIC	ASTC
	1. 10-mm thick laminated flooring 2. 3.5-mm thick rubber mat (InsonoFloor) 3. 19-mm thick Maxxon Gyp-Crete® 4. 5-mm thick Maxxon nylon mesh 5. 175-mm thick CLT 6. 13-mm deep resilient channels at 600 mm O.C.[(1)] 7. 19-mm thick Type X acoustical panels (QuietRock®) 8. Dropped ceiling on metal grillage ("Armstrong drywall grid system") 380 mm below the QuietRock® surface 9. 76-mm thick rock fibre insulation (Roxul-AFB®) 10. 15.9-mm thick Type X gypsum board	53	54
	1. 10-mm thick laminated flooring 2. 2.4-mm thick felt (AcoustiTECH Premium™) 3. 2x18-mm thick OSB 4. 38-mm x 89-mm lumber sleepers at 1.2 m O.C., connected to the OSB 5. 10-mm thick rubber pad of 38-mm x 38-mm under the lumber sleepers at 1.2 m O.C. along the length of each sleeper 6. Rock fibre insulation (Roxul-AFB®) 7. 175-mm thick CLT	57	55

9.10.5 Solutions to Achieve AIIC ≥ 45 with CLT Floor/Ceiling Assemblies Tested in FPInnovations' Mock-Up, Using a 1.2-m by 1.2-m Topping Patch

Table 16 **Solutions to achieve AIIC ≥ 45 with CLT floors with apparent ceiling tested in FPInnovations' mock-up, using a 1.2-m by 1.2-m topping patch**

Side view of the floor cross-section	Description of the 1.2-m by 1.2-m topping patch, from top to bottom	AIIC
	1. 10-mm thick laminated flooring 2. 5.5-mm thick felt (ThermaSonHD™) 3. 10-mm thick Fermacell 4. 2x10-mm thick Fermacell with 12.7-mm thick wood fibreboard on back 5. 175-mm thick CLT	45
	1. 6-mm thick carpet 2. 2x10-mm thick Fermacell with 12.7-mm thick wood fibreboard on back 3. 175-mm thick CLT	46
	1. 10-mm thick laminated flooring 2. 3.5-mm thick rubber mat (InsonoFloor) 3. 2x10-mm thick Fermacell with 12.7-mm thick wood fibreboard on back 4. 175-mm thick CLT	46
	1. 6-mm thick carpet 2. 10-mm thick Fermacell 3. 2x10-mm thick Fermacell with 12.7-mm thick wood fibreboard on back 4. 175-mm thick CLT	48

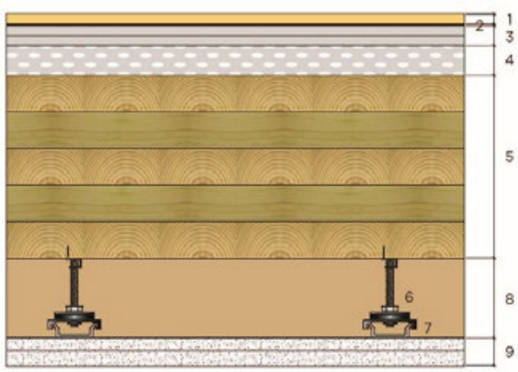
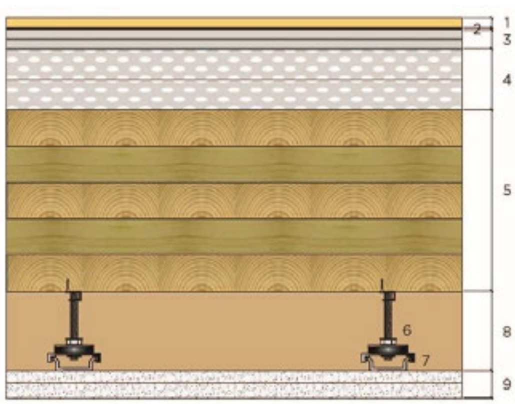
(Continued in Table 17)

Table 17 **Solutions to achieve AIIC ≥ 45 CLT floors with apparent ceiling tested in FPInnovations' mock-up, using a 1.2-m by 1.2-m topping patch (continuing from Table 16)**

<table>
<tr><th>Side view of the floor cross-section</th><th>Description of the 1.2-m by 1.2-m topping patch, from top to bottom</th><th>AIIC</th></tr>
<tr><td rowspan="3">
</td><td>1. 10-mm thick laminated flooring
2. 3.5-mm thick rubber mat (InsonoFloor)
3. 2x16-mm thick Fiberock®
4. 15-mm thick rubber mat (InsonoMat)
5. 175-mm thick CLT</td><td>45</td></tr>
<tr><td>1. 10-mm thick laminated flooring
2. 2.4-mm thick felt (AcoustiTECH Premium™)
3. 2x16-mm thick Fiberock®
4. 12.7-mm thick wood fibreboard
5. 175-mm thick CLT</td><td>48</td></tr>
<tr><td>1. 10-mm thick laminated flooring
2. 5.5-mm thick felt (ThermaSonHD™)
3. 2x13-mm thick cement fibreboard (PermaBase®) or 2-layer 16-mm thick Fiberock®
4. 18-mm thick felt (S125)
5. 175-mm thick CLT</td><td>49</td></tr>
<tr><td>
</td><td>1. 10-mm thick laminated flooring
2. 3.5-mm thick rubber mat (InsonoFloor)
3. 2x10-mm thick Fermacell with 12.7-mm thick wood fibreboard on back
4. 15-mm thick rubber mat (InsonoMat)
5. 175-mm thick CLT</td><td>51</td></tr>
</table>

(Continued in Table18)

Table 18 Solutions to achieve AIIC ≥ 45 with CLT floor/ceiling assemblies tested in FPInnovations' mock-up, using 1.2-m by 1.2-m topping patch ceiling assemblies (continuing from Table 17)

Side view of the floor cross-section	Description of the 1.2-m by 1.2-m topping patch, from top to bottom	AIIC
	1. 10-mm thick laminated flooring 2. 3.5-mm thick rubber mat (InsonoFloor) 3. 38 mm thick normal-weight concrete 4. 15-mm thick rubber mat (InsonoMat) 5. 175-mm thick CLT	48
	1. 10-mm thick laminated flooring 2. 5.5-mm thick felt (ThermaSonHD™) 3. 38 mm thick normal-weight concrete 4. 18-mm thick felt (S125) 5. 175-mm thick CLT	52
	1. 10-mm thick laminated flooring 2. 2.4-mm thick felt (AcoustiTECH Premium™) 3. 2x10-mm thick Fermacell 4. 30-mm thick Fermacell honeycomb filled with sand 5. 175-mm thick CLT 6. 100-mm high sound isolation clips (RSIC-1ADM® Multi-Clip) 7. Metal hat channels 8. Rock fibre insulation (Roxul-AFB®) 9. 15.9-mm thick and 12.7-mm thick Type X gypsum board for base and face layer, respectively	49
	1. 10-mm thick laminated flooring 2. 2.4-mm thick felt (AcoustiTECH Premium™) 3. 2x10-mm thick Fermacell 4. 2x30-mm thick Fermacell honeycomb filled with sand 5. 175-mm thick CLT 6. 100-mm high sound isolation clips (RSIC-1ADM® Multi-Clip) 7. Metal hat channels 8. Rock fibre insulation (Roxul-AFB®) 9. 15.9-mm thick and 12.7-mm thick Type X gypsum board for base and face layer, respectively	54

9.11 SOUND INSULATION OF WOOD ELEVATOR SHAFTS

The 2015 NBC requires that construction separating a dwelling unit from an elevator hoist way or a refuse chute shall have a STC of no less than 55. In our experience, CLT wall designs with a minimum STC of 60 or ASTC of 55 provide a good sound insulation for wood elevator shafts. Decoupling elevator shafts from the building structures is also a solution.

9.12 SOUND INSULATION OF WOOD STAIRWELLS

Occupants in the units next to stairwells have raised concerns about footstep noise transmission through the walls of the stairwell, when people are walking on the stairs. This is a common concern that occurs not only in wood multi-family buildings, but also in concrete and steel buildings.

The root of the problem is the poor impact sound insulation of the stairs and the connections between the stairwell and the supporting walls. Therefore, highly sound insulated walls for stairwells with a minimum rating of 60 for STC or 55 for ASTC are a solution to solve this problem; decoupling the stairs from the walls is also a solution. It might be best to have details to float the connection between the stairwell and the wall, so there will be no direct rigid sound channel between them. In addition, use of sound insulated stairs is another solution to reduce the impact sound noise generated by the footstep forces on the stairs when occupants walk on the stairs.

9.13 SOUND INSULATION OF STEPPED-STORY WOOD BUILDINGS

Figure 6 shows an example of a stepped-story building, in which the terraces of some units are above the roofs of the units below. Such a configuration challenges the sound insulation design of the units below, in part because the NBC contains no requirements for sound insulation of dwelling units from noise generated outside the building.

Figure 6 Stepped-story building

Considering that extreme activities such as jumping, dancing, etc., may occur on the terrace floors, the design for the impact and airborne sound insulation of the terrace floors and the airborne sound insulation of the roofs of the units below the terrace should have ratings of at least 55 for STC and IIC, for occupant satisfaction. Low frequency footstep noise should be seriously considered for the terrace floor acoustic design. It is also recommended to decouple the terrace floors from the roofs of the units under the terrace. Controlling all the flanking paths around the wall-floor and wall-roof junctions, etc., will ensure that the target STC and IIC may be achieved. Development of design examples for terrace floors decoupled from the roofs of the units below the terrace is underway.

9.14 FINAL REMARKS

The following points should be considered:

- At the design stage, when selecting a solution that takes into account sound insulation, a trade-off is often required between material and labor costs, ease of installation, and impact on other performance aspects, such as those related to deformation, fire, thermal insulation, and structural integrity.
- To successfully apply the proposed design and construction solutions listed in this Chapter to building projects, there must be onsite quality control pertaining to flanking. The solutions cannot guarantee the same ratings if best practices are not followed.
- Best practices for noise control consist of three components: a) flanking sound transmission control; b) field measurement of ASTC and AIIC ratings of floors and walls after the project is completed, and the application of corrective measures if needed; c) subjective evaluation by promoters, developers, architects, engineers, and producers, using the procedure described in this Chapter. If these evaluators do not like the sound insulation, they can hardly expect the occupants to be satisfied. Therefore, the remedy should be implemented before the occupants move in.
- Acquiring experience in the correlation of measured ASTC and AIIC ratings to subjective evaluation results will be very useful to developers when selecting ASTC and AIIC ratings as design goals for the wall and floor/ceiling assemblies used in future projects. In addition, it will be helpful to use the correct wording to describe the sound insulation performance of the building, in any advertisement. Thus, a “quiet” building should most likely have ratings above 60, a “sound resisting” building should most likely have ratings between 50-60, and finally, a building characterized as “meeting codes” will most likely have ratings of 45-47, depending on the codes in effect.

This Chapter is a compilation of the current knowledge, data, and experience of noise management in CLT buildings. The description of the design examples to achieve satisfactory sound transmission ratings through wall or floor/ceiling assemblies given in this Chapter have listed the trade names of finish, membrane, topping and underlayment products, due to the lack of generic assessment and classification of these products in the current standards. This was done only to provide the details of reasonably good and functional assemblies, without any intention to promote specific products or manufacturers. The Chapter will evolve in the future with the development of such generic standards and criteria for product sound insulation properties.

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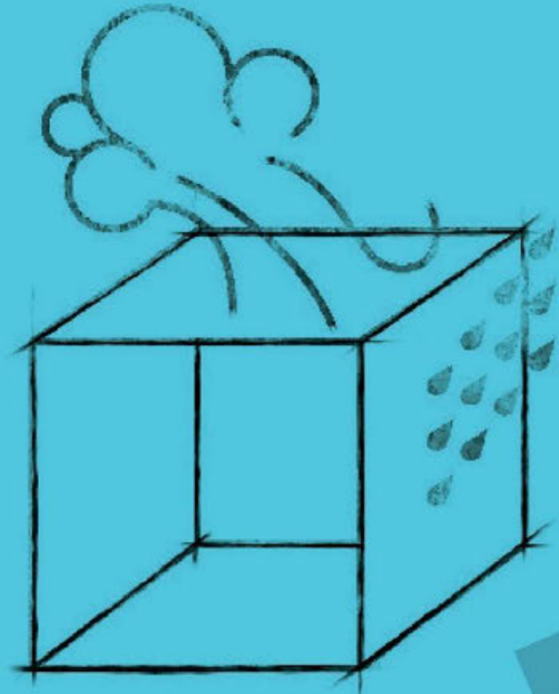

CHAPTER 10

Building enclosure design of cross-laminated timber construction

Lead Authors
Graham Finch
Jieying Wang

Reviewers
Robert Jonkman
Michael Lacasse
Mark Lawton

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ABSTRACT

When the first edition of the Canadian CLT Handbook was being written between 2009 and 2011, there were very few built examples of CLT construction in Canada and the United States. Research had started and those seriously interested in CLT were looking at the early examples in Europe. The 2013 U.S. Edition of the CLT Handbook provided additional information for the climates in the United States. Fast forward to late 2019 and CLT has now been used in the construction of hundreds of small to large buildings in Canada and the United States. This includes the 18-storey UBC Tallwood House (Brock Commons) in Vancouver, British Columbia where CLT was used within the floor system, and the13-storey Origine building in Québec City where CLT was used both in the floor and the wall systems.

The building enclosure (also known as the building envelope) system—the focus of this Chapter—is the component of mass timber buildings which protects the structure from moisture and environmental elements, separates the indoors from outdoors, and is a key passive design element within energy-efficient and sustainable buildings. The building enclosure may incorporate CLT structural elements or be placed in a position outside of a structure. The proper design and long-term performance of the building enclosure is therefore critical to the sustainability of mass timber buildings.

This Chapter provides building science guidance on best practices for the design of building enclosures incorporating CLT panels. This guidance is based on a combination of research, testing and experience with the construction of buildings with CLT building enclosure systems. A brief primer on relevant building code requirements and the building science of heat, vapour, air, and moisture control for CLT walls and roofs is followed by sections on CLT wall and roof designs and detailing. The final section covers strategies and solutions for addressing construction moisture, service moisture, and preservative treatment to ensure long-term durability.

10.1 INTRODUCTION

This Chapter covers best practices for cross-laminated timber (CLT) building enclosure assembly design and construction, for Canadian climates. These practices are based on research, laboratory and field testing, and years of project experience with CLT panels in wall, roof, and floor applications across Canada and the United States. The purpose of this Chapter is to provide guidance for achieving long-term durability and energy efficiency, while maintaining the desirable aesthetic and functional aspects of the CLT integrated into the building enclosure. Figure 1 provides an example where CLT panels were incorporated into the floor and roof assemblies of the first tall mass timber building in North America, the Wood Innovation and Design Centre in Prince George, British Columbia.

Figure 1 CLT beams within roof and floor assemblies at the Wood Innovation and Design Centre in Prince George, BC by Michael Green Architecture (courtesy of Ema Peter)

The building enclosure is a system of materials, components, and assemblies that physically separates conditioned (interior) from unconditioned (exterior) spaces (Figure 2). It controls the flow of heat, air, and moisture in the form of both water vapour and liquid water, in addition to providing other functions, such as fire and acoustic separation, as described throughout this Handbook. With respect to mass timber and specifically CLT buildings, the CLT panels used for the building's structure may either be incorporated as part of the building enclosure or be independent. It is also common for CLT structures to utilize other load- or non-load-bearing wall and roof systems (i.e., steel stud or wood-framed walls/roofs, aluminum and glass curtain wall, or even precast concrete panels) (Figure 3) (Finch and Hubbs, 2017). The focus of this Chapter is on the building science for enclosures that incorporate CLT panels. This typically includes using CLT in load-bearing platform-type and balloon-type exterior walls or non-load bearing hung panels (Figure 4). CLT floors, although they are often entirely indoors and do not separate dissimilar spaces, are also discussed, because they form part of the enclosure during the construction process and have similar considerations as the roof with respect to moisture protection.

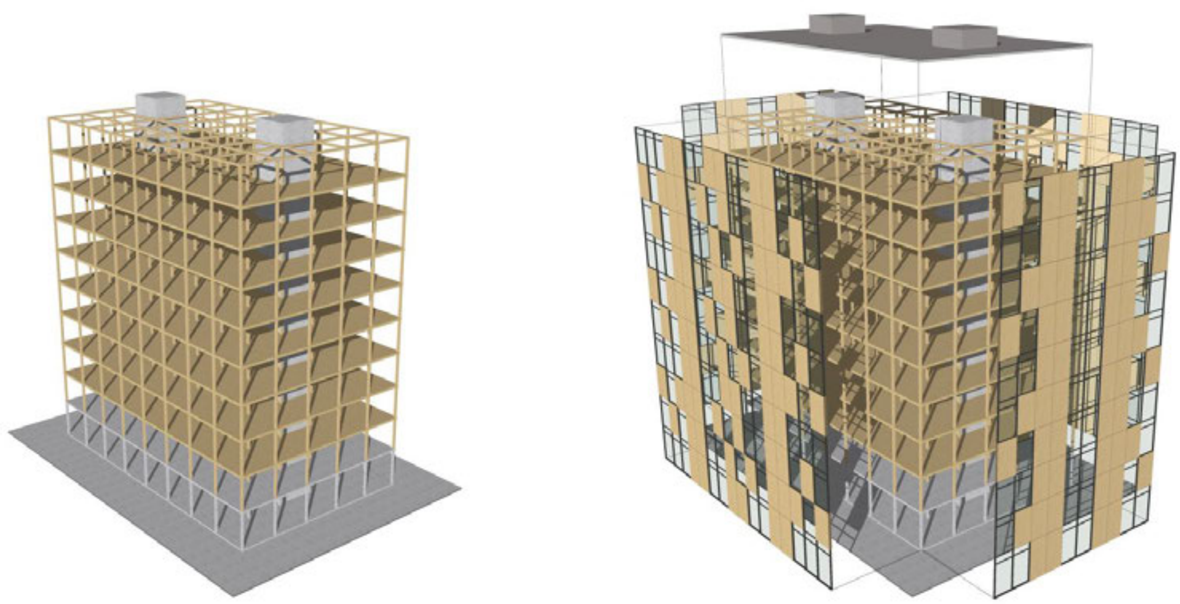
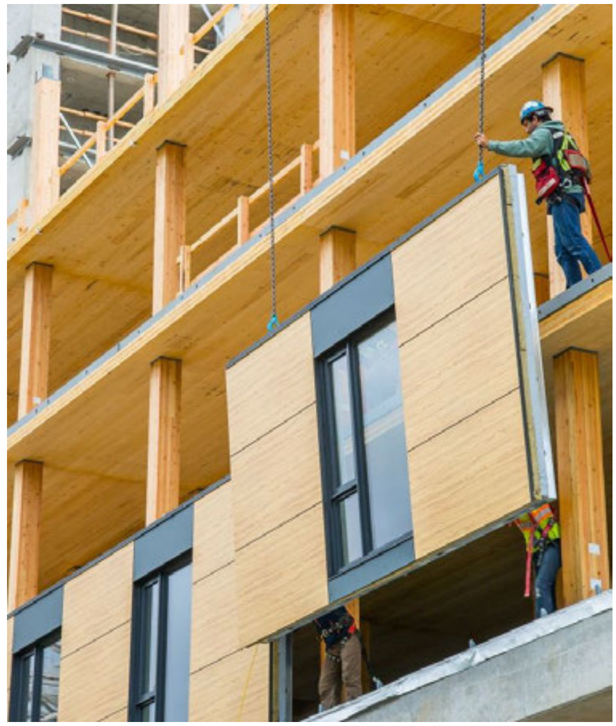


Figure 2 CLT mass timber structure shown without the building enclosure (left) and with the building enclosure (right)

Section 10.2 of this Chapter provides a brief overview of the energy and building code requirements for CLT building enclosures. Section 10.3 covers best practices for insulating CLT walls and roofs, incorporating air barrier systems within CLT assemblies, designing for vapour control and drying ability, and providing details for the control of rainwater and exterior moisture. Section 10.4 provides design examples of CLT walls and roofs, and special considerations for detailing window installations within CLT panels, as well as the transitions from below-grade to above-grade CLT interfaces. Section 10.5 provides guidance on managing construction moisture for CLT walls, roofs, and floor systems through the use of pre-applied protection and on-site measures.

CLT was integrated into the exterior wall and flat roof building enclosure assemblies at the 3-storey Ronald McDonald House, Vancouver

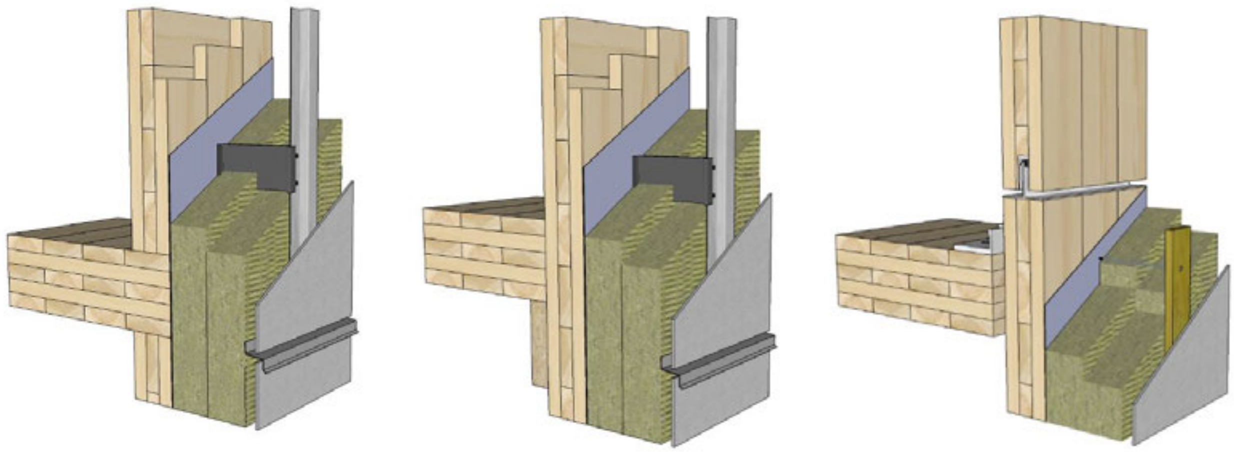


CLT was integrated into the floor structure independent of the prefabricated steel stud enclosure of the 18-storey UBC Tallwood House, Vancouver

Large CLT exterior wall panels installed in a balloon framed manner at the Origine project in Québec (photo courtesy: Stéphane Groleau)

CLT exterior wall panels installed in a platform frame manner at the Cité Verte project (photo courtesy: Stéphane Groleau)

Figure 3 Photos showing integration of CLT panels into buildings



Load-bearing platform-type CLT wall — Non-load-bearing balloon framed-type CLT wall — Non-load-bearing hung curtainwall-type CTL wall

Figure 4 Examples showing use of load or non-load bearing CLT panels in exterior walls of CLT buildings. Consideration for the structural design and wood-movement are different for each of these three systems and factored in the design of the building enclosure

10.2 ENERGY AND BUILDING CODE REQUIREMENTS FOR CLT

This Handbook focuses on large buildings. The design of building enclosures of large buildings is primarily governed by Part 5 of Division B of the National Building Code of Canada (NRC, 2015a). Part 5 provides general requirements about design for environmental loads and control of heat transfer, air leakage, vapour movement, and ingress of precipitation and ground moisture, for an environmental separator (i.e., the building enclosure, or known as the building envelope). Climate conditions across Canada range widely, as illustrated below in a map showing the major climate zones in Canada (Figure 5). Therefore, properties and placement of control layers and components used in the CLT building enclosure may vary greatly based on the project location and the building type. Building-related energy codes and standards also have different performance requirements for different climate zones, in particular with respect to the minimum effective insulation levels in the building enclosure.

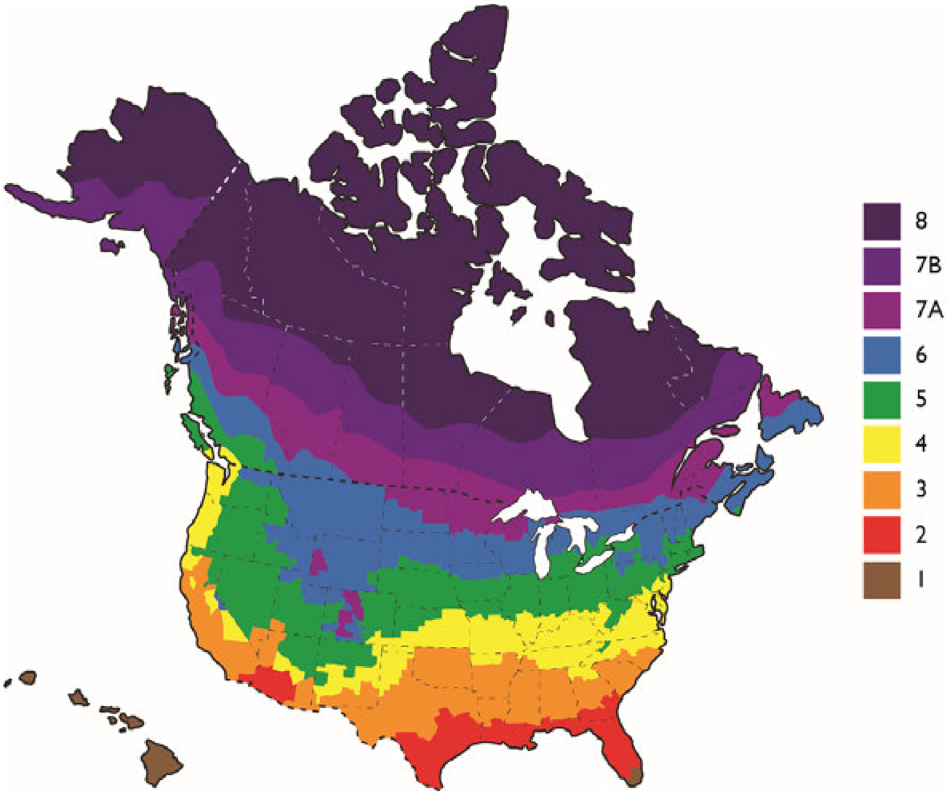


Figure 5 Climate zones in Canada and the United States based on the 2015 National Energy Code of Canada for Buildings and on the U.S. Department of Energy Climate Zones

The National Energy Code of Canada for Buildings (NECB) was first published in 2011, substantially updated from the previous 1997 Model National Energy Code for Buildings; it was again updated in 2015 (NRC, 2015b). This model energy code has been adopted by provinces, such as British Columbia, Ontario, and Alberta. The compliance options include a prescriptive path, a trade-off path, and a performance path. The maximum thermal transmittance, i.e. the minimum effective R-value requirements for above-grade assemblies and roofs are provided in the code for the prescriptive path. Effective R-values, instead of nominal insulation values, take into account thermal bridging caused by the more conductive materials, such as structural framing, metal fasteners, and other penetrations through the installed insulation. When CLT or other wood framing bypasses thermal insulation, it reduces thermal bridging. However, wood's thermal bridging is much smaller compared to other structural materials, such as concrete and steel, and it usually is much easier for wood to meet the minimum requirements for effective R-values (BC Hydro and BC Housing, 2014).

The NECB requires that all opaque building assemblies acting as environmental separators include a continuous air barrier. Materials used as part of the air-barrier systems must be air impermeable (i.e. less than 0.02 L/s·m^2; 0.004 cfm/ft^2 at 75 Pa), free of holes and cracks, and compatible with adjoining materials. The air-barrier continuity between opaque assemblies and fenestration must also be maintained. Currently, there are no requirements for whole building airtightness testing within the NECB. Aside from the NECB, some jurisdictions, such as British Columbia and Ontario, also allow the use of ASHRAE 90.1 (ASHRAE, 2010) for large buildings to meet the energy requirements (Part 3).

The requirements for buildings conforming to Part 9 of Division B of the NBC are provided in its Section 9.36: Energy Efficiency of the National Building Code of Canada (NRC, 2015a). From a building envelope perspective, the NBC provisions have a scope similar to that of NECB.

In British Columbia, a new energy step code was enacted in 2017 (BC, 2017). This code requires new construction of both Part 9 and Part 3 buildings to be zero energy ready by 2032, through several steps of improvement depending on the climate and progress in each jurisdiction. Airtightness testing and energy modelling have become mandatory to ensure compliance with the energy codes. In essence, the step energy code requires that all building types be built with building enclosures having higher levels of thermal insulation and airtightness compared to the minimum requirements for enclosures provided in the existing codes and standards.

Energy codes and standards, as referenced within Canadian codes or project-specific energy performance targets, will provide the required thermal resistance (R-values) or thermal transmittance (U-factors) values for building enclosure assemblies and the tolerance for loss in thermal performance due to the effects of thermal bridging. Within Canadian reference energy codes and standards for buildings (NBC, NECB and ASHRAE 90.1), the minimum effective prescriptive IP unit R-value (ft^2 ^{o}F/Btu/h) for an exterior CLT wall ranges from R-15 to over R-30, depending on the climate zone, with more insulation being required in the far north. For roofs, the range of minimum R-values is between R-20 and R-60 depending on climate. Where prescriptive R-values are not met, alternate simple component or building enclosure trade-off or energy modeling paths may be used, as outlined within the energy code. As will be illustrated in the next section, despite the relatively high thermal resistance properties of wood compared to other structural materials, CLT building enclosure assemblies in Canada will require thermal insulation in conjunction with the wood panels themselves.

There are many industry resources on the design of building enclosures and a number of them are focused on wood buildings. The major design guides relevant to the design of CLT building enclosures include:

- The Guide for Energy-Efficient Wood-Frame Building Enclosures in Marine to Cold Climates in North America (FPInnovations, 2013)
- Building Envelope Thermal Bridging Guide: Analysis, Applications & Insights (BC Hydro and BC Housing, 2014)
- The Technical Guide for the Design and Construction of Tall Wood Buildings in Canada (FPInnovations, 2014)
- Building Enclosure Design Guide–Wood Frame Multi-Unit Residential Buildings (BC Housing, 2018).

10.3 BUILDING SCIENCE FOR CLT BUILDING ENCLOSURES

Where CLT is used as part of the building enclosure, it works together with several other layers or components to control the flow of heat, air, water and vapour, and to provide fire, smoke and acoustic separation. The relevant design guides available (Section 10.2) provide a more in-depth appraisal of loads and control functions for CLT building enclosures. The information provided in this Chapter provides a summary of the building science as it relates specifically to CLT enclosures.

To ensure that the enclosure will perform adequately, designers should choose an appropriate CLT assembly, carefully prepare details of enclosure interfaces and transitions, and account for climate-specific conditions and building occupancy requirements as they may arise, both during construction and over the service life of the building.

10.3.1 Control of Heat Flow and Selection of Thermal Insulation

Properly managing heat flow across the enclosure is important to reduce energy consumption, minimize condensation risk, and increase occupant thermal comfort. For building enclosure assemblies incorporating CLT panels, the heat flow path is controlled by the inherent thermal resistance of the CLT (depending on the thickness and species) together with added thermal insulation and the nominal thermal resistance of other enclosure layers, surface air films, and finish materials. The effective R-value of the assembly of components can then be calculated to determine whether the assembly meets the energy code requirement or the final performance target of a specific project. The thermal resistance provided by the wood itself is also beneficial to the CLT building enclosure for other reasons beyond energy efficiency, as covered in the following sections.

Wood has a relatively low (i.e., good) thermal conductivity compared to other structural building materials such as steel or concrete. Thermal conductivity and resistance values for common CLT lamination thicknesses are given in Table 1 (Kumaran, 2002; NRC, 2015), based on example 3-, 5-, 7-, and 9-ply panels available in Canada. It should be noted that suppliers offer many other CLT panel thicknesses, of different lamination depths. It is also important to note that the adhesives used within the CLT panels do not significantly affect the thermal resistance of the panels.

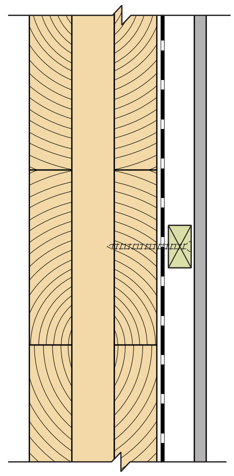
Table 1 Thermal Conductivity, Thermal Resistance and Equivalent R-values for CLT Panels Using North American Softwood Species

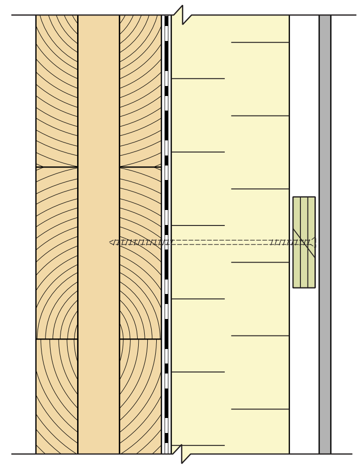
CLT Panel Species	Thermal Conductivity at 12% Moisture Content		Thermal Resistance per Inch	
	W/m·K (Btu·in/hr·ft²·°F)		ft²·°F·hr/Btu	
Spruce-Pine-Fir (SPF)	0.12 (0.82)		1.2	
Hemlock-Fir (Hem-Fir)	0.12 (0.82)		1.2	
Douglas Fir-Larch (DFL)	0.15 (1.0)		1.0	
CLT Panel Species	**Approximate R-value for CLT of Various Thicknesses (ft²·°F·hr/Btu)**			
	3-ply, 3.5”	**5-ply, 5 5/8”**	**7-ply, 7 ¾”**	**9-ply, 10.5”**
Spruce-Pine-Fir (SPF)	4.2	6.8	9.3	12.6
Hemlock-Fir (Hem-Fir)	4.2	6.8	9.3	12.6
Douglas Fir-Larch (DFL)	3.5	5.6	7.8	10.5

In all climate zones, it is best practice to locate the thermal insulation of a CLT assembly on the outboard side of the CLT panel. This placement helps ensure the wood remains warmer than the ambient environment, thus ensuring decreased wetting potential and improved drying capacity. It also protects the wood from temperature fluctuations and related changes in relative humidity, resulting in increased long-term durability and a consistent thermal resistance. From an aesthetics perspective, this location also allows the CLT to remain exposed to the interior, where allowable by fire safety provisions.

When all or a portion of the thermal insulation is located on the interior side of the CLT, it is advised to carefully evaluate the project-specific assembly for long-term moisture performance and durability. The placement inboard or outboard of the CLT and the selection of insulation properties is further discussed in Section 10.4.1. The support of claddings and thermal bridging through exterior insulation is also discussed. Figure 7 provides a summary of the considerations for calculating effective R-values for CLT building enclosure assemblies.

Figure 6 CLT exterior wall covered with a self-adhered vapour permeable air-barrier and semi-rigid exterior mineral wool insulation at the 13-storey Origine building in Québec City (courtesy of Stéphane Groleau for Nordic Structures)





Most softwood species used in North American CLT panels have an equivalent IP unit R-value of R-1.2 per inch. Therefore, a 3-ply 3.5” CLT panel has a material R-value of R-4.2. This is added together with the thermal resistance of the surface air films and cladding to provide an effective center of wall R-value of approximately R-7. Alone, this does not comply prescriptively with Canadian energy code requirements and, therefore, insulation is required (right).

The material R-value of a 3-ply CLT panel is R-4.2 for SPF or Hem-Fir. To reach typical component R-value targets of R-15 to R-30, several inches of insulation are required. Thermal insulation products recommended for CLT walls have material R-values in the range of R-4 to R-5/inch and therefore 3” to over 8” of exterior insulation may be required. This will also be impacted by the thermal effectiveness of the cladding support strategy used to connect the cladding to the backup CLT structure.

Figure 7 CLT wall assembly without insulation (left) and with exterior thermal insulation (right), including discussion of effective R-value calculations.

Wood also has a low thermal diffusivity due to its relatively low thermal conductivity (high R-value per inch of thickness), and moderate specific heat capacity and density. As a result, CLT panels may contribute to moderation or potential reduction of heating and cooling loads in some climates (e.g., mixed climates with large temperature swings) and may contribute to overall thermal comfort. Hourly whole-building energy modeling can be used to assess the potential benefit from the increased mass of CLT systems as compared to other construction types. However, in Canadian climates the overall benefit is expected to be small.

10.3.2 Control of Airflow and Air Barrier Systems

Managing airflow across the building enclosure is a key element in reducing energy consumption, increasing thermal comfort, and minimizing the movement of water vapour through the assembly. CLT enclosure assemblies have very unique airflow management considerations, as discussed in this Section. Managing airflow also minimizes the transfer of sound, smoke, fire, and airborne particulates and contaminants between environments.

Managing airflow across the building enclosure is a requirement of Canadian building codes (Section 10.2) and is accomplished by using an air barrier system – i.e., a three-dimensional system of materials designed, constructed, and acting to control airflow across and within the building enclosure. Interior air barrier systems may also be incorporated into buildings for smoke and fire protection reasons. Air barrier systems are integrated with CLT components; in most cases, it is recommended that membranes and components be adhered directly to the CLT panel to help ensure air barrier materials are adequately supported and to improve constructability. Examples of the many parts of air barrier systems applied to CLT assemblies are shown in Figure 8.

Figure 8 Photos of self-adhered air barrier membranes, flashings, sealants, tapes, roofing membranes, etc. applied to the exterior of CLT wall and roof panels for air barrier continuity (these components are installed prior to the installation of exterior insulation and claddings. Interfaces and details are the focus of attention for achieving acceptable performance of the air barrier system (and also for water control))

An air barrier system has five basic requirements; these requirements, with a specific focus on CLT wall, roof, and floor assemblies, are as follows:

1. Air Impermeability

The air barrier system must resist airflow (not to be confused with vapour flow as covered in Section 10.3.3). Typically, CLT panels alone are not part of the air barrier system. While CLT panels may initially have very low air permeability when tested in a laboratory setting, the actual interfaces between weathered panels in the field and the small spaces or gaps between each lamination of the CLT allow for the passage of air when the panels are structurally connected together. Examples of typical interfaces are shown in Figure 9 (left) for a floor to platform wall panel connection, and in Figure 9 (right) for a corner condition. Figure 10 shows a balloon-framed parapet condition, also common with CLT designs where even roof membranes applied to the CLT cannot block off these pathways.

The size of these gaps between laminations depends largely on the grade of CLT, shrinkage/swelling of the laminations resulting from moisture content changes, and to a lesser extent on the presence of edge gluing. Finish grade CLT manufactured with tight moisture control, and with minimal checking or gaps between laminations, will typically have less leakage than structural grade CLT, though it can still be significant. Project experience across North America has found that gaps and checks will be present at the time of erection or will gradually open up after moisture cycling and drying of the CLT during the first year. As a result of this inherent property of CLT, an air barrier system independent of the CLT is needed for long-term airtightness. This system is comprised of an appropriately detailed air barrier membrane that meets air transmission rates outlined in the building code and is applied to the exterior of the CLT panels. Other solutions to make the CLT panels airtight have been tried on different projects, including drilling and sealing of all the gaps/checks with expanding sealants; however, this option is labour intensive and challenging to detail. As an alternative to an exterior applied air barrier, an interior applied air barrier membrane system could be detailed, provided that all interfaces and transitions are sealed. The use of tape only at CLT panels joints or loosely applied, mechanically attached membranes is not recommended as primary air barrier systems for CLT assemblies.

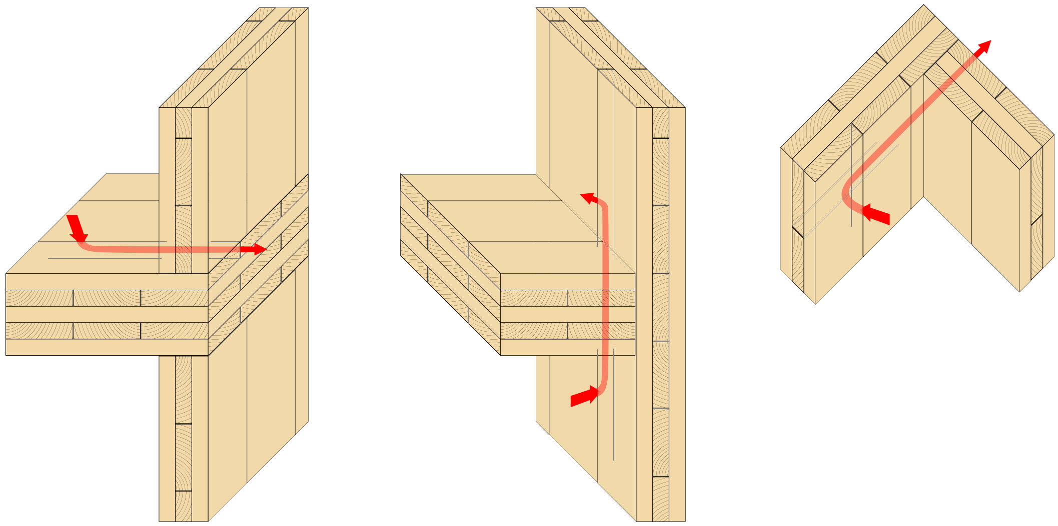


Figure 9 CLT panel interfaces which create potential airflow pathways that can be difficult to seal utilizing only the panels as an airtight element (LEFT: platform-framed wall panels sitting on a CLT floor. RIGHT: exterior/interior corners through CLT laminations. Both can be resolved with a separate air barrier membrane applied to the exterior side of the panels. MIDDLE: Balloon-framed CLT wall panel with CLT floor/roof)

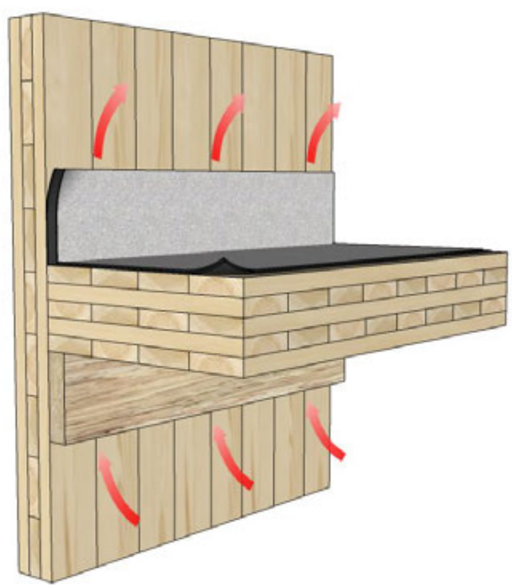


Figure 10 Parapet of balloon-framed CLT wall panel with CLT roof (or other framed roof deck) showing air leakage pathway through vertical gaps in the CLT.

In this scenario, the detail is difficult to air seal with the ledger support and roof membrane interfaces. The best method to seal this interface is to wrap the parapet from the exterior to the interior side of the wall with a self-adhered (vapour permeable) membrane (Figure 11).

2. Continuity

The materials within the air barrier system must be continuous. This requires that the air barrier system of the CLT assembly be continuous at all joints, penetrations, and interfaces with other assemblies. This means that many different materials acting together will be relied upon to provide airtightness. Examples showing air barrier continuity are shown in Figure 11 for a parapet interface at a CLT roof and for a CLT floor soffit to exterior wall application.

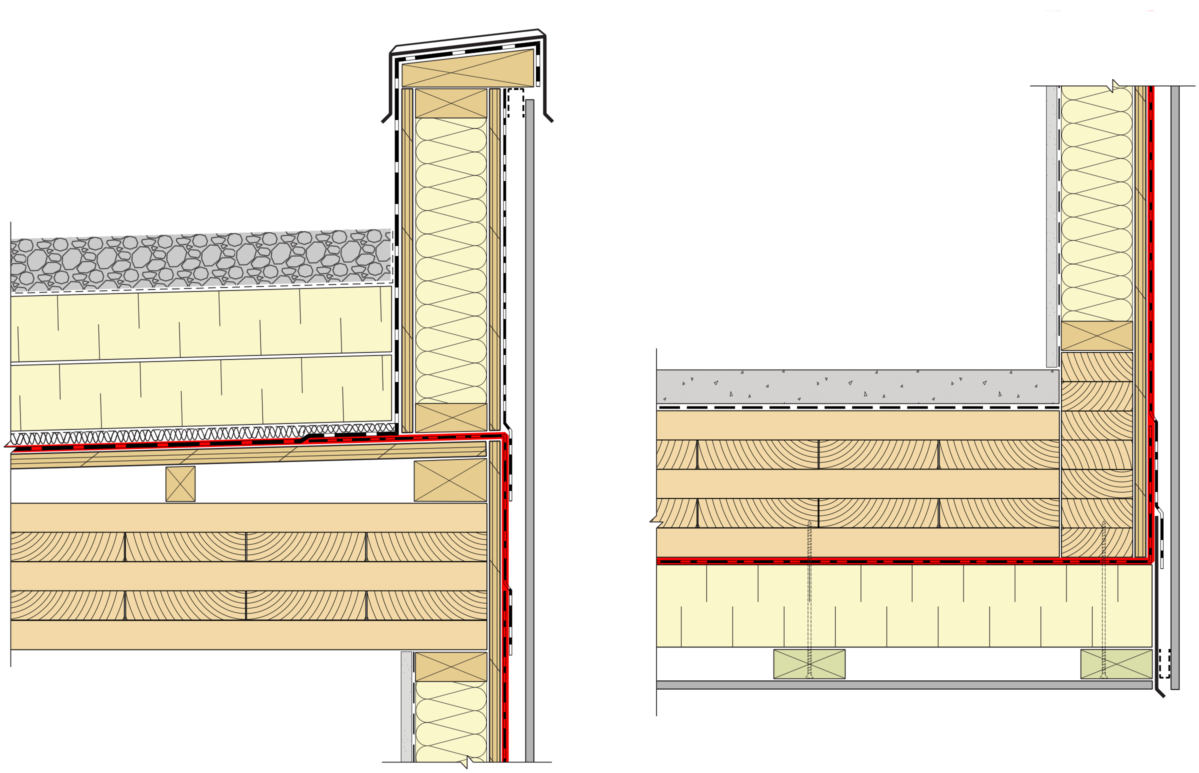


Figure 11 Example of CLT air barrier continuity details showing roof parapet (left) and exterior floor soffit (right) (Note the use of membranes from roof to wall and applied to the underside of the floor. In these applications these membranes and components are also used to control water)

3. Stiffness

The air barrier system must be resistant to any structural loads that will be applied to it, without significantly distorting, delaminating, or becoming damaged. These loads are due mainly to the air pressure differential acting across the air barrier. In a CLT assembly, this is best overcome by providing an air barrier membrane that is fully adhered to the CLT panels, as both positive and negative pressures can be resisted by the adhesion to the CLT. In low-rise applications with decreased wind exposures, a mechanically attached membrane can also be used when sandwiched between exterior insulation and the CLT panel, though more care must be taken to address airflow behind the membrane, at membrane details and interfaces. Figure 12 demonstrates this scenario at a roof parapet, and shows that achieving airtightness with a mechanically attached membrane is significantly more difficult than with a self-adhered membrane applied to the CLT panels.

Figure 12 Example parapet detail where air barrier membrane stiffness has a significant impact on the performance of the system (The use of a self-adhered membrane or sandwiching this membrane behind exterior insulation would help reduce the billowing and sealing challenges at this interface)

4. Strength

The air barrier system must be strong enough to transfer air pressures back to the supporting structure. Whereas the CLT structure is strong enough to carry this load, the membrane and components that serve as the air barrier system should be fully adhered or mechanically attached to the CLT. Mechanically attached systems are cautioned against in higher wind exposure applications.

5. Durability

The air barrier system must be durable enough to perform over the design service life of the building enclosure. In a CLT assembly, this requires that the air barrier system withstand temperature fluctuations, building movement, air pressure differentials, and environmental exposures (e.g., UV and site contaminants), which may occur during the building's life cycle.

The properties discussed above are specific to the building service life; however, air barrier system materials must also demonstrate durability and strength during the construction phase to ensure long-term performance. UV exposure, moisture exposure, wind pressures/gusts, and trade activities must all be considered. Placement of the air barrier system at a protected location, for example on the exterior of the CLT behind exterior insulation, helps to address these requirements. An example of such a wall assembly is shown in Figure 13.

The location of the air barrier membrane within CLT wall and roof assemblies is further discussed in Sections 10.4.1 and 10.4.2.

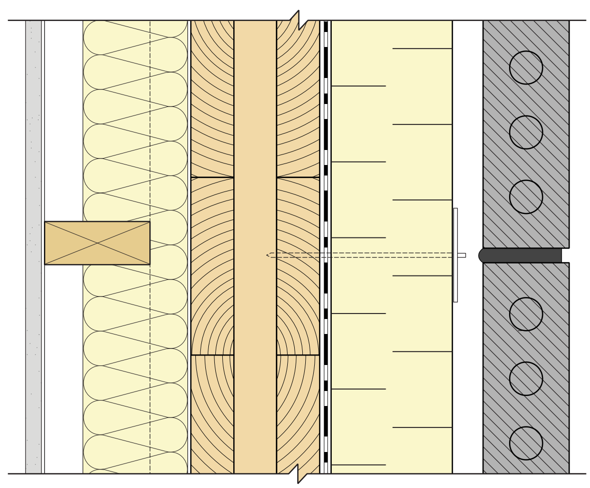


Figure 13 Example exterior-insulated CLT wall with masonry cladding, with air barrier membrane sandwiched between CLT and exterior insulation (courtesy of Ronald McDonald House, Vancouver, BC by Michael Green Architecture)

10.3.3 Controlling Vapour Flow and Maintaining Drying Capacity

10.3.3.1 Vapour Sorption and Dimensional Changes

CLT, being made of wood, is hygroscopic and therefore has an inherent moisture-storage capacity. CLT panels exchange moisture with the surrounding air under ambient conditions. The amount of moisture gain or loss largely depends on the relative humidity, but also on the temperature and other factors. When the wood no longer gains or loses moisture, it reaches equilibrium moisture content under a specific set of environmental conditions. Figure 14 illustrates the relationship between equilibrium moisture content and relative humidity at a few select temperatures. The ANSI/APA PRG 320 (2018) CLT manufacturing standard requires that the moisture content of lumber at the time of CLT manufacturing be 12% ± 3%. Typical equilibrium moisture content of wood materials within building enclosures are from 8% to 18%, depending on the interior relative humidity conditions. This means that to adjust to typical building service conditions, CLT panels exhibit only small changes in moisture content after installation, depending on the outdoor and indoor conditions. Wood has a delayed response to changing environment, depending on its size, vapour permeability, the environmental conditions, and its surface finishing or other treatment, if present. For a large CLT panel, the surface can change moisture content quickly, but it takes a much longer time (e.g., weeks or months) for the centre of the panel to show a response to the changing environmental conditions. The hygroscopic nature of CLT can be advantageous in that CLT enclosures can buffer or accommodate short-term changes in humidity and temperature, unlike metal-framed enclosures.

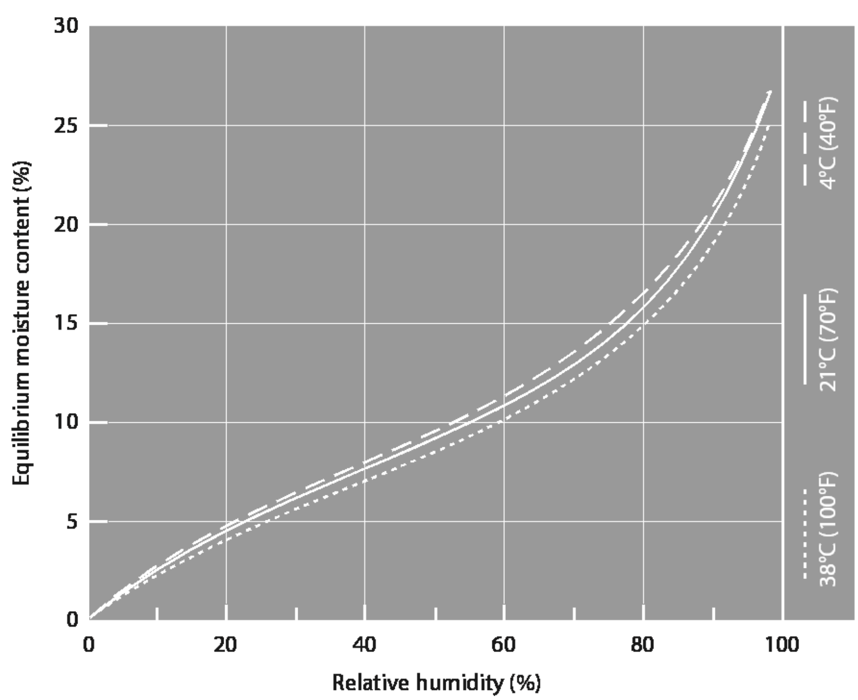


Figure 14 Generic sorption isotherms for wood, adapted from the Wood Handbook (FPL, 2010)

When the moisture content is below its fiber saturation point, wood shrinks when it loses moisture and swells when it gains moisture. Dimensional changes are greatest in the direction of the annual growth rings (tangential), about half as much across the growth rings (radial), and usually very small along the grain (longitudinal) (FPL, 2010). For example, the average shrinkage of spruce from fibre saturation point to oven-dry state (i.e. a moisture content change from 30% to 0%) is about 7-8% in the tangential direction, 4% in the radial direction, and 0.1-0.2% in the longitudinal direction (FPL, 2010). Wood used in construction and similarly in CLT manufacturing always has a mixture of growth ring orientations. It is recommended to use an average shrinkage coefficient of 0.20% to 0.25% per 1% change in moisture content, for cross-sections of most softwood lumber.

When care is taken in the manufacturing, transport, storage, and construction stages, the moisture content of CLT will only change within a small range, and consequently the shrinkage (or swelling) will be much smaller. For example, if the CLT has an average moisture content of 12% during manufacturing and the equilibrium moisture content in service is 10%, the moisture content change is 2%, which is associated with potential shrinkage of around 0.4-0.5% in the thickness direction of the CLT panel. Although the potential shrinkage in the width direction of the individual boards would be similar to that in the thickness direction, the cross lamination of boards in CLT panels minimizes the in-plane dimensional changes, due to the good longitudinal stability of the adjacent lamina, as in plywood. This has been proved by field measurements, which showed that the vertical movement of CLT walls is about 2-4 mm per storey (Wang et al., 2016), a level not much higher than that measured for glulam columns. However, the shrinking and swelling of individual boards can cause warping and checking in the CLT panel surface if large cyclic moisture content changes occur.

10.3.3.2 Vapour Permeability and Vapour Barriers

Managing water vapour transport across a CLT assembly can be accomplished by virtue of using the intrinsic properties of wood to control water vapour diffusion (e.g., vapour retarder) and by managing airflow with an air barrier system (discussed above). Airflow transports significantly larger amounts of water vapour than vapour diffusion alone; however, both transport mechanisms should be carefully considered in relation to the building's interior and exterior climatic conditions. Vapour permeance should be considered as much for allowing drying as it is for controlling or stopping vapour movement.

The vapour permeance of North American softwood lumber species used for CLT manufacturing at normal indoor RH levels of 30-50% generally ranges from less than 10 ng/Pa·s·m^2 to as much as 125 ng/ Pa·s·m^2 (0.17 to 2.0 U.S. perms) dry cup values, for a 25-mm (1-in.) thick piece of lumber (Kumaran, 2002; Alsayegh et al., 2013). For a CLT panel thickness of approximately 89 mm (3½ in.), the total vapour permeance ranges from less than 3 ng/Pa·s·m^2 to 35 ng/Pa·s·m^2 (0.05 to 0.6 U.S. perms) dry cup. See Table 2 for approximate vapour permeance for CLT made with Canadian softwood species, based on limited testing. A typical 3-ply CLT panel itself meets the requirement for vapour barrier (e.g., Class II vapour retarder), based on the Canadian code requirement (NRC, 2015a). Thicker panels are even less permeable. A supplemental vapour barrier is therefore not necessary when designing CLT building enclosures.

Table 2 Vapour permeance of CLT at different thicknesses and relative humidity levels (based on Alsayegh et al., 2013)

Relative Humidity	Vapour permeance ng/Pa·s·m^2 (U.S. perm)		
	100 mm (4 in.)	150 mm (6 in.)	200 mm (8 in.)
20%	3.4 (0.06)	2.3 (0.04)	1.7 (0.03)
50%	18 (0.31)	12 (0.21)	9.0 (0.15)
80%	59 (1.0)	39 (0.68)	30 (0.51)

10.3.3.3 Maintaining Drying Capacity

Vapour flow control is important in building enclosure design since it is associated with two major strategies of moisture management: to minimize moisture accumulation within the building enclosure, and to maximize drying capability by generally using materials with high vapour permeability. These two strategies may conflict, and it is important to coordinate them in the design. Because a built-up CLT panel is inherently a vapour retarder, and for all practical purposes inhibits the flow of water vapour across it, **any additional vapour barrier is not desirable within CLT assemblies in order to maintain the drying capacity**. This is fairly straightforward for wall designs, though complicated in roof assemblies, as these feature impermeable water-resistant layers. This is further discussed in Sections 10.4.1 and 10.4.2 for walls and roofs. The inclusion of impermeable materials within a CLT assembly can minimize its drying ability, should the panel become wet during construction. As such, it is important that CLT panels be sufficiently dry prior to the installation of any enclosure layers, or that the assembly be specifically designed to allow for drying of construction moisture. This is particularly relevant for the application of roofing over CLT roofs or concrete toppings, or of membranes over CLT floors.

When designing a CLT assembly, the vapour impermeability of the CLT should be considered in relation to the assembly's insulation placement and type, as well as the placement and properties of other air and water control membranes, to avoid water vapour accumulation within the panel and to ensure the panel's long-term durability. Hygrothermal modeling (Wang et al., 2010) followed by field monitoring research at the University of Waterloo (Lepage, 2012; McClung et al., 2014) and by FPInnovations (Wang, 2014; 2018) has shown the potential risk of encapsulating an initially wet CLT behind vapour impermeable materials installed on either side of the panel. Therefore, for CLT assemblies, the selection of vapour control materials with properties that allow drying is as important as the vapour retarder properties, to ensure long-term performance.

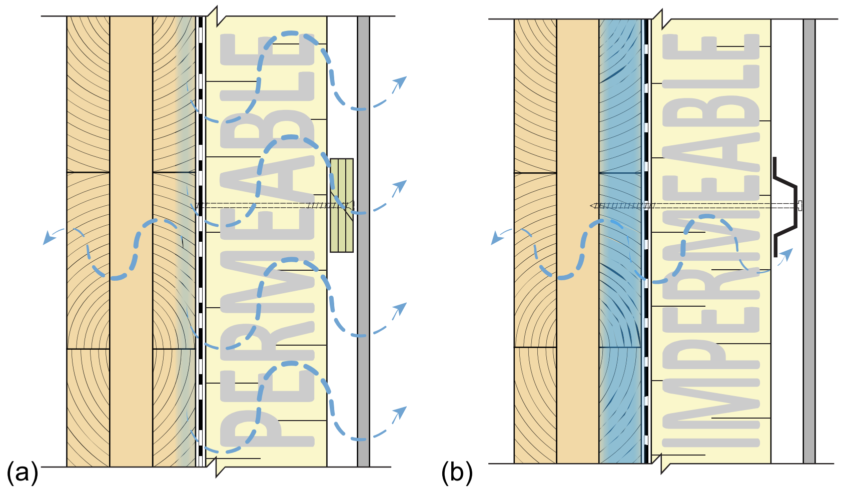


Figure 15 Example showing the relative rate of drying towards the exterior (a) vapour permeable exterior insulation and a permeable air barrier (AB)/water resistive barrier (WRB) membrane; (b) vapour impermeable exterior insulation or impermeable AB/WRB. While the interior laminations of the CLT can dry out relatively quickly when they are allowed to dry towards the interior, the outer lamination behind the impermeable membrane will take months to years to fully dry out.

10.3.4 Controlling Rainwater and Exterior Moisture

CLT exposure to liquid water can occur both during construction and over the service life of the building. Absorption of water can rapidly change the moisture content of the wood as compared to vapour sorption. Consequently, this increases the risk not only of dimensional changes, but also of mould growth, decay, and fastener corrosion. The major factors affecting the rate of water absorption and the amount of water absorbed are the wood species, grain orientation, and time of exposure. For example, wood absorbs moisture much more rapidly through its end grain (i.e., longitudinal direction) than through the transverse directions (Wang, 2018). Exposure can increase risks of dimensional changes, as a result of shrinking and swelling, and can create gaps between CLT laminations, between panels, and between panels and penetrating or surrounding elements such as columns or wall structures. Rapid dimensional changes can also cause surface checking to occur. Minimizing exposure to liquid moisture and maintaining the moisture content of the CLT consistent with the in-service equilibrium moisture content is critical to ensuring the integrity of the CLT assembly and preserving the expected service life of the building.

Liquid water at the roof is managed by the roof membrane and roof drainage system; the location of this membrane and additional considerations are discussed in Section 10.4.2. To help ensure the long-term performance of the CLT roof during building occupancy, this guide recommends that a durable fully adhered (e.g., multi-ply) roof membrane be installed on the CLT roof, especially where temporary roof membranes are not used. Additional best practice guidance is provided in the related roofing manuals developed by the National Roofing Contractors Association.

Within wall assemblies, water is managed first by the water-shedding surface, which consists of claddings, flashings, and other surface water management features, and secondly, by the water-resistive barrier (WRB) system. The best practice strategy for rainwater penetration control in CLT walls is a drained and ventilated cladding (i.e., rainscreen cladding), which is a common construction practice in the wetter regions of North America. While this rainwater control strategy may seem excessive in some climates, it helps provide redundancy in the water management design of CLT structures. This practice of back-ventilating cladding is also recommended by many manufacturers, to help ensure the long-term performance of various claddings and cladding finishes and coatings. This design approach is also beneficial in providing an outlet for inward-driven moisture from more absorptive claddings (i.e. reservoir claddings), such as stucco, brick and stone masonry, and other porous cladding materials.

The cladding surface sheds the majority of the rainwater load from the exterior surface of the wall; however, it is not the only line of resistance to water penetration. Moisture that does penetrate beyond the water shedding surface will either run down the backside of the cladding, the strapping, the surface of the exterior insulation, or the final line of protection, i.e., the lapped and air/water-sealed WRB. The WRB is a secondary plane of protection against liquid water and the innermost plane that can safely manage and drain an incidental moisture load. The WRB is typically in the form of a waterproof sheathing membrane installed on the exterior of a CLT wall panel. In most cases this same membrane will also be sealed and detailed as the primary air barrier system. With exterior insulation, the CLT is also somewhat buffered from wetting by the moisture resistant exterior insulation. Any moisture that penetrates the cladding must then be drained back out of the assembly using flashings attached behind the WRB at floor levels and around through-wall penetrations, such as windows. Figure 16 provides a summary of design recommendations for a drained and ventilated exterior-insulated wall assembly.

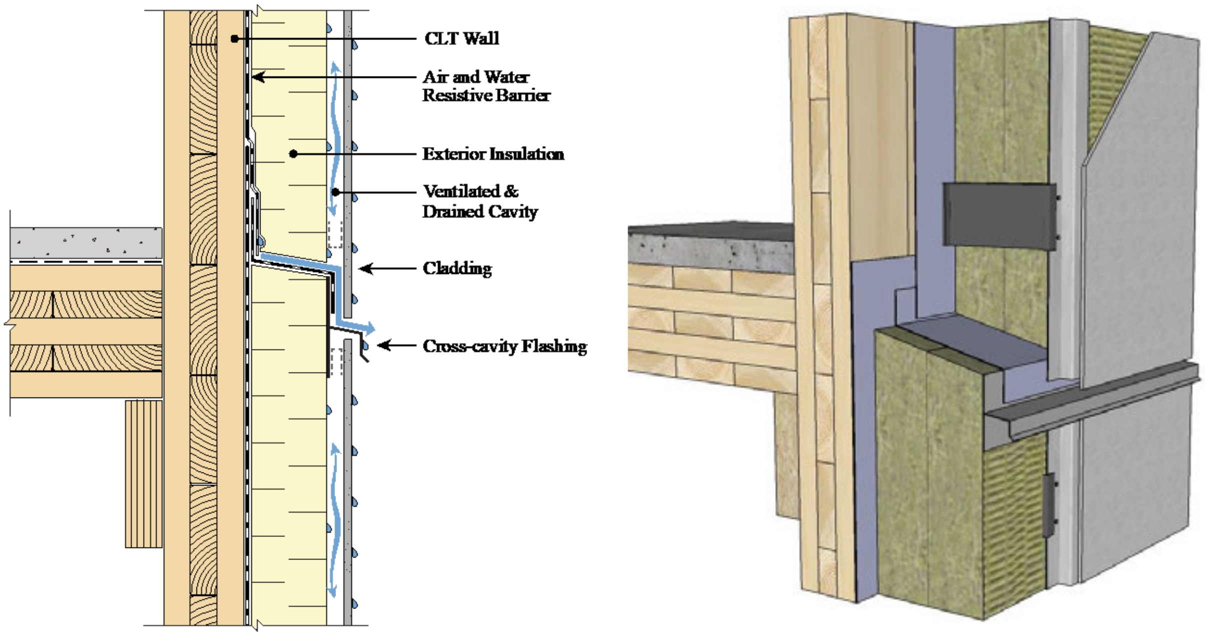


Figure 16 Best practice rainwater management strategy for CLT wall assembly (Detail shows a ventilated and drained cladding rainscreen system where primary cladding and secondary drainage planes are provided in addition to ventilation behind the cladding. This is typical of other exterior-insulated rainscreen wall assemblies).

Floor assemblies are relatively protected from liquid water exposure during the building's service life except for plumbing and appliance failures and other wet in-service building conditions. Where the risk of wet interior conditions exists, a waterproof floor coating and drainage is recommended. When cementitious toppings are to be installed on floor assemblies, the moisture content of the CLT panel should be maintained below a maximum of ~16% prior to placement of the concrete (Wang, 2018, RDH, 2016). Moisture-laden concrete toppings are a source of moisture that can become trapped within the CLT for extended periods of time; thus, coatings or membranes applied on the top side of the panel are typically recommended prior to concrete placement, unless it can be shown that the CLT will not negatively be impacted by this large moisture source, either by design or by low initial moisture levels. Managing liquid water at CLT soffit assemblies is also accomplished by appropriately managing water at the interfaces of adjacent perimeter walls. Protection of floors against construction and in-service moisture is discussed in detail in Section 10.5.

10.3.5 Protection of CLT Panels from Wetting at Grade

CLT is intended for above-grade applications only; it is not suitable for below-grade applications due to the potential for moisture-related damage. CLT also needs to be protected from wetting sources at and above grade. Similar to wood-frame construction, the NBC requires that sill plates be treated with preservatives if the vertical clearance to the finished ground level is less than 150 mm (6”) and a damp-proof membrane is not used (NRC, 2015a). A good practice is to elevate the CLT at least 200 mm (8”) above the finished ground, together with the use of vapour-impermeable capillary break/waterproofing to separate the wood from the concrete. An example of a detail showing the protection of a CLT shear wall at a podium slab and at a base-of-wall interface is shown in Figure 17. In areas where termites and other wood boring pests are present, measures should be taken to prevent damage from insects, as required by local building codes. Further information is provided in Section 10.5.3.

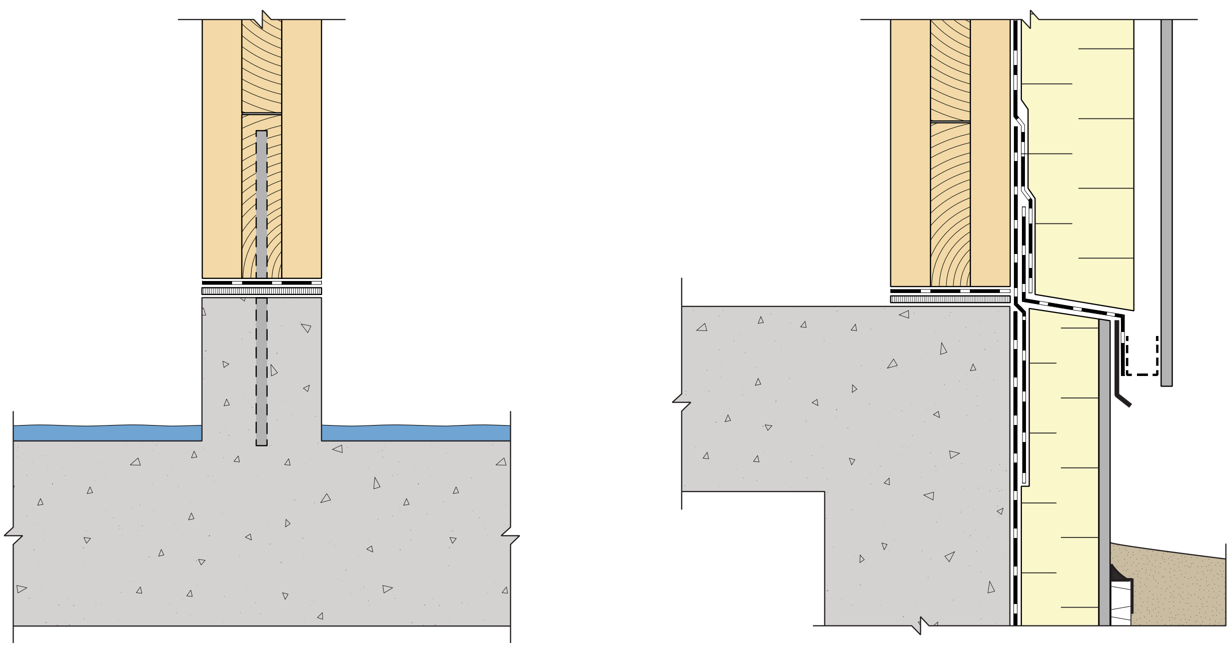


Figure 17 Examples of a detail showing elevation of CLT above potential construction and in-service wetting mechanisms at a concrete podium slab and at an exterior wall detail

10.4 BUILDING ENCLOSURES INCORPORATING CLT

10.4.1 Exterior Wall Assemblies

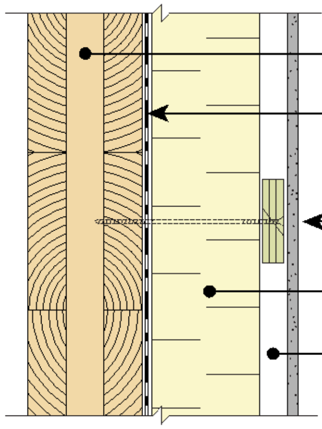
The exterior wall assemblies of buildings with CLT structures may or may not be constructed of CLT panels. Where exterior walls are designed as load-bearing or lateral shear walls, CLT panels are commonly used; however, where exterior walls are non-load-bearing and only transfer lateral loads and their self-weight, a number of options exist. For non-load-bearing applications, walls tend to be lightly framed and may even be hung to the primary structural frame as a panelized curtain wall. Thicker 3- or 5-ply CLTs in these lighter load applications may not be the most cost-effective choice, unless thinner 3-ply or 5-ply panels are used, potentially even in non-load-bearing curtain wall or precast-like applications. In some cases, the fire safety provisions and other site-specific requirements may also dictate the use of non-combustible materials such as steel stud/gypsum framing for exterior walls. In some applications, it may also be required to install gypsum sheathing over the CLT panels as a thermal barrier.

10.4.1.1 Best Practices for CLT Wall Designs

CLT walls should be designed in relation to the climate zone in which the building is located and the use of the indoor space. CLT panels will perform best when they are kept dry and ideally near room temperature. In most cases, this suggests that all or most of the required thermal insulation be installed on the exterior of the CLT wall assemblies. Based on building science fundamentals for heat, air, vapour, and water control presented in Section 10.3, the design of wall assemblies incorporating CLT panels was determined and a summary is given in Figure 18 for cold, heating-dominated climate zones in Canada (as well as in the Northern United States, i.e., Climate Zones 4-8).

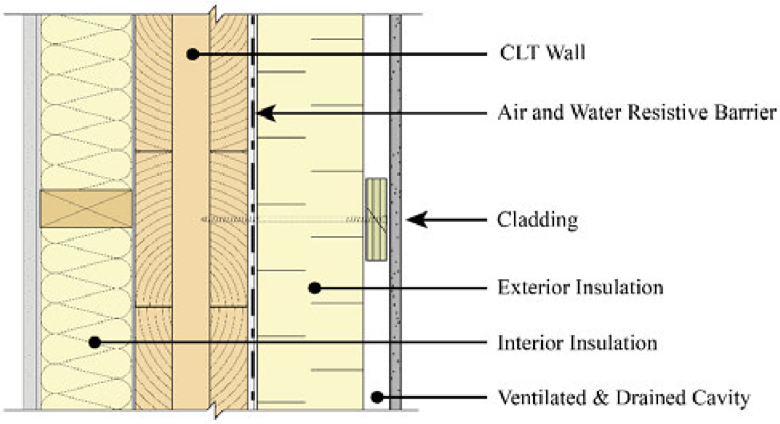
Wall Design Guidance for Cold, Heating-Dominated Climate Zones in Canada

Exterior-Insulated



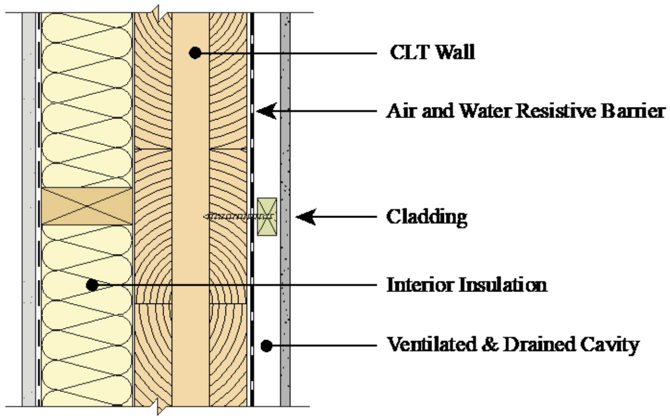
Exterior-insulated is the preferred design approach for CLT wall assemblies in Climate Zones 4 through 8; this also allows the CLT to be left exposed on the interior, where allowed by the fire safety provisions. Exterior insulation keeps the CLT warm and dry; the CLT on the interior provides sufficient vapour resistance to outward vapour drive, eliminating the need for further vapour control. The use of vapour permeable exterior insulation (>10 perms) and a self-adhered vapour permeable AB/WRB membrane outboard of the CLT is the most durable approach, as it allows for outward drying of initially wetted CLT or the drying of small leaks in service. The use of either vapour impermeable exterior insulation or a vapour impermeable AB/WRB membrane will practically stop outward drying and therefore should only be used with extreme caution. Caution must also be taken where the indoor RH is expected to be elevated such as in densely occupied housing, museums, and swimming pools. The use of a drained and ventilated rainscreen cladding is recommended for all building types and will also help to dissipate inward solar-driven moisture. Vertical strapping is recommended for cladding support, and where horizontal strapping is required for cladding support, it should be installed on the exterior of the vertical strapping. Long screws are shown as cladding attachments though other strategies exist. To achieve current minimum prescriptive code R-values, 3-8" or more of exterior insulation will be required with a thermally efficient cladding attachment strategy.

Exterior- & Interior-Insulated



The use of both exterior and interior insulation may be desirable in some wall designs to either improve the overall effective R-value with less exterior insulation and to meet acoustic performance requirements where the interior of the CLT is not left exposed. The above guidance for exterior-insulated walls must also consider assessing the ratio of exterior to interior insulation (and hence resulting temperatures) and the material properties of the selected materials (mainly vapour and air permeability). The addition of interior insulation means that the CLT panel itself will not be as warm as in the exterior insulated case. Too much interior insulation could mean that the interior surface of the CLT could drop below the dewpoint of the indoor air and be at risk for condensation or moisture accumulation. As a general rule, to reduce the risk of formation of condensation on the interior panel surface and promote drying, the exterior-to-total insulation ratio should exceed 50% for low humidity spaces, over 65% for moderate humidity, and should be greater for high humidity spaces. This ratio can be determined by assessing the dewpoint temperature or with the use of hygrothermal modeling with specific material properties for long-term wetting and drying potential. The interior insulation should be vapour permeable, typically mineral fiber batts, and no supplemental vapour control layer should be included on the interior. This type of wall assembly should typically be assessed by a design professional; current Canadian codes do not cover this type of assembly with respect to insulation ratios or vapour control.

Interior-Insulated



Interior-insulated CLT wall assemblies are not generally recommended for building enclosures located in Climate Zones 4 through 8, except for unique indoor climatic situations and with special attention to the selection of materials and details. These exceptions may include cold-storage facilities and other low temperature or semi-conditioned indoor uses. In such an assembly, the use of interior insulation would maintain the CLT in cold and damp conditions (will come into approximate equilibrium with average outdoor RH) but, if prone to air leakage, the possibility for the formation of condensation due to vapour diffusion must be considered. Therefore, the use of a vapour impermeable insulation on the interior will allow control of outward wetting. However, this means that interior drying is not possible; note that drying outwards through the cold damp CLT is very slow. In all cases, a rainscreen cladding is highly recommended with this highly sensitive design. This type of wall assembly should always be assessed by a design professional, given the risk it presents to the long-term performance of the CLT.

Figure 18 Design guidance for exterior-insulated, exterior- and interior-insulated, and interior-insulated CLT wall assemblies in heating-dominated climates (Climate Zones 4-8 in Canada and Northern USA)

10.4.1.2 Cladding Attachment through Exterior Insulation and CLT

Cladding attachments can be a source of significant thermal bridging in exterior-insulated CLT wall assemblies if not properly implemented. It is critically important to adequately consider the gravity, wind, and seismic loads to ensure the claddings will perform in service without excessive deflection, cracking, or detachment from the structure. Optimizing structural cladding attachments is also important for achieving the thermal efficiency of exterior-insulated wall assemblies, while minimizing exterior insulation and overall wall thickness. Within the past few years many strategies have been developed for the use of different products to support cladding through exterior insulation in a thermally efficient manner, for several different wall types including CLT backup walls. These include the use of long screws in a truss configuration and various metal and fiberglass clip and rail systems that project through the exterior insulation. Examples of these different systems are shown in Figure 19. In comparison to framed stud walls, CLT has the benefit of allowing screws to be easily placed anywhere, without the need to ensure that fasteners are located in the vertical studs, as in the case of a wood- or steel-framed walls.

Long screws through exterior insulation over CLT. Cladding to be attached to the vertical strapping.

Fibreglass clips attached with screws to CLT backup with vertical metal girts outboard of the insulation to attach cladding.

Stainless steel brick ties (lower) and thermally improved brick shelf angle supports to allow for continuous insulation behind the angle.

Adjustable stainless steel clips through exterior insulation. Cladding to be attached to the vertical strapping (not added yet in photo).

Figure 19 Examples of various thermally efficient cladding support systems for use over CLT backup walls

For CLT wall assemblies, the most cost-effective and thermally efficient strategy for most cladding systems will be long screws through the insulation (Figure 20). This typically consists of #10 to #14 screws installed through vertical wood strapping or metal girts placed on the face of rigid insulation; it is recommended that the wood strapping be preservative-treated in damp areas (NRC, 2015a; CSA, 2015). Screws are typically placed every 305 to 406 mm (12”-16”), penetrating at least 25 mm (1”) into the CLT structure. Rigid mineral wool (density > 128 kg/m^3 (8 pcf)), rigid wood-fibre insulation, expanded polystyrene (EPS), extruded polystyrene (XPS), and polyisocyanurate are all sufficiently rigid for this approach. In this configuration, the reduction in thermal performance of the exterior insulation is typically less than 5% for stainless steel screws, and less than 10% for galvanized steel screws, depending on gauge and spacing. These options are far more thermally efficient than the reductions in thermal performance evident with continuous steel girts, which are typically in the range of 50% to over 70% loss in thermal resistance.

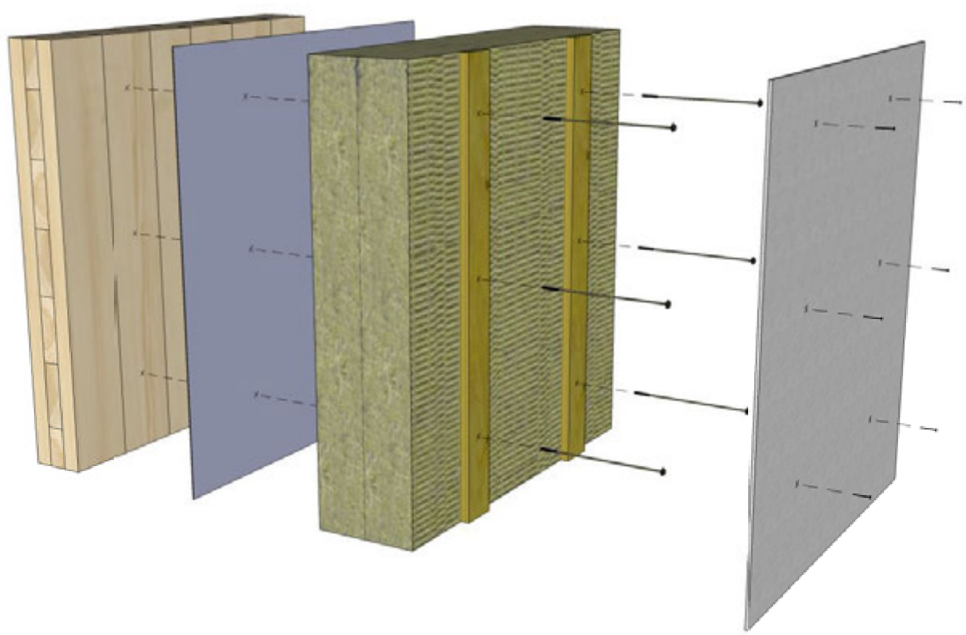


Figure 20 Best practice and most cost-effective light to medium weight cladding attachment for CLT walls

More information about designs for thermally efficient cladding attachments can be found in relevant design guides (see, e.g., BC Hydro and BC Housing, 2014; BC Housing, 2017; RDH, 2017).

10.4.1.3 Integration of Windows into Exterior-Insulated CLT Walls

The installation of windows into an exterior-insulated CLT wall assembly is similar to the integration of windows into other exterior-insulated wood stud or steel stud walls, though extra care must be taken not to trap moisture within an initially wet CLT or when incidental leaks to the CLT panel occur in service.

There are several potential methods to prepare the rough opening of the CLT and to install and seal the window in place, depending on the desired placement and aesthetic of the window, and the type of window frame. In general, the installation of windows should follow the directives provided in CAN/CSA-A440.4-07 (R2016): Window, Door, and Skylight Installation, with a specific focus on protection of the CLT in the rough opening and provisions for air and water management at the window-to-wall interface. Several examples, including some for CLT walls, are shown in the *Guide for Designing Energy-Efficient Building Enclosures for Wood-Frame Multi-Unit Residential Buildings in Marine to Cold Climates in North America* (FPInnovations, 2013); a general schematic of a punched window installation is provided in Figure 21 below. Key points to consider when detailing the rough opening and window installation include:

- Continuity of the air barrier and water-resistive barrier (AB/WRB) membrane from the exterior surface of the CLT into and through the rough opening is critical to protect the CLT and to seal the window, though this interface can be detailed in a number of different ways.
- The membrane used as the sill flashing should be thick and sufficiently robust not to be damaged when installed against the cut edge of the CLT or plywood liner. This membrane should also be vapour impermeable and resistant to ponding water, as the CLT surface will typically be flat. The use of a back-dam or sill angle is preferred to air- and water-sealing of the sill, and no fasteners should penetrate the sill membrane into the CLT, unless inboard of the air and water seal.
- The membranes used at the jamb and head of the window into the CLT rough opening should be vapour permeable. It is suggested that the same vapour permeable self-adhered membrane recommended for the exterior of the walls be used in this instance. The window is to be sealed to this membrane using a compatible sealant, tape, or self-adhered membrane.
- At the window sill, the installation of a supplemental flashing “skirt” to divert water from the rough opening and over the exterior insulation is preferred over allowing water to drain behind the exterior insulation. This skirt is typically installed at the time of window installation, with the release liner left intact until the exterior insulation is installed.

- A plywood box liner installed within the CLT panel is not necessary for most window installations, where installed towards the middle or inner part of the wall. Where a window installed towards the exterior cladding is desired from an aesthetic standpoint, it can be attached to a plywood box liner, which is attached to the CLT. If a plywood box liner is used, it needs to be protected with appropriate membranes. From a simple construction and thermal optimization standpoint, the window would ideally be installed on the CLT panel at the outermost point for support, with insulation detailed around and potentially over the exterior of the frames.

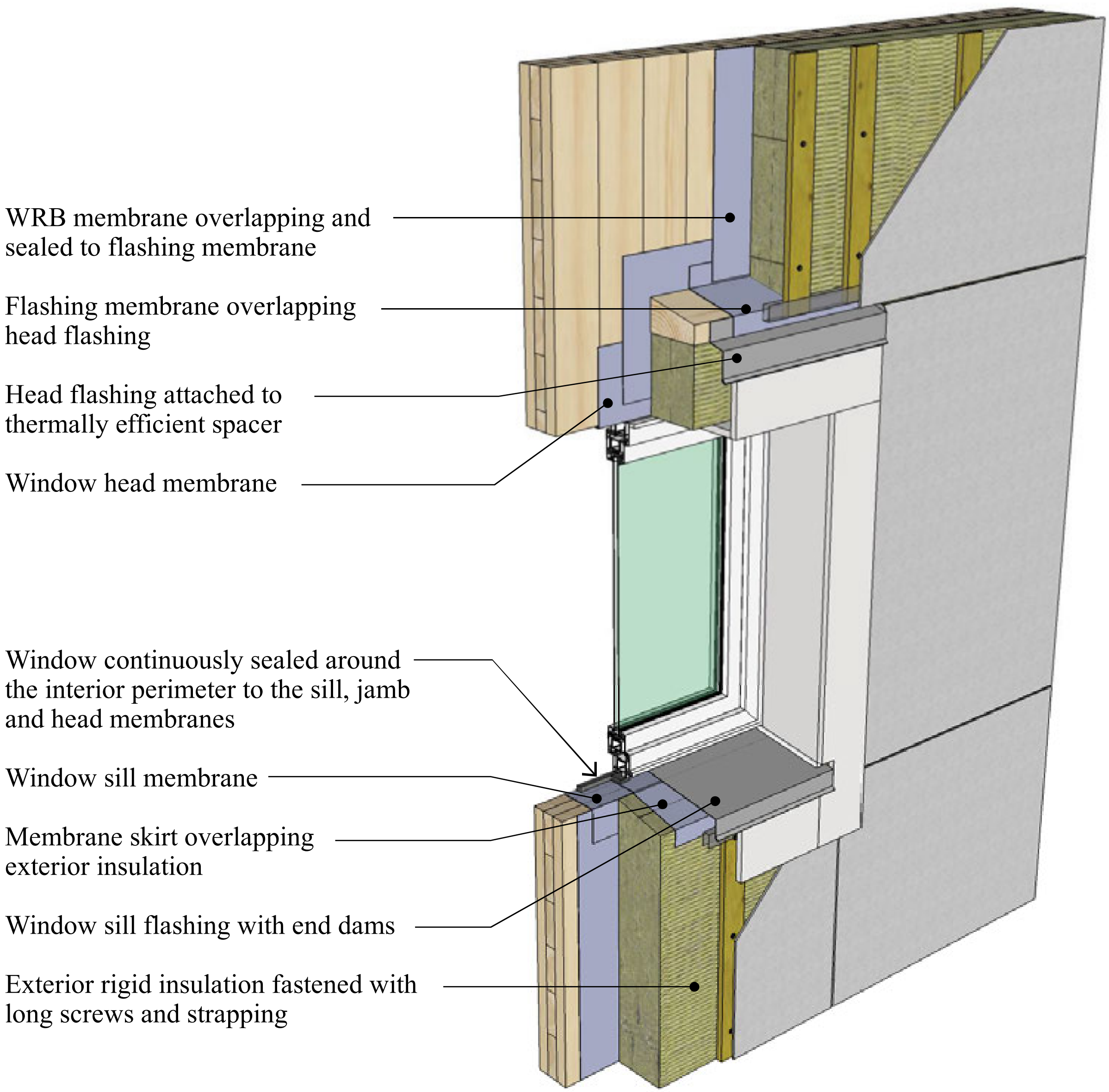


Figure 21 **Example window installation details for exterior-insulated CLT wall**

10.4.2 Roof Assemblies

Best practice for the design of low-slope insulated roof assemblies incorporating CLT panels is summarized in Figure 22 for a conventional insulated roof, and in Figure 23 for a protected membrane (i.e., inverted) insulated roof. The design of interior-insulated vented or unvented CLT roofs is not recommended, except in very unique circumstances of building use and is not covered in this guide. A conventional insulated roof has an exposed roof membrane and is common in applications where the roof is not accessible. An example of a protected membrane/inverted insulated roof is where extruded polystyrene insulation together with ballast or pavers sit on top of the membrane, protecting it from mechanical damage and exposure to solar radiation. This roof assembly is more durable, though tends to be more expensive to construct; it is also heavier due to the need for ballast to weigh down the insulation to prevent the wind uplift pressure from lifting the insulation and to prevent water from floating the insulation. Green or vegetated roofs tend to be applied over protected membrane assemblies for protection of the roof membrane from plant roots and also for ease of maintenance.

While much of the building science guidance for walls could also apply to roofs, since the physics is fundamentally the same with respect to heat flow, vapour flow, and air control, guidance for roofs is unique in that roofing membranes by their nature are very impermeable to water and vapour diffusion. Thus, most low-slope CLT roof assemblies will not allow moisture to dissipate to the exterior; as such, the only direction for drying is to the interior. CLT roof panels wetted from a roof leak will dry very slowly and this may lead to damage and decay, should the leak be persistent (Wang, 2014; 2018). It is therefore critical in the design of roof assemblies that robust roofing materials be utilized; redundancy is typically incorporated in CLT roof design through the use of additional waterproofing strategies, or installation of permanent leak detection and monitoring systems. Venting and drying strategies may also be used to help ensure the long-term performance of the roof assembly.

For example, as shown in Figure 22 to Figure 24, for both roof assemblies (i.e. conventional insulated and protected membrane), a 19-mm (¾") or larger gap/vented space is left between the CLT roof panel and the roof sheathing panel. The roof sheathing may be plywood, OSB, LVL or other wood panel product. This vented space is provided to add redundancy to the CLT structure in the event that the CLT is wetted in the construction stage or during in-service conditions, because of a small inadvertent water leak to the assembly. It also allows for ease of re-roofing without damaging the CLT panels later on, in the life of the roof assembly. Figure 24 shows two different venting options. Where venting is being considered, the temporary roof membrane located directly on top of the CLT is omitted or relocated on top of the sheathing to facilitate panel drying. Careful consideration should be given to omitting the temporary roof membrane based on project-specific climate conditions.

Given its massive nature and the ability of CLT structural sheathing to absorb moisture, the source of roof leaks may be difficult to identify, should a water leak occur through the roof membrane. For this reason, an active electronic leak detection system may be recommended depending on the design of the assembly, to help identify and locate leaks. This recommendation would be strengthened when a temporary roof membrane over the CLT is not provided or when a vegetative or green roof system is used. If a leak detection system is used, it should be located below the roof membrane or as recommended by the roof manufacturer.

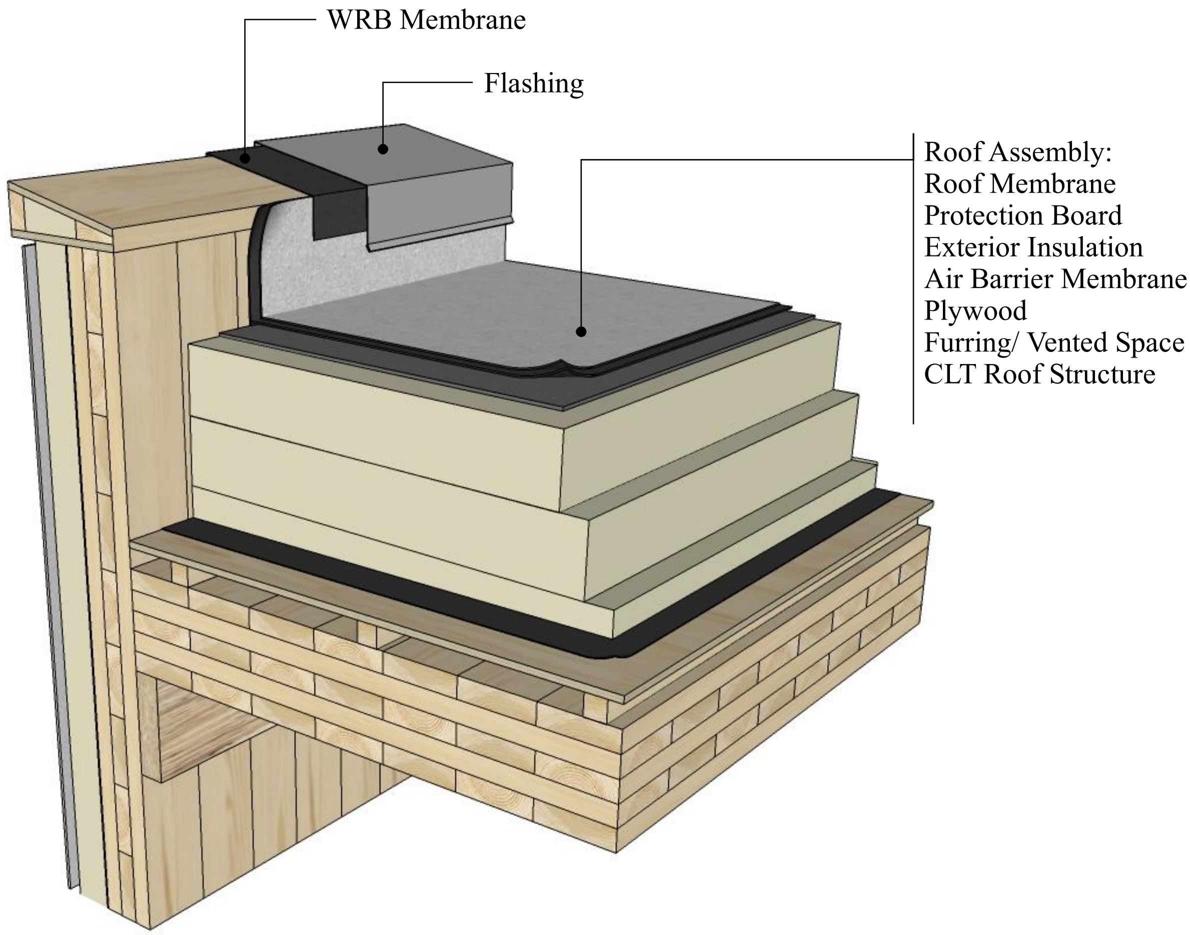


Figure 22 Conventional insulated CLT roof assembly. Conventional roof assembly over CLT with exposed membrane and rigid insulation. Note the incorporation of a vent space between the roof sheathing (structural diaphragm) and the CLT structure, which provides built-in redundancy and facilitates drying in the event of initially wetted CLT or a small leak. The vent space can also be used with a taper package to provide slope in lieu of tapered insulation or sloped CLT. A temporary roof membrane is applied over the sheathing which also serves as an air barrier and vapour barrier in service.

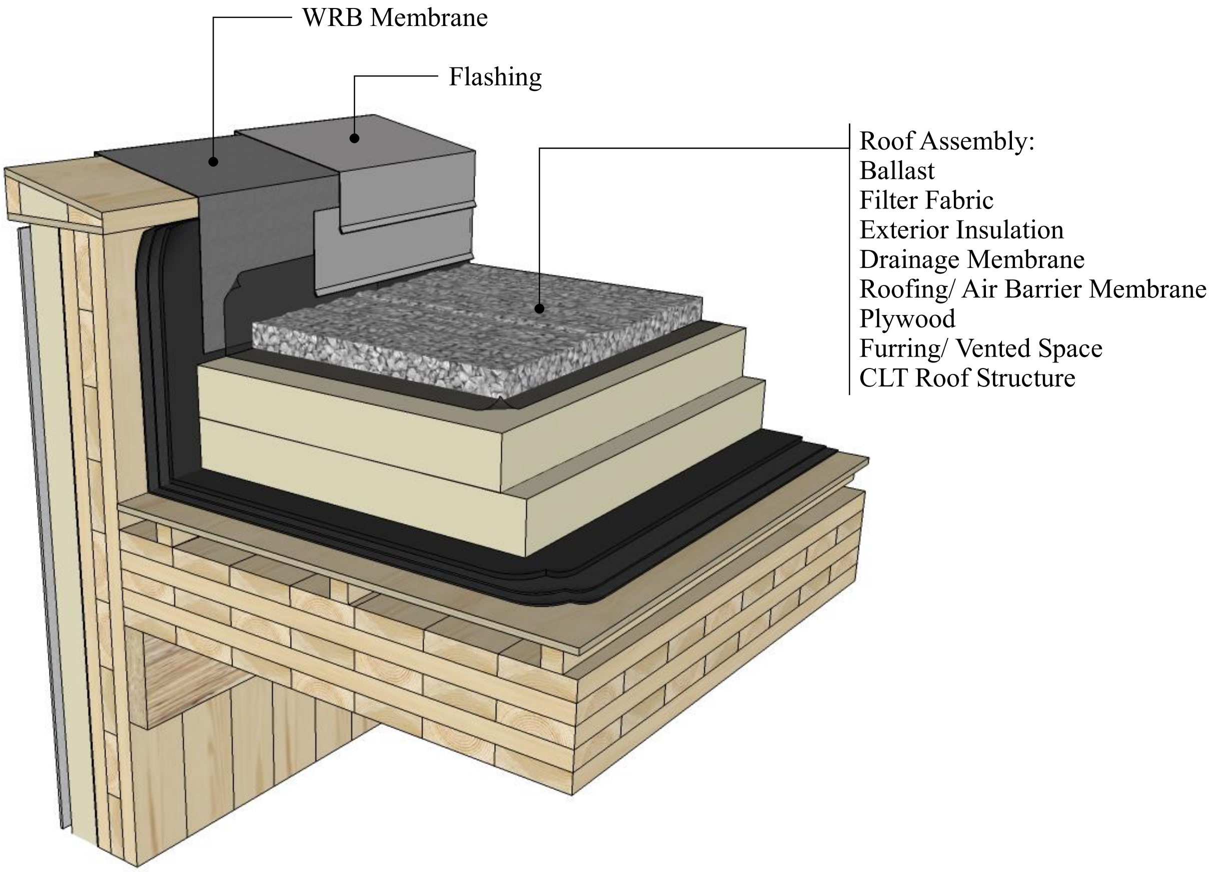


Figure 23 Protected membrane/inverted insulated CLT roof assembly. Protected membrane/inverted roof assembly over CLT with extruded polystyrene insulation and gravel ballast over a membrane applied to sheathing above the CLT. Note the use of a vent space between the roof sheathing (structural diaphragm) and the CLT structure, which provides built-in redundancy and facilitates drying in the event of initially wetted CLT or a small leak. The vent space also allows for the roof membrane to be sloped in lieu of sloping the CLT or incorporating a layer of tapered insulation below the membrane (modified inverted roof assembly). The base sheet of the roof membrane serves as a temporary roof membrane during construction and when complete is also the air barrier and vapour control layer in this assembly.

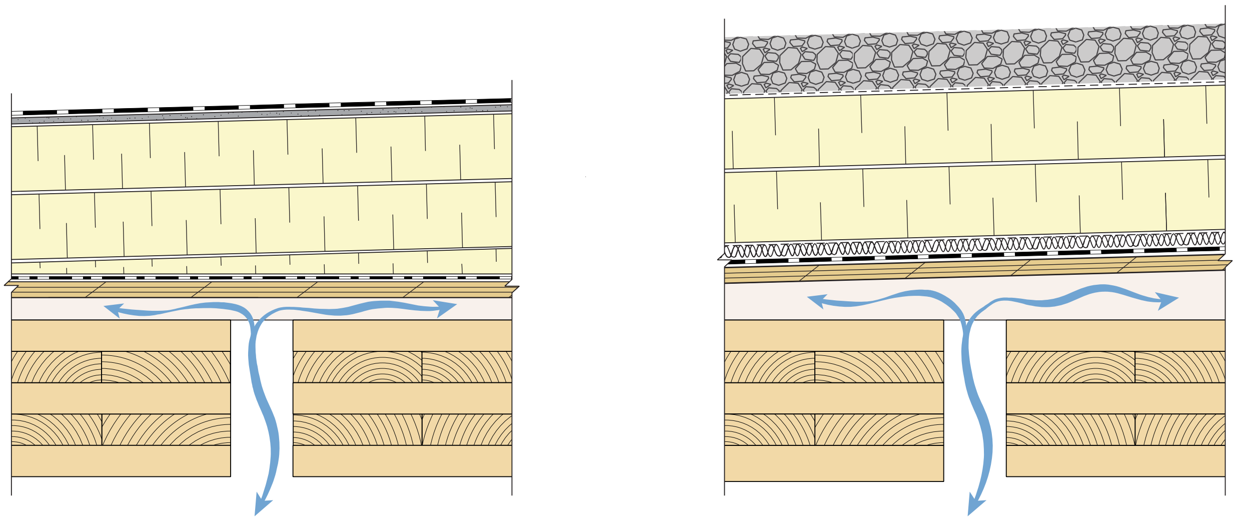


Figure 24 Interior venting options for conventional or protected membrane CLT roof assemblies. Where no intentional gaps are present between the CLT panels, some beneficial venting can still be expected to occur through butt joints and other penetrations.

However, other concerns need to be addressed in the design of such interior-vented roof assemblies. More specifically:

- Concerns about fire safety. In some instances, there may be a requirement that the air cavity be filled completely or partially with non-combustible insulation, which would negate the purpose of the vented cavity.
- The integration of a cavity requires the diaphragm to be robust and not compromise structural integrity.
- Requirements for perimeter attachment of the structural sheathing may obstruct the passage from the clear air cavity to the interior, thereby preventing any benefits as may be derived from venting the panel to the interior.
- While the amount of airflow within this gap may be small and have a negligible impact on thermal performance, some jurisdictions may not permit the inherent R-value of the CLT material to be included in calculation of the assembly effective R-value, when the air cavity is vented to the interior.
- In no cases should the cavity be inadvertently vented to the exterior, as this will negate the thermal insulation value and the airtightness and increase the chance for condensation within this concealed space.

Figure 25 Example implementation of a topside interior-vented CLT panel roof at the Wood Innovation and Design Centre in Prince George. (In the design of this roof, plywood sheathing needed for the structural diaphragm is separated from the CLT with strips of ¾” × 3½” plywood. As shown in the photos, this allowed for the roofing work to proceed, while some areas of the CLT were still drying from construction wetting/snow; it also allowed for drying of small construction leaks through defects in the temporary roof membrane).

Care must be taken during construction to keep CLT panels dry, so that roofing work is not delayed because of the need to induce drying of the panels. Self-adhered vapour impermeable barrier membranes applied to dry CLT or the sheathing panel above the CLT (often used as a temporary roof) can be used to protect the panels from direct wetting during construction and in service (Figure 26). Pre-application of temporary waterproofing membranes may be considered, as long as these membranes are immediately sealed on site (ideally torch-sealed with heat-welded joints at penetrations). The use of factory-applied coatings on the CLT may also help reduce water penetration in the short term. Further information on construction moisture management is provided in Section 10.5.

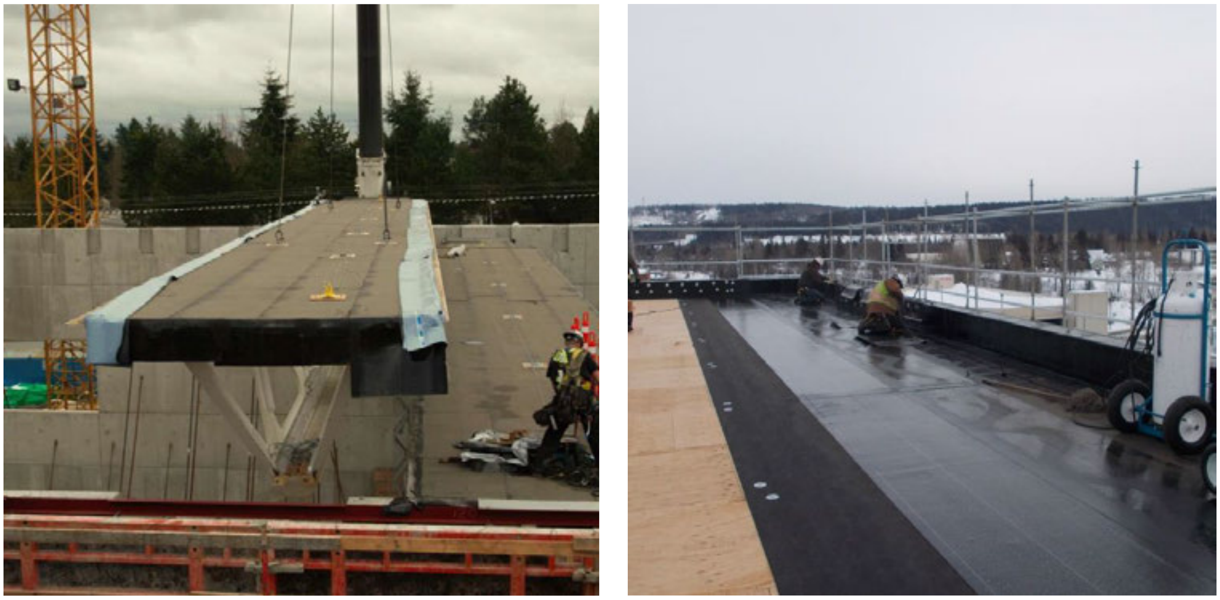


Figure 26 Temporary roof membrane pre-applied to mass timber roof panel with torch lap joints (left) and site-applied temporary roof membrane over CLT roof assembly (right)

10.5 CONSTRUCTION AND IN-SERVICE MOISTURE PROTECTION

10.5.1 Construction Moisture Management

In reviewing the construction of many CLT buildings built to date in North America and Europe, it was found that a number of different strategies have been utilized to protect CLT panels from getting too wet during construction. This includes delivering CLT panels just-in-time to the site, maximizing prefabrication to minimize on-site construction time and exposure to the elements, protecting CLT face and edge grain with factory-applied water-repellant coatings, pre-applying temporary or permanent water-resistant membranes, and using temporary hoarding or canopies during construction (Finch, 2016; Wang, 2016a).

As with other wood products, CLT panels should always be protected from exposure to rain, snow, and wet ground during the construction process. CLT panels are vulnerable to damage from wetting due to the nature of their laminated construction and because they are capable of absorbing a considerable amount of water through their exposed ends (e.g., in the longitudinal direction) and the gaps between the panel laminations. Absorption of water through the faces (i.e., in the transverse direction) is slower due to the nature of wood but will happen slowly when exposed to prolonged wetting. The on-site drying of CLT is possible using natural and/or mechanical means, though it can delay construction schedules and add unnecessary expense.

CLT panels are much more massive than plywood or standard dimension lumber and can take a longer time to dry out if water is allowed to penetrate deeply. Therefore, prevention of wetting during construction should be a priority to maintain scheduling. CLT product standards and building codes require that the moisture content of CLT be less than 16% to 19% at any location within a panel (surface, core, or edge), before it is closed in. Experience has shown that a moisture content of less than 16% is more desirable than the maximum of 19% usually allowed in light wood framing, due to the difficulty for moisture to dissipate, particularly under impermeable materials. In addition, it is important to keep the panels at a stable moisture content from construction into service, because moisture-related expansion and contraction may damage the laminations and lead to distortion of the panels. Cyclical wetting and drying can also be damaging to CLT, as repeated swelling and contraction can lead to the opening up of joints and seams, allowing water to penetrate past the surface, thus increasing the length of time required for the CLT to dry out before it can be covered; this can be a real scheduling issue with the application of roofing and flooring materials on horizontal panels. Figure 27 illustrates these wetting mechanism considerations for CLT panels during construction.

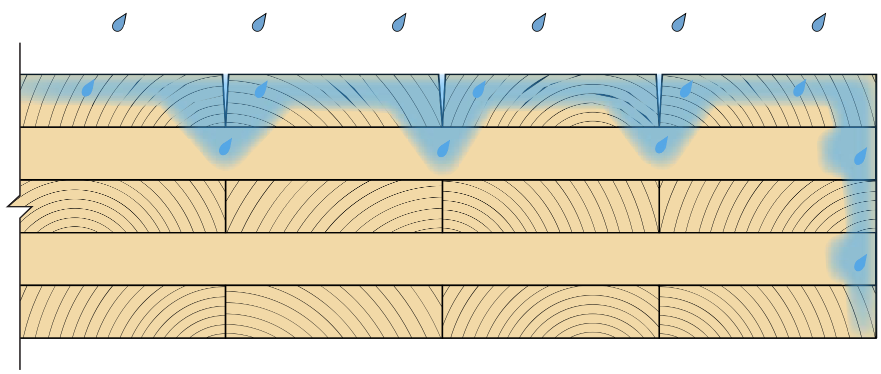


Figure 27 Wetting mechanisms of CLT panels during construction (The edges of CLT are the fastest to wet up during wetting events, though they can also dry quickly if exposed to air (i.e., away from panel joints). The top surface of CLT is also fairly resistant to wetting, though if significant gaps and checks are present in the laminations, then water can get down into lower laminations and be more difficult to dry out. The use of an edge grain sealer and some surface protection combined with a good on-site moisture management strategy can help minimize overall wetting of CLT during construction).

CLT panels can be temporarily protected by use of water-repellant coatings (especially on the edges), water-resistive sheet membranes, and/or other effective methods to reduce environmental moisture uptake until they are protected by the building roof. Temporary protection can be applied in the CLT manufacturing facility and should be maintained during shipping and on-site storage. This protection should also be maintained as the panels are erected in place to protect the panels until the roof or other elements such as the WRB provide adequate protection. The membrane or coating should also be selected considering construction traffic and how penetrations will be sealed on site. It is critical that liquid water not be allowed to bypass sheet membranes for example, as these membranes, whether vapour impermeable or permeable in nature, will significantly retard surface evaporation and drying rates.

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, as discussed in the previous section, will keep the CLT panel warm (i.e., exterior insulated) and allow for excess moisture to readily escape from the assembly (i.e., vapour open concept) to prevent damage and deterioration.

10.5.2 Construction Moisture Management Planning

The construction of CLT buildings requires careful planning to manage potential moisture risks, though when properly executed, it presents no more of a risk than building with other construction materials. CLT does need to be protected from excessive wetting so that the need for natural or mechanical drying does not impact the construction schedule. CLT that is closed in when too wet can be at risk for fungal growth and/or decay, which could lead to indoor air quality, serviceability, or structural concerns (Wang, 2016b). When these risks are mitigated properly by the construction process and design, then a long service life can be expected.

Figure 28 Construction of the structure and panelized building enclosure of the UBC Tallwood House (Brock Commons), an 18-storey mass timber building with CLT floors (Key to the moisture management plan of the CLT floors at this project was a factory- and site-applied water-repellant coating, in addition to a prefabricated exterior wall/window system. While construction occurred in what are typically drier months, significant rain events occurred during erection. Wetting was reduced by the CLT coatings, site surface water management, and fast enclosure. Challenges arose with the schedule when the CLT floor panels had to be dried prior to the application of concrete floor toppings. A combination of natural and mechanical drying was used. Moisture monitoring of the building found that while some areas of the CLT did get wet during construction, the moisture content rarely exceeded 20% at the wettest locations, with the majority of panels having a moisture content less than the conditions at fabrication (Mustapha et al., 2017; Wang and Thomas, 2016).

Experience with the construction of CLT buildings (Finch, 2016) has shown that it is critical to develop a construction moisture management plan early in the design development stage (an example shown in Figure 28). This plan does not need to be elaborate, though it is intended to document a thinking and decision process by the parties involved in the design and construction of the building. This helps to better understand how certain design decisions will potentially impact the need for different types of construction protection, how different assemblies can be dried out if inadvertently wetted, and whether certain designs may be challenging to construct during certain site conditions or times of the year. The following subsections document the process to consider when developing a moisture management plan, including some questions to be asked of the design and construction teams during the design phase.

The higher the section number below, the more involved and potentially costly the management plan; therefore, there is an advantage in focusing on the simpler early stage items discussed in the first few sections.

10.5.2.1 Design and Construction Risk Evaluation

The first step in developing a moisture management plan is to evaluate the risk of wetting for the CLT components utilized in the project. Consider the climate and timing of seasonal rainfall, and whether the construction schedule will potentially fall within this period. Assess the chances that the CLT components will get wet during construction or from rainfall or snow landing on the roof or floors above. Once this is determined, estimate a construction schedule and the length of time the various CLT roof, floor, and wall panels could potentially get wet.

The use of high levels of prefabrication to speed up the erection of CLT components, or of prefabricated, prefinished building enclosure components will impact the duration of potential wetting. The ability to install a roof quickly over the top of a CLT structure also has a significant bearing on the potential for wetting.

Experience has shown that horizontal CLT panels are fairly tolerant of a few weeks of rain in coastal climates and will safely dry out without harm, provided conditions for drying prevail afterwards. Wetting of horizontal CLT panels for several months with limited opportunity for drying will be a concern for construction scheduling, especially when the CLT needs to be encapsulated or covered. Vertical CLT elements are much more resistant to wetting, though protection of the edges is important, especially if the panels are in contact with a wet substrate such as at the base of the building. Snow and ice fortunately are easier to deal with and will not wet up CLT to a great extent, unless they melt before they can be swept or shoveled away.

With a basic risk evaluation in place as to the potential for CLT used on the project to get wet, any necessary solutions can be developed, as discussed in the following sections.

10.5.2.2 Risk Mitigation and Scheduling

The second step is to evaluate the impact that scheduling can have on the moisture risk and how accelerating installation of certain building components (such as windows or a roof) can help to mitigate the potential for the CLT to get wet and ensure it is kept below 16% moisture content prior to encapsulating.

One critical path scheduling item to consider includes roofing over CLT panels. As discussed in Section 10.4.2, the use of pre-applied temporary roof membranes can help to avoid roof scheduling issues over wet CLT, as long as the pre-applied membranes are well applied and detailed against leaks. Leaks under impermeable roof membranes will not be able to dry out without removing the membrane.

Another critical path scheduling item can be the application of acoustic membranes or concrete toppings to CLT floor systems. While interior CLT floors are not typically part of the building enclosure, they are exposed to wetting during the construction process until the roof and vertical enclosure components are installed and sealed. With respect to flooring, some self-leveling concrete screeds/gypcrete toppings contain an extra amount of moisture for the hydration reaction in the concrete and, therefore, may require the CLT panels below to be protected against the higher moisture level in this type of concrete (Figure 29). That being said, if the concrete topping has a low water to cement ratio, such as in a composite structure, or the CLT is very dry when the concrete is applied, then a membrane or coating may have little initial benefit (RDH, 2016; Wang, 2018).

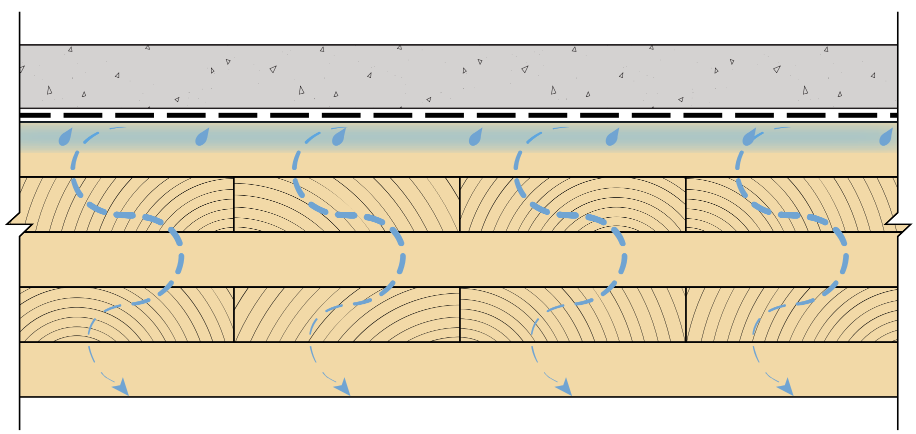


Figure 29 Protection of CLT against wet concrete topping using a membrane applied to the CLT. (If concrete toppings or impermeable floor finishes are applied to wet CLT, drying through the CLT is very slow and it can take months to years for the CLT to fully dry out. Therefore, moisture levels in the CLT should be below a maximum of ~16%, prior to encapsulating. The use of vapour semi-permeable or impermeable coatings or membranes applied to dry CLT may help mitigate the uptake of moisture from self-leveling floor concrete/gypcrete and from short-term flooding events).

10.5.2.3 Considerations for Factory and Early Site Protection of CLT Panels

The third step in moisture management planning is to consider the use of factory- or site-applied temporary or permanent moisture protection for the CLT panels. The need for protection is enhanced for horizontal CLT applications, in particular in roofing applications, but even for floors where the panels may be exposed to rainwater for extended periods of time and deemed at risk in steps 1 and 2 above (Figure 30).

Figure 30 Surface wetting of CLT floors from a rainstorm and subsequent wetting of the cut end grain at all panel joints and openings

One of the simplest methods to minimize the uptake of moisture into CLT panels during construction is to incorporate a factory-applied water-repellant coating, although their effectiveness varies and may not be relied on entirely, particularly under prolonged wetting conditions (RDH, 2016; Wang, 2018). The most important parts of a CLT panel that can benefit from factory protection are the edges, where the end grain of the wood is exposed. The most effective coatings for stopping moisture migration into the end grain are available from the log-home and glulam industries and are typically a high-build paraffin-based product. This should be applied to all edges including interior cut outs and cored holes etc. The exposed topside surface of horizontal CLT floor and roof panels may also benefit from the pre-application of a water-repellant coating to slow down uptake of water into the CLT, and potentially beyond any voids or checks between laminations. The best coatings for this application are highly water-repellant penetrating coatings or surface coatings that have a moderate permeability, to allow for drying after wetting events. In addition to protection of the CLT, consideration should be given to the protection of the panel spline joints, if utilized. For example, plywood splines will absorb a much larger amount of moisture than CLT panels and can be more difficult to dry out when severely wetted. Figure 31 summarizes the factory or site protection of CLT floors below a concrete floor topping.

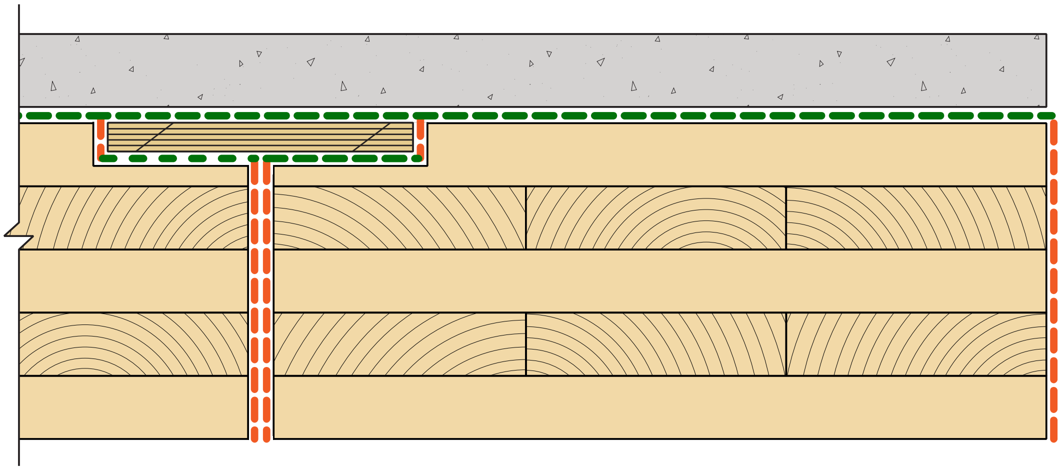


Figure 31 Factory- or field-applied coating strategy for CLT floor panels below a concrete topping includes a coating on the top surface to protect from the concrete topping moisture, the edge/end grain to protect from water running through during construction and at the panel spline joints

Figure 32 Field-applied semi-permeable water-repellant coating applied to CLT floor assembly to protect the CLT from possible excessive wetting due to the application of a self-leveling concrete topping and future wetting events. This type of application is less desirable that a factory coating due to scheduling but may be necessary if the CLT supplier cannot apply a coating within the constraints of their factory

As an alternate to liquid-applied coatings, a range of temporarily applied or permanently applied sheet membranes (which ideally later perform a building enclosure function) can also be used; these can be applied to CLT panels in the factory or on-site. Sheet membranes have an added benefit in that they also span over the many gaps of the CLT laminations and address issues with water penetration deeper into the CLT panels, as discussed in Figure 27. In terms of selection guidance for CLT applications, several types of membranes can be used that can be classified into the following four groups, based on their vapour permeance and whether they are mechanically attached or self-adhered.

- ***Vapour permeable, mechanically attached (temporary or permanent)*** – Products such as vapour permeable house-wraps or lumber-wraps can provide some degree of protection from rainwater in the short term, especially during shipping and on-site storage. The wraps are stapled in place and the laps, joints, and penetrations should be sealed with tape, membrane, or sealant. While cost-effective, the majority of these products are not resistant to ponding water for extended periods or after being walked on, and most are easily damaged by construction activities. Many membranes are also slippery, so the surface texture should be carefully considered for site safety. Mechanically attached membranes are challenging to seal effectively around penetrations and panel joints and can be easily damaged, allowing water to bypass the surface. Even if vapour permeable, these membranes will slow the drying of any water that gets beneath them, relative to exposed CLT. Because of these challenges, these products are only recommended to protect CLT panels during shipping and short periods of exposure on-site. They are more suitable for walls than for floors with high traffic or roof panels.

- ***Vapour permeable, self-adhered (permanent)*** – Products such as vapour permeable self-adhered house-wraps bonded to the CLT can provide an improved level of protection from rainwater and also tend to be more resilient to construction damage. The membranes are self-adhered, and the laps and joints are self-adhered and/or taped. These membranes are more expensive than mechanically attached sheets but can make for good pre-applied permanent protection. The majority of these products have limited long-term resistance to ponding water and will degrade after being walked on and exposed to rain and dirt. Most of these membranes have a texture, though care should be taken to select one that is suitable to walk on. Self-adhered as compared to mechanically attached sheets are easier to seal around penetrations and at panel joints but require special tapes and adhesives for effective seals. Water that gets beneath these membranes at laps or panel edges will be slower to dry out than water in exposed CLT. This type of membrane is typically recommended for permanent CLT wall assembly air barrier and water-resistive barrier functions and may already be in use on-site. These membranes are recommended to protect prefabricated CLT wall panels, though they can also be used in floor or roof panel applications to reduce wetting.

- ***Vapour impermeable, mechanically attached (temporary or permanent)*** – Products such as vapour impermeable polyethylene or other plastic sheets/tarps can be used to provide protection from rainwater in short-term applications. These plastic membranes are stapled in place and the laps, joints, and penetrations are taped or sealed. While very cost-effective and resistant to ponding water, these membranes are easily damaged by construction activities and very slippery to walk on, posing a safety risk. Loose sheet membranes are challenging to seal effectively around penetrations and panel joints and can be easily damaged, allowing water to bypass the surface and travel underneath the loose membrane. Water that gets beneath these membranes is a significant risk, as the vapour impermeable membrane prevents any outward drying from the CLT. The only method of drying the CLT is then to completely remove the membrane. As there is only limited use for these membranes in permanent building enclosure applications, they are often only used temporarily for floor or roof applications, or just for on-site storage.

- ***Vapour impermeable, self-adhered (permanent)*** – Products such as vapour impermeable peel-and-stick or roofing membranes bonded to the CLT provide an improved level of protection from rainwater and also tend to be reasonably resilient to construction damage. The membranes are self-adhering and the laps, joints, and penetrations are torched, welded, self-adhered and/or taped. Torched or welded joints are preferred where standing water is expected. These membranes are more expensive than mechanically attached sheets but can make for good pre-applied permanent protection. These products tend to have inherently good resistance to ponding water though they can still be damaged by construction activities and foot traffic. Many suitable roofing membranes have a texture, though care should still be taken to select one that is suitable to walk on. Self-adhering, as compared to mechanically attached sheets are easier to seal around penetrations and at panel joints, and also limit the risk for water to travel beneath the membrane. Water that gets beneath these membranes is a significant risk, as the vapour impermeable membrane prevents any outward drying from the CLT. The only method of drying the CLT is then to completely remove the membrane. Therefore, the use of robust, well-sealed membranes coupled with an appropriate CLT edge seal is recommended. These membranes are often recommended to protect CLT roof panels (as discussed in Section 10.3.2), as the membrane can also serve as a temporary construction roof and later as an air/vapour barrier.

The use of factory-applied coatings or membranes is an important design decision and the specification or requirements for coatings, as determined by the site team, should be documented in the CLT performance requirement specifications. Permanent coatings or self-adhered membranes are often preferred, as they serve dual duty in construction and in service and reduce construction wastage. Ideally, testing should be conducted for the specific applications conditions to ensure the measures will be effective.

10.5.2.4 CLT Panel Joints and Penetrations

The fourth step after evaluating temporary or permanent moisture protection membranes for CLT is to consider how all the joints and penetrations of the membranes will be sealed on-site to protect the CLT and the rest of the building beneath.

As discussed for the four different classifications of membrane type in the previous section, joints within these membranes typically consist of sealed tape, membrane, sealant, or heat-welded connections. These joints maintain the continuity of the water-deflecting elements to protect the CLT from wetting. This is critically important to consider, as a membrane with poorly sealed and leaky joints will be potentially pose more risk to the CLT than a bare CLT panel with no membrane at all. Detailing and sealing of these joints, including sequencing, should be part of the moisture management plan.

With bare or factory-coated CLT panel-to-panel joints, the spline joints deserve extra attention and should be part of the moisture management plan, especially for floor and roof panels. Spline connections for CLT typically consist of plywood or OSB, though plywood is preferred structurally, since it typically swells less than OSB when wetted. These CLT panel joints are sensitive to wetting and can be slower to dry after severe wetting in this concealed space, especially if the end grain of the CLT panels is not factory-coated. Taping of the spline connection is only moderately effective at preventing water penetration through the CLT. This is due to the presence of lamination gaps in CLT and of paths for water to flow into and around sealed spline joints, which exist at almost all penetrations (illustrated in Figures 33 and 34). For this reason, where bare or factory-coated CLT is used as roof or floor panels without a supplemental membrane, water penetration should be expected through the panels and onto the floors below until the building is completely closed in. This may be problematic if gypsum or other moisture-sensitive components are being installed below.

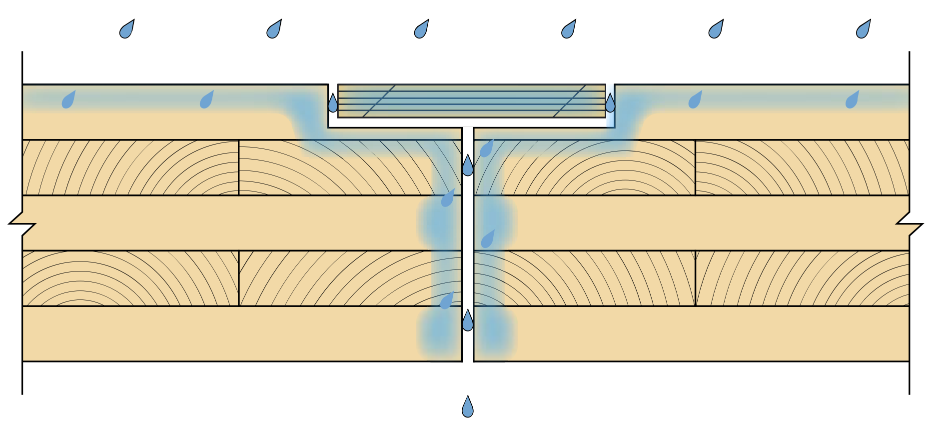


Figure 33 CLT panel joint and potential for wetting at end grain of the panels, and at the spline joint. (The taping of splines alone is not completely effective at preventing the penetration of water through this CLT joint and should not be considered to be watertight during construction. Where 100% water tightness is necessary, e.g., to install moisture sensitive gypsum below, a membrane or topping on the entire CLT floor is necessary).

Figure 34 Typical CLT panel joints and various practically un-sealable penetrations that will allow for water penetration through the gaps and checks in the CLT, to the floor below

10.5.2.5 Tarping, Hoarding and Other Protection Systems

The fifth step in the moisture management strategy is to consider the need for and feasibility of temporary or permanent tarping, hoarding, or tenting of the CLT building and to weigh costs and risks compared to the previously discussed passive protection measures. The design of the building enclosure and the use of prefabricated wall and roof elements to quickly close in a CLT structure may also be considered in this evaluation.

The erection of temporary moisture protection structures including hoarding or tents (Figure 35) does come at a significant cost and also impacts the construction schedule, though if planned and budgeted from early on, it can potentially offset the need for most of the other CLT protection measures described earlier. That being said, having to add an unplanned temporary moisture protection system during construction, e.g., to allow for a wetted CLT structure to dry out, not only delays the schedule but also adds a significant cost. A critical risk evaluation should therefore be performed early in the project as to whether tenting is possible or not for the site, and how the cost compares against other alternate protection measures. Experience shows that most mass timber construction teams have been forced into adding roof protection to dry out wetted mass timber and that this had not been planned from the outset. While the best outcome of this step is potentially to design out the need for this additional protection, incorporating hoarding and roof protection as part of the erection strategy may also allow for year-round construction using CLT in challenging conditions, such as cold climates.

Figure 35 Examples of whole roof scaffolding tents used in these instances to protect mass timber roofs. Similar measures can allow for wetted mass timber roofs to dry out by natural or mechanical means

10.5.2.6 Active Site Water Management Strategy

The sixth step in the development of a moisture management strategy is to develop a plan for managing site water, if and when it occurs. This plan is heavily influenced by decisions and design choices made previously in steps 1 through 5, and the wetting risk posed to the CLT in question. For example, if a permanent membrane is applied to the CLT roof panels or the building is covered by a large tent, then few moisture management activities may be needed to protect the CLT. In contrast, if it is planned that construction in the summer is to proceed without protection of the CLT components, and the summer is indeed beset with repeated rain events, then the project will be delayed, to ensure either the installation of protection or the drying out of the CLT components. Clearly, a more elaborate site water management strategy is necessary and will provide opportunities to protect the CLT during installation.

Several passive CLT moisture protection measures were previously discussed up to and including covering the building to protect it from rain. This current step in the management plan focuses on active measures that may or may not need to be implemented prior to, during or after rain events, when the CLT has been wetted. While the planning for these measures may use a team approach, accountability should rest with the general contractor or their delegated subcontractor, given their direct control over the construction site.

This sixth strategy involves actively monitoring and planning for rainfall events and being familiar with the local weather patterns. From this information, one can devise a means to manage the expected wetting of the various CLT panels, given the occurrence of rain events on-site, and if and when rainfall enters in contact with the various CLT panels.

As CLT floor and roof panels tend to be installed flat, drainage during rainfall events relies on displacement to the perimeter, which is a slow process compared to gravity-driven drainage in a sloped application. Therefore, active strategies may be necessary to direct water away from the CLT surfaces during extreme rain events, to reduce moisture uptake into the CLT. Active management strategies may also include sweeping, squeegeeing, or vacuuming surface water from CLT panels or diverting water from the base of panels at grade. To collect rainwater from the surfaces of CLTs and drain it safely away from other CLT components, temporary drains and rainwater leaders may also be needed (see Figure 36, for example). Water-repellant coatings can help reduce the rate of water absorption into the CLT panels before water can be removed by other means.

Figure 36 Example use of temporary floor drains and rainwater leaders with a waterproofed concrete floor topping to limit the amount of water cascading down this high-rise CLT structure under construction. (The use of more than one moisture protection regime in multi-storey CLT buildings provides for redundancy and protection of the many floors below, in case one fails to perform adequately).

10.5.2.7 CLT Drying Mechanisms

The seventh and final step in the moisture management plan is to develop a plan for the drying of CLT panels when and if the need occurs on-site. This step may never be implemented, though it is helpful to have a plan should the need arise. The plan may incorporate either natural or mechanical methods of drying or both may be implemented together, depending on the situation and environmental conditions. It is important to consider the rate of moisture dissipation, given that a wetted CLT cannot be dried rapidly; in addition, the drying of wetted CLT panels typically ought to take into consideration both scheduling and budgeting issues, as may arise on-site. Prolonged periods of wetting or cyclical and repeated wetting and drying events can cause delamination and distortion of the CLT that, consequently, may degrade its performance. Therefore, the need for providing drying should be weighed against the six earlier measures of this plan.

In most scenarios, drying of the CLT following a wetting event can proceed by natural means, through evaporation from the exposed surface of the CLT to the surrounding air. The rate of moisture dissipation from the CLT will depend on the degree of wetness of the CLT (i.e. moisture content), the surface temperature, air temperature and relative humidity, and wind speed. Higher temperatures, lower relative humidity levels, and higher wind speeds are all more conducive to drying of the wetted CLT. Solar radiation significantly heightens the drying rate, as it increases the surface temperature, which increases the rate of evaporation from their surface. However, in coastal climates, persistent rainfall during the winter months limits periods for natural drying and mechanical methods must then be employed to dry out the CLT surface. Mechanical methods include the use of fans with or without the addition of heat that provides for dry and desiccated air to enhance the evaporation process. To avoid causing damage to the CLT when using mechanical methods, the rate of drying should be controlled by adjusting the different heating and air movement variables, although this approach should be performed by a contractor experienced in drying out wood buildings, as to not damage the CLT.

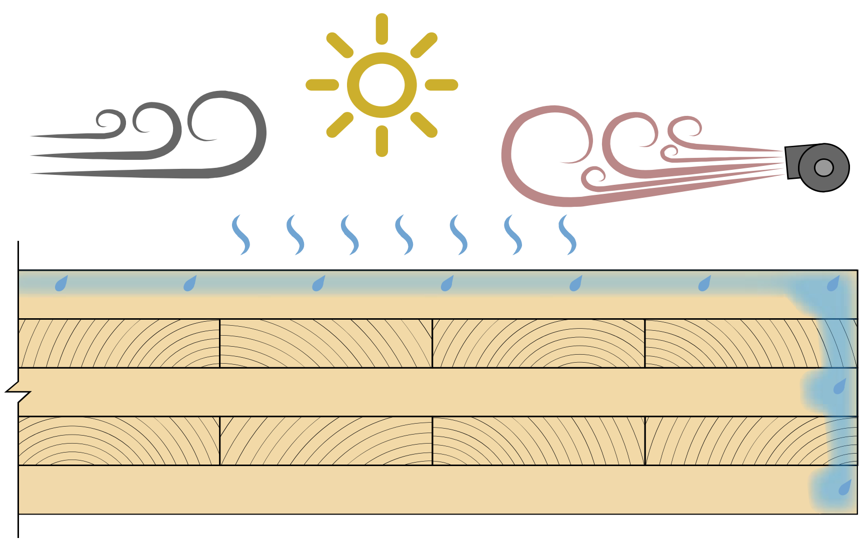


Figure 37 Drying mechanisms for wetted CLT include wind, sun, temperature and heated/dried air by mechanical means

10.5.3 Applications for Wood Treatment

In most wall, roof, and floor applications, CLT panels do not need to be constructed of treated wood laminations, and the panels need not be treated with wood preservatives. Experience across the industry has shown that for the majority of CLT building enclosure and floor designs, untreated wood has an acceptable level of performance with respect to durability. That being said, there may be design applications, certain building enclosure designs, or construction scheduling pressures where the use of a preservative-treated CLT may be beneficial. In wet or more humid climates, or where termites are prevalent, CLT panels (especially any exposed portions of the panels and parts in contact with foundations) will benefit from wood preservative treatment, such as borate or copper-based preservatives. While best practice construction and design strategies attempt to minimize exposure of the wood panels to wetting, it is inevitable that portions of some of the CLT panels may be exposed to moisture during their lifetime. The additional factor of safety provided by wood preservatives can be beneficial to durability in certain applications.

In terms of treatment, preservatives used for treatment of lamina prior to the 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 the boards on the exterior lamination, applying post-lamination surface treatments to the exterior and end grains, or using boron rods for local protection may all help. The requirements for wood preservative treatment are provided in the CSA O80 standard (CSA, 2015). Note that CSA O86 (8.3.3.) does explicitly forbid CLT to be pressure-treated with water-borne preservatives after gluing, and therefore, the use of post-treatments should be carefully evaluated by the manufacturer of the CLT and the engineer for the project.

In areas with a high termite hazard (e.g., the Southeastern United States), multiple lines of defense should be taken 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, measures for termite control should also be provided to below-grade insulation materials.

The use of fire retardants may help meet fire safety requirements and allow the use of exposed CLT panels for aesthetic purposes. Some fire retardants contain boron and will also provide decay and termite resistance. Again, with respect to CLT treatments, CSA O86 (8.3.3) states that “for CLT treated with fire-retardant or other potentially strength reducing chemicals, strength and stiffness can be based on documented results of tests that shall take into account the effects of time, temperature and moisture content”.

10.5.4 Management of Interior Water Leaks

Interior water leaks may occur in building operation due to, for example, leaks of water pipes and activation of sprinkler systems. Although floors and interior walls are relatively protected from ingress of rainwater during the building's service life, they are typically the most affected when there is a plumbing failure or an accidental activation of the sprinkler system. No large differences exist in dealing with an interior water leak among CLT, other mass timber, light wood-frame, or even light steel-frame assemblies. Most floors have concrete topping covering the structural members (e.g., CLT, plywood, OSB) with finishing materials (e.g., flooring, carpet etc.) on the top. Most interior walls are covered with drywall, except for exposed bare CLT, where permitted. When a leak occurs, bulk water should be removed (e.g. by vacuum) as quickly as possible to minimize wetting exposure time. Remaining small amounts of moisture may be removed by heating and ventilation, such as by blowing hot air directly towards the wet areas to hasten the evaporation of moisture. Non-structural components, such as drywall, batt insulation inside framed walls or floors, carpet, and wood or vinyl flooring may need to be removed to accelerate drying. Many interior components, such as drywall, carpet, and wood may develop mould under damp conditions and that may thereafter affect the indoor air quality. Warm environments enhance the growth of fungi, and prolonged wetting may even allow decay to start in wood members. This may compromise the structural integrity of the structure and make rehabilitation a more complicated and challenging process. For the floors and walls of indoor damp spaces, such as those that may be found in a bathroom, special measures should be taken to resist the higher indoor moisture loads. For example, when CLT is used for floors, these should be water-proofed to provide extra protection against water uptake. The installation of an additional interior vapour barrier may become necessary to reduce vapour diffusion into CLT walls.

10.6 CONCLUDING REMARKS

It is intended that these guidelines should assist practitioners in designing CLT building enclosures for Canadian climates, to ensure long-term durability and energy efficiency. However, these guidelines are not intended to substitute for the input of a professional building scientist.

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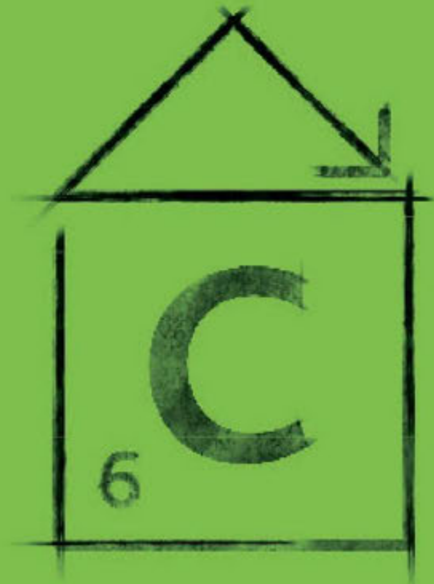

CHAPTER 11

Environmental performance of cross-laminated timber

Lead Author
Blane Grann

Co-Authors
Alpha Barry
Diane Schorr

Reviewers
Dave Patterson
Lal Mahalle
Antje Wahl
Helen Goodland

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GLOSSARY

acidification potential

The deposition and accumulation of acidic substances in the environment contributes to acidification, which can affect buildings (e.g. corrosion), and the productivity and diversity of ecosystems.

albedo

Albedo describes the fraction of the sun's energy reflected by an object. Designers may be familiar with the term in the context of urban heat island effect. However, at northern latitudes, there is a potential cooling "landscape albedo" effect (negative global warming potential) related to forests that have been recently disturbed which are more reflective in the winter as a result of seasonal snow cover.

biomass

Material of biological origin, excluding material embedded in geological formations and material transformed to fossilized material (ISO 14067, 2019).

biogenic carbon

Carbon derived from biomass (ISO 14067, 2019).

concrete carbonation

Carbonation is a chemical reaction in which CO_2 reacts with cementitious material to produce calcium carbonate.

embodied carbon

The embodied carbon of a building is the total life cycle greenhouse gas emissions generated to construct the building. This includes emissions caused by extraction, manufacture, transportation and assembly of every product and element in the building. A life cycle assessment reporting results for the single indicator global warming potential is the approach to quantify embodied carbon for a building.

environmental product declaration (EPD)

An environmental product declaration **(**EPD) is a document that reports a set of environmental impact data for a product based on a life cycle assessment that has been conducted in compliance with ISO standards (ISO 14025). An EPD includes information about the environmental impacts such as GWP and smog potential. EPDs may be used by designers on a product-by-product basis to compare a range of environmental impacts.

eutrophication potential

Nutrient enrichment of aquatic ecosystems contributes to eutrophication that can cause algal blooms which can alter species diversity in the affected area, and which can also produce toxins that have human health effects (e.g. shellfish poisoning) (Knockaert, 2014).

fuel substitution

Combustible waste materials are used to produce energy at the end of life, which avoids the use of other energy sources when compared to landfilling.

global warming potential (GWP)

GWP is a measure of how much radiative energy a greenhouse gas traps in the atmosphere over a specific time horizon, relative to carbon dioxide. GWP was developed to allow comparisons of the global warming impacts of different greenhouse gases. Specifically, it is a measure of how much radiative energy the emissions of 1 ton of a gas to the atmosphere will absorb over a given period of time, relative to the emissions of 1 ton of carbon dioxide.[1]

LEED

Leadership in Energy and Environmental Design (LEED) is the dominant voluntary green building rating system in North America and is also used extensively around the world. Certification is on a scale, ranging from Certified, Silver, and Gold, to Platinum at the highest level, and is based on the total points achieved.[2]

life cycle assessment (LCA)

Life cycle assessment is a standardized framework (ISO 14040, 2006) for quantifying the potential environmental impacts of a product system (such as a whole building) on air, land, and water over its entire life cycle, from resource extraction to its end-of-life disposition. Examples of environmental indicators reported by LCA include global warming potential, smog potential and ozone depletion potential.

mass balance

A mass balance is an application of the principle of conservation of mass in physical systems. A mass balance in the forest sector accounts for key flows of wood from the forest to industries (wood input and outputs) including waste and residuals. A mass balance can be used to validate input and outputs of a system.

[1] www.epa.gov/ghgemissions/understanding-global-warming-potentials
[2] www.cagbc.org

particulate matter (PM) criteria air pollutants

Particulate matter (PM) and other criteria air pollutants are linked to increased incidence of cardiovascular disease (Gakidou et al., 2017).

ozone depletion potential

Ozone in the stratosphere provides a protective layer that helps block damaging radiation from reaching the earth's surface. Ozone depleting substances can contribute to a reduction of this layer, which can have implications for human health (e.g. skin cancer and cataracts), and plant health (including agricultural crops) (US EPA, 2015; USDA, 2016).

smog potential

Photochemical smog refers to ground-level ozone, a form of air pollution that is known to contribute to lung disease (Gakidou et al., 2017).

ABSTRACT

This Chapter evaluates several important dimensions of the environmental performance of CLT. Section 1 presents results from a life cycle assessment study comparing a 4-storey CLT apartment building to a functionally equivalent building with concrete slab and column structure and light gauge steel stud walls. Results from this comparison show that the CLT building provides a reduction in life cycle greenhouse gas emissions, a finding that is consistent with results from other case studies. Section 1 concludes with an overview of the climate effects of wood use that are not always addressed in current LCA practice.

Section 2 addresses the topic of wood availability in Canada. It highlights the gap between the annual allowable cut and current harvesting levels, and the wide availability of certified wood products to consumers of Canadian wood products. Section 2 also uses a mass balance framework to identify the potential effects of expanding CLT production. This framework can be used to explore some of the indirect effects of expanding the production of CLT. It shows that wood supply for an increase in CLT production in Canada can come from several sources: the forest, other wood product markets, and critically resource efficiency in forest management, wood processing, manufacturing, consumption, reuse, and recycling.

Finally, Section 3 summarizes results for indoor air emissions from CLT samples. It shows that CLT panels can easily achieve the most stringent indoor air quality standards.

11.1 INTRODUCTION

Exploring the environmental performance of CLT requires that we examine how CLT affects environmental problems that are driven by pressures from expanding global population and consumption.

A few of the important dimensions of global environmental change include climate change, air pollution, water pollution, land use change, biodiversity loss, and an alteration in (or disruption to) ecosystem services. In this Chapter, we will:

1. Examine climate change and several air and water quality aspects of CLT use by presenting results from a life cycle assessment case study which compares a CLT building to a functionally equivalent building with a concrete structure and light gauge steel stud walls,
2. Discuss the availability of wood supply in Canada, and
3. Discuss results for common indoor air quality metrics for CLT.

Presenting this information helps to demonstrate the relevant merits of CLT in addressing sustainability objectives.

This Chapter deals specifically with the environmental performance of CLT. For information related to design and construction applications of CLT and mass timber systems generally in the context of 1) reducing the impacts of buildings (and building products) on the environment, and 2) improving occupant health and well-being in buildings, refer to Chapter 3 in the Technical Guide for the Design and Construction of Tall Wood Buildings in Canada (Karacabeyli & Lum, 2014, and the forthcoming 2020 editions).

11.2 COMPARATIVE LIFE CYCLE ASSESSMENT STUDY

The methodology that is used by scientists and industry to understand and objectively compare the environmental properties of CLT is life cycle assessment (LCA). LCA is a framework guided by international standards (ISO 14040, 2006; ISO 14044, 2006), to evaluate the environmental performance (e.g., global warming potential, acidification potential, and smog potential) of a product or service throughout its life cycle (e.g. manufacture, use, disposal)[3].

Manufacturers can use LCA to identify environmental hot spots in the life cycle of their products and pursue strategies to reduce these impacts. In addition, designers can perform an LCA for whole buildings and compare the impact of different material decisions (EN 15978, 2011; ISO 21931-1, 2010). The predominant green building rating systems (such as LEED) and model codes in North America recognize and encourage the use of LCA in building design and materials selection and

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[3] A broader explanation can be found at: www.lifecycleinitiative.org/starting-life-cycle-thinking/life-cycle-approaches/environmental-lca

guidance to help support this is available from Athena Sustainable Materials Institute (Bowick, O'Connor & Meil, 2014). Details on the state of the technical infrastructure for LCA can be found in Appendix 2 of Embodied Carbon of Buildings and Infrastructure: International Policy Review (Zizzo et al., 2017).

The aim of this Section is to demonstrate the environmental properties and benefits of CLT. It summarizes findings reported in an ISO 14044 (2006) compliant LCA study (Grann, 2013), comparing a CLT building with a similar building designed with a **c**oncrete **s**lab and column structure, and light gauge **s**teel stud **w**alls (*CSSW*). It should be noted that whole-building LCA is an evolving field. The terminology and considerations used in the following LCA analysis are considered emerging practice today and may not yet be in general use within the codes and standards familiar to the design and construction community. Additional information on the data and methods can be found in the original report (Grann, 2013).

Two general LCA approaches are used based on whether one is interested in a) the fraction of total environmental burdens linked to a product system where normative rules are used to partition burdens between multiple products[4] (attributional LCA), or b) the environmental effects of a decision (e.g., a policy or purchasing decision) (Sonnemann et al., 2011).

A common *functional unit* is defined in Table 1 to provide a basis for comparing the buildings. A brief description of the building products and equipment included in the assessment is summarized in Table 2, and the life cycle stages that were evaluated are depicted in Figure 1. Building elements that were not included in the assessment are assumed to be equivalent for the two buildings. Three end-of-life scenarios were considered in the assessment: 1) sending waste to landfill, 2) reuse of 50% of the CLT panels and sending the remainder of the waste to landfill, and 3) energy recovery for combustible waste.

Table 1 Functional equivalent unit used for the building comparison

Building type	4 storey multi-unit residential building
Floor area	Gross floor area: 4060 m^2 (including underground parkade) Heated floor area: 3270 m^2
Technical and functional requirements	The National Building Code of Canada (National Research Council of Canada, 2005) Comparable acoustic and fire ratings of assemblies and equivalent effective R-values for the roof, exterior walls, and main level floor
Assumed service life	60 years
Year of construction	2013

[4] For example, a sawmill produces chips, sawdust and lumber. Revenue from each product can be used to attribute burdens from operating the sawmill across these three products.

Table 2 Building products and equipment

Building Element	CLT	CSSW
Exterior Walls	105-mm CLT	6" light gauge steel studs
Floors	245-mm / 208-mm CLT floors	230-mm reinforced concrete slab
Roof	208-mm CLT roof	230-mm reinforced concrete slab
Shear Walls	CLT, steel shear plates	Reinforced concrete
Columns and Beams	Glulam columns and beams (above ground)	Reinforced concrete columns
	Reinforced concrete columns (below ground)	
Stairwells/Elevator Shaft	105-mm CLT walls	Reinforced concrete walls
Interior Walls Enclosing Apartments	105-mm CLT walls	6" light gauge steel studs
Stairs	Wide flange steel supporting concrete slabs	Reinforced concrete
Foundation Walls	Not included	
Foundation Slab	Not included	
Windows & Doors	Not included	
HVAC	Not included	
Plumbing & Electrical	Not included	
Furniture	Not included	
Finishing (e.g., paint, flooring, moldings)	Not included	
Minor Interior Walls	Not included	
Exterior Siding	Not included	
Lifts	Not included	

Building Life Cycle																		Supplementary Information		
Product Stage			Construction Stage		Use Stage					End of Life Stage								Beyond the System Boundary		
Raw Material Supply (A1)	Transport (A2)	Manufacturing (A3)	Transport (A4)	Construction (A5)	Building Use (B1)*	Maintenance (B2)*	Repair (B3)*	Replacement (B4)	Refurbishment (B5)*	Demolition (C1)	Transport (C2)	Waste Processing (C3)	Disposal (C4)					Reuse (D)**	Recovery (D)***	Recycling Potential (D)
Scenario	Scenario	Scenario	Scenario	Scenario				Scenario		Scenario	Scenario	Scenario	Scenario					Scenario	Scenario	
					Operational energy use (B6)*															
					Operational water use (B7)*															

Included in the LCA
Excluded from the LCA

* Operational energy and water use as well as maintenance (e.g. cleaning), repair and refurbishment activities are assumed to be identical between the two building types and are excluded from the system boundary.
** Reuse of CLT panels is considered.
*** Energy recovery from combustible waste is considered.
Note that the planning and design stages are also outside the system boundary.

Figure 1 System boundary: building life cycle stages according to ISO 21930

11.2.1 Comparison of Environmental Impacts of CLT Versus Concrete / Steel

The environmental impact indicators included in the assessment, which are described in the Glossary, are based on the U.S. EPA's Tool for the Reduction and Assessment of Chemical and Other Environmental Impacts v2.0[5] and include:

- Global warming potential (GWP)
- Acidification
- Particulate Matter (PM) criteria air pollutants
- Eutrophication potential
- Ozone depletion potential
- Smog potential

[5] See www.epa.gov/chemical-research/tool-reduction-and-assessment-chemicals-and-other-environmental-impacts-traci

Differentiating between Embodied Carbon and Global Warming Potential

Designers are becoming increasingly familiar with the term embodied carbon – which is appearing in building policies and green building rating systems (such as LEED) across Canada.

Embodied carbon is the total GHG emissions (in tonnes CO_2e) generated to produce a product, assembly or entire building. This includes emissions caused by extraction, manufacture, transportation and assembly of every product and element in the asset. In other words, embodied carbon quantifies the life cycle GHG emissions (in tonnes CO_2e).

By comparison, GWP is a relative measure of how much heat a greenhouse gas traps in the atmosphere up to a specific time horizon, relative to carbon dioxide (whose GWP is standardized to 1). The embodied carbon of a building is the result of quantifying the global warming potential of life cycle GHG emissions.

GWP was assessed dynamically (i.e. accounting for the effect of the timing of emissions and removals) (Grann, 2013), which is useful for comparing climate effects from different end-of-life disposal options for wood product (landfill, waste-to-energy, re-use). Results for the various end-of-life scenarios are presented below only for the CLT building, as results for the CSSW building were similar across these end-of-life scenarios.

Activities involving the extraction and manufacture of materials were found to be the dominant source of environmental impacts. For the CLT building, key impacts were due to rockwool insulation and CLT, and to a lesser extent to gypsum board. For the CSSW building, key impacts were due to concrete and rockwool insulation. Rebar was important for eutrophication potential and gypsum board was important for acidification potential.

The results presented in Figure 1 are labelled according to the life cycle stages used in EN 15978. Results for biogenic carbon, albedo, concrete carbonation, and fuel substitution (see definitions in the Glossary) are presented separately from other life cycle impacts based on recommendations to a) present impact results that have different signs (i.e. positive and negative results) separately using stacked bars (Verones et al., 2016), and b) separately report biogenic carbon emissions and removals (ISO 14067, 2018).

Two summary measures of life cycle impacts are presented in Figure 2. "Traditional" life cycle impacts exclude biogenic carbon, albedo, concrete carbonation, and fossil fuel substitution. The summary term "net" is used to refer to the sum of all positive and negative effects. This separation may be useful for the reader when interpreting the results, since biogenic carbon, albedo, concrete carbonation, and substitution effects are often not included in building LCA results.

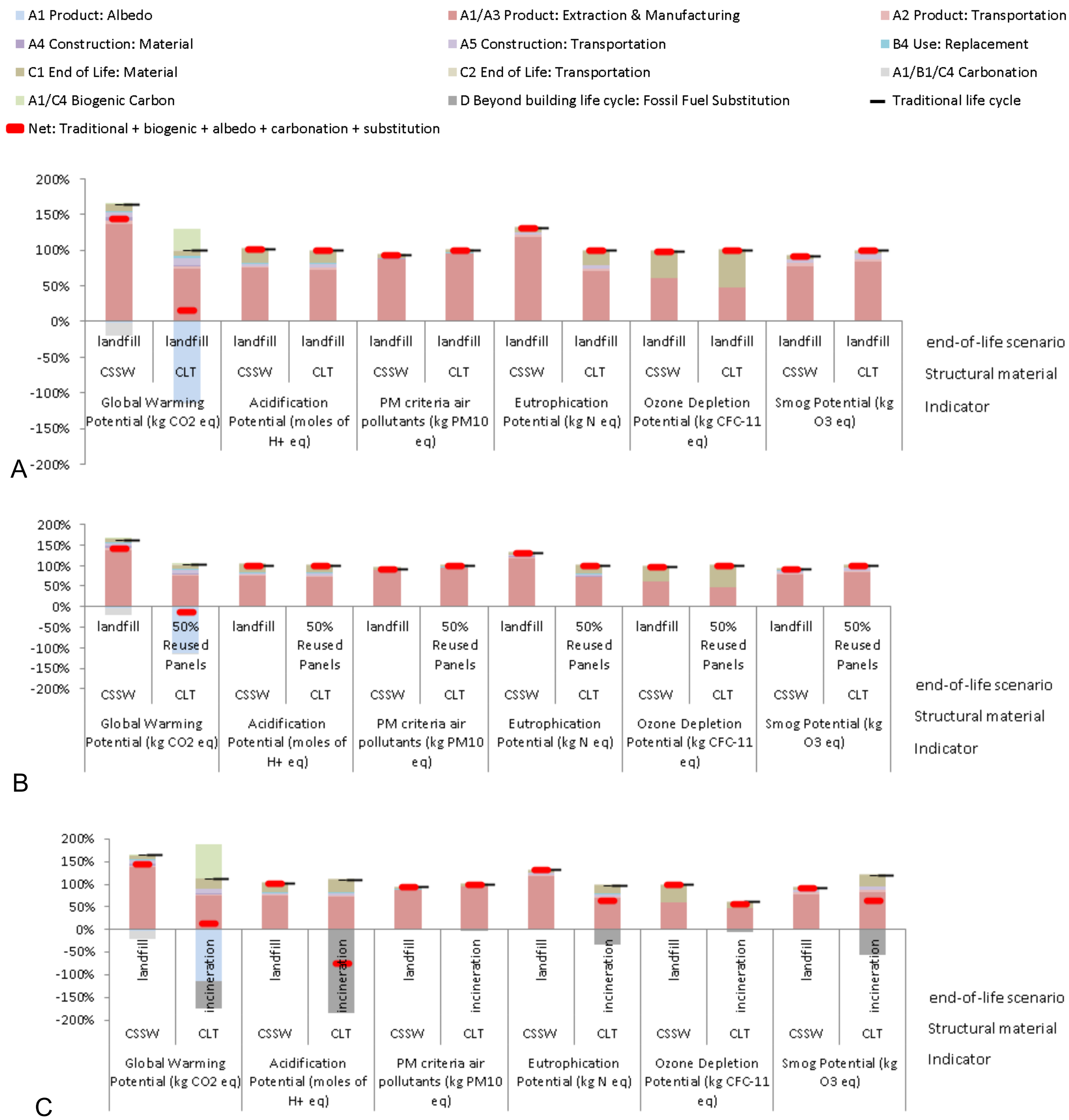

Figure 2 Environmental impact comparative results: sensitivity for end-of-life management

NOTES RELATING TO FIGURE 2:

This Figure shows the performance of the two building systems (CLT and CSSW) across six environmental impact measures (TRACI 2.0) considering different scenarios for end-of-life waste management for the CLT building (A: landfilling waste; B: recycling of 50% of CLT panels; C: incineration of combustible waste). Letters in stacked bar charts correspond to standard life cycle stages in EN 15978 (Figure 1). All results are normalized to ''traditional' life cycle impacts for the CLT building scenario with landfilling. For greater transparency, ''traditional' life cycle results exclude results for biogenic carbon, albedo, fossil fuel substitution, and concrete carbonation. Results are based on data from the Athena Impact Estimator (AIE) and end-of-life modelling using the U.S. EI database in SimaPro v7.3.3.

Carbonation: includes the effect of CO_2 reabsorbed by concrete, which mostly occurs when the concrete is crushed for reuse at the end of the building lifetime.

Net: the net effect of all life cycle environmental impacts including "traditional" life cycle impacts plus results from biogenic carbon, albedo, and fossil fuel substitution.

Ecotoxicity and Human Toxicity impacts are not provided by the AIE due to greater uncertainty in these impact assessment methods. Further indicators such as biodiversity, land use, and indoor air quality are not included in the TRACI methodology.

Long-term emissions (>100 years) are excluded from the results.

Landfilling includes leachate collection and treatment and assumes 23% of the carbon in the wood is released through decomposition, and 47% of the landfill gas is captured and flared. The 23% assumption for degradable carbon leads to conservatively high GHG emissions from landfilling compared to empirical results summarized in Lavoie et al. (2016), which ranged from 0-8.3% (interquartile range) for softwoods.

GWP results for concrete carbonation and the net GWP of wood products are presented separately due to greater uncertainty in the underlying scenarios. A detailed assessment of the effect of the timing of GHG emissions and removals associated with biogenic carbon, concrete carbonation, and concrete calcination can be found in Section 11.4.1 of the original report.

GWP for biogenic carbon and albedo is specific to black spruce harvested in Northern Québec and includes life cycle dynamics of biogenic carbon from the harvest site to end-of-life treatment, plus changes in surface albedo throughout the harvest cycle.

Results are based on the system boundary depicted in Figure 1, which excludes operational energy consumption as well as building assemblies listed in Table 2.

Global Warming Potential

The GWP identified from traditional life cycle results was found to be 40%-60% lower for the CLT building system compared to the CSSW building, based on the commonly modelled building elements. Considering the additional effects from biogenic carbon, albedo, and fuel substitution led to additional GWP savings for the CLT building. Fuel substitution for the waste to energy end-of-life scenario also had benefits for the CLT building in several other impact categories. A limitation of the CLT end-of-life scenario for reuse is that the results only capture the benefits of extending the carbon storage period (i.e. reduced GWP of biogenic carbon) and do not consider the benefits from avoiding the manufacture of materials that would result if CLT was reused.

Concrete carbonation, a process that re-absorbs some of the CO_2 emitted during cement manufacturing, was found to reduce GWP results by 10% for the CSSW building. For concrete carbonation, most of the CO_2 was found to be absorbed at the end of life. Where energy produced in waste-to-energy facilities at the end of life can substitute for the use of fossil fuels (substitution of heat from natural gas is presented in Figure 2) this can provide large GWP benefits for the CLT building.

GWP results for albedo were found to be of similar magnitude, but with an opposite sign (i.e. albedo results in a negative GWP) compared to traditional life cycle GWP for the CLT building. Albedo and biogenic carbon GWP results should be interpreted with caution when making comparisons between the CLT and CSSW buildings. Since the results for biogenic carbon and albedo are site-specific (Smith et al., 2014), these GWP results should not be generalized to other case studies. The case study results for albedo and biogenic carbon are based on harvesting in Chibougamau, QC, where the supplier of CLT for the building was located.

A key limitation of results for the CSSW building scenario is that it does not consider potential GHG removals/emissions that could occur if a forest stand is not harvested, such as increased carbon emissions linked to natural disturbance risks (forest fires, pests, wind, etc.), or the potential for continued forest growth (Cherubini, Guest & Stømman, 2013; Holtsmark, 2013; Nunery & Keeton, 2010).

Acidification Potential and Particulate Matter

Results for acidification potential and particulate matter were similar between the CLT and the CSSW buildings for all of the end-of-life scenarios except for the waste-to-energy scenario. For acidification, traditional life cycle impacts increased by 10% for the CLT building when waste-to-energy was used for waste disposal at the end of life. However, if the use of natural gas is avoided because of the heat produced from incinerating wood products, avoided acidification potential was found to be almost twice the acidification impacts associated with the CLT building life cycle and the net impact was negative.

Eutrophication Potential

Net eutrophication potential was found to be lower for the CLT building, especially if the bioenergy from the wood products can be used as a substitute for natural gas at the end of life.

Ozone Depletion Potential

Ozone-depleting emissions for the two systems were found to be minimal and these results are not discussed further.

Smog Potential

Finally, net smog potential was higher for the CLT building with landfilling and incineration, although there is again a large potential for avoiding smog emissions when energy recovery at the end of life can substitute for heat produced from natural gas.

Other Results

Other results presented in the original report included estimates of life cycle energy consumption, freshwater withdrawal, process waste, and site waste. Life cycle energy consumption was found to be similar for both buildings. The CLT building was found to use less water and produce less process waste during the production of materials. Wastes from the building site ending up in the landfill were lower for the concrete building because the scenario assessment conservatively assumed that all concrete would be used on site as filler material after demolition. Incinerating wood products at the end of life in a waste-to-energy facility significantly reduces landfilled waste for the CLT building, as wood is the primary mass in the CLT building.

Results from the comparative assessment are shown in Figure 2. The *y*-axis of this figure is expressed as a percentage because all of the results presented in Figure 2 are normalized using “traditional” life cycle impacts for the CLT building with landfilling for each indicator.

For additional details about these results, see Grann (2013). For other whole building LCA results evaluating CLT and other mass timber buildings, the reader can refer to Athena Sustainable Materials Institute (2015), Ruuska & Häkkinen (2012), and Teshnizi, Pilon, Storey, Lopez & Froese (2018).

11.2.2 Further Considerations

This Section expands on findings from the LCA study (Grann, 2013) that had important GWP effects, but which are not always addressed in current building LCA practice:

1. the potential for energy substitution at the end of life using waste wood,
2. contributions to GWP from biogenic carbon emissions, and
3. increased surface reflectivity at the harvest site (cooling effect from increased albedo).

The potential for energy substitution at the end of life is an approach that is standardized in the core LCA guidance standard (ISO 14044), but is optional in building LCA standards (ISO 21930, EN 15978, EN 15804). The climate effects of biogenic carbon emissions/removals and albedo for biomass systems have been recognized by the Intergovernmental Panel on Climate Change[6]. Current standardization efforts[7] are looking to address the effects from near-term climate drivers like albedo supplementing the long-lived GHGs covered by existing global agreements.

The following overview explains the GWP results from energy substitution, biogenic carbon and albedo while Appendix A briefly describes model assumptions and provides additional references for the reader.

11.2.2.1 Biogenic Carbon Accounting

Dynamic Carbon Accounting

The study (Grann, 2013) applies dynamic carbon accounting[8] to evaluate the climate effects from biogenic carbon emissions, concrete carbonation, and fossil fuel substitution, which occur at different points throughout the building lifetime. Dynamic carbon accounting of biogenic carbon involves tracking the time profile of emissions and removals linked to the use of wood products from a cut-block[9] including:

1. emissions at the harvest site from slash burning and decomposition,
2. removals at the harvest site from tree re-growth,
3. carbon emissions from bioenergy used to manufacture wood products, and
4. emissions at the end of life from wood product disposal.

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[6] see Section 11.13.4 in Smith et al. (2014)
[7] c.f. ISO TC207/SC7/WG13 working draft on 'radiative forcing management'.
[8] Cherubini, Peters, Berntsen, Strømman & Hertwich, 2011; Levasseur, Lesage, Margni, Deschênes & Samson (2010)
[9] Cherubini et al. (2013) show that biogenic carbon GWP results from a single harvest stand are equivalent to results from a wider landscape perspective.

Contributions to GWP from biogenic carbon differ between the end-of-life scenarios for the CLT building as a result of differences in the time profile of GHG emissions and removals, and because of differences in the carbon balance over the assessed time period (when landfilling, a fraction of landfilled carbon remains stored in the landfill). CLT reuse extends the period of carbon storage so that emissions from disposal of 50% of the CLT do not occur in the assessment time horizon (100 years). Therefore, CLT reuse decreases GWP from biogenic carbon relative to the landfilling scenario. For the waste-to-energy scenario, energy production results in transferring 100% of carbon contained in wood products to the atmosphere at the end of the building lifetime, whereas the landfilling scenario assumed that 23% of the carbon contained in wood products was emitted to the atmosphere through anaerobic decomposition.[10]

Baseline Scenario for Forest Carbon Accounting

Considering the benefits of using wood products as substitutes for other products implies that benefits can be achieved by expanding the use of wood products relative to a baseline scenario. This requires taking into account:

a. the potential effects on forest carbon from an incremental increase in harvesting rates compared to a baseline (e.g. current or recent historical),

b. the constraints on harvesting rates dictated by the annual allowable cut in Canada (discussed in more detail in Section 11.2 of this Chapter), as well as

c. the substitution effects when increasing the use of wood products for CLT, which results in a decrease in the use of wood products in other markets (e.g., formwork in China, or lumber in single-family homes).

For completeness, accounting for forest carbon effects should also consider the risks of losing carbon stored in forests due to natural disturbances (forest fire, pests, wind, etc.), which is a cause of considerable carbon emissions in Canadian forests (see Figure 6-3 in Environment and Climate Change Canada, 2017) that could otherwise be stored in long-lived wood products.

A key limitation of the results discussed above is the lack of accounting for the change in carbon storage in forests and long-lived wood products when CLT is used instead of concrete. Several previous studies[11] have explored the GHG implications of using wood products, considering changes to carbon stored in forests, products, and landfills, as well as the potential substitution effects when wood products are used instead of other types of products. One of the main findings

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[10] In a recent review of empirical evidence for anaerobic decomposition wood products in landfills (Grann, 2015; Lavoie, Grann & Mahalle, 2016), it was found that for softwoods, the fraction of carbon released through anaerobic decomposition is less than 10%, implying that the landfill GHG emissions reported here are conservatively high.

[11] Some of these studies include: Chen, Ter-Mikaelian, Yang & Colombo (2018); Dymond (2012); Hennigar et al. (2013); Hennigar, MacLean & Amos-Binks (2008); Perez-Garcia, Lippke, Comnick & Manriquez (2005); Sathre & O'Connor (2010); Skog (2008); Smyth, Rampley, Lemprière, Schwab & Kurz (2017); Suter, Steubing & Hellweg (2017); Wang, Padgett, De la Cruz & Barlaz (2011); Xu, Smyth, Lemprière, Rampley & Kurz (2018).

from these studies has been to highlight the GHG benefits when wood products are used to substitute for alternative products. Results have also consistently found that carbon is expected to continue to accumulate in products in use (e.g., from growth in the wood building stock) and in landfills. Harvesting sustainably managed forests in Canada and producing long-lasting wood products can therefore contribute to GHG mitigation.

Limitations of this previous research include the inherent challenge in identifying product substitutes, and the reality that substitution effects are likely to change over time as a result of innovation within the economy. Additionally, most of the research to date has focused on substitution effects linked to primary wood products like lumber, plywood, and oriented strand board (OSB), although it may also be important to explore substitution effects in markets that use wood co-products (e.g. pellets, particleboard, medium density fibreboard, etc.). This is a common limitation of whole building LCA studies which are often biased to focus on the primary wood products ignoring the potential GHG effects from substitution that can result from the use of wood co-products generated from lumber production. For example, Canadian wood pellet exports are often co-fired with coal in power plants (Strauss, 2017), and substitution effects in end-use markets for wood co-products like pellets could be an important consideration for a sensitivity analysis. Where wood residues from expanding the use of CLT are used to produce pellets that substitute for fossil fuel, this would be an additional climate benefit that is rarely accounted for in single building LCAs.

11.2.2.2 Albedo Effect

The cooling albedo effect (negative GWP) for the CLT building was due to an increase in surface reflectivity at the harvest site in winter months, compared to an un-harvested site. Albedo effects are site-specific and the results presented above are based on a harvest site in Chibougamau, QC, where the supplier of CLT was located. While albedo effects are not yet commonly considered in LCA, the Intergovernmental Panel on Climate Change (Smith et al., 2014) highlights that albedo effects are an important climate consideration for the agriculture, forestry, and land use sectors.

11.2.2.3 Energy Substitution Associated with Waste-to-Energy

If waste-to-energy is used to dispose of wood when the building is demolished, all the carbon stored in the biomass is released to the atmosphere. At the same time, if energy can be produced from wood waste and then that energy is used to reduce fossil fuel use, then this represents a climate mitigation strategy. Results from energy substitution are sensitive to the fuel that is assumed for the substitution (natural gas in the case study presented above). Some LCA standards (e.g., EN 15804 and EN 15978), do not require substitution effects that may occur at the end of life to be considered, although they can be presented “if relevant and available” (EN 15978, 2011, p. 36).

Scenarios for exploring energy substitution might include business as usual (e.g. projecting the current energy system into the future) or an energy system scenario consistent with global commitments for climate change mitigation, such as the Paris Agreement.

As a sensitivity analysis, substitution of coal and oil, in addition to natural gas, is considered in Grann (2013). However, given the long-time horizon relevant for a building's lifespan (e.g. 30-100 years), other scenarios may be appropriate for considering energy substitution at the end of life. For example, if waste-to-energy is used to produce heat, it is also possible that, in the future, this heat may substitute for other sources of "low carbon" / renewable heat such as heat pumps or solar. If fossil fuel substitution benefits are not likely to be achieved, scenario results suggest that landfilling wood products might provide a better strategy for reducing GHG emissions, as a result of continued carbon storage in the landfill.

11.3 RESOURCE AVAILABILITY TO SUPPORT CLT PRODUCTION

This Section explores the topic of wood availability in Canada. It highlights the gap between the annual allowable cut and current harvesting levels and demonstrates the wide availability of certified wood products to consumers in Canada and key markets abroad. It uses a mass balance (see Glossary) to identify the activities that are potentially affected by increased CLT production. This mass balance approach can help explore some of the indirect effects that might arise from expanding the production of CLT, including substitution of end-use products fabricated at the harvest site and in sawmills as a consequence of expanding CLT production.

11.3.1 Forest Management in Canada

Around 90% of Canadian forests are found on publicly owned (Crown) land (Canadian Council of Forest Ministers, n.d.-a; Natural Resources Canada, 2018). Forest management in Canada is within the jurisdiction of provinces, with regulatory frameworks developed to address social, economic, and ecological aspects of sustainable forest management (Rostad & Sleep, 2014). For forestry operations on Crown land, laws are in place that require companies to develop a management plan, consult with communities, and receive provincial or territorial approval prior to harvesting (Natural Resources Canada, 2017). Similarly, there are legal requirements for reforestation to ensure forests are replenished, and for stream buffer zones to prevent erosion and damage to fish habitat. Compared with the rest of the world, forest management in Canada has been found to operate under some of the most stringent sustainability laws and regulations (Cashore & McDermott, 2004; Indufor, 2009; Indufor, 2016).

11.3.2 Wood Supply

"Wood supply" is the term used to describe the volume of timber that can be harvested from public and private lands in Canada over a specified period, based on current regulations (National Forestry Database, n.d.-b). The total available wood supply in Canada includes the Annual Allowable Cut (AAC) from provincial Crown lands, and the estimated wood supply from federal, territorial, and private lands (National Forestry Database, n.d.-b).

Most of the wood supply in Canada is from provincial Crown lands. Provincial governments regulate harvest levels by determining an AAC, which is the annual amount of wood that can be harvested from a managed unit of Crown land. In practice, annual harvest volumes may be above or below the AAC, but they must balance out over the regulation period (generally 5-10 years) (National Forestry Database, n.d.-b). Across all of Canada, less than one half of one percent of the managed forest area is harvested annually (Natural Resources Canada, 2018).

In 2015, Canada harvested just over 127 million cubic metres (m^3) of softwood, well below the estimated wood supply level of 169 million cubic metres (m^3) (National Forestry Database, 2017b) (Figure 3). As the global population and demand for forest products increases, the volume of timber harvested may increase, narrowing the gap between harvest and the sustainable wood supply. Despite this narrowing gap, harvest levels remain below the sustainable wood supply levels, given the strong provincial regulatory AAC regimes in place (Natural Resources Canada, 2018).

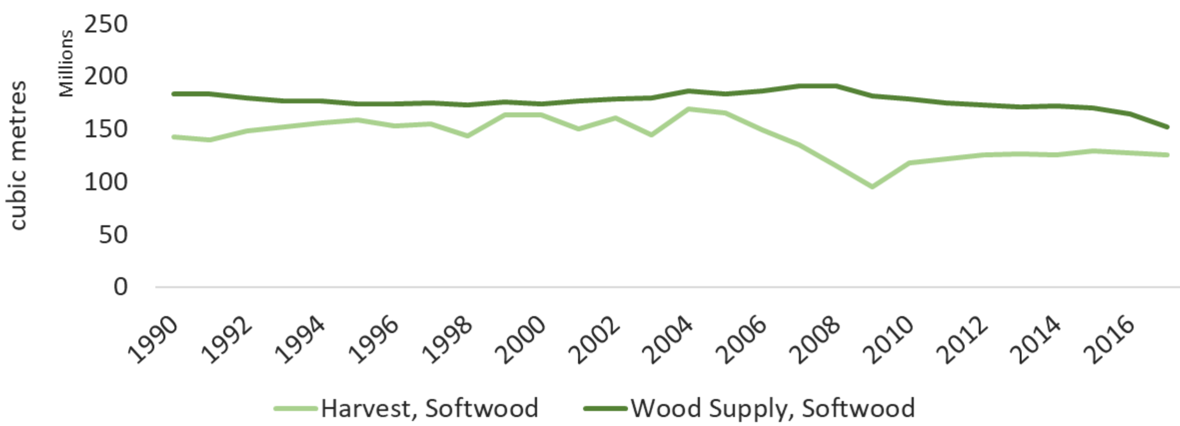

Figure 3 Annual softwood supply vs annual softwood harvest in Canada

Between 1990 and 2015, the differences between the available supply (annual allowable cut) and harvest of softwood in Canada ranged from 11-89 million m^3 / year with an average of 38 million m^3/year.

Source: National Forestry Database, (n.d.-a, n.d.-b)

The wood supply for an increase in CLT production can come from several sources:

1. increased harvesting from domestic forests, considering harvesting constraints in the form of the AAC,
2. supplying less wood to other domestic markets for solid wood products,
3. decreasing lumber/roundwood exports to foreign markets and/or increasing their import from foreign markets, and
4. increasing resource efficiency in all wood supply chains (Figure 4).

Over time, increased demand for forest products can also stimulate forest management practices that increase roundwood supply from forests through mechanisms such as increased productivity on existing forest lands through planting, use of better seed and management of competing vegetation (Smyth et al., 2014), or through expanding forest area through afforestation of agricultural lands or other unproductive ecosystems.

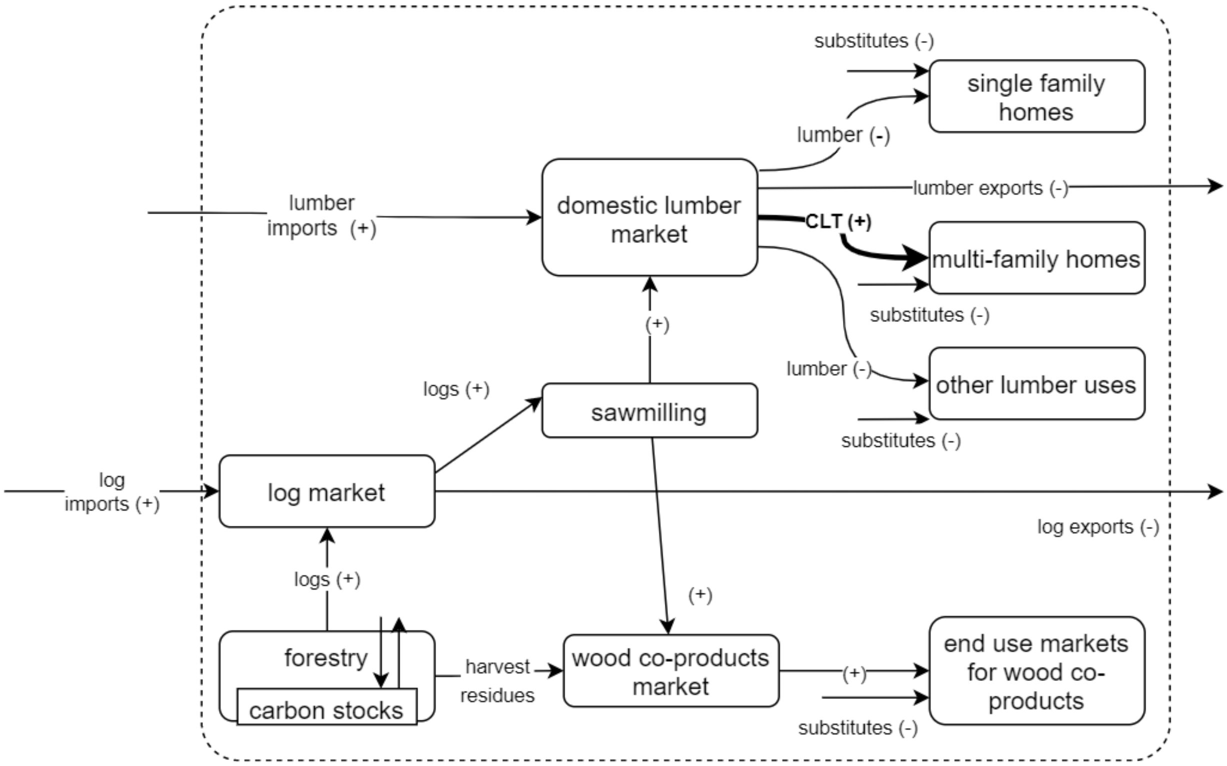

Figure 4 **Potential wood supply for increased CLT production**

11.3.3 Resource Efficiency

Resource efficiency in forest management, wood processing, manufacturing, consumption, reuse, and recycling is an important strategy for reducing the demands on natural ecosystems from human consumption. From a resource efficiency perspective, a CLT panel can appear to consume significantly more wood compared to a traditional light-frame wood wall or floor. However, it is important to consider the metric used to estimate resource efficiency. Examples of resource efficiency metrics that are relevant for buildings include: quantity per m^2 of building area, and quantity per occupant.

Table 3 Structural wood use in mass timber buildings

Building	Structure	Location	Wood Use		Storeys	Height (m)
			m^3/m^2 floor area	$m^3/$ occupant		
Wood Innovation and Design Centre	CLT floors; CLT elevator and stair cores; glulam columns and beams	Prince George, BC	0.32	14	7	29.5
Brock Commons	CLT floors; glulam and parallel strand lumber columns; reinforced concrete podium, concrete stairway, and elevator cores	Vancouver, BC	0.15	7	18	54
Stadthaus	CLT floors; CLT interior and exterior partition walls; reinforced concrete podium	London, UK	0.33	18	9	29
Origine	CLT floors; CLT interior and exterior partition walls; glulam columns; reinforced concrete podium and underground parkade	Quebec, Qc	0.22	18	13	41

Sources: Forestry Innovation Investment (2015, 2016); "Stadhaus", 2018; Wood WORKS! and Canadian Wood Council, n.d.; Cecobois (n.d.-a, n.d.-b)

Wood use intensity (m^3/occupant) for structural wood products in Canadian single-family homes is around 13 m^3/occupant compared to 8 m^3/occupant for multi-family light-frame wood apartment buildings[12]. Considering the range of wood use factors for CLT and mass timber buildings shown in Table 3, *per capita* wood use in apartments using CLT is 7-18 m^3/occupant[13]. Cross-laminated timber apartment buildings therefore represent a per capita wood use that is similar to wood use for residential construction in Canada.

[12] This assumes 1) 0.20 m^3/m^2 floor area for single family homes and 0.18 m^3/m^2 for multi-family homes (McKeever & Elling, 2015); 2) 2.8 persons per household for single family homes and 1.8 persons per dwelling for apartments with 5 or more storeys (Statistics Canada, 2012), and 3) 180 m^2/unit for the average size of a new single family home and 83 m^2/unit for new condos (Canadian Mortgage and Housing Corporation, 2013; Marr, 2016), scaled up to account for common areas (hallways, etc.). In McKeever & Elling, floor area represents the finished floor area measured from the outside of exterior walls (gross floor area). For multi-family, this includes hallways and lobbies and for all buildings garages and unfinished basements are excluded.

[13] Assuming an average of 75 m^2/unit for new condos (Canadian Mortgage and Housing Corporation, 2013; Marr, 2016), and 1.8 persons per unit (Statistics Canada, 2012).

11.3.4 Forest Certification

As an additional layer of reassurance to consumers of CLT and other Canadian forest products, Canada leads the world in third-party certification, with more land certified to voluntary, market-based forest sustainability programs than any other country (Figure 5). As of the end of 2018, Canada had over 184 million hectares of independently certified forest land (Certification Canada, 2019). This represents 49% of Canada's forests and 37% of all certified forests worldwide, the largest area of third-party certified forests of any country (Natural Resources Canada, 2018).

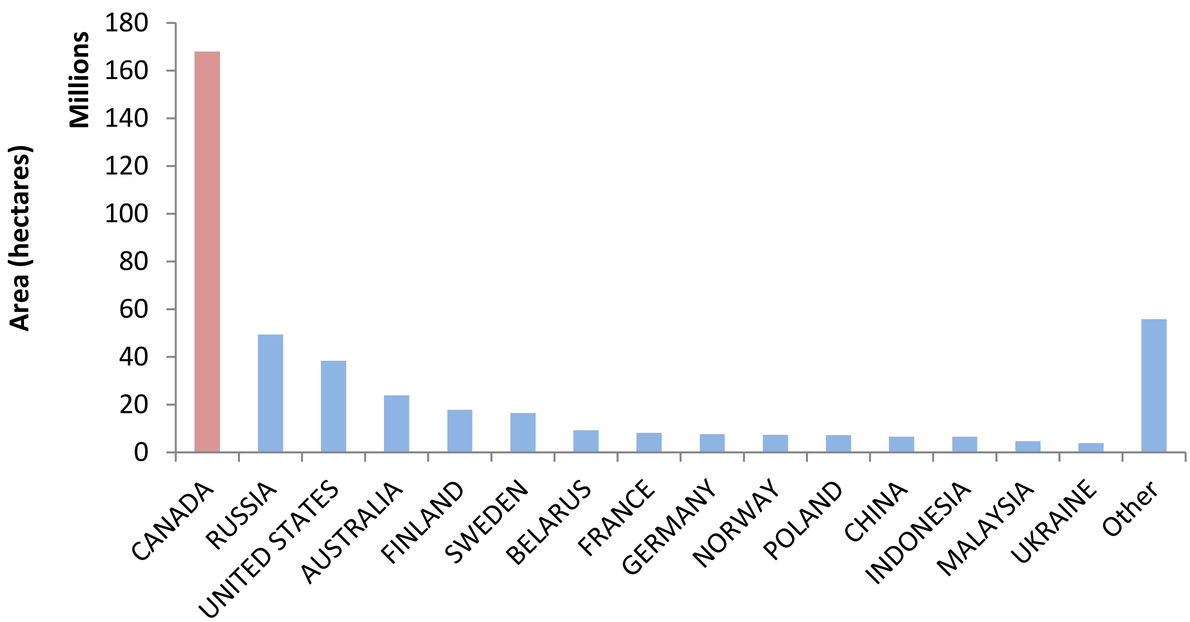

Figure 5 Forest certification by country, 2017

Chart includes forests certified to the standards of the Forest Stewardship Council and the Programme for the Endorsement of Forest Certification. In 2017, Canada had 37% of the world's certified forest.

Data sources: Forest Stewardship Council International (2018), PEFC (2017) (PEFC, 2018).

Voluntary forest certification allows forestry companies to demonstrate, via independent third-party assessments, that their practices meet the environmental, economic and social standards required by the certification standards. For many aspects of sustainable forest management (e.g., forest health, water protection, old growth management, prohibition of GMOs), requirements in Canadian forest management regulatory frameworks have been found to be similar to, or go above and beyond, the responsibilities set out in sustainable forestry certification standards (Cashore & McDermott, 2004; Indufor, 2009; 2016). In other aspects (e.g., habitat and species protection), certification can go beyond regulatory requirements (Indufor, 2016). Third-party forest certification in Canada helps to demonstrate the strengths of Canada's regulatory framework for forest management (Canadian Council of Forest Ministers, n.d.-b).

Three forest certification systems are used in Canada. The Canadian Standards Association (CSA) and the Sustainable Forestry Initiative (SFI) systems are endorsed by the international umbrella organization called the Programme for the Endorsement of Forest Certification Schemes (PEFC). The third system is the Forest Stewardship Council Canada (FSC). Canada has more than half of the world's PEFC-endorsed certifications and almost a third of the world's FSC certifications.[14]

11.4 INDOOR AIR QUALITY IMPACTS FROM USING CLT IN BUILDINGS

This Section outlines the potential indoor air quality impacts from using CLT due to the emission of volatile organic compounds (VOCs). We present test results for VOC emissions from Canadian CLT. Details of the test procedures are provided in Appendix B.

VOCs are substances that off-gas from some solids and liquids. VOC emissions to indoor air are one of many types of pollutants that can affect indoor air quality (U.S. EPA, 2014b). Potential sources of VOCs include common household products (paint, household cleaners) and building products (furniture, carpet, etc.) (U.S. EPA, 2014a). VOC emissions to indoor air are commonly addressed by green building standards such as LEED and, increasingly, by government regulations.

While Canada does not currently have regulations related to formaldehyde emissions and indoor air quality, Health Canada has a guideline to help limit formaldehyde exposure (Health Canada, 2006) and there is a regulatory plan for formaldehyde emissions from composite wood products (Health Canada, 2017). The formaldehyde emission limits set forth by the California Air Resources Board's (CARB, 2007) Composite Wood Products Airborne Toxic Control Measure are some of the most rigorous emission limits in the world for composite wood products. The U.S. EPA (U.S. EPA, 2016) introduced a regulation providing formaldehyde emission standards for composite wood products that are generally consistent with the Californian CARB limits.

Historically, formaldehyde emissions from the adhesives used in engineered wood products have been the main source of VOC emissions linked to wood components in buildings. Another source of VOCs can be fire retardants used to treat wood products, although fire retardants are not commonly used with CLT. VOCs that may be associated with the wood itself include acetaldehyde, acetone and various carbonyl compounds. Formaldehyde is known to be irritating to the eyes and the respiratory system at high concentrations and is a known human carcinogen (California Air Resources Board, 2018).

Five CLT products were tested for their VOC emissions, including formaldehyde and acetaldehyde. The tested laminated products had different thicknesses and a different number of glue lines. Emissions were collected after 24 hours of sample exposure in the environmental chamber. Please see Appendix B for details on the test methods used and the detailed results.

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[14] www.nrcan.gc.ca/our-natural-resources/forests-forestry/sustainable-forest-management/forest-certification-canada/17474

While CLT is outside the scope of CARB and U.S. EPA formaldehyde emission standards, which apply to particleboard, hardwood plywood and medium density fibre board, the results documented in Appendix B show that the CLT samples that were tested easily met the most stringent CARB limits. In addition, the results were generally lower than limits set forth by European emission labeling systems. In fact, the 24-hour CLT test results were lower than European limits intended for measurement after three days. This is in line with other research which found the VOC emissions from CLT to be lower after three days compared to the first 24 hours (Höllbacher, Rieder-Gradinger, Stratev & Srebotnik, 2014). These results are also within the limits set for Greenguard[15] certified building products and interior finishes.

The test results documented in Appendix B apply to CLT made from two Canadian wood species (spruce and pine) – the only North American CLT commercially available at the time of the original sampling in 2010 – with a polyurethane glue (Purbond HB E202) used for bonding the faces of the board and another polyurethane glue (Ashland UX-160/WD3-A322), which was used for finger jointing. While polyurethane adhesives are the most widely used for CLT (as indicated in Chapter 2 of this Handbook, *Cross-laminated Timber Manufacturing*), these test results may not apply to CLT products made by other manufacturers using different adhesives and wood species. Information about specific adhesives used by CLT manufacturers can often be found on their websites, documented in environmental and health certifications, such as Declare, Environmental Product Declarations (EPDs), Health Product Declarations, etc.

It is important to note that different formaldehyde-based products have different levels of chemical stability that either reduce (high stability) or increase (low stability) their emissions of VOCs under different environmental conditions. For example, in contrast to the more volatile urea formaldehyde (which are not used for glued wood products with a structural rating), phenol formaldehyde, resorcinol formaldehyde, phenol resorcinol formaldehyde, and melamine formaldehyde polymers "do not chemically break down in service; thus no detectable formaldehyde is released" (Frihart and Hunt, 2010).

Due to regulations requiring lower formaldehyde emissions not only in the finished product but also during manufacturing for workplace safety, new adhesive formulations have been—and are being—developed to significantly reduce levels of formaldehyde emissions, both during manufacturing and in bonded wood products. Refer to CWC (2013) for an overview of emissions from wood product adhesives and Chapter 3 in Karacabeyli & Lum (2014, and forthcoming 2020 editions) for further information about the variety of additives that must be considered by the design team, including adhesives, treatments for protection against wood destroying organisms and wood rot, and treatments for fire protection.

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[15] UL Environment's Greenguard certification program helps manufacturers create--and helps buyers identify and trust--interior products and materials that have low chemical emissions, improving the quality of the air in which the products are used. All certified products must meet stringent emissions standards based on established chemical exposure criteria. UL Environment is a business unit of UL (Underwriters Laboratories). For more information visit www.greenguard.org.

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APPENDIX A

11A.1 KEY MODEL DESCRIPTIONS FOR CLT LCA STUDY

Table 11A.1 Model Descriptions

Aspect	Description
Biogenic carbon accounting	Model uses dynamic biogenic carbon accounting to consider the effect of the timing of GHG emissions and removals linked to the harvest site (Cherubini et al., 2011; Grann, 2013).
Albedo of harvesting activities	Model results consider the change in surface reflectivity (albedo) linked to harvesting activities near Chibougamau, Québec (Bright, Cherubini & Strømman, 2012; Grann, 2013).
Concrete carbonation	Results from the Athena Impact Estimator did not include estimates of concrete carbonation. Dynamic carbon accounting was used to estimate the effect of the timing of GHG emissions associated with concrete carbonation when concrete is crushed and used as filler at the end of life.
End of life	Scenario results for landfilling assume 47% landfill gas captured and flared; 23% of the carbon in landfilled wood products is assumed to decompose with 50% released as methane and 50% as CO_2. Scenario results for incineration with energy recovery is considered for all combustible waste. Heat produced from energy recovery is assumed to substitute for heat produced with natural gas, with a sensitivity considering potential substitution of heat produced from other fossil fuels.

APPENDIX B

11B.1 DETAILS OF THE VOC TESTING

In this Section, we provide preliminary findings regarding emissions to indoor air from cross-laminated timber (CLT). This data applies to CLT made from two Canadian species (spruce and pine).

11B.1.1 Objectives and 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. The World Health Organisation (WHO) has defined VOCs as organic compounds with boiling points between 122°F (50°C) and 500°F (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 well documented, very little if any data exist for multi-ply products (in other words, products with multiple wood layers like CLT and plywood).

11B.1.2 Procedures and Results

All measurements were made in general agreement with the specified standards and protocols. The precision levels were in accordance with the technical requirements.

11B.1.2.1 Material Sampling, Packaging, Transportation, and Conditioning

Duplicate test samples measuring 11 inches x 30 inches (280 mm x 760 mm) (Figure 11B.1) were cut 12 inches (300 mm) from each end of an 18-foot (5.5-meter) long original CLT panel. In order to avoid any potential contamination of the samples, latex gloves were worn during the entire sampling and packaging processes. In addition, a towel was used to clean the saw blade before cutting the samples. Samples were wrapped in plastic foil without 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. All samples were tested within one month after production.

The VOC and formaldehyde tests were performed on the same sample and under similar conditions at a loading ratio of 0.44 m^2/m^3 with all edges sealed with a non-emitting aluminum tape material and leaving the two flat surfaces exposed.

Figure 11B.1 A prepared sample with edges sealed, ready to be put in the chamber for emissions testing

11B.1.2.2 Method

A constant and adjustable airflow, conditioned for relative humidity, was fed through a small environmental chamber at a rate corresponding to an air change rate of one per hour. The VOC sampling procedures, except for formaldehyde, were similar to those described in the ASTM D5116-97 and ANSI/BIFMA M 7.1-2007 standards. The chamber (Figure 11B.2) was constructed 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.

Figure 11B.2 A general view of the 1 m³ 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 the VOCs and derivatized DNPH cartridges were used to sample the low molecular weight aldehydes, formaldehyde and acetaldehyde. Higher molecular weight aldehydes were sampled with the Tenax® tubes used for sampling the VOCs. VOCs were analyzed by desorbing the VOCs from the sample tubes through a thermal desorption system and then injecting them into a gas chromatograph equipped with a mass detector (GC/MS). The aldehyde tubes were desorbed with acetonitrile solvent, which was then injected into a high-performance liquid chromatograph (HPLC). Table 11B.1 describes the small chamber operating conditions, while Table 11B.2 summarizes the GC/MS and the HPLC operating conditions.

Table 11B.1 Small chamber operating conditions

Parameter Symbol Unit Value	Parameter Symbol Unit Value	Parameter Symbol Unit Value	Parameter Symbol Unit Value
Chamber volume V m^3 1.0	Chamber volume V m^3 1.0	Chamber volume V m^3 1.0	Chamber volume V m^3 1.0
Loading ratio Lr m^2/m^3 0.44	Loading ratio Lr m^2/m^3 0.44	Loading ratio Lr m^2/m^3 0.44	Loading ratio Lr m^2/m^3 0.44
Temperature T ºC 23±1	Temperature T ºC 23±1	Temperature T ºC 23±1	Temperature T ºC 23±1
Relative humidity RH % 50±5	Relative humidity RH % 50±5	Relative humidity RH % 50±5	Relative humidity RH % 50±5
Air exchange rate ACH h-1 1.0	Air exchange rate ACH h-1 1.0	Air exchange rate ACH h-1 1.0	Air exchange rate ACH h-1 1.0
Sampling time Hours 24	Sampling time Hours 24	Sampling time Hours 24	Sampling time Hours 24

Table 11B.2 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 to 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
Mobile phase	70% ACN:30% water
Flow rate	1.0 mL/min
Total injected volume	25 µL
Column temperature	20ºC
Detector	DAD 360 nm

11B.1.2.3 Quantification of Formaldehyde and Other Carbonyl Compounds

Formaldehyde, acetaldehyde and acetone emissions were quantified according to a modified version of ASTM D5197-03 entitled "Standard Test Method for Determination of Formaldehyde and Other Carbonyl Compounds in Air (Active Sampler Methodology)". The method can be summarized as follows: the sampled air is drawn through a cartridge containing silica gel coated with 2,4-dinitrophenylhydrazine (DNPH) reagent. Carbonyl compounds readily form stable derivatives with the DNPH. After sampling, the cartridge is desorbed into a known volume of acetonitrile. The DNPH derivatives are analyzed for parent aldehydes and ketones utilizing high performance liquid chromatography (HPLC). Formaldehyde and other carbonyl compounds emitted by the sample are identified and quantified by comparison of the retention time and peak areas of their corresponding DNPH derivatives with those of a standard solution. The concentration obtained is back-calculated to the original concentration via an aliquot factor and presented as micrograms of analyte per cubic meter of air sample (µg/m^3). This concentration is then converted to parts per billion (ppb) and emission factor as µg/m^{2}h

11B.1.2.4 Quantification of the Total Volatile Organic Compounds

VOC measurements from panel samples were conducted in accordance with the ASTM D5116-97 guide and as described in great details 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 Gas Chromatograph (GC) column in order to "cryofocus" the thermally desorbed chemicals prior to their injection into the GC. The GC oven was programmed for 10 minutes at 70°C, followed by ramping up the heat to 200°C at a rate of 8°C/min, and then holding 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.

11B.2 RESULTS AND DISCUSSION

Tables 11B.3 and 11B.4 summarize the emitted VOCs including formaldehyde, acetaldehyde, and acetone, expressed in micrograms per cubic meter (µg/m^3). To better illustrate the variation of the emissions as a function of the product category, the results are graphically shown in Figures 11B.3 and 11B.4; the same scale was applied to both figures for an easy comparison. As can be seen in these figures, no correlation exists between emission results and the number of glue lines involved in each product category, or the product thickness. In addition, most of the emitted VOCs, except for formaldehyde and acetaldehyde, are those usually emitted from softwood species, indicating that only formaldehyde and acetaldehyde could really be associated with the product manufacturing processes. Figure 11B.5 compares the total volatile organic compounds (TVOC), excluding formaldehyde, acetaldehyde and acetone, emitted from the five different products tested. With respect to the individual VOCs, no correlation can be established between TVOCs, and the thickness or the number of plies in the cross-laminated lumber products.

Table 11B.3 Samples 24-hour individual VOCs (iVOCs), TVOC as toluene, between n-C6 and n-C16 including formaldehyde (µg/m³)

		114-3S			95-3S		
VOCs	**CAS #**	**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	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	35	3.6	8.3	6
Para-cymene	99-87-6	78.6	5.9	42.3	43	45.4	44.2
Limonene	95327-98-3	7.6	11.7	9.6	3.3	2.8	3
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	79.1
Acetone	67-64-1	33.2	65.3	49.2	45.7	24.4	35

* Compound for which concentration (µg/m³) is below the quantification limit allowed by ANSI BIFMA.

Table 11B.4 Samples 24-hour individual VOCs (iVOCs), TVOC as toluene, between n-C6 and n-C16 including formaldehyde (µg/m³)

		190-5S			152-5S			210-7S		
VOCs	**CAS #**	**A**	**B**	**Mean**	**A**	**B**	**Mean**	**A**	**B**	**Mean**
Acetic	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
Beta-pinene	18172-67-3	14	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	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	81.7
Formaldehyde	50-00-0	9.4	8.8	9.1	5.7	6.5	6.1	6	5.4	5.7
Acetaldehyde	75-07-0	71.5	68.5	70	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

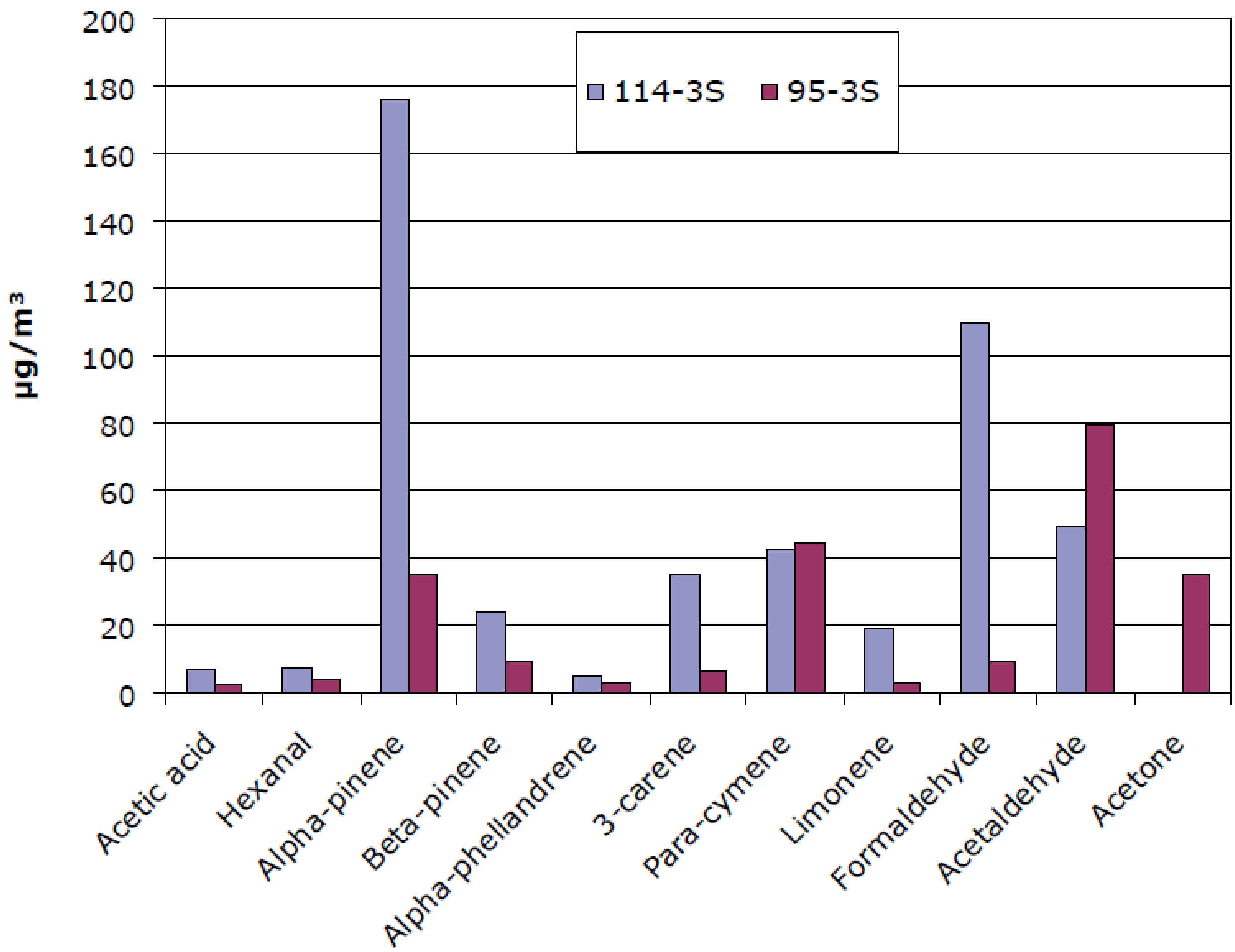

Figure 11B.3 24-hour VOCs including formaldehyde and acetaldehyde off-gassing as a function of sample type

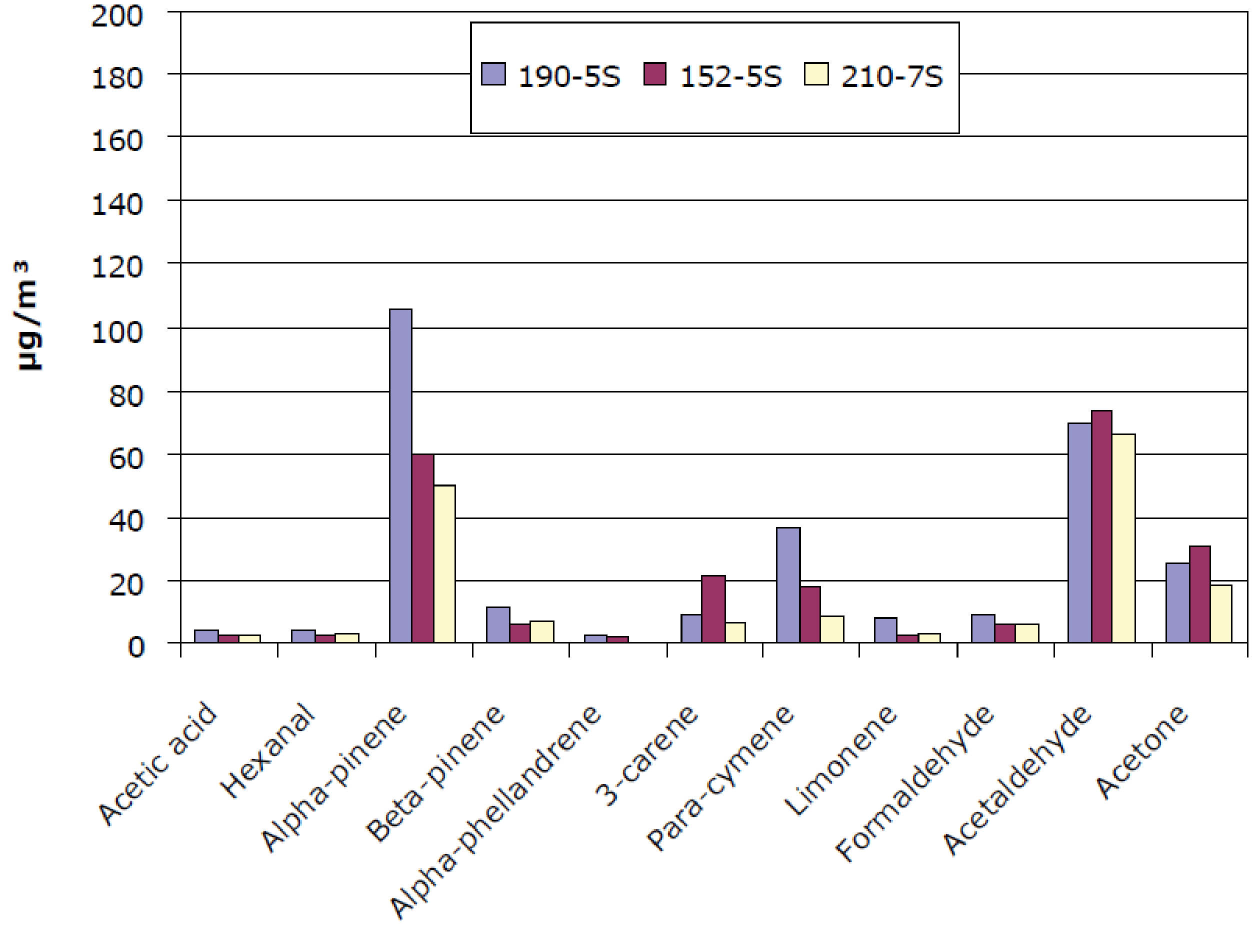

Figure 11B.4 24-hour VOCs including formaldehyde and acetaldehyde off-gassing as a function of sample type

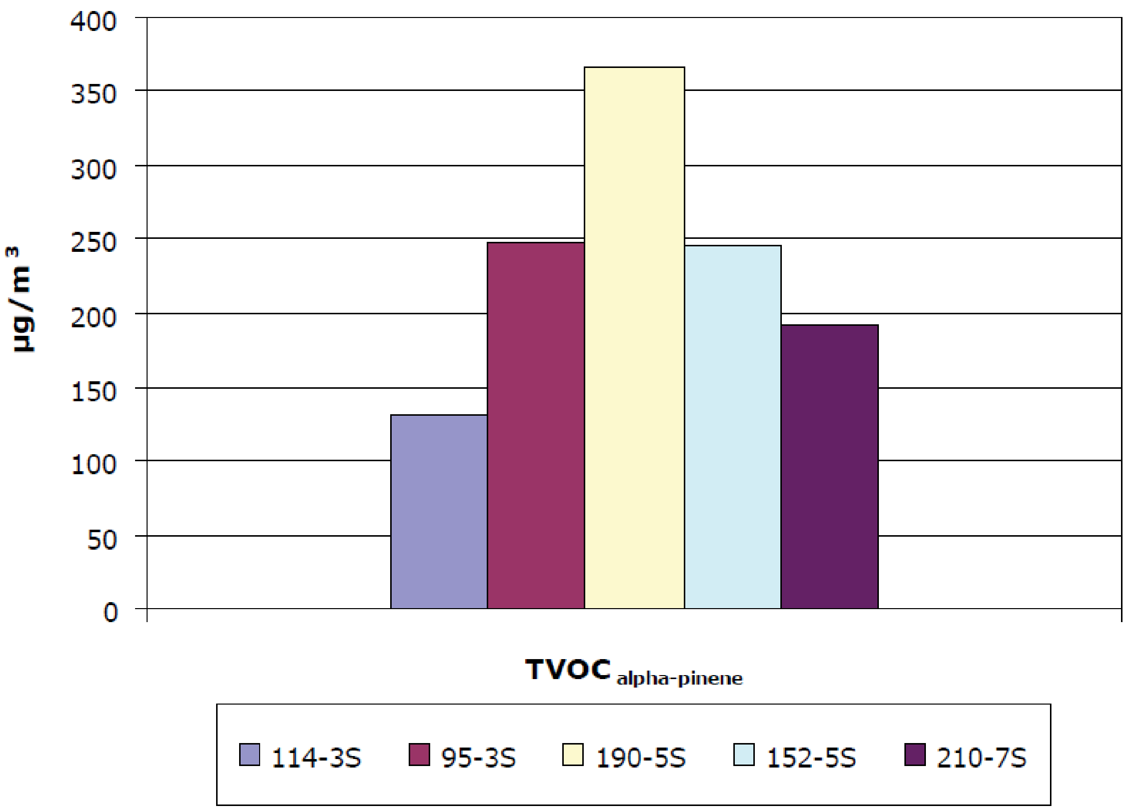

Figure 11B.5 24-hour TVOC emissions as a function of cross-laminated product

Examples of VOC emission labeling systems in Europe, including formaldehyde and acetaldehyde, are summarized in Table 11B.5 in order to put the tested CLT products emissions in context and to help manufacturers interested in labeling their products for overseas markets. Because few individual VOC emission limits are expressed in emission factors (EF), i.e., mass of the emitted VOC per square meter of the product tested per hour (µg/m²h), the CLT product emission results have been converted into emission rates. These are summarized in Tables 11B.6 and 11B.7. Results of emission factors reported in these tables were calculated from the 24-hour sampling time. On the other hand, the voluntary limits listed in Table 11B.5 were calculated after 3, 10 or 28 days of sample exposure in the environmental chamber. One should expect that the CLT emission factors would be much lower if their exposure were to be prolonged for an additional 3, 10 or 28 days and thus, should meet the most stringent Blue Angel or GUT (Germany) TVOC emission limits that were not met after only 24 hours of exposure.

To convert measured individual or total VOC emissions (both expressed in µg/m³) in the environmental chamber into emission factors, knowing the flow rate Q(m³/h) and the total exposed surface area of the sample A(m²), the following equation can be used:

$$EF\ (\mu g/m^2h) = C(\mu g/m^3) * Q(m^3/h)/A(m^2)$$

Table 11B.5 Example of some European emission labeling systems

Label	Origin	TVOC	Aldehydes Additional Requirements
AgBB	Germany	10 mg/m (3 days) 1 mg/m (28 days)	DIBt: 120 µg/m^3 (28 days)
CESAT	France	5000 µg/m (3 days) 200 µg/m (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 h (3 days) 200 µg/m h (28 days)	Formaldehyde: 10 µg/m^3 (28 days)
Natureplus	Germany	5000 µg/m h (3 days) 200 µg/m h (28 days)	Formaldehyde: 36 µg/m after 3 days or 28 days
Blue Angel	Germany	200 or 300 µg/m (28 days)	Formaldehyde: 60 µg/m^3 (28 days)
Austrian Ecolabel	Austria	1.2 mg/m (3 days) 0.36 mg/m (28 days)	Hexanal: 70 µg/m h (28 days), nonanal: 20 µg/m h after 3 days
GUT	Germany	300 µg/m^3 (3 days)	Formaldehyde: 10 µg/m^3 after 28 days
EMICODE EC1 such as adhesives	Germany	500 µg/m^3 (10 days)	Formaldehyde and acetaldehyde: 50 µg/m each after 24 hours
Scandinavian Trade Standards	Sweden	Declaration of TVOC after 28 days and 26 weeks no limits specified	Formaldehyde and acetaldehyde according to WHO

Table 11B.6 Samples 24-hour iVOCs, TVOC as toluene, between n-C6 and n-C16 emission factors including formaldehyde (µg/m²h)[16]

		114-3S			95-3S		
VOCs	**CAS #**	**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	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	109.6	145.8
Acetone	67-64-1	16.7	32.9	24.8	77.5	52.6	65.1

* Compound for which concentration (µg/m³) is below the quantification limit allowed by ANSI BIFMA.

[16] 1 µg/m²h corresponds to 1.55x10³ µg/po²h

Table 11B.7 Samples 24-hour individual VOCs and TVOC as toluene, between n-C6 and n-C16 emission factors including formaldehyde (µg/m²h)

		190-5S			152-5S			210-7S		
VOCs	CAS #	A	B	Mean	A	B	Mea n	A	B	Mean
Acetic acid	64-19-7	8.7	9	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	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	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
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	132.9	139.5	136.2	172.3	140.2	156.3
Acetone	67-64-1	45	62.9	54	58	56.6	57.3	50.3	38.1	44.2

* Compound for which concentration (µg/m³) is below the quantification limit allowed by ANSI BIFMA.

The levels of emitted formaldehyde converted into parts per billion (ppb) are summarized in Table 11B.8 and, as can be seen, emissions are just of the order of a few parts per billion. Compared to the European E1 wood products formaldehyde emission limit of 0.1 ppm (100 ppb), all five CLT products tested had emissions 6 to 20 times lower than the E1 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 labeling (Table 11B.5), three of the five samples meet the formaldehyde emission limits. The two samples encoded 114-3S and 190-5S would need to be tested for longer periods of time (ranging from two to three days) in order to be qualified for the most stringent GUT (Germany) labeling system, which has its formaldehyde emission limit set at 10µg/m³ after three days of sample exposure in the controlled environmental chamber.

Table 11B.8 24-hour formaldehyde emissions as a function of product type

Formaldehyde	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
	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 Californian government, known under the acronym of CARB Phase I and Phase II, for wood composite products including particleboard, MDF, thin MDF, and hardwood plywood (HWPW) with composite core (HWPW-CC) or veneer core (HWPW-VC) have been in effect since July 1st, 2012. The final formaldehyde emission limits are: 0.13 ppm (130 ppb) for thin MDF, 0.11 ppm for regular MDF, 0.09 ppm for particleboard and 0.05 ppm for both hardwood plywood (HWPW) with veneer core (VC) or composite core (CC). By comparing these limits to the results obtained for the CLT products shown in Table 11B.8, it can be concluded that the CLT products easily meet the most stringent CARB limits of 50 parts per billion (ppb).

11B.3 CONCLUSIONS AND RECOMMENDATIONS

Five CLT products were tested for their emitted VOCs, including formaldehyde and acetaldehyde emissions, in order to assist architects, engineers, and builders to better select construction materials with low-emitting characteristics, which have less impact on indoor air quality. The tested laminated products had different thicknesses and a different number of glue lines. Emissions were collected after 24 hours of samples exposure in the environmental chamber.

Results did not show any correlation between individual VOCs (iVOCs), including formaldehyde and acetaldehyde, or TVOC and the thickness of the CLT panel or the number 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 material used in the manufacture of the CLT products.

In terms of evaluating the impact of CLT on indoor air quality, one can easily conclude that it would be negligible, if any. The five CLT product TVOCs and formaldehyde 24-hour results were generally lower than those set forth by some European emission labeling systems; it is worth noting that the European limits are based on emissions measured after 3, 10 or 28 days of sample exposure. In addition, the European E1 grade for wood product 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 CLT products.

Comparing the results obtained with the CARB limits, one can conclude that the CLT products tested in this study would easily meet the most stringent CARB limit of 50 ppb. However, when architects, builders and engineers are using CLT products other than those tested in this study, we recommend conducting a validation that the emissions of the products to be used meet the requirements, because emissions are a wood species characteristic.

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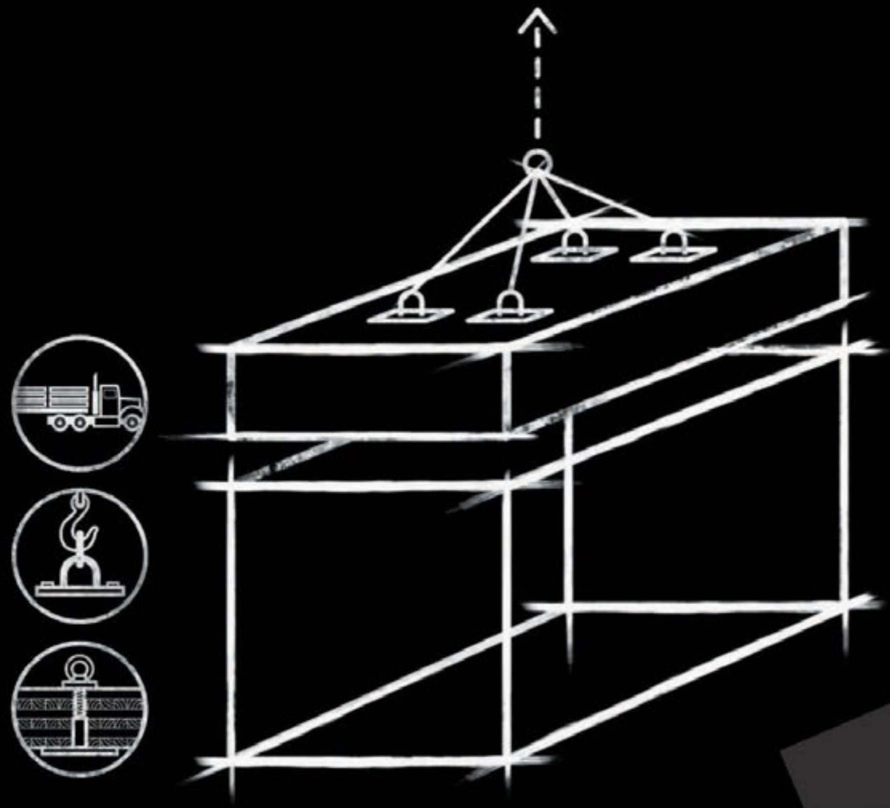

CHAPTER 12

Lifting and handling of CLT elements

Lead Author
Sylvain Gagnon

Co-Author
Jan Michaelsen

Reviewer
Erol Karacabeyli

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ABSTRACT

Cross-laminated timber (CLT) construction is now a reality throughout North America. There are several CLT producers in North America and more are expected to enter the market in the coming years. Many buildings erected in whole or in part with CLT that allow us to observe the construction techniques that are currently preferred in Canada and the United States. This Chapter presents a wide range of lifting systems that can be used in the construction of structures made of cross-laminated timber (CLT) panels, some of them being currently used while others are suggested. A number of systems discussed in this Chapter are directly inspired by the systems used in precast concrete construction.

We also discuss the basic theory required or suggested for proper lifting techniques. In addition, various tools and accessories that are frequently required for CLT construction, as well as good building practices to help contractors build safe and efficient CLT panel structures are presented. Finally, issues related to the transportation of CLT assemblies from factory to building site are discussed along with regulatory aspects of transportation.

It is important to note that the lifting, handling, and installation of CLT panels involve multiple interest groups including design professionals, contractors/erectors and CLT manufacturers, each with different areas of interest and expertise. Therefore, the information presented in this Chapter is broad in scope and may or may not be applicable entirely to each interest group.

12.1 INTRODUCTION

CLT construction is a relatively new process in North America. Although current CLT manufacturers located in Canada and the USA provide some recommendations on lifting systems for the installation of prefabricated wood assemblies, there is still very little specific technical documentation for the erection of structures designed and built with CLT panels and adapted to the North American reality. The aim of this Chapter is to add to the current available documentation, including information on the handling and transportation of CLT panels.

This Chapter presents a wide range of lifting systems that can be used in the construction of structures made of cross-laminated timber (CLT) panels, some of them being currently used while others are suggested. The basic theory required for proper lifting techniques is also discussed. In addition, various tools and accessories that are frequently required during CLT construction are introduced, as well as good building practices to help contractors build safe and efficient CLT panel structures. Finally, issues related to the transportation of CLT assemblies from factory to building site, as well as regulatory aspects of transportation are also discussed.

12.1.1 Parallel with the Precast Concrete Industry

A close look at Figure 1 reveals that techniques used in precast concrete construction using large concrete slabs is, in many ways, similar to the current techniques used in CLT construction. As the precast concrete construction industry is still more developed and experienced, it is advantageous for CLT designers and contractors to obtain or use systems and lifting accessories adapted to this more mature industry and to build on their experience.

For example, certain systems discussed in this Chapter, which are sometimes used in CLT construction, are directly inspired by the systems used in precast concrete construction. In addition, a large amount of technical data contained in the following sections has been adopted from documentation developed and provided by major producers of precast concrete, or by manufacturers of specialized lifting devices for precast concrete.

Figure 1 Lifting and handling of precast concrete elements

12.1.2 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 a 12-story building in a downtown area typically requires more preparation and precaution than a small office building built in the suburbs or surrounding rural areas. But if that country house is to be perched high in the mountains, the techniques that are 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 the lifting and handling processes of CLT panels on construction sites. The techniques and lifting systems used are discussed in detail later in this Chapter.

Figure 4 Lifting and handling of a relatively light CLT element, in Norway (courtesy of Brendeland and Kristoffersen, Architects)

Figure 5 Lifting and handling of a CLT floor element, in Montréal, Québec, Canada

Figure 6 **Lifting and handling of a CLT wall element, in Québec City, Québec, Canada**

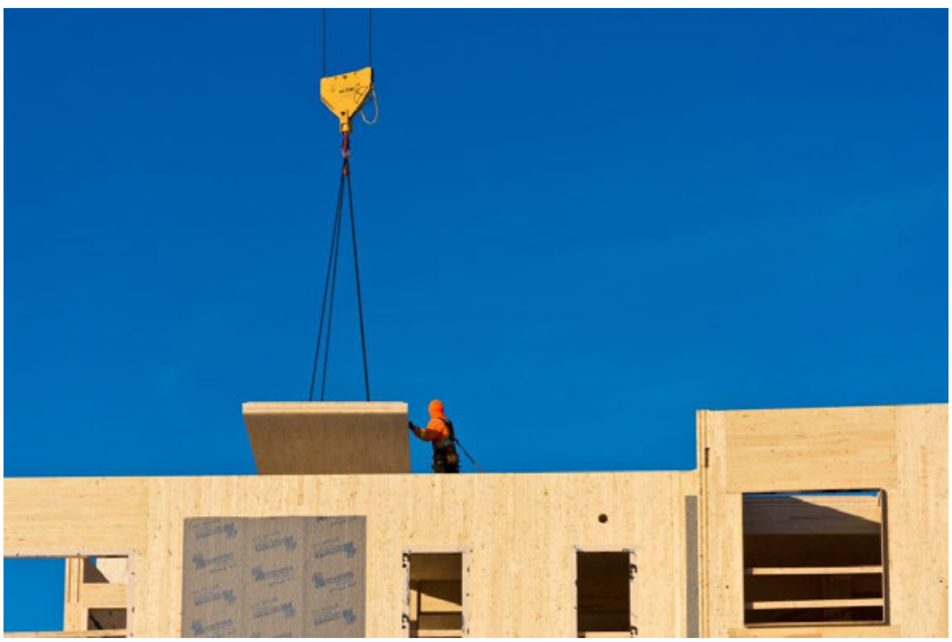


Figure 7 **Lifting and handling of a CLT floor element in Québec City, Québec, Canada**

Figure 8 Lifting and handling of a CLT floor element in Lac Etchemin, Québec, Canada

Figure 9 Lifting and handling of a CLT floor element in Växjö, Sweden

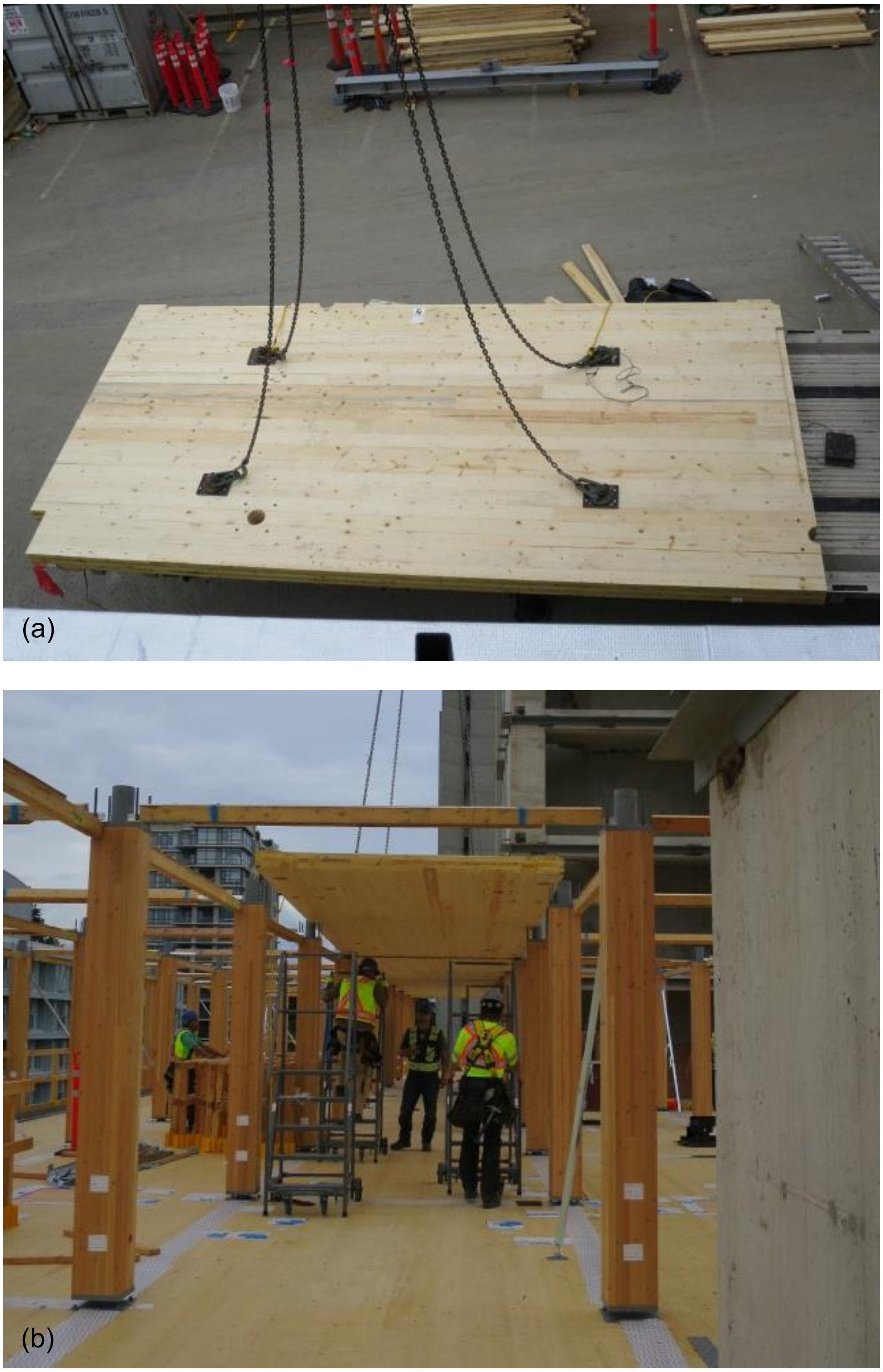


Figure 10 Lifting and handling of CLT elements, in Vancouver, BC, Canada

12.2 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 inspired by systems used in the precast 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. Also, the inspection of all lifting devices is the responsibility of the user and must be conducted by qualified personnel.

12.2.1 Contact Lifting Systems

Lifting systems using steel plates that provide compressive resistance on the lower face of the panels during lifting are generally considered the safest CLT panel handling methods. However, it appears that this system has rarely been used in Canada and the United States until now.

To avoid accidents on the lower levels of the building once the panels are in place, great care must be taken when removing the lifting system, as the steel plates are usually not secured once the system is unbolted.

This lifting technique typically requires in-plant drilling to allow the insertion of dowels or threaded sleeves with nuts. This technique uses the wood’s inherent strength in compression perpendicular to the grain. 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; cutaway views are shown for simplicity and clarity.

12.2.1.1 Single Lifting Loop with Threaded Sleeve Used with Socket Steel Tube Welded onto Flat Steel Plate

This system, comprised of a single lifting loop with threaded sleeve, is widely used in the construction of precast concrete. The system shown in Figure 11 is a modification of the system commonly used to lift precast concrete. Instead of enclosing the welded plate socket in concrete at the plant, the socket is welded to 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 precast concrete industry can be reused but the contractor must verify its ongoing performance through rigorous design and have a careful inspection and quality control system to ensure safety. When using this system, the recommended maximum angle (β) is 30° (Figure 35). 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.

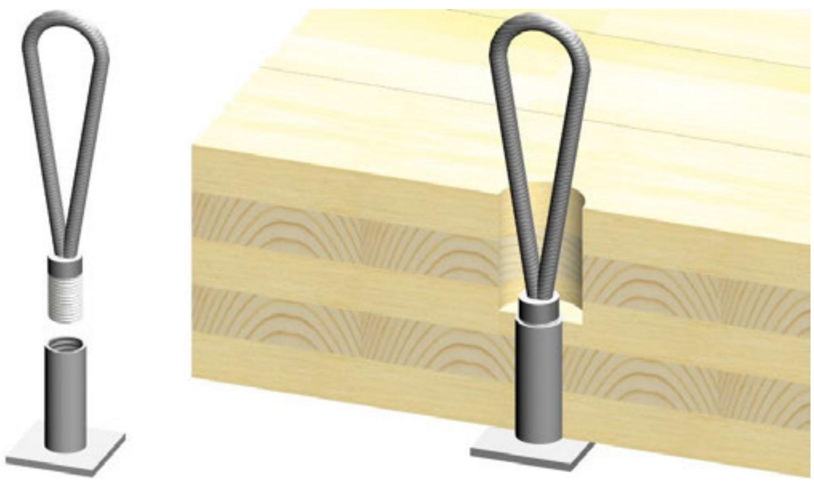


Figure 11 Single lifting loop with threaded sleeve used with socket steel tube welded onto flat steel plate

12.2.1.2 Articulated Lifting Loop with Threaded Sleeve Used with Socket Steel Tube Welded onto Flat Steel Plate

This system, made of an articulated lifting loop with threaded sleeve, also comes from the precast concrete industry and is installed in the same manner as the previous system (Figure 12). One advantage of this system is the ability of the steel cable to rotate in all directions around the threaded sleeve. However, the lifting angle should still be limited to 30°. When handling is completed, the two components must be removed carefully.

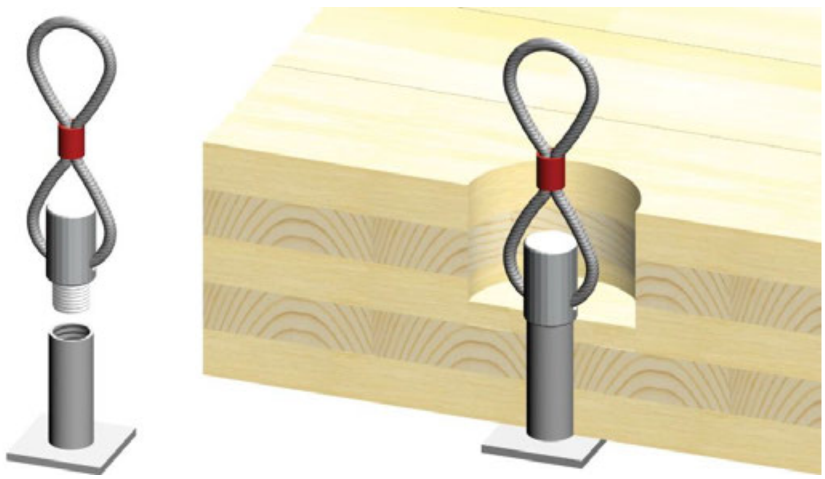


Figure 12 Articulated lifting loop with threaded sleeve used with socket steel tube welded onto flat steel plate

12.2.1.3 Articulated Lifting Hook with Threaded Sleeve Used with Socket Steel Tube Welded onto Flat Steel Plate

This system, with articulated lifting hook and threaded sleeve used with a socket steel tube welded onto a flat steel plate, also comes from the precast concrete industry (Figure 13). The hook allows for quick installation on the lifting system and has the ability to rotate around the steel ring. The lifting angle should still be limited to 30°. When handling is completed, the two components must be removed carefully.

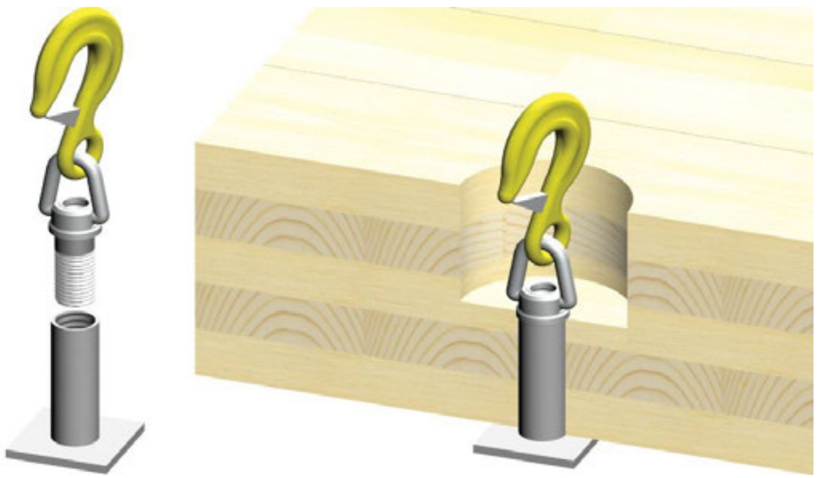


Figure 13 Articulated lifting hook with threaded sleeve used with socket steel tube welded onto flat steel plate

12.2.1.4 Threaded Eyelet Bolt Used with Socket Steel Tube Welded onto Flat Steel Plate

A 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 to install it correctly (Figures 14 and 15). It is recommended to use an eyelet base bolt when lifting heavy loads at an angle, and ensure there is proper contact between the base and the wood panel, as well as sufficient thread engagement between the eyelet and threaded sleeve. Plain or regular eyelet bolts (without base) are normally used in straight tension when lifting light loads; that is, when used with a spreader beam or with only one attachment point. In addition, in accordance with good practice, the eyelet bolts must be oriented in the same direction as the tensioned slings, to prevent the eyelet from bending under heavy oblique loads. When handling is completed, the two components must be removed carefully.

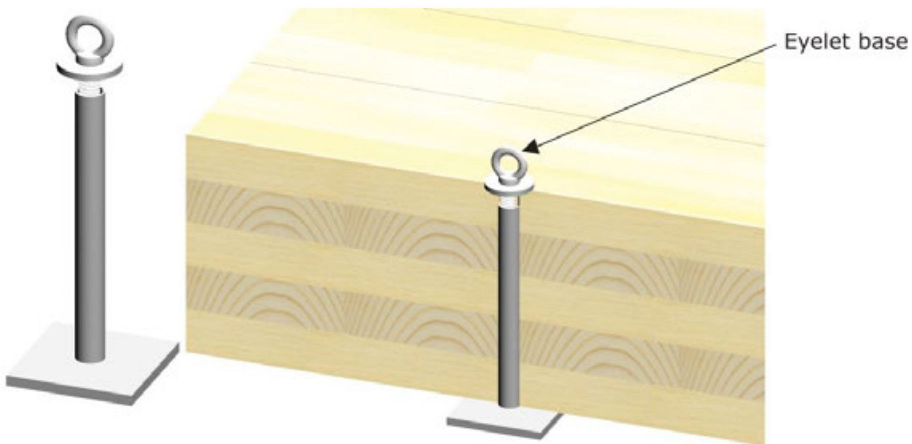


Figure 14 Threaded eyelet bolt (with base) used with socket steel tube welded onto flat steel plate

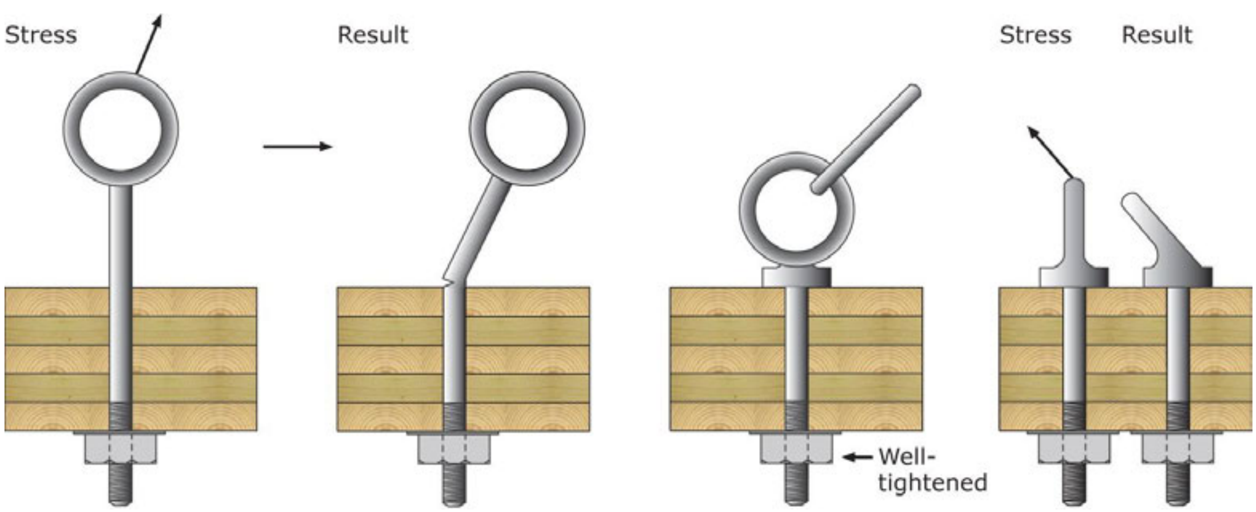


Figure 15 Correct use of threaded eyelet bolt (with and without eyelet base)

12.2.1.5 Threaded Eyelet Bolt Used with Steel Plate and Nut

This system, which uses a threaded eyelet bolt in conjunction with a steel plate and nut, is sometimes used in CLT construction (Figure 16). It is important to choose the proper eyelet bolt and to install it correctly. The use of an eyelet base bolt when lifting at an angle is also strongly recommended. In addition, in accordance with good practice, the eyelet bolts must be oriented in the same direction as the tensioned slings to prevent the eyelet from bending under heavy oblique loads. When handling is completed, the system must be entirely removed.

Figure 16 Threaded eyelet bolt used with steel plate and nut

12.2.1.6 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 a 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 entirely and carefully removed.

Figure 17 Eyelet used with threaded bolt or sleeve and steel plate (courtesy of Nordic Structures)

12.2.1.7 Threaded Eyelet Bolt, Threaded Socket, Threaded Bolt and Steel Plate

A threaded eyelet bolt can be used with a threaded socket, a bolt, or a threaded rod and steel plate (Figure 18). The threaded socket is normally pre-installed in the CLT plant for future use on site. On the construction site, the eyelet bolt and the single bolt or the threaded rod are easily screwed to the plate. Again, it is important to choose the right eyelet bolt and to 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 to increase the adaptability of the building. For instance, dismantling of a building, as well as repairing and upgrading operations, require the presence of elements or tools that facilitate the handling process of the building components.

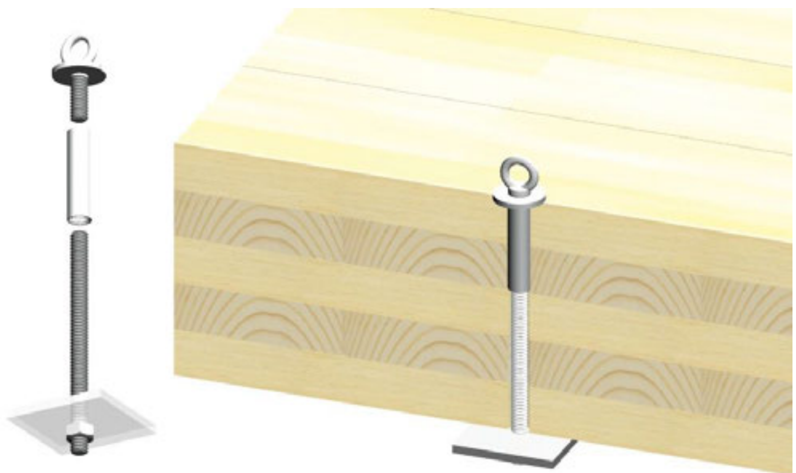


Figure 18 Threaded eyelet bolt, threaded socket, threaded bolt or sleeve and steel plate

12.2.1.8 Threaded Eyelet Bolt, Threaded Socket and Steel Round Rod

Another product that comes from the precast concrete construction industry can inspire a new system for lifting light CLT panels (Figure 19). A threaded socket with holes at the tip is inserted into the CLT slab. This socket would normally be 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. This proposed system is suitable for lightweight CLT panels only (e.g., less than ½ ton).

Figure 19 Threaded eyelet bolt, threaded socket and steel round rod

12.2.1.9 Soft Lifting Sling Used with Support

Another system sometimes used in CLT construction is shown in Figure 20. A hole is drilled into the panel, usually at the CLT plant (2~3 inches or 50~75 mm in diameter). 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 this system being used with a piece of dimensional lumber. However, it is important to ensure that the locking parts are properly fixed and will not slip during handling. This proposed system is suitable for small and lightweight CLT panels only (e.g., less than ½ ton).

Figure 20 Single lifting sling used with support

12.2.1.10 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 or two holes and a flexible sling is often used, as can be seen in Figure 21. The sling must be load rated for the panels being tilted and/or lifted. Since wall elements are often lighter than thick floor slabs, this system is often appropriate for tilting up, lifting, and placement of the panels. The holes must be plugged once handling is completed, especially those in the exterior walls and partition walls between units.

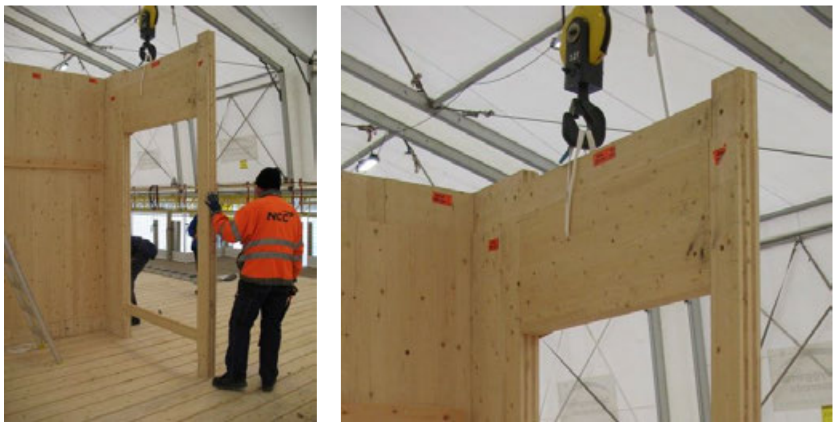


Figure 21 Lifting sling without support (with hole)

12.2.1.11 Soft Lifting Sling without Support for Horizontal Elements

The simple lifting system shown in Figure 22 does not require holes to be drilled in the panels. However, this technique comes with a real risk of instability due to the possibility of the 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 (Sections 12.4.3 to 12.4.5).

Figure 22 Lifting sling without support (without hole)

The next technique requires two holes to be drilled at the CLT plant for each anchor point. These holes have a diameter of approximately 2 inches (50 mm) and are relatively close together but not less than a distance equal to the thickness of the CLT panel. A soft sling is inserted as shown in Figure 23.

Figure 23 Lifting sling without support (with holes)

12.2.2 Screw Hoist Systems

There are several lifting techniques that rely only on the withdrawal resistance of fasteners. Although these techniques are simple and effective, they require a careful design analysis for the loads involved and 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.

12.2.2.1 Screwed Anchor

One of the most widely used screw hoist system in Europe is shown in Figure 24. This system is based on an anchor used in precast 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 self-tapping screw is usually installed in the plant by the CLT manufacturer as a recess is normally required in order to embed the fixed piece. 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 for the design professional to refer to the manufacturer's technical data in order to determine the allowable loads and for usage and installation specifications. Figure 25 (a) and (b) show screwed anchors in service.

Figure 24 Lifting system with self-tapping screw


(a)



(b)


Figure 25 Screwed anchor for wall (a) and floor (b)

12.2.2.2 Screwed Plate and Lifting Ring

There are various lifting systems that use screws or lag screws in combination with steel plates with holes. This is currently the most common lifting technique used in Canada. Figure 26a shows a system that uses only two self-tapping screws. 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 the lifting capacity. Figure 26b shows a much more flexible system. The plate has a sufficient number of pre-drilled holes to accommodate several lag screws or wood screws. Thus, the plate provides the professional 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 to ensure they have not been damaged during previous uses. Figure 26c shows a CLT panel ready to be lifted.

Lag screws should only be installed in a properly sized lead hole (see CSA O86 standard for more information) and care must be taken during installation of the lag screws to prevent stripping out of the wood. When pneumatic or electric tools are used to drive lag screws, proper calibration and maintenance of torque-limiting clutch systems is essential.

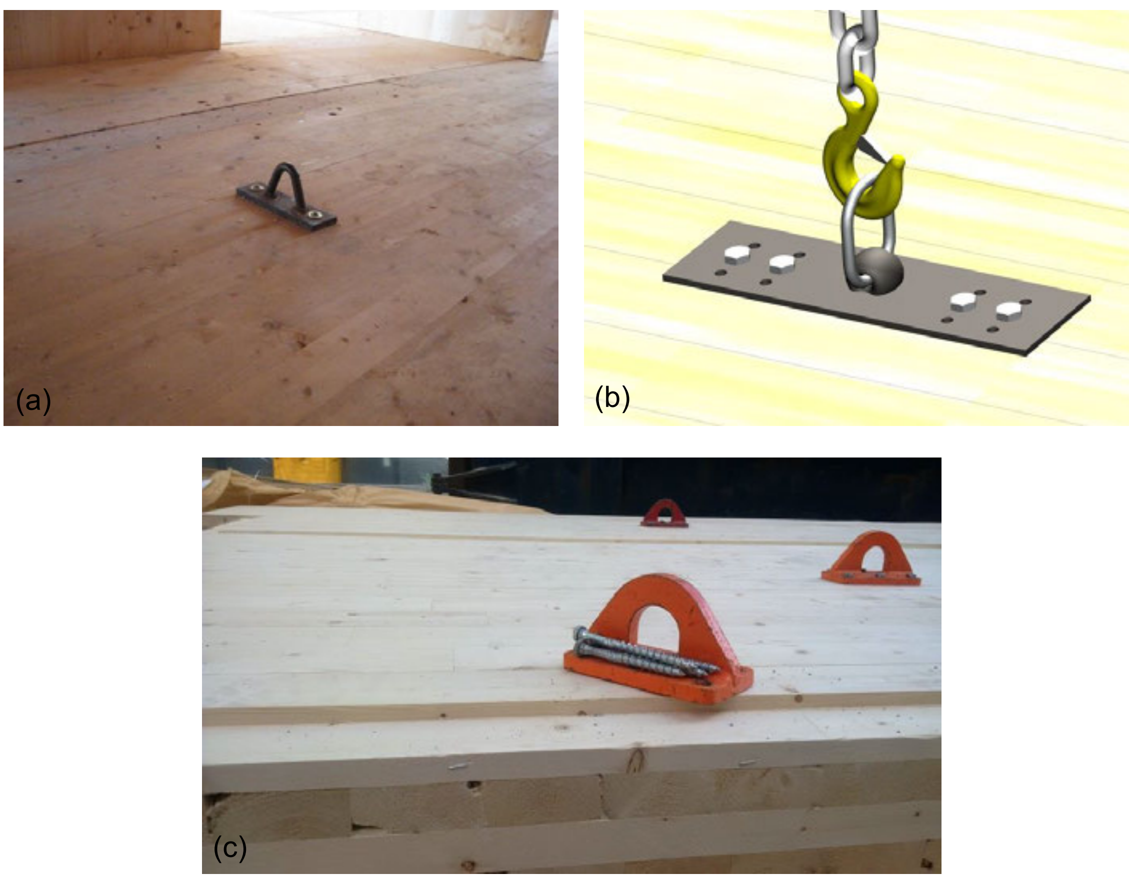


Figure 26 Screwed plate and lifting ring

12.2.2.3 Double-Threaded Socket with Eyelet Bolt or Lifting Loop with Threaded Sleeve

Another lifting method involves using a double-threaded socket (i.e., threaded inside and outside) together with an eyelet bolt or lifting loop (Figure 27). The socket is screwed into the panel at the CLT plant. As with a lag screw, a hole with a diameter equal to 75~90% of the socket diameter must first be drilled in the wood. It is important that the design professional refers to the manufacturer's technical data to determine the acceptable installation and usage of these proprietary lifting systems.

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 sides of the panels. It is important to refer to the manufacturer's technical data to determine the allowable loads and for usage specifications.

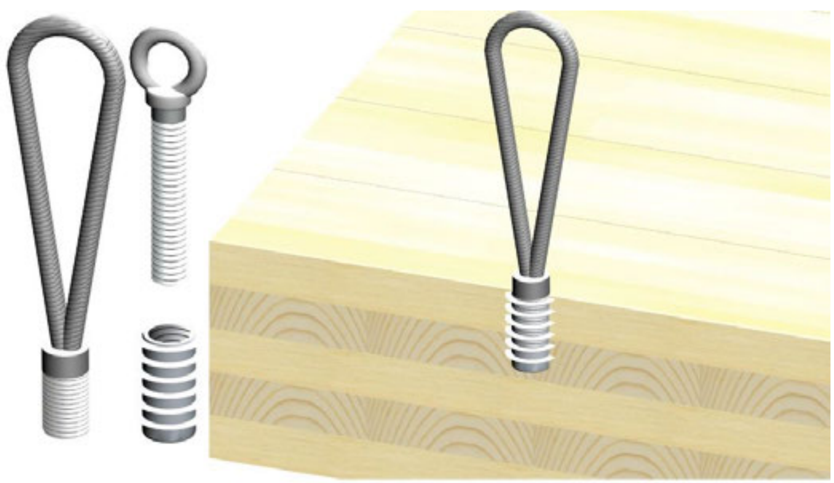


Figure 27 Double-threaded socket with eyelet bolt or lifting loop with threaded sleeve

12.2.2.4 Innovative Lifting System with Wood Screws and Eyelet Bolt

Some manufacturers may offer other innovative anchoring systems. An example is shown in Figure 28. Wood screws are used to screw a cylindrical steel component to the panel. This piece is usually attached on the top of the panel. However, a recess can be made into the panel at the CLT 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. Again, it is important that the design professional refers to the manufacturer's technical data to determine the acceptable installation and usage of these proprietary lifting systems.

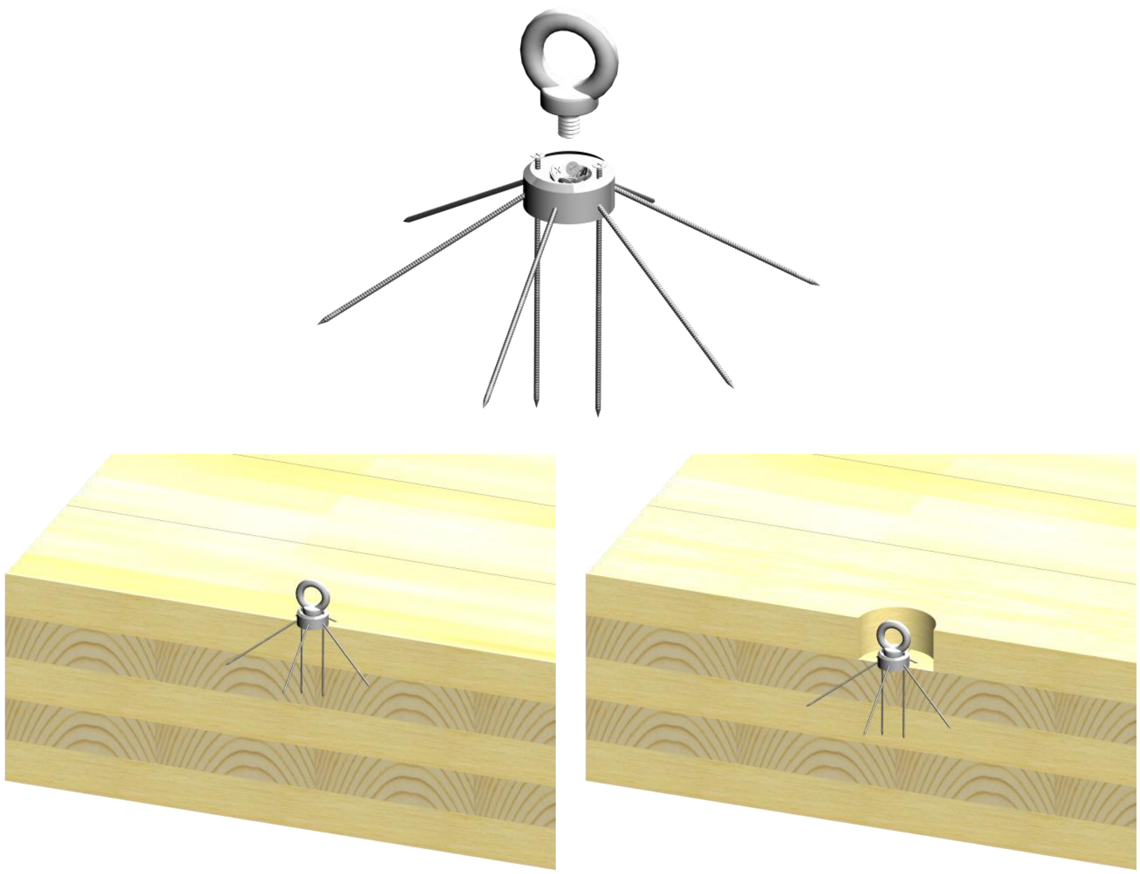


Figure 28 Innovative lifting system with wood screw (with or without recess)

12.2.3 Lifting Systems

The principle of using plant-integrated support parts lends speed to jobsite execution on construction sites. These systems are simple and safe. In addition, if the ceilings of the building 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.

12.2.3.1 Inserted Rod with Soft Sling

This technique is sometimes used in Europe and Canada. It consists of first drilling one hole on the top of the panel a few inches from the edge, depending on the dowel bearing strength of the CLT panel (see Chapter 5). This hole, which has a diameter of about 2~3 inches (50~75 mm), is drilled at the CLT plant by a CNC machine 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 inserted into the hole. It is possible to use smooth rods or steel reinforcing bars. Upon insertion of the rod, a soft sling is installed (at the plant) and held by the rod. The hole should be large enough to position the sling within the hole for easy stacking during transportation. Once the lifting and handling steps have been completed, the sling is either cut or reinserted into the hole for future use. Figure 29a shows the first system.

Figure 29b shows a similar system. However, instead of drilling a hole, a groove is made on the top of the panel, a few inches from the edge. The alteration is performed using a CNC machine at the CLT plant, 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 inserted into the hole. Once the panel is on the construction site, a soft sling is simply slipped 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.

(a)

(b)

Figure 29 Inserted rod with soft sling

12.2.3.2 Inserted Rod with Lifting Hook

This next system is once again inspired by techniques used in the precast concrete construction industry. This method consists of drilling one hole on the top of the panel (re-entrant), a few inches from the edge, depending on the dowel bearing strength of the CLT panel used (see Chapter 5). 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 drilled at the CLT plant using a CNC machine 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 inserted into the hole. Once the panel is on the construction site, the hook is attached to the rod for the lifting and handling phase. The steel rod remains in place and the hole should be sealed.

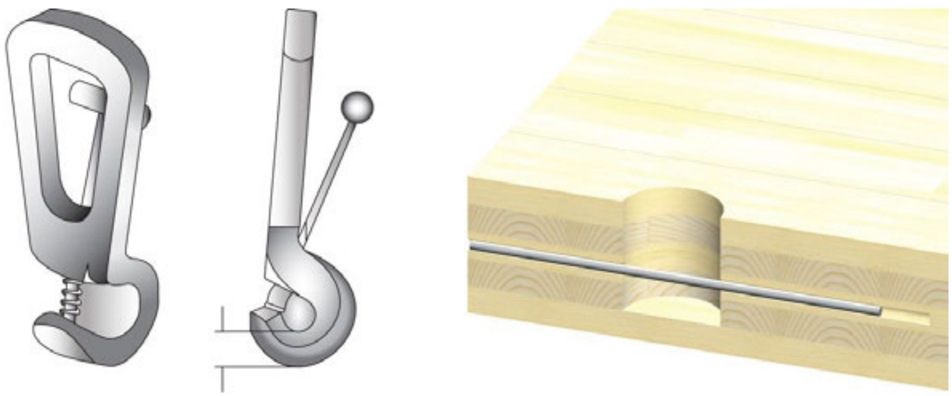


Figure 30 Inserted rod with lifting hook

12.2.3.3 Friction System

The system shown in Figure 31 was observed recently in Europe during the construction of a seven-story building. A hole the size of the diameter of the bottom portion of the system (i.e. once it is closed) is drilled in the CLT to the depth required for the total insertion of the steel part, all the way to the ring. Once inserted, the upward movement of the system when lifted causes the lower part to open and enter into contact with the CLT, because of friction. This prevents the system from detaching from the element and allows the CLT to be lifted. It should be noted that this system presents certain risks. It is thus recommended that this system be studied in detail, before it is used.

Figure 31 Inserted friction lifting hook

12.3 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 utmost importance that lifting equipment be positioned and operated properly. Several criteria must be verified and validated prior to, and during work on site. Design professionals and contractors/builders in charge of a construction project involving CLT panels need to consider several important points. Some recommended considerations are presented in the following sections.

12.3.1 Lifting Station and Devices

The lifting station is undoubtedly a key location on the construction site. The lifting device must be selected and positioned according to several criteria. Certain construction sites may require more than one lifting device 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 the construction:
  - types of loads may vary on the same construction site
  - if possible, the lifting device should not be moved; however, it should be possible to move the lifting device if the jobsite and/or the erection conditions require it to be moved.
- reach appropriate heights and distances with the required maximum load:
  - appropriate range must be attained for all required distances, from point A to point B;
  - the travel path of the element to be lifted to reach the desired location must be clear of any obstacles.
- be efficient, capable of maintaining the needed working 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 unplanned consequences, 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 an excessive load;
- the ground can degrade under the device's bearing points;
- a device that is too close to a slope can become unstable and tip over; and
- the device's range may allow it to come into contact with obstacles (e.g., buildings, trees, a second crane, power lines, etc.).

Despite taking all these precautions, 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.

12.3.2 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. The whole weight of the load is then considered concentrated at this center of gravity.

Although the density of wood varies greatly depending on the wood species (specific gravity) and moisture content, i.e. between 320 and 720 kg/m^3 (Wood Handbook, 2010), it is recommended to use an average density ranging between 400 and 600 kg/m^3 $(or\ 4{\sim}6\ kN/m^3)$ for the calculation of the total weight of CLT elements made of softwood lumber. Note that this density is about five times lower than the density used for precast reinforced concrete elements, which is usually about 2400 kg/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 6,000 kg (6 tons or about 60 kN). It is strongly suggested that the weight of the particular CLT panels to be used be obtained from the CLT manufacturer, as the total weight will vary depending on the wood species used. The following section illustrates how to calculate the weight of a CLT panel.

The total weight of a CLT slab is simply calculated as follows:

$$P = V \times \rho_{CLT} \tag{1}$$

$$V = b \times L \times h \tag{2}$$

where:

P = CLT slab weight (kN)

V = Volume of slab to be lifted and handled (m^3)

b = Slab width (m)

L = Slab length (m)

h = Slab thickness (m)

ρ_{CLT} = CLT slab average density (4~6 kN/m^3)

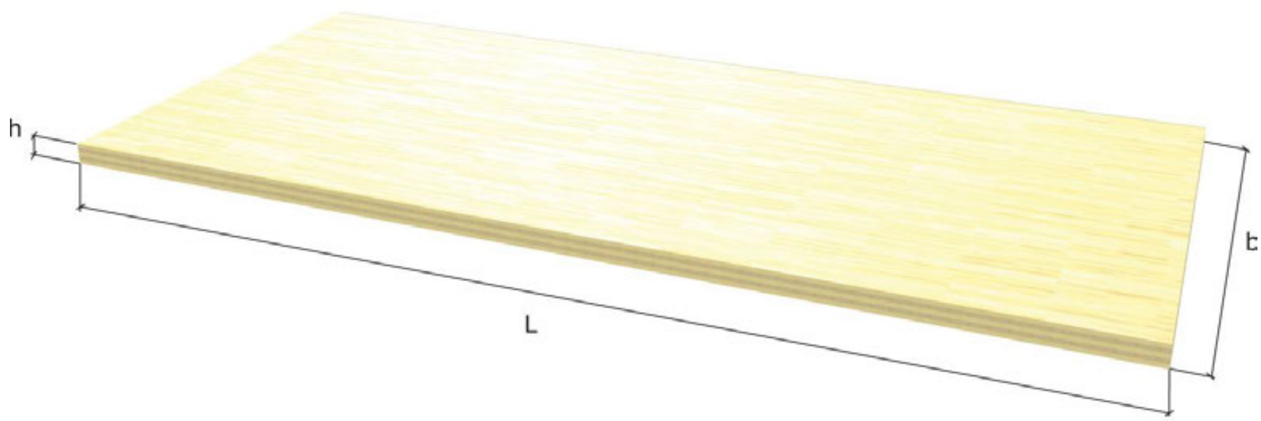


Figure 32 Dimensions for calculating the weight of a CLT slab

12.3.3 Dynamic Acceleration Factors

12.3.3.1 Lifting System Used

During lifting and handling maneuvers, 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 suggested lifting and handling dynamic acceleration factors for specific devices used in construction. These factors should be taken into account for the calculation of forces.

IMPORTANT: Note that the tabulated values are provided for informational purposes only. It is important to refer to normalized values, if any, as provided by the relevant authorities (e.g., Provincial, federal, municipal, etc.).

Table 1 Dynamic acceleration factors (f)

Lifting device	Dynamic acceleration coefficient f
Fixed crane	1.1 ~ 1.3
Mobile crane	1.3 ~ 1.4
Travelling crane	1.2 ~ 1.6
Lifting and moving on flat ground	2.0 ~ 2.5
Lifting and moving on rugged ground	3 ~ 4 and +

Sources: Pfeifer, Snaam, Halfen, Peikko, Arteon

12.3.3.2 Other Effects to Consider

Wind can significantly increase forces in lifting systems. CLT manufacturers, contractors, and design professionals in charge of a project should 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 unwise to lift loads when weather conditions are deemed dangerous. Prefabricated CLT elements are subject to wind movement and this phenomenon should not be underestimated, especially in tall wood building construction.

The use of guide ropes is recommended 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 the (sequence of) operations intended by the engineer.

12.3.4 Asymmetrical Distribution of Load According to Center of Gravity

It is always better to fix anchors in a way that limits the eccentricity due to the center of gravity of the element to be lifted. If anchors are asymmetric with regards 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 panels 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 33 shows the appropriate method. However, if the lifting of an element is conducted 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.

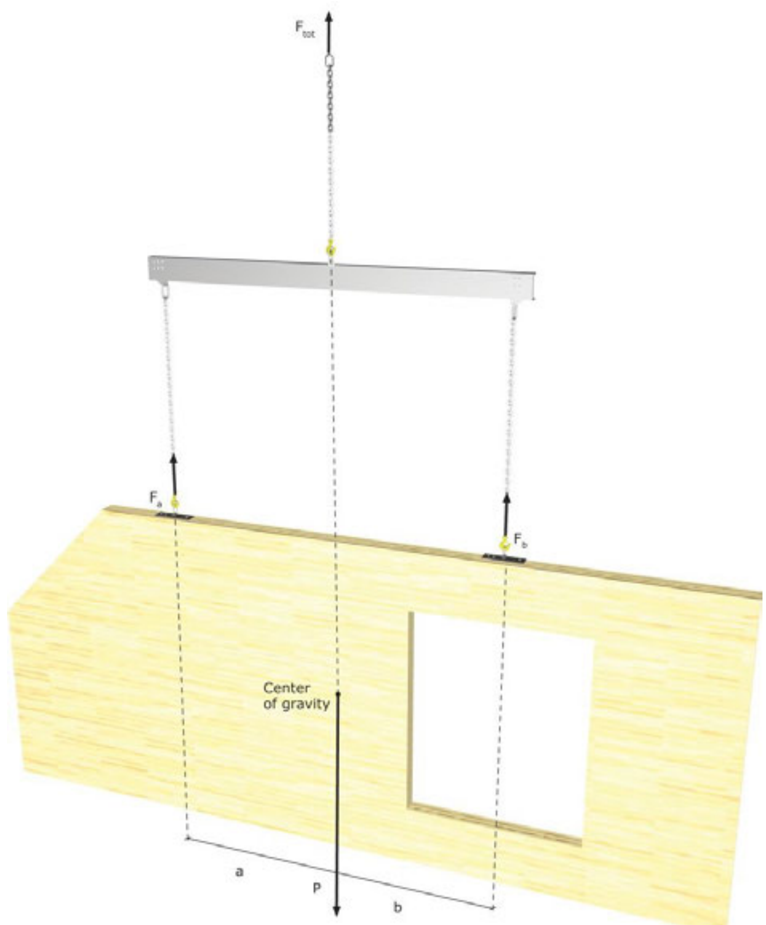


Figure 33 Element lifted with a spreader system

For example, the next equations are used to calculate forces in two anchors placed asymmetrically from the center of gravity of an element that is being lifted with a spreader system. The center of gravity required for determining measures "a" and "b" may be given by the CLT manufacturer when CAD software and CNC machines are used.

$$F_a = \frac{P \times b}{(a+b)} \quad [3]$$

$$F_b = \frac{P \times a}{(a+b)} = P - F_a \quad [4]$$

12.3.5 Determining Forces According to Lifting Angles

When a spreader system similar to that shown in Figure 33 is not used for lifting 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 for various inclination angles β. Please refer to Figure 35 for more details about angles. These coefficients are used in Equation 5 presented later in this Chapter.

Table 2 Coefficient of lifting angle (β)

Angle of cable β (1)	Angle α (2)	Coefficient of angle 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

12.3.6 Determining Load Distribution According to the Number of Effective Anchors (Suspension in Several Effective Points « N »)

It is common practice to use only two anchor points when CLT wall or beam elements are handled on the construction site. In these cases, it is normally sufficient to determine forces at 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 in relation to the center of gravity. Indeed, there is no guarantee that the load will be perfectly symmetrical in relation to the center of gravity or that the slings will be of exactly the same length. It is therefore strongly suggested 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, the calculations should be made so that each anchor would be capable of supporting the total load of the assembly (N = 1).

Furthermore, to ensure proper distribution of forces for each anchor considered to be effective, it is important to use systems with minimal 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 in order to avoid instability or creating 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 34 to 41 present typical cases of CLT element lifting and the number of effective anchors suggested in the calculations.

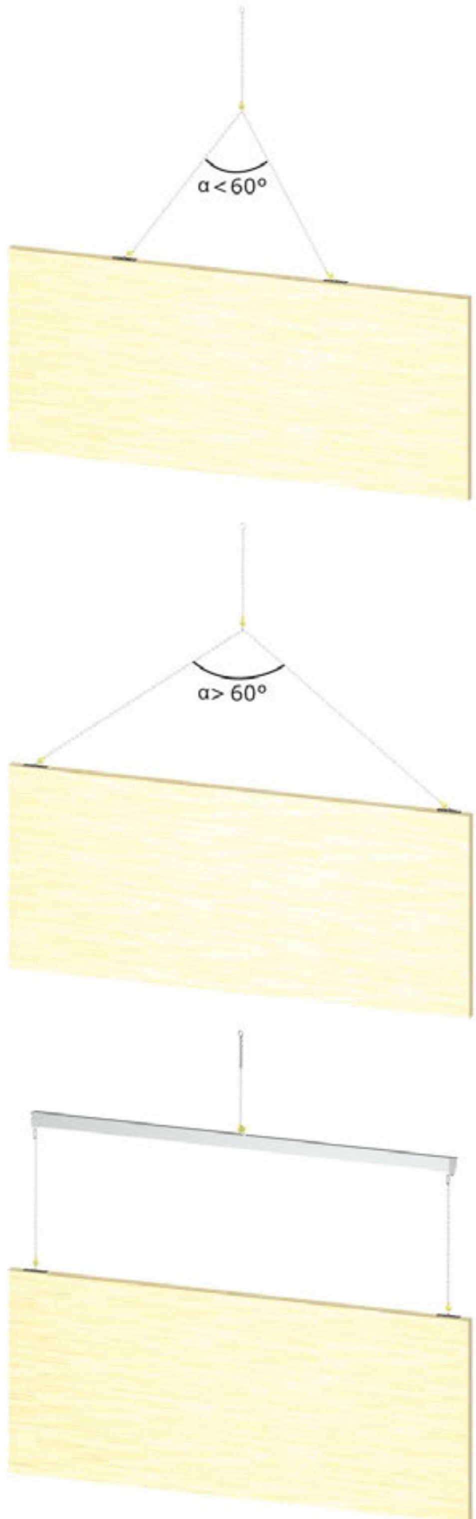
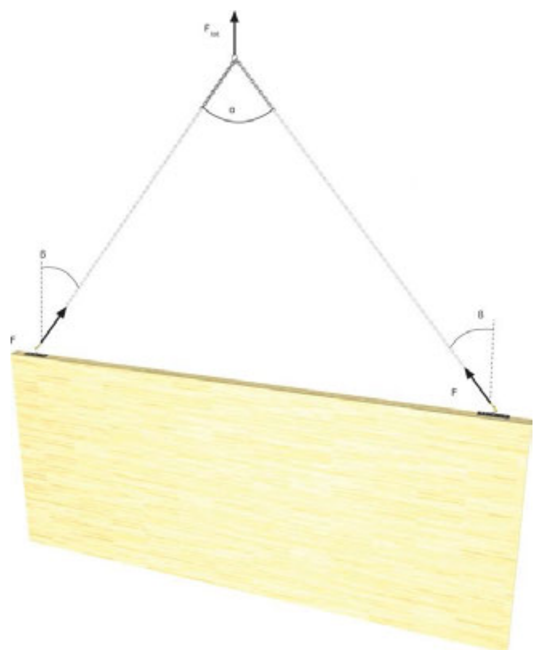
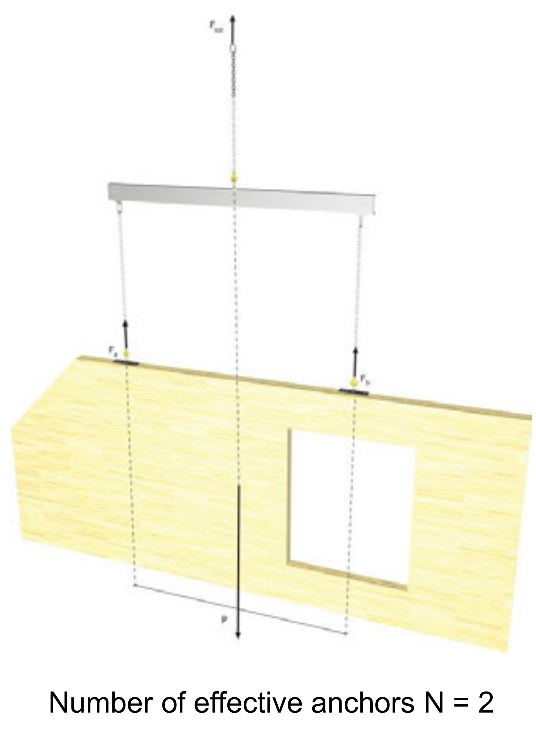


Figure 34 CLT wall lifted with two slings symmetrically positioned – Good and bad practices



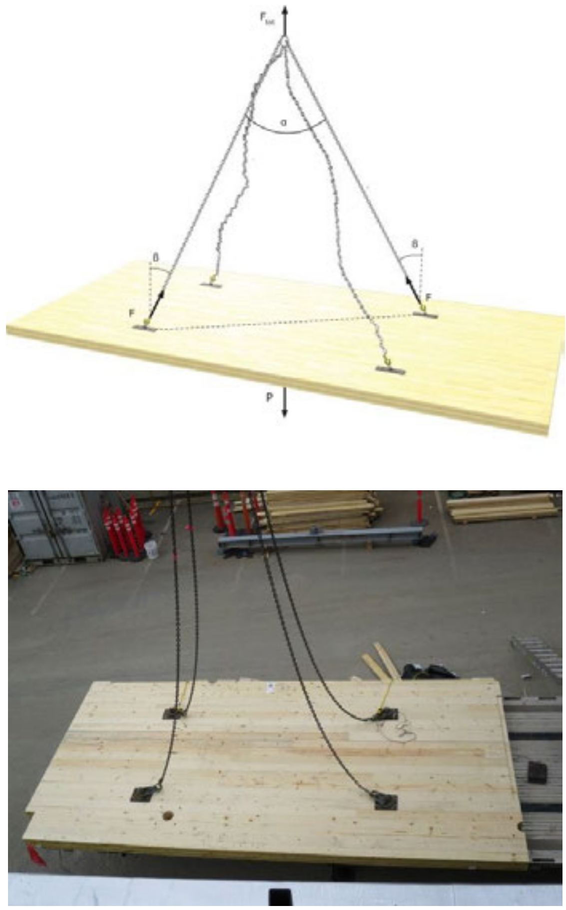
Number of effective anchors = 2

Figure 35 CLT wall lifted with two slings symmetrically positioned with F_{tot} in line with the center of gravity (N = 2)



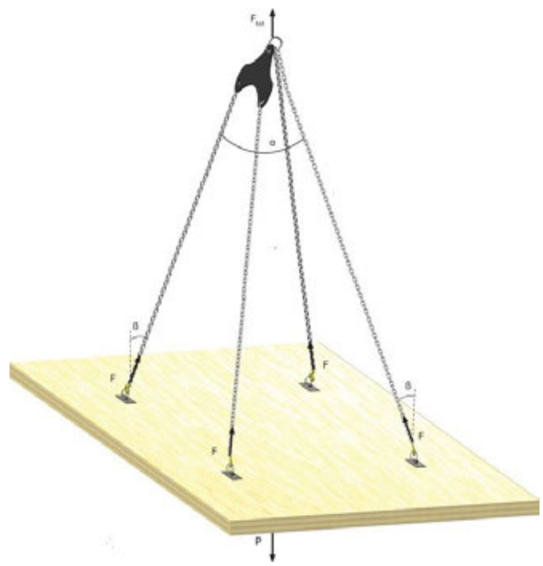
Number of effective anchors N = 2

Figure 36 CLT wall lifted with two slings asymmetrically positioned with F_{tot} in line with the center of gravity, with single spreader (N = 2)



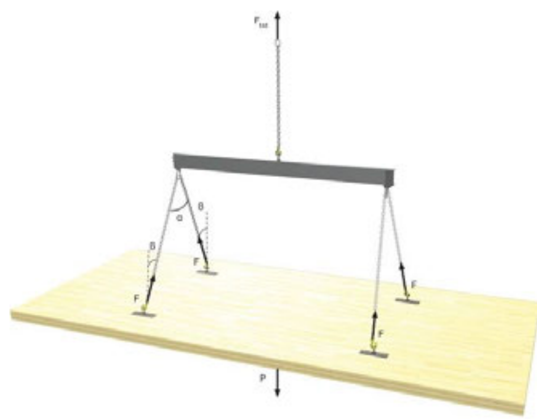
Number of effective anchors = 2

Figure 37 CLT slab lifted with four slings symmetrically positioned in relation to the center of gravity, without spreader and without compensation system (N = 2)



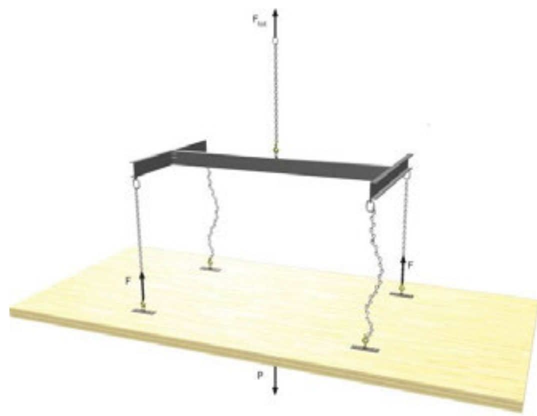
Number of effective anchors = 4

Figure 38 CLT slab lifted with four slings symmetrically positioned in relation to the center of gravity, with compensation system (N = 4)



Number of effective anchors = 4

Figure 39 CLT slab lifted with four slings symmetrically positioned in relation to the center of gravity, with single spreader (N = 4)



Number of effective anchors = 2

Figure 40 CLT slab lifted with four slings symmetrically positioned in relation to the center of gravity, with single spreader, with three fixed spreaders (N = 2)

Number of effective anchors = 4

Figure 41 CLT slab lifted with four slings symmetrically positioned in relation to the center of gravity, with single spreader, with three free spreaders (N = 4)

12.3.7 Calculation of Forces Resulting from Lifting with Anchors

The maximum forces resulting from lifting using anchors must be evaluated at each stage of the lifting and handling processes. 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. stationary 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 the final destination of the element, it may be necessary to use an oversize anchor to accommodate the effects of repetition.

For loads that require lifting with slings placed symmetrically in relation to the center of gravity, the force per anchor is calculated as follows:

$$F_i = \frac{F_{tot} \times f \times z}{N} \qquad [5]$$

Where:

F_i = Resultant anchor force (kN)

F_{tot} = *P* = Total weight of assembly to be lifted (kN)

f = Dynamic acceleration factor (Table 1)

z = Angle coefficient (Table 2)

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 then be correctly designed by the design professional or by the manufacturer.

Important notes:

- → If the anchors are not symmetrical in relation to the center of gravity, the calculation of the resultant forces must be adjusted by using the appropriate static equations (see Equations [1] and [2]).
- → Other effects, such as wind, may significantly influence load movement on lifting systems.
- → If the same lifting system if used more than once during the same handling/lifting operation, it may be necessary to adjust the allowable anchor capacity to account for previous stressing of the system.
- → It is important to ensure that the calculated and provided capacities of anchorage systems are compatible.
- → Laboratory tests may be required (e.g., when proprietary lifting devices are used).

12.4 OTHER ACCESSORIES AND MATERIALS

Numerous construction accessories and materials are required on a construction site. This Section presents products, tools, and accessories that may be useful or essential on a construction project using CLT panels, in addition to the items and tools normally required in conventional wood construction. Figure 42 shows common accessories and materials required on a CLT construction site.

Figure 42 Common accessories and materials

12.4.1 Fire-Resistant "Rope" (Fibrous Caulking Material) and Joint Sealing Tapes

To ensure proper sealing of CLT panel joints (i.e., floor-to-floor or floor-to-wall joints), it is recommended to use products that are specifically intended for this purpose. There are a variety of acceptable products on the market.

Typically, the proposed products should perform the following in-service functions:

- Help reduce sound transmission through floors and walls;
- Ensure effective protection against fire and hot combustion gases;
- Improve energy efficiency by reducing heat loss and by limiting air flow (for CLT elements that are part of the enclosure).

Fire

Fire-resistant materials used to seal joints and openings are typically flexible. Some products are made from non-combustible mineral fiber inserted into a fiberglass wire netting. These materials must provide effective protection against fire and hot combustion gases.

Intersecting fire resistance-rated assemblies may require the joints or intersections to be protected by a fire resistance joint system complying with ASTM E1966. Please also refer to Section 8.8 of Chapter 8 – *Fire Performance of CLT Assemblies* for further details.

Acoustics

Acoustic membranes or tapes are specially 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 43 to 45 show some examples of tight joints between CLT elements. Figure 46 shows a membrane installed between a steel staircase and a CLT wall, for acoustics purpose.

Figure 43 Sealing joint between floor, wall, and connectors

Figure 44 Joint between floor and wall made with a semi-rigid membrane

Figure 45 Joint between two floor slabs made with a flexible membrane

Figure 46 Joint installed between a steel staircase and a CLT wall, for acoustics purpose

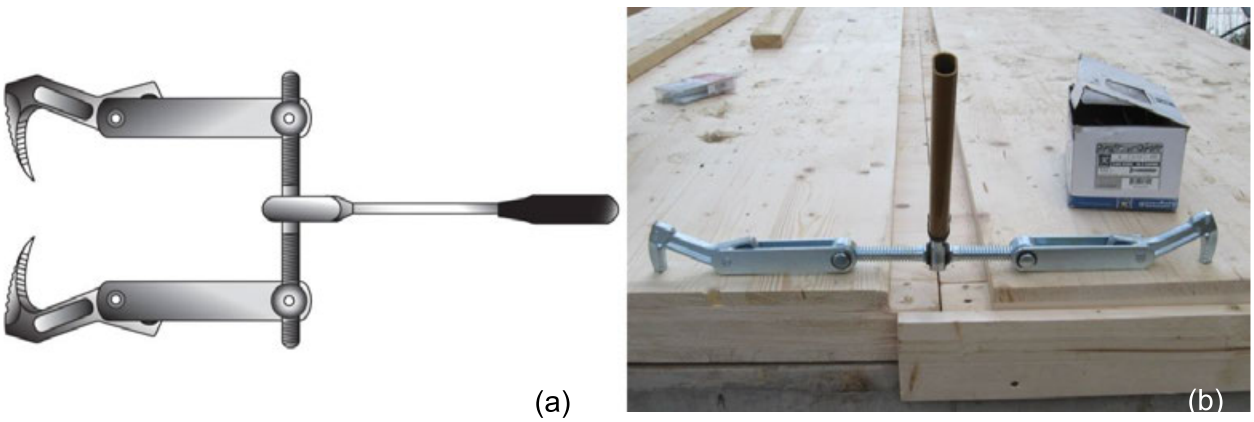
12.4.2 Adjustable Steel Shoring

During frame assembly, it is crucial to have the right tools at hand. Figures 47(a) and (b) show examples of the use of adjustable steel shoring to ensure that the walls or columns are plumb. Shoring can be adjusted with screws or with steel dowels that can be placed at frequent intervals. This type of system is essential to ensure a precise angle of installation. The fastening at both ends is done with screws. If the CLT panels or Glulam columns are to remain visible, repairs may be required when the operation is complete.

Figure 47 Adjustable shoring for walls (a) and columns (b)

12.4.3 Beam Grip with Ratchet and Hooks

Figure 48(a) 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 48(b) shows a beam grip being used to bring two floor panels together. It is important to notice 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.



(a) (b)

Figure 48 Beam grip with ratchet and hooks

12.4.4 Beam Grip with Ratchet and Screw Plate

A beam grip can also be used to ensure proper contact between two panels that are installed perpendicularly. Instead of hooks, the beam grip is equipped 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 system (refer to Chapter 5 for more information). Tightening will ensure proper contact between the elements to limit air infiltration and sound transmission. Note that in Figure 49, a weatherproofing membrane is used at the junction of the panels.

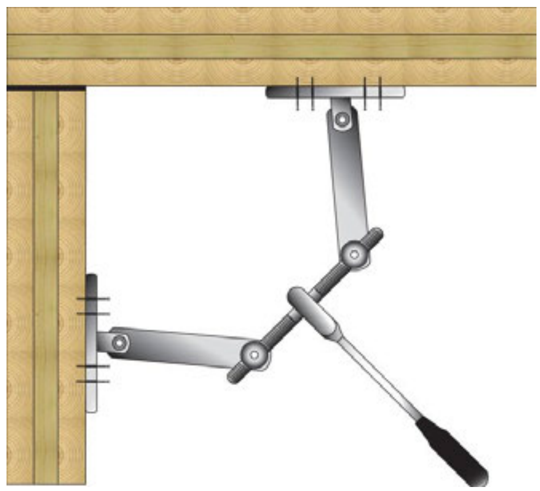


Figure 49 Beam grip with ratchet and screw plate

12.4.5 Manual Winch with Cables or Slings

Instead of a beam grip, a hand winch attached to cables or slings can be used to bring the CLT panels together. Figure 50(a) and (b) show this type of 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 50 Manual winch used with soft slings

12.4.6 Steel Shims and No-Shrinkage Cement-Based Grout

It is sometimes necessary to use steel shims of different thicknesses under CLT loadbearing walls, at the junction with concrete foundations for the walls 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 51 Junction between concrete foundation and CLT walls with steel winch and no-shrinkage cement-based grout

12.5 TRANSPORTATION OF CLT ELEMENTS

Before undertaking the design of a CLT building, consideration must be given 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. It is important to understand that the transportation of CLT panels may involve the design professional, the contractor/erector and the CLT manufacturer; therefore, the information that follows is intended to address the concerns of each of these team members when applicable.

As shown in Section 12.1, CLT panels can be quite large. Typical panel widths are 1.2 m (4 ft), 2.4 m (8 ft) and 3 m (10 ft), while maximum lengths are dependent on the press type and may reach 18 m (60 ft). 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.

12.5.1 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 weight and dimension limits within their jurisdictions. More information on the MoU may be obtained by visiting the Council of Ministers Responsible for Transportation and Highway Safety website.

In the United States, vehicle weights and dimensions fall under the Federal Motor Carrier Safety Regulations (FMCSR) and are regulated by the Federal Motor Carrier Safety Administration (FMCSA).

While each of the States may have varying rules and regulations for weights and dimensions, the FMCSR have been adopted by all States and take precedence over any individual State regulations. Under the Motor Carrier Safety Assistance Program (MCSAP) and the Safety Management System (SMS), the FMCSA allows truck travel restrictions on certain roads and bridges, if the size and weight of such roads and bridges will not safely accommodate commercial vehicles. Keep in mind that States are allowed (and many do) to set more liberal or more stringent weight and dimension restrictions within their jurisdictions and may also require special permitting for loads considered over-dimensional or those that exceed the maximum allowable gross vehicle weight rating. The motor carrier being selected should have previous experience in safely securing and transporting flatbed shipments and efficiently handling cross border traffic. Motor carriers may be called upon to deliver in any State within the United States, so they must have operating authority in the U.S. territory and be familiar with and comply with State and federal regulations governing interstate motor carriers.

It is also recommended that the motor carriers' Compliance, Safety & Accountability (CSA) record be reviewed prior to contracting for the movement of products. A motor carrier's safety record is available online through FMCSA's website and can be searched by either the motor carrier's name or their assigned Department of Transportation (DOT) number.

12.5.1.1 Dimension Limits

In terms of dimension limits, the main points to consider with regards to road vehicles (according to dimensional limits applicable to the U.S. territory, which are slightly more restrictive than Canadian limitations) are:

- vehicle height, including load, is limited to 4.11 m (13 ft 6 in);
- vehicle width, including load but excluding load covering or securing devices, cannot exceed 2.6 m (8 ft 6 in);
- semi-trailer length, including load, cannot exceed 16.15 m (53 ft).

The FHWA website discusses in detail the size and weight limitations of commercial motor vehicles.

Figure 52 presents these limits in a graphical format.

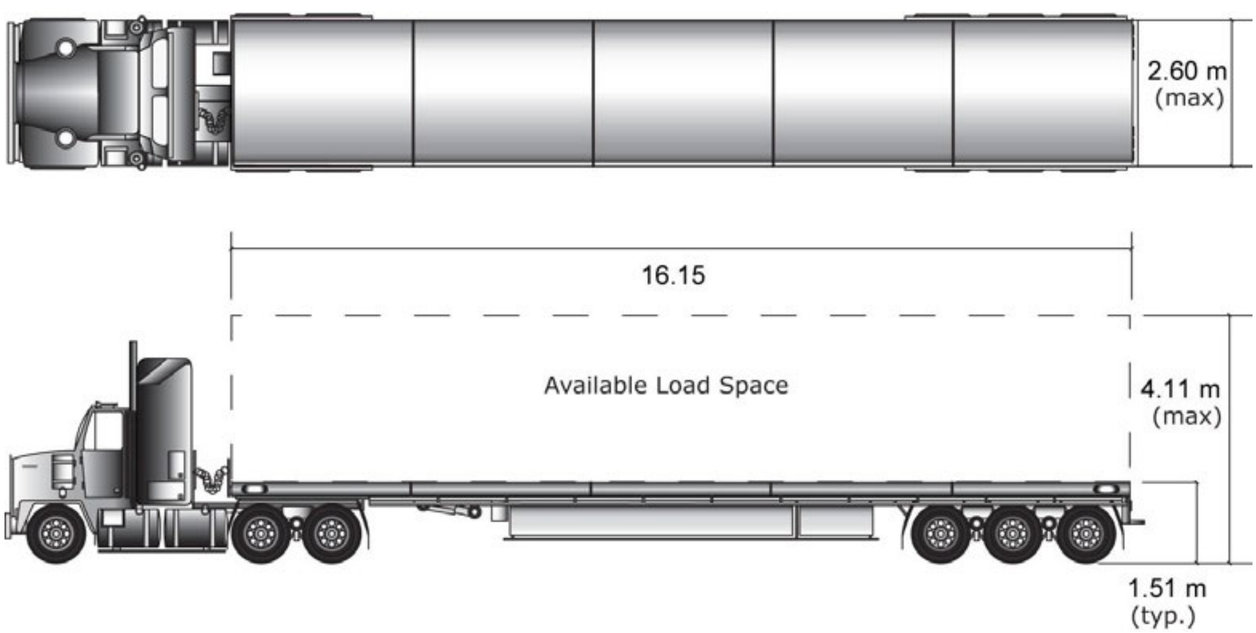


Figure 52 Available load space on a flatbed semi-trailer

Exceptions

In the United States, some States allow what are called over-dimensional loads to be hauled with special restrictions and permitting. Over-dimensional loads (or OD loads as they are commonly referred to) generally require the following, at a minimum:

1. OD loads may consist only of indivisible products. Definitions and exceptions to this rule may be found by visiting the FHWA website.
2. In most cases, OD loads may only be transported during daylight hours.
3. Special vehicle markings are typically required with placards or banners showing oversized or over-width loads.
4. Special permits must be ordered from each State well in advance, with specific routes traveled being strictly adhered to.
5. Some OD loads may require a safety escort service to lead and, in some cases, follow the OD load, depending on the routes to be traveled.

Motor carriers with experience in transporting OD loads are responsible for obtaining the proper markings, permits and any safety escorts to comply with all Federal, State and other municipal rules and regulations.

The majority of CLT panels are transported using a flatbed semi-trailer (Figure 53). These trailers have the advantage of being open on all sides, which facilitates loading, and of 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 (4 ft 11 in) (at the front of the trailer, which is the highest point), this permits load heights of 2.6 m (8 ft 6 in). 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 (8 ft 6 in), a width of 2.6 m (8 ft 6 in), and a length of 16.15 m (53 ft), if it is to be transported by a flatbed semi-trailer. This type and size of trailer is the most commonly utilized in the USA, although some motor carriers still have 14.6 m (48 ft) length trailers in their fleet. It is recommended, when ordering a truck, to be specific about your length requirements.

For taller structures, drop deck (also called step deck) semi-trailers can also be used. However, as can be seen in Figure 54, unlike flatbed semi-trailers, the deck of a drop deck is not continuous. A drop deck flatbed with smaller 255/70R22.5 type tires (but still using normal axle hubs and brakes) can be used to allow a 3 m (9 ft 10 in) -tall load on the rear 12.8 m (42 ft) section and a 2.6 m (8 ft 6 in) -tall load on the front 3.35 m (11 ft) section.

Other semi-trailers with even greater load heights are available, such as double drop decks (Figure 55), 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 (29 ft 6 in) and a deck height of 0.55 m (1 ft 9 in), allowing products of up to 3.56 m (11 ft 8 in) in height.

Although all these semi-trailer types can be as long as 16.15 m (53 ft), many are 14.63 m (48 ft) in length. The dimensions given here are presented as guidelines.

IMPORTANT: It is important that you check with your transportation provider to verify the dimensions of their vehicles before going forward with any transportation plan.

Figure 53 Flatbed semi-trailer

Figure 54 Drop deck semi-trailer

Figure 55 Double drop deck semi-trailer

12.5.1.2 Weight Limits

When it comes to weight limits, the situation is more complex, since the CLT panels may be crossing the Canada/U.S. border and weight limits and axle configurations vary between the two countries. Legal Gross Vehicle Weight (GVW) is the weight of the vehicle and its load. Legal GVW varies not only by province in Canada, 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 United States, tractor/semi-trailer combinations are limited to 5 axles. In 2014, a configuration of a tridem drive tractor pulling a three-axle semi-trailer was introduced in the Canadian MOU. It should be noted that although all provinces had committed to introducing this configuration in their respective W&D regulations, not all had done so by the end of 2017.

Table 3 presents the maximum payloads authorized with 5- and 6-axle flatbed combinations by jurisdiction, taking into account the typical light tare weights for these units (13.0 t {28,600 lbs.} for a 5-axle unit, 14.5 t {31 900 lbs.} for a 6-axle unit, and 16 t {38,000 lbs.} for tri-drive 7-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 and a tri-drive 7-axle combination (t)

Jurisdiction	5-axle combinations	6-axle combinations	Tri-drive 7-acle combination
MOU†	26.5	32	35
Atlantic Provinces + Québec	28.5	35	TBD*
Ontario‡	28.5	36.6	38
USA	23.4 (51.400)	N.A.	N.A.

† Manitoba, Saskatchewan, Alberta, and British Columbia limits all follow the MOU
‡ Although higher GVW may be allowed in the regulation, we have included the highest practical GVW
* At the time of publication, not all of the provinces had introduced this configuration in their regulations

12.5.1.3 Other Canadian Legal Configurations

All provinces except for Manitoba, Saskatchewan and British Columbia allow the use of 4-axle semi-trailers, while Ontario also allows 5-axle semi-trailers with much higher payloads. Given that these vehicles have limited 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 56) at a GVW of 62.5 tonnes (125,000 lbs.). However, the length of both trailers combined is 20 m (65.62’), with a lead trailer typically having a deck length of 9.75 m (32 ft) and a rear trailer with a deck of 8.5 m (27 ft 9 in). 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 (32 ft). Typical tares are in the range of 18 t (36,000 lbs.), so loads of up to 44.5 t (89,000 lbs.) are possible. Gross vehicle weight in the U.S. is 80,000 lbs. or 40 t.

Finally, most provinces allow or plan to allow tri-drive configurations with combinations having more than 7-axles, such as tri-drives pulling B-trains, 4-axle semi-trailers, or quad trailers.

Different possible configurations are also available in the United States, 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 57 presents a typical U.S. motor carrier flatbed with spread axle configurations and a 53' trailer.

Figure 56 Super B-train flat deck combination

Figure 57 U.S. motor carrier flatbed

12.5.2 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 exceed the normal limits permitted by legislation. Larger CLT panels may 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 the CLT manufacturers and transport companies that specialize in this type of work. If it is determined that CLT elements are not within the standard legal dimensions or weights described in Section 12.5.1, it is important to contact a specialist. For more information on oversize and overweight permitting, refer to local State or provincial authorities. A complete understanding of the size and weight limits as well as State by State lists may be obtained on the U.S. Federal Highway Administration website.

12.5.3 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 weight and dimension regulations. First of all, the contractor, working in conjunction with the CLT manufacturer and their selected transport company, must ensure that the route from the plant to the construction site will allow movement of the truck, including its load, without any obstacles that would interfere with the transport of the CLT panels. 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 shorter 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 or piles of materials, for example, to make driving space changes can disrupt and delay deliveries and increase costs.

This can be a challenge when working in tight urban areas where the space for storing 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 wheel bases, 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 from the Society of Automotive Engineers.

Awareness of local municipal regulations and pre-planning to match construction site challenges are advisable to ensure a smooth, efficient delivery without delays and cost overruns.

12.5.4 Other Transportation Considerations

Having the design professional work in concert with the CLT manufacturer to design loads to fit on normal equipment presents significant advantages, as is provides the option to use for-hire carriers to deal with long distance one-way hauls when many loads must arrive and be staged at a jobsite, within a close period of time. It also reduces vulnerability by allowing access to replacement vehicles when a specialized vehicle has downtime and helps dealing with swings in demand.

When normal flatbeds are used, it is generally best to lay the load horizontally for easier tarping, and to have the load center 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 (10 ft) 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 there is 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 also best to have a physical tarp over the load as primary protection against rain, ice and debris.

In addition to the general standards described here, U.S. federal law includes provisions, exemptions, and variations applicable to particular States, routes, vehicles, or operations. For more details, please consult 23 CFR Part 658, available on the FHWA's Office of Freight Management and Operations website.

12.6 POSITIONING OF MATERIALS ON CONSTRUCTION SITE AND PROTECTION AGAINST WEATHER

12.6.1 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 that proper logistics control is in place during construction, as there are costs associated with handling each piece or 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 at least 6 inches above the ground; skids must be in sufficient numbers to protect panels from water runoffs and appropriate tarpaulins should be used to protect them from direct exposure to the elements.

Figure 58 shows small 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 placed on wood skids to protect them from water runoffs. Although this packaging practice may be adequate, it is recommended that high-quality tarpaulins also be used. Every effort should be made to ensure that the packs remain sealed since, if there are openings, water could infiltrate the packs and become trapped. Therefore, the bottom of the wrapping must be slit at the jobsite to permit any moisture that may become entrapped to escape. 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.

Figures 59(a) and (b) show a truck platform left on a construction site with large CLT panels completely wrapped (six faces). It will be recovered on the next trip. This can reduce costs by allowing independent scheduling of transportation and unloading.

Finally, it should be noted that the stacking of the panels on the construction site should match the planned installation sequence, when possible. Unnecessary handling leads to additional costs and risks of accidents or damage.

Figure 58 Storage on construction site – individually wrapped bundles stacked on lumber skids

Figure 59 **Truck platform left on construction site with CLT panels wrapped on six faces – it will be recovered on the next trip**

12.6.2 Construction Load on Frame

Stacking and storage of CLT elements or other heavy materials must take 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 60).

Figure 60 CLT slabs temporarily stored on a floor

12.6.3 Temporary Protection during Construction

As indicated in Section 12.6.1, wood components should be protected against the elements as much as possible 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, and long exposure to the elements should be avoided. Otherwise, the wood may become discolored or dirty during construction.

In addition, due to the hygroscopic nature of wood, exposure of the CLT panels to the elements may result in slight variations in size during construction due to swelling which may cause problems at the joints. For example, connections can be difficult to perform on the construction site, especially if accuracy is important.

There are some effective techniques that can be used to provide adequate protection against the elements during frame set-up operations. Figures 61 to 64 show techniques used mainly in Europe to protect components from the weather during construction. While these erection techniques are not commonly used in the United States, there may be applications where such protection could be beneficial.

Figure 61 Use of a temporary tarpaulin (courtesy of Fristad Bygg, Sweden)

Figure 62 Use of a weather-proof tarpaulin outside scaffoldings – Germany

Figure 63 Use of an adjustable tent – Sweden

Figure 64 Use of a weather-proof tarpaulin outside scaffolding – Europe

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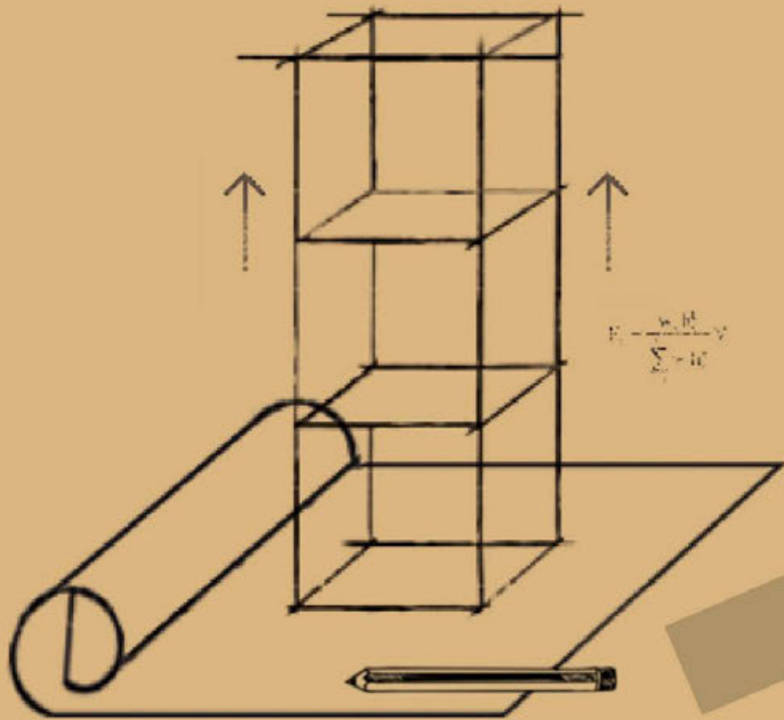
FPInnovations



CHAPTER 13

Design Example

Lead Authors
Mark Prescott
David Moses

Reviewers
Zhiyong Chen
Samuel Cuerrier-Auclair
Christian Dagenais
Julie Frappier
Sylvain Gagnon
Simon Gallagher
David Hanna
Lin Hu
Erol Karacabeyli
Chun Ni
Marjan Popovski

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ABSTRACT

The use of cross-laminated timber (CLT) alongside glulam products are demonstrated with a design example for an eight-storey platform-framed mass timber building located in Ottawa, Ontario. The example illustrates the key components of gravity, fire, and lateral design, and provides example calculations and design guidance for CLT roof and floor panels, CLT loadbearing and shear walls, and glulam post and beam framing. In addition, example connections are given for the transfer of gravity and lateral forces, as well as a consideration of wind-induced vibration. An Annex is included containing plans and elevations of the example building along with examples of connection details.

The design examples provided are in line with the specifications in CSA O86-14 Update 2 (CSA, 2017) and the National Building Code (NRC, 2015), as well as incorporating the latest research findings that have been implemented in the 2019 Edition of CSA O86-19 (CSA, 2019).

13.1 INTRODUCTION

An eight-storey residential mass timber building is presented in this Chapter to illustrate key components of gravity, fire, and lateral design. The first storey of the building is concrete and the upper seven storeys are mass timber (i.e. CLT roof panels, CLT floor panels, CLT elevator and stair cores, and a glulam post-and-beam frame).

The elevator and stair cores, as well as the additional shear walls are platform-type – per the method for CLT described in CSA O86-14 Update 2 (CSA, 2017) to resist seismic and wind loads. Where indicated, the CLT lateral design went beyond this standard by making use of the latest research findings that have been implemented in the 2019 Edition of the CSA Standard O86-19 (CSA, 2019).

The layout is shown on plans and elevations in Annex A to Chapter 13. A typical floor plan is shown below in Figure 1. This floor plan is repeated on all levels and is 15.2 m x 50 m, with nine regular structural grids spaced at 6.1 m on-centre. There are two stair cores and a single elevator core. The storey height is 3.6 m for the first storey (the concrete storey) and 3 m floor-to-floor for the upper seven storeys (mass timber storeys). The total height is 24.6 m.

The building is oriented north-south along the building short-axis direction and east-west along the building long-axis direction.

The building is located in Ottawa, Ontario.

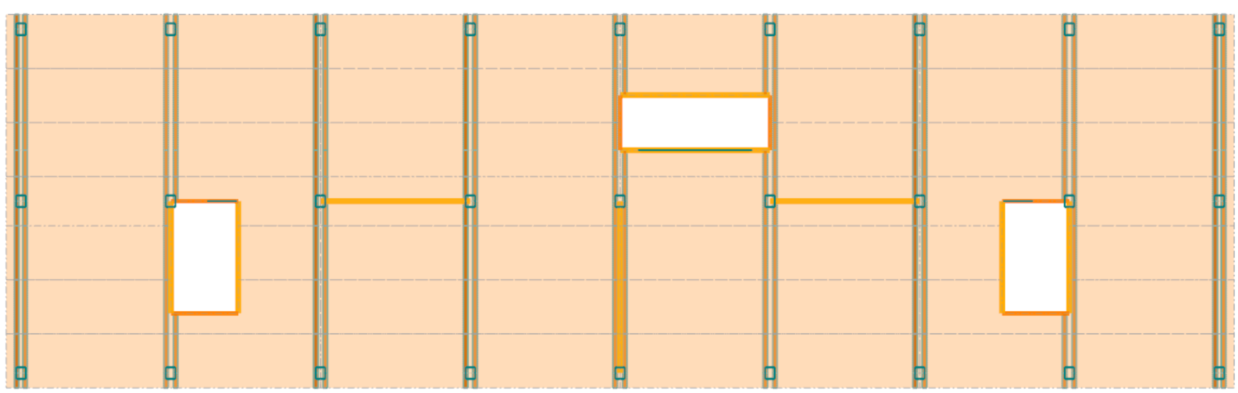

Figure 1 A typical floor plan

13.1.1 Geotechnical Data

Soil conditions are assumed to be bedrock with a bearing capacity of 500 kPa SLS and 1000 kPa ULS. Site Class C is assumed.

13.1.2 Gravity Load Resisting System

The vertical load resisting system comprises the following elements:

a) CLT floor and roof panels, spanned one-way between beam lines.

b) Post-and-beam glulam with columns on a 6.1-m x 7.0-m grid, and double continuous glulam beams in the North-South direction.

c) Elevator and stair core CLT walls support beam point loads, and the CLT floor and roof panels.

d) Three CLT walls outside of the core, of which two are in the East-West direction and one is in the North-South direction.

13.1.3 Lateral Load Resisting System

The lateral load resisting system comprises the following elements:

a) Elevator and stair core CLT platform-type CLT walls designed to resist lateral forces through rocking.

b) CLT shear walls in each orthogonal direction, in addition to the elevator and stair cores: two in the long direction and one in the short direction, designed to dissipate lateral forces through rocking.

c) CLT floor and roof panels connected with diaphragms modelled as semi-rigid elements (actual in-plane stiffness considered).

d) Continuous steel rod tie-down system or discrete hold-downs.

e) Concrete podium with conventionally constructed concrete shear walls.

13.1.4 Acoustic Performance

This example building is designed for student residence occupancy. The targeted sound insulation performance per code is an ASTC of 47 or a STC of 50, and an IIC ≥ 55, as recommended in the NBC (NRC, 2015). The floor assembly is assumed to be:

- 5-mm carpet or 3-mm luxury vinyl tile
- 2.3-mm INSONO AF-3 membrane or similar
- 38-mm normal weight concrete
- 25-mm SonusWave underlayment or similar
- 175-mm five-ply CLT
- Exposed ceiling

The Sound Transmission Class (STC) rating describes the performance of the separating wall or floor/ceiling assembly, whereas the Apparent Sound Transmission Class (ASTC) takes into consideration the performance of the separating element as well as the flanking transmission paths. Building professionals should also ensure that floors are designed to minimize impact transmission. Selecting an appropriate separating assembly is only one part of the solution for reducing airborne sound transmission between adjoining spaces: to fully address the sound performance of the whole system, flanking assemblies must be connected to the separating assembly. For more details, see Note A-5.8. in NBC (NRC, 2015).

It is important to note that the acoustic design of CLT floor systems is likely to control the dead weight of mass timber floors. It is advisable to consult with the architect and an acoustical engineer prior to deriving the dead weight of mass timber floors.

Refer to Chapter 9 of this Handbook for an in-depth explanation of the acoustic performance of CLT assemblies.

13.1.5 Fire Performance

The eight-storey building's major occupancy is residential. The NBC (NRC, 2015) requires that buildings with residential occupancy exceeding six storeys have:

a) floor assemblies that behave as fire separations with a minimum fire resistance rating of 2 hours;

b) load bearing walls and columns that have a fire resistance rating at least equal to the supported floor assemblies; and

c) exit stair cores that have a minimum fire resistance rating of 2 hours and which prevent the ingress of contaminated air over this same timeframe

For this example, therefore, exposed wood elements will be required to have a fire resistance rating of 2 hours.

13.2 REFERENCE CODES AND STANDARDS

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13.3 MATERIALS

Cross-laminated timber (CLT): Grade E1 per ANSI/APA PRG 320 (2018) and CSA O86-14 Update 2 (2017)

Glulam beams: D. Fir-L 24f-EX per CSA O86-14

Glulam columns: D. Fir-L 16c-E per CSA O86-14

Note that other proprietary glulam products are available and approved for use in Canada through the Canadian Construction Materials Centre (CCMC) and APA – The Engineered Wood Association.

13.4 LOADS

Climatic loads are for Ottawa City Hall, Ontario in accordance with the NBC (NRC, 2015) data.

S_s = 2.4 kPa
S_r = 0.4 kPa
One day rainfall (1 in 50 years) = 86 mm
q_{50} = 0.41 kPa

$S_a(0.2)$	0.439	$S_a(2.0)$	0.056
$S_a(0.5)$	0.237	$S_a(5.0)$	0.015
$S_a(1.0)$	0.118	$S_a(10.0)$	0.0055
PGA	0.281	PGV	0.196

13.4.1 Gravity Loads

Floor 1 – Ground floor

DEAD LOAD
Finishes 0.20 kPa
Partition load 1.00 kPa
150-mm concrete slab-on-grade 3.60 kPa
TOTAL 4.80 kPa

LIVE LOAD
Retail and wholesale areas 4.8 kPa [NBC Table 4.1.5.3]

LIVE LOAD
Exits and fire escapes 4.8 kPa [NBC Table 4.1.5.3]

Floor 2

DEAD LOAD
Finishes 0.20 kPa
Partition load 1.00 kPa
350-mm concrete suspended slab 8.40 kPa
Mechanical/electrical allowance 0.25 kPa
TOTAL 9.85 kPa

LIVE LOAD
Residential areas 1.9 kPa [NBC Table 4.1.5.3]

LIVE LOAD
Exits and fire escapes 4.8 kPa [NBC Table 4.1.5.3]

Floors 3 to 8

<u>DEAD LOAD</u>

Finishes incl. sound insulation	0.20 kPa
Partition load	1.00 kPa
38-mm concrete topping	0.91 kPa
175-mm CLT floor slab (515 kg/m^3)	0.88 kPa
GL post-and-beam	0.20 kPa (not included for floor panel design)
Mechanical/electrical allowance	<u>0.25 kPa</u>
TOTAL	3.44 kPa

<u>LIVE LOAD</u>

Residential areas — 1.9 kPa [NBC Table 4.1.5.3]

<u>LIVE LOAD</u>

Exits and fire escapes — 4.8 kPa [NBC Table 4.1.5.3]

Roof

<u>DEAD LOAD</u>

Roofing, insulation and finishes	0.43 kPa
175-mm CLT roof slab (515 kg/m^3)	0.88 kPa
GL post-and-beam	0.20 kPa (not included for roof panel design)
Mechanical/electrical allowance	<u>0.25 kPa</u>
TOTAL	1.76 kPa

<u>SNOW LOAD</u>

S_s = 2.4 kPa
S_r = 0.4 kPa
C_s = 1.0 slope less than 15° [NBC 4.1.6.2(5)]
I_S = Importance factor: normal = 1.0 [NBC Table 4.1.6.2.-A]
$S = I_s [S_s (C_b \times C_w \times C_s \times C_a) + S_r] = 1.0 [2.4 (0.8 \times 1.0 \times 1.0 \times 1.0) + 0.4] = 2.32$ kPa
Note: Loads due to snow drifting have been ignored in this example.

<u>RAIN PONDING LOAD</u>

Rain ponding has been ignored in this example.

<u>LIVE LOAD</u>

Roof minimum — 1.0 kPa [NBC Table 4.1.5.3] – snow load governs
1.3 kN concentrated load applied over 200- x 200-mm area

13.4.2 Wind Loads

There are three procedures outlined in NBC (NRC, 2015) 4.1.7.2 for determining wind load on structures. Typically, the static approach for deriving wind loads can be used for structures with a natural frequency higher than 1 Hz. For structures with a natural frequency between 0.25 to 1 Hz, the dynamic procedure is to be used, and where the natural frequency is less than 0.25 Hz, the wind tunnel procedure should be used.

To determine whether the dynamic procedure is required or not, the lowest natural frequency of the example building can be determined using Rayleigh's Method, or from the finite element model created for the linear dynamic analysis described in Section 13.8.3.

As in NBC Structural Commentary I, the lowest natural frequency, f, can be estimated using Rayleigh's method, with the following equation:

$$f = \frac{1}{\left(2\pi\sqrt{\frac{\sum_{j=1}^{N} W_j X_j^2}{g\sum_{j=1}^{N} F_j X_j}}\right)}$$

where:

f = frequency (Hz)

W_j = weight at each floor (kN)

F_j = static lateral load applied at each floor to produce static deformation (kN)

X_j = static deformation produced by static loading on structure determined by linear elastic analysis model (mm)

N = number of vertical levels

g = gravitational acceleration (m/s^2)

Table 1 shows the input values for the North-South direction, for the equation above.

Table 1 Examples of building size relative to occupancy group, as per Division B of NBCC

Level	Drift X_j (mm)	Inter-Storey Drift x_j (mm)	$F_j X_j$ (kN.mm)	W_j (kN)	$W_j X_j^2$ (kN.mm^2)
Roof	14.20	1.90	1380	1427	287740
8	12.30	2.00	2299	3119	471874
7	10.30	2.20	1841	3119	330895
6	8.10	2.40	1373	3119	204638
5	5.70	2.30	906	3119	101336
4	3.40	1.90	533	3119	36056
3	1.50	1.30	235	3119	7018
2	0.2	0.20	31	7916	317
Total			8599	28057	1439872

Therefore, the natural frequency in the N-S direction is:

$$f = \frac{1}{\left(2\pi\sqrt{\frac{1439872}{9.81 \times 1000 \times 8599}}\right)} = 1.22\ Hz$$

Similarly, it can be shown that the natural frequency in the E-W direction using the same method is:

$$f = \frac{1}{\left(2\pi\sqrt{\frac{69952}{9.81 \times 1000 \times 465}}\right)} = 1.29\ Hz$$

From these calculations, it can be concluded that the lowest natural frequency is greater than 1 Hz in both directions; thus, per NBC, the building need only be analyzed using the static wind procedure, i.e. the dynamic procedure is not required. Note that the finite element model developed for this building was found to have lowest natural frequencies of 1.13 Hz and 1.25 Hz in the North-South and East-West directions, respectively.

Static base shear due to wind load is as follows:

Rough terrain

Building height = 24.6 m

Exposure factor $C_e = 0.7\ (h/12)^{0.3}$, with minimum $C_e = 0.7$ [NBC 4.1.7.3.(5)(b)]

C_e varies with height.

Gust effect factor $C_g = 2.0$ for the building as a whole and for main structural members [NBC 4.1.7.3(8)].

C_p calculated per NBC 4.1.7.5(2) for the main structural system considering external pressure coefficients on the windward and leeward faces of the building.

In the North-South direction, $H/D = 24.6/15.2 = 1.62 \geq 1.0$, therefore $C_p = 0.8$ on the windward face, and $C_p = -0.5$ on the leeward face.

In the East-West direction, $H/D = 24.6/50 = 0.49$, therefore $C_p = 0.27 \times (0.49 + 2) = 0.67$ on the windward face, and $C_p = -0.27 \times (0.49 + 0.88) = -0.37$ on the leeward face.

Topographic factor, $C_t = 1.0$ [NBC 4.1.7.4.(1)]

Importance category, I_W = normal = 1.0 (0.75 for SLS) [NBC Table 4.1.7.3]

Reference velocity pressure, $q_{1/50} = 0.41$ kPa [NBC climatic data for Ottawa, Ontario]

Specified external pressures calculated for the windward and leeward faces:

$p = I_W q C_e C_t C_g C_p = 1.0 \times 0.41 \times C_e \times 1.0 \times 2.0 \times C_p = 0.82 C_e C_p$ kPa

Net wind load calculated as absolute sum of the windward and leeward external pressures, as shown in Tables 2 and 3.

Table 2 Wind loading distribution per floor – North-South (N-S)

Distribution of base shear to floors N-S									
Level	Storey Height (m)	Accum. Height (m)	Bldg Weight (m)	C_e	C_g	C_p, wind	C_p, lee	P (kPa)	Factored Wind Load (kN)
Roof	3.00	24.6	50.0	0.87	2	0.80	-0.50	0.87	97
8	3.00	21.6	50.0	0.83	2	0.80	-0.50	0.82	187
7	3.00	18.6	50.0	0.80	2	0.80	-0.50	0.76	179
6	3.00	15.6	50.0	0.76	2	0.80	-0.50	0.69	170
5	3.00	12.6	50.0	0.71	2	0.80	-0.50	0.63	159
4	3.00	9.6	50.0	0.70	2	0.80	-0.50	0.63	157
3	3.00	6.6	50.0	0.70	2	0.80	-0.50	0.63	157
2	3.00	3.6	50.0	0.70	2	0.80	-0.50	0.63	157
Ground	3.60	0	50.0	0.70	2	0.80	-0.50	0.63	0
TOTAL base shear WIND N-S									1261

Table 3 Wind loading distribution per floor – East-West (E-W)

Distribution of base shear to floors E-W									
Level	Storey Height (m)	Accum Height (m)	Bldg Weight (m)	C_e	C_g	C_p, wind	C_p, lee	P (kPa)	Factored Wind Load (kN)
Roof	3.00	24.6	15.2	15.2	2	0.67	-0.37	0.74	24
8	3.00	21.6	15.2	15.2	2	0.67	-0.37	0.71	46
7	3.00	18.6	15.2	15.2	2	0.67	-0.37	0.68	44
6	3.00	15.6	15.2	15.2	2	0.67	-0.37	0.65	41
5	3.00	12.6	15.2	15.2	2	0.67	-0.37	0.61	39
4	3.00	9.6	15.2	15.2	2	0.67	-0.37	0.60	38
3	3.00	6.6	15.2	15.2	2	0.67	-0.37	0.60	38
2	3.00	3.6	15.2	15.2	2	0.67	-0.37	0.60	38
Ground	3.60	0	15.2	15.2	2	0.67	-0.37	0.60	0
TOTAL base shear WIND E-W									308

13.4.3 Earthquake Load and Effects

Site Class C is assumed, as noted above in Section 13.1.2.

Site specific earthquake parameters for Ottawa, Ontario:

Table 4 5% damped spectral response

$S_a(0.2)$	$S_a(0.5)$	$S_a(1.0)$	$S_a(2.0)$	$S_a(5.0)$	$S_a(10.0)$	PGA	PGV
0.439	0.237	0.118	0.056	0.015	0.0055	0.281	0.196

Per NBC 4.1.8.1.(2):
$I_EF_sS_a(0.2) = 1.0 \times 1.0 \times 0.439 = 0.439 > 0.16$
$I_EF_sS_a(2.0) = 1.0 \times 1.0 \times 0.056 = 0.056 > 0.03$
Thus, the provisions of NBC 4.1.8.2 to NBC 4.1.8.22 apply. [NBC 4.1.8.1.(2)]

NBC 4.1.8.4. – Site Properties
$S_a(0.2)$ / PGA = 0.439 / 0.281 = 1.56 < 2.0 thus $PGA_{ref} = 0.8 \times PGA = 0.8 \times 0.281 = 0.225$
[NBC 4.1.8.4.(4)]

For site class C, the Site Coefficients, F, determined from NBC Tables 4.1.8.4.-B to 4.1.8.4.-I, are:

F(0.2)	F(0.5)	F(1.0)	F(2.0)	F(5.0)	F(10.0)	F(PGA)	F(PGV)
1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

F_a = F(0.2) = 1.00 [NBC 4.1.8.4.(7)]
F_v = F(1.0) = 1.00 [NBC 4.1.8.4.(7)]

The design spectral acceleration values are [NBC 4.1.8.4.(9)]:
$S(T \leq 0.2\text{ s}) = \max[F(0.2)S_a(0.2), F(0.5)S_a(0.5)] = \max[1.00 \times 0.439, 1.00 \times 0.237] = \underline{0.439}$
$S(T = 0.5\text{ s}) = F(0.5)S_a(0.5) = 1.00 \times 0.237 = 0.237$
$S(T = 1.0\text{ s}) = F(1.0)S_a(1.0) = 1.00 \times 0.118 = 0.118$
$S(T = 2.0\text{ s}) = F(2.0)S_a(2.0) = 1.00 \times 0.056 = 0.056$
$S(T = 5.0\text{ s}) = F(5.0)S_a(5.0) = 1.00 \times 0.015 = 0.015$
$S(T \geq 10.0\text{ s}) = F(10.0)S_a(10.0) = 1.00 \times 0.0055 = 0.0055$

Importance Factor [NBC 4.1.8.5.]:
Normal
ULS $I_E = 1.0$
$I_EF_aS_a(0.2) = 1.0 \times 1.00 \times 0.439 = 0.439 > 0.35$

Method of Analysis [NBC 4.1.8.7]

The Equivalent Static Force Procedure (ESFP) may be used when any of the following three criteria is satisfied:

a) $I_E F_a S_a(0.2) = 1.0 \times 1.00 \times 0.439 = 0.439 > 0.35$ [NBC 4.1.8.7.(a)]
NOT SATISFIED.

b) Regular structures (i.e. no structural irregularities) less than 60-m tall and the period is less than 2.0 s [NBC 4.1.8.7.(b)]
NOT SATISFIED. Structure has mass and torsional irregularities.

c) Building is less than 20-m tall, period less than 0.5 s and Irregularities 1, 2, 3, 4, 5, 6 or 8 [NBC 4.1.8.7.(c)]
NOT SATISFIED. Structure is taller than 20 m and period is greater than 0.5 s.

Since none of these conditions are met, the Dynamic Analysis Procedure of NBC 4.1.8.12. must be used.

Prior to carrying out the dynamic analysis procedure (see Section 13.8 in this Chapter), the following can be determined from NBC:

h_n = 24.6 m
$T_a = 0.05 (h_n)^{3/4} = 0.05 (24.6)^{3/4} = 0.55$ s, in both directions

Design spectral response acceleration:
Interpolating between S(0.5) = 0.237 and S(1.0) = 0.118:
$S(0.552) = 0.237 + (0.55 – 0.5) \times (0.118 – 0.237) / (1.0 – 0.5) = 0.225$

Seismic Force Resisting System (SFRS)

Platform type cross-laminated timber (CLT) shear walls
Ductility modification factor: $R_d = 2.0$
Over-strength modification factor: $R_o = 1.5$

Minimum lateral earthquake force [NBC 4.1.8.11.(2)]:

Using $R_d = 2.0$ and $R_o = 1.5$ for CLT shear walls and $R_D = 1.5$ and $R_O = 1.3$ for conventional concrete shear walls for podium-type structure.

$V = S(T_a)M_v I_E W/(R_d R_o) = 0.225 \times 1.0 \times 1.0 \times W / (2.0 \times 1.5) = 0.076 W$

Note M_v calculation below.

For walls, V shall not be less than:
$V_{min} = S(4.0)M_v I_E W/(R_d R_o)$ [NBC 4.1.8.11.(2)(a)]

Interpolating between S(2.0) = 0.056 and S(5.0) = 0.015:

S(4.0) = 0.056 + (4.0 – 2.0) x (0.015 – 0.056) / (5.0 – 2.0) = 0.029
V_{min} = 0.029 x 1.0 x 1.0 x W / (2.0 x 1.5) = <u>0.01 W</u>

The lateral earthquake force need not be greater than [NBC 4.1.8.11.(2)(c)]:

V = max [2/3 S(0.2)I_EW/($R_d$$R_o$), S(0.5)$I_E$W/($R_d$$R_o$)] = max [2/3 x 0.439 x 1.0 x W / (2.0 x 1.5), 0.237 x 1.0 x W / (2.0 x 1.5)] = max [0.098, 0.079] W

Direction of Loading:

since the components of the seismic force resisting system are orientated along orthogonal axes, independent analyses about each of these axes will be performed, i.e. in each direction [NBC 4.1.8.8.(a)].

Thus, for design, the controlling base shear V = <u>0.076 W,</u> in each orthogonal direction.

NBC Table 4.1.8.11. for Higher Mode Factor, M_v, and Base Overturning Reduction Factor, J:
S(0.2) / S(5.0) = 0.439 / 0.015 = 29.3
Since T_a = 0.552 s, from NBC Table 4.1.8.11 and interpolating:
M_v = 1.01
Interpolating between S(0.2)/S(5.0) = 20 and S(0.2)/S(5.0) = 40, and J for T_a <= 0.5 s and J for T_a = 1.0 for T_a = 0.552 s:
J = 0.97

Per NBC 4.1.8.11.(7), since T_a = 0.552 s < 0.7 s, F_t = 0.

Table 5 Calculation of seismic weight

Level	Area (m^2)	DL (kPa)	LL (kPa)	SL (kPa)	Weight (kPa)	Weight (kPa)
Roof	760	1.76	1.00	2.40	2.36	1794
8	760	3.44	1.90		3.44	2614
7	760	3.44	1.90		3.44	2614
6	760	3.44	1.90		3.44	2614
5	760	3.44	1.90		3.44	2614
4	760	3.44	1.90		3.44	2614
3	760	3.44	1.90		3.44	2614
2						
TOTAL						17478

Table 6 Earthquake loading distribution per floor from ESFP

Level	Floor Area (m²)	Storey Weight (kPa)	Storey Height (m)	Accum. Height (m)	Weight x Height (kNm)	Lateral Force (kN)
Roof	760	1794	3.00	21.0	37674	247
8	760	2614	3.00	18.0	47052	309
7	760	2614	3.00	15.0	39210	257
6	760	2614	3.00	12.0	31368	206
5	760	2614	3.00	9.0	23526	154
4	760	2614	3.00	6.0	15684	103
3	760	2614	3.00	3.0	7842	51
2						
TOTALS		17478			202356	1327

Summary:

Earthquake load control in both directions (not wind) as per calculation of base shears.

From NBC Table 4.1.8.6, structural irregularities are as follows:

<u>Type 1 Vertical Stiffness Irregularity</u>

"Vertical stiffness irregularity shall be considered to exist when the lateral stiffness of the SFRS in a storey is less than 70% of the stiffness of any adjacent storey, or less than 80% of the average stiffness of the three storeys above or below."

For this design example, the second-floor concrete structure has a higher lateral stiffness than the storey above. From the analytical model, the ratio of stiffness is: 10% between the concrete podium and timber storey above.

Conclusion: Present (podium versus mass timber above).

Type 2 Weight (Mass) Irregularity

"Weight irregularity shall be considered to exist where the weight, W, of any storey is more than 150% of the weight of an adjacent storey. A roof that is lighter than the floor below need not be considered."

For this design example, the second-floor concrete structure has a higher weight than the storey above. From the dead loads, the weight of the second floor compared to the third floor is 274% > 150%.

Conclusion: Present (podium versus mass timber above).

Type 3 Vertical Geometric Irregularity

"Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the SFRS in any storey is more than 130% of that in an adjacent storey."

For this design example, the CLT shear walls and CLT cores have a constant horizontal dimension at each storey, throughout the height of the building.

Conclusion: Not present.

Type 4 In-Plane Discontinuity in Vertical Lateral-Force-Resisting Element

"Except for braced frames and moment-resisting frames, an in-plane discontinuity shall be considered to exist where there is an offset of a lateral-force-resisting element of the SFRS or a reduction in lateral stiffness of the resisting element in the storey below."

For this design example, there is no in-plane discontinuity in the vertical concrete and CLT walls.

Conclusion: Not present.

Type 5 Out-of-Plane Offsets

"Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements of the SFRS."

For this design example, there are no out-of-plane offsets of the concrete and CLT walls.

Conclusion: Not present.

Type 6 Discontinuity in Capacity - Weak Storey

"A weak storey is one in which the storey shear strength is less than that in the storey above. The storey shear strength is the total strength of all seismic-resisting elements of the SFRS sharing the storey shear for the direction under consideration."

For this design example, all storeys have identical shear strength, except for the first storey concrete structure, which has a higher shear strength than the mass timber storeys above.

Conclusion: Not present.

Type 7 Torsional Sensitivity (to be considered when diaphragms are not flexible)

"Torsional sensitivity shall be considered to exist when the ratio B calculated according to Sentence 4.1.8.11.(10) exceeds 1.7."

For this design example, the torsional sensitivity was measured in the 3D finite element model; the results are shown in Tables 7 and 8 below:

Table 7 Torsional sensitivity in North-South direction

Level	Δ, max (mm)	Δ, min (mm)	Δ, avg (mm)	B= Δ, max/ Δ, avg
Roof	34.9	4.1	19.5	1.79
8	29.8	3.4	16.6	1.80
7	24.3	2.7	13.5	1.80
6	18.4	2.1	10.3	1.79
5	12.6	1.4	7	1.80
4	7.2	0.8	4	1.80
3	3.0	0.3	1.7	1.76
2	0.3	0	0.2	1.50
Ground				

Table 8 Torsional sensitivity in East-West direction

Level	Δ, max (mm)	Δ, min (mm)	Δ, avg (mm)	B= Δ, max/ Δ, avg
Roof	18.8	15.2	17	1.11
8	16.4	13.2	14.8	1.11
7	13.6	11	12.3	1.11
6	10.5	8.5	9.5	1.11
5	7.4	6	6.7	1.10
4	4.3	3.5	3.9	1.10
3	1.9	1.6	1.8	1.06
2	0.2	0.2	0.2	1.00
Ground				

Conclusion: Torsional sensitivity is present in the North-South direction but not in the East-West direction.

<u>Type 8 Non-Orthogonal Systems</u>

"A non-orthogonal system irregularity shall be considered to exist when the SFRS is not oriented along a set of orthogonal axes."

For this design example, the concrete and CLT walls are orientated North-South and East-West.

Conclusion: Not present.

<u>Type 9 Gravity-Induced Lateral Demand Irregularity</u>

"Gravity-induced lateral demand irregularity on the SFRS shall be considered to exist where the ratio, α, calculated in accordance with Sentence 4.1.8.10.(5), exceeds 0.1 for an SFRS with self-centering characteristics and 0.03 for other systems."

For this design example there are no conditions, including inclined columns and cantilevered floor plates, that would result in significant ratcheting behaviour and the amplification of drifts.

Conclusion: Not present.

Overall conclusion: Building is irregular.

13.5 LOAD COMBINATIONS

13.5.1 Ultimate Limit State

The following load combinations are considered at the ultimate limit state for the design of members and connections:

1.4**D**
1.25**D** + 1.5**L** + 1.0**S**
1.25**D** + 1.5**S** + 1.0**L**
1.25**D** + 1.5**L** + 0.4**W**
1.25**D** + 1.5**S** + 0.4**W**
1.25**D** + 1.4**W** + 0.5**L**
1.25**D** + 1.4**W** + 0.5**S**
1.0**D** + 1.0**E** + 0.25**S**

As per NBC (NRC, 2015) Clause 4.1.3.2 (5), the counteracting factored dead load 0.9D shall be used when the dead load acts to resist overturning, uplift, sliding, failure due to stress reversal, and to determine anchorage requirements and the factored resistance of members.

13.5.2 Serviceability Limit State – Gravity

1.0**D**
1.0**D** + 1.0**L** + (0.9)0.5**S**
1.0**D** + (0.9)1.0**S** + 0.50**L**

13.5.3 Serviceability Limit State – Wind

1.0**D** + 1.0**L** + (0.75)0.4**W**
1.0**D** + (0.9)1.0**S** + (0.75)0.4**W**
1.0**D** + (0.75)1.0**W** + (0.75)0.5**S**
1.0**D** + (0.75)1.0**W**

13.5.4 Ultimate Limit State – Fire

According to CSA O86-14, Clause B.1.4, the actual specified gravity loads are to be used when evaluating the structural fire resistance of timber elements, resulting in the following combinations:

1.0**D** + 1.0**L**
1.0**D** + (0.9)1.0**S**
1.0**D** + 1.0**L** + (0.9)0.5**S**
1.0**D** + (0.9)1.0**S** + 0.5**L**

13.6 GRAVITY DESIGN

The mechanical properties of CLT panels and glulam are typically published by manufacturers. This design example uses the mechanical properties published in ANSI/APA PRG 320 (2018). The designer must select the correct mechanical properties according to the manufacturer's literature for the product being used.

The following assumptions are made:

K_D (load duration factor)	=	1.0 for standard duration
K_H (system factor)	=	1.0 for CLT per CSA O86
K_{Sb} (service condition factor for bending)	=	1.0 for dry service conditions
K_{Sv} (service condition factor for shear)	=	1.0 for dry service conditions
K_T (treatment factor)	=	1.0 for untreated

In the following gravity design examples, it is assumed that the members are protected from fire by a method of encapsulation. Design for fire safety of exposed members is considered in Section 13.7.

Refer to Chapter 3 of this Handbook for background information on the structural design of CLT elements.

13.6.1 Roof CLT Panel – Simply Supported, Single Span

Design of a single span, simply supported CLT roof panel of a 6.1-m length, as shown on the roof plan. Panel tributary width is 1 m.

Factored loads:

w_f = 1.25 DL + 1.5 SL = 1.25 x 1.76 kPa + 1.5 x 2.32 kPa = 5.68 kPa x 1 m = 5.68 kN/m

$M_f = 1/8 \times w_f \times L^2 = 1/8 \times 5.68 \times 6.1^2 = 26.42$ kNm

$V_f = 1/2 \times w_f \times L = 1/2 \times 5.68 \times 6.1 = 17.32$ kN

Considering 175-mm thick, Grade E1, five-layer CLT:

Bending resistance:

f_b = 28.2 MPa

$F_b = f_b K_D K_H K_{Sb} K_T$ = 28.2 x 1.0 x 1.0 x 1.0 x 1.0 = 28.2 MPa

E = 11,700 MPa

$(EI)_{eff,y} = 4166x10^9$ Nmm^2 (Table A4, PRG 320-2018)

$S_{eff,y} = (EI)_{eff,y}/E$ x (2/h) = $4166x10^9$/11700 x (2/175) = $4.07x10^6$ mm^3

$K_{rb,y}$ = 0.85 = adjustment factor for bending moment resistance of CLT panels

$M_{r,y} = \Phi F_b S_{eff,y} K_{rb,y}$ = 0.9 x 28.2 x $4.07x10^6$ x 0.85 x 10^{-6} = 87.8 kNm > M_f = 25.58 kNm (29%)

Shear resistance:

$$V_r = \emptyset F_s \frac{2 A_g}{3}$$

A_g = 1000 x 175 x $175x10^3$ mm^2

$F_s = f_s (K_D K_H K_{SV} K_T)$, where f_s is the specified strength in rolling shear

f_s = 0.5 MPa (CSA O86-14 Table 8.2.4)

V_r = 0.9 x 0.5 x 2/3 x $175x10^3$ x 10^{-3} = 52.5 kN > V_f = 16.78 kN (32%)

Deflections:

Uniformly distributed load, per A.8.5.2 of CSA O86-19

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

EI_{eff} = 4166 x 10^9 Nmm^2/m (Table A4, PRG 320-2018)

GA_{eff} = 15 x 10^6 N/m (Table A4, PRG 320-2018)

K (kappa) = form factor = 1.0 for rectangular cross-sections

K_{creep} = creep factor = 2.0 for dry service conditions

Specified dead load = 1.76 kN/m

Specified snow load = 2.32 kN/m x 0.9 = 2.08 kN/m

L = 6100 mm

$$\Delta_{s,perm}=\frac{5}{384}\cdot\frac{1.76\times(6100)^4}{4166(\times 10)^9}+\frac{1}{8}\cdot\frac{1.76\times(6.1)^2\times 1.0}{15\times(10)^6}\times 10^6=7.6+0.55=8.15\text{ mm}=\frac{L}{748}<\frac{L}{360}$$

$$\Delta_{s,snow}=\frac{5}{384}\cdot\frac{2.08\times(6100)^4}{4166(\times 10)^9}+\frac{1}{8}\cdot\frac{2.08\times(6.1)^2\times 1.0}{15\times(10)^6}\times 10^6=9+0.64=9.64\text{ mm}=\frac{L}{632}<\frac{L}{240}$$

$$\Delta_{s,total}=\Delta_{s,snow}+\Delta_{s,perm}\ \times K_{creep}=9.64+(8.15\times 2)=25.94\text{ mm}=\frac{L}{235}<\frac{L}{180}$$

Note: where the shear deformation component of the total deformation of the CLT panel under out-of-plane standard term loading such as snow, and live loads is significant (i.e., in short spans, short span cantilevers, etc.), as determined by the designer, the shear deformation under these loads should be increased by 30%, to account for a time-dependent effect associated with rolling shear.

13.6.2 Roof CLT Panel – Two-Span Continuous

Design of a two-span continuous CLT panel of a 12.2-m length, as shown on the roof plan. Panel tributary width is 1 m.

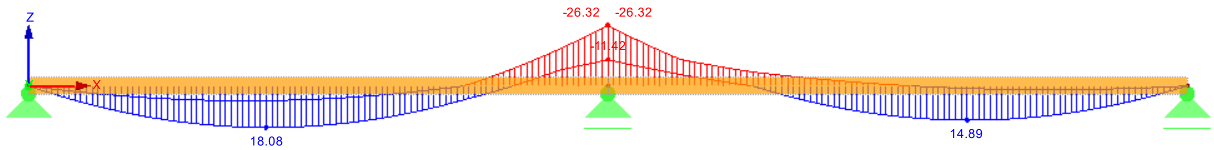
Factored loads:

w_f = 1.25 DL + 1.5 SL = 1.25 x 1.56 kPa + 1.5 x 2.32 kPa = 5.68 kPa x 1 m = 5.43 kN/m

Using pattern loading.

Considering 175-mm thick, Grade E1, two-span continuous, five-layer CLT:

Results from analysis:

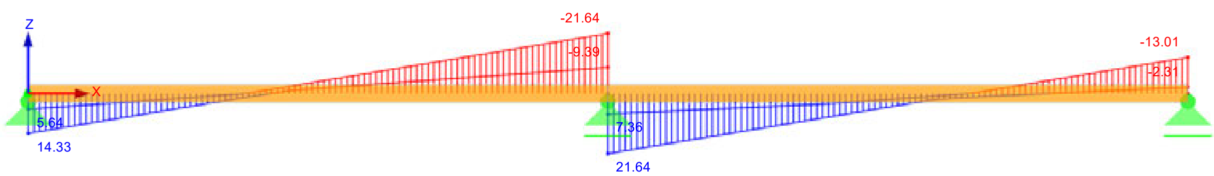


Moments:

M_f +ve = 18.1 KNm

M_f -ve = 26.3 kNm

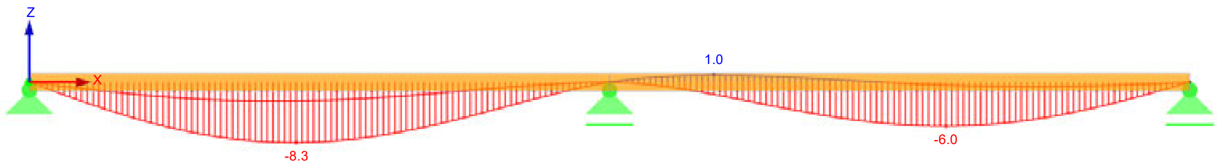
Shear:



V_f – 21.6 kN

Deflections:

Analysed using EI_{eff}.



$\Delta s_{,perm}$ = 2.6 mm = L/2346 < L/360 (15%)

$\Delta s_{,snow}$ = 5.7 mm = L/1070 < L/240 (22%)

$\Delta s_{,total}$ = 8.3 mm = L/735 < L/180 (24%)

Bending resistance:

f_b = 28.2 MPa

$F_b = f_b\ K_D\ K_H\ K_{Sb}\ K_T$ = 28.2 x 1.0 x 1.0 x 1.0 x 1.0 = 28.2 MPa

E = 11700 MPa

$(EI)_{eff,y} = 4166\text{x}10^9$ Nmm^2 (Table A4, PRG 320-2018)

$S_{eff,y} = (EI)_{eff,y}/E$ x (2/h) = $4166\text{x}10^9$/11700 x (2/175) = $4.07\text{x}10^6$ mm^3

$K_{rb,y}$ = 0.85 = adjustment factor for bending moment resistance of CLT panels

$M_{r,y} = \Phi\ F_b\ S_{eff,y}\ K_{rb,y}$ = 0.9 x 28.2 x $4.07\text{x}10^6$ x 0.85 x 10^{-6} = 87.8 kNm > M_f = 24.4 kNm (28%)

Shear resistance:

$$V_r = \emptyset\ F_s\ \frac{2\ A_g}{3}$$

A_g = 1000 x 175 x $175\text{x}10^3$ mm^2

$F_s = f_s\ (K_D\ K_H\ K_{SV}\ K_T)$, where f_s is the specified strength in rolling shear.

f_s = 0.5 MPa (CSA O86-14 Table 8.2.4)

V_r = 0.9 x 0.5 x 2/3 x $175\text{x}10^3$ x 10^{-3} = 52.5 kN > V_f = 20 kN (38%)

Therefore, 175-mm thick, Grade E1, two-span continuous CLT can be used.

13.6.3 Floor CLT Panel – Simply Supported, Single Span

Design of a single span, simply supported CLT floor panel of a 6.1-m length, as shown on the roof plan. Panel tributary width is 1 m.

Factored loads:

w_f = 1.25 DL + 1.5 SL = 1.25 x 3.24 kPa + 1.5 x 1.90 kPa = 6.9 kPa x 1 m = 6.9 kN/m

$M_f = 1/8 \times w_f \times L^2 = 1/8 \times 6.9 \times 6.1^2 = 32.1$ kNm

$V_f = 1/2 \times w_f \times L = 1/2 \times 6.9 \times 6.1 = 21.0$ kN

Considering 175-mm thick, Grade E1 CLT:

Bending resistance:

f_b = 28.2 MPa

$F_b = f_b K_D K_H K_{Sb} K_T = 28.2 \times 1.0 \times 1.0 \times 1.0 \times 1.0 = 28.2$ MPa

E = 11,700 MPa

$(EI)_{eff,y} = 4166 \times 10^9$ Nmm^2 (Table A4, PRG 320-2018)

$S_{eff,y} = (EI)_{eff,y}/E \times (2/h) = 4166 \times 10^9/11700 \times (2/175) = 4.07 \times 10^6$ mm^3

$K_{rb,y}$ = 0.85 = adjustment factor for bending moment resistance of the CLT panels

$M_{r,y} = \Phi F_b S_{eff,y} K_{rb,y} = 0.9 \times 28.2 \times 4.07 \times 10^6 \times 0.85 \times 10^{-6} = 87.8$ kNm > M_f = 32.1 kNm (37%)

Shear resistance:

$$V_r = \emptyset F_s \frac{2 A_g}{3}$$

$A_g = 1000 \times 175 \times 175 \times 10^3$ mm^2

$F_s = f_s (K_D K_H K_{SV} K_T)$, where f_s is the specified strength in rolling shear

f_s = 0.5 MPa (CSA O86-14 Table 8.2.4)

$V_r = 0.9 \times 0.5 \times 2/3 \times 175 \times 10^3 \times 10^{-3} = 52.5$ kN > V_f = 21.0 kN (40%)

Deflections:

Uniformly distributed load per A.8.5.2 of CSA O86-14

$$\Delta = \frac{5}{384} \cdot \frac{wL^4}{(EI)_{eff}} + \frac{1}{8} \cdot \frac{wL^2 k}{(GA)_{eff}}$$

$EI_{eff,0}$ = 4166 x 10^9 Nmm^2/m (Table A4, PRG 320-2018)

$GA_{eff,0}$ = 15 x 10^6 N/m (Table A4, PRG 320-2018)

K (kappa) = form factor = 1.0 for rectangular cross-sections

K_{creep} = creep factor = 2.0 for dry service conditions

Specified dead load = 3.24 kN/m

Specified live load = 1.9 kN/m

L = 6100 mm

$$\Delta_{s,perm} = \frac{5}{384} \cdot \frac{3.24 \times 6100^4}{4166 \times 10^9} + \frac{1}{8} \cdot \frac{3.24 \times 6.1^2 \times 1.0}{15 \times 10^6} \times 10^6 = 14.89 + 1.00 = 15.89 \text{ mm} = \frac{L}{384} < \frac{L}{360}$$

$$\Delta_{s,live} = \frac{5}{384} \cdot \frac{1.90 \times 6100^4}{4166 \times 10^9} + \frac{1}{8} \cdot \frac{1.90 \times 6.1^2 \times 1.0}{15 \times 10^6} \times 10^6 = 8.22 + 0.59 = 8.81 \text{ mm} = \frac{L}{692} < \frac{L}{240}$$

$$\Delta_{s,total} = \Delta_{s,live} + \Delta_{s,perm} \times K_{creep} = 8.81 + (15.89 \times 2) = 40.59 \text{ mm} = \frac{L}{150} > \frac{L}{180}\ NG$$

Note: where the shear deformation component of the total deformation of the CLT panel under out-of-plane standard-term loading such as snow, and live loads is significant (i.e., in short spans, short span cantilevers, etc.), as determined by the designer, the shear deformation under these loads should be increased by 30%, to account for the time-dependent effect associated with rolling shear.

Thus, the 175-mm thick, five-layer, E1, single span CLT panel is not sufficient, and a thicker panel should be considered. It is recommended that if a 175-mm thick CLT is to be used, then all floors should be two-span continuous.

The designer must check bearing perpendicular to grain from loads transferred from the wall(s) above.

Refer to Section 13.6.4 for two-span continuous floor design including consideration of vibration.

13.6.4 Floor CLT Panel – Two-Span Continuous

Design of a two-span continuous CLT panel of a 12.2-m length, as shown on the floor plan. Panel tributary width is 1 m.

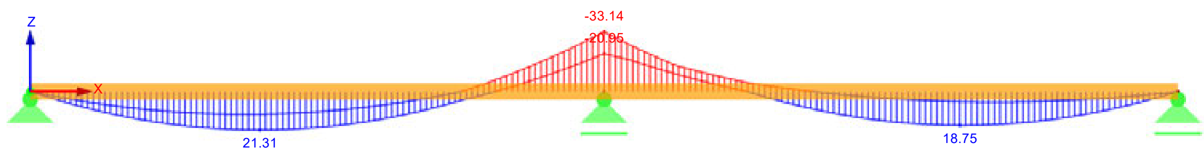
Factored loads:

w_f = 1.25 DL + 1.5 LL = 1.25 x 3.24 kPa + 1.5 x 1.90 kPa = 6.9 kPa x 1 m = 6.9 kN/m

Using pattern loading.

Considering 175-mm thick, Grade E1, two-span continuous, five-layer CLT:

Results from analysis:

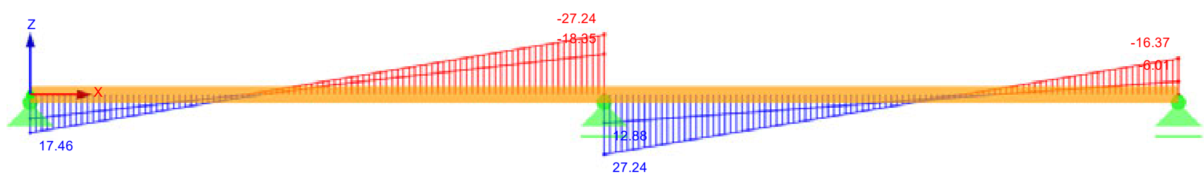


Moments:

M_f +ve = 21.3 kNm

M_f -ve = 33.1 kNm

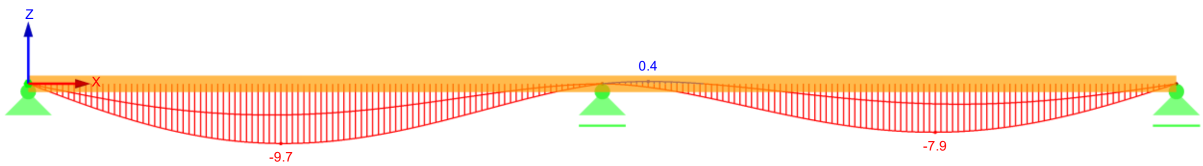
Shear:



V_f = 27.2 kN

Deflections:

Analysed using EI_{eff}.



$\Delta s_{,perm}$ = 5.1 mm = L/1196 < L/360 (30%)

$\Delta s_{,live}$ = 4.7 mm = L/1298 < L/240 (18%)

$\Delta s_{,total}$ = 9.8 mm = L/622 < L/180 (29%)

Bending resistance:

f_b = 28.2 MPa

$F_b = f_b K_D K_H K_{Sb} K_T$ = 28.2 x 1.0 x 1.0 x 1.0 x 1.0 = 28.2 MPa

E = 11,700 MPa

$(EI)_{eff,y} = 4166x10^9$ Nmm^2 (Table A4, PRG 320-2018)

$S_{eff,y} = (EI)_{eff,y}/E$ x (2/h) = $4166x10^9$/11700 x (2/175) = $4.07x10^6$ mm^3

$K_{rb,y}$ = 0.85 = adjustment factor for bending moment resistance of the CLT panels

$M_{r,y} = \Phi F_b S_{eff,y} K_{rb,y}$ = 0.9 x 28.2 x $4.07x10^6$ x 0.85 x 10^{-6} = 87.8 kNm > M_f = 32.8 kNm (37%)

Shear resistance:

$$V_r = \emptyset F_s \frac{2 A_g}{3}$$

A_g = 1000 x 175 x $175x10^3$ mm^2

$F_s = f_s (K_D K_H K_{SV} K_T)$, where f_s is the specified strength in rolling shear

f_s = 0.5 MPa (CSA O86-14 Table 8.2.4)

$V_r = 0.9 \times 0.5 \times 2/3 \times 175 \times 10^3 \times 10^{-3} = 52.5$ kN > $V_f = 27$ kN (51%)

Therefore, 175-mm thick, Grade E1, two-span continuous CLT can be used.

The designer must check bearing perpendicular to grain from loads transferred from the wall(s) above.

Vibration:

The Wood Design Manual (CWC, 2017) Panel Selection Table (Serviceability – Floor Vibration) for CLT gives a maximum vibration-controlled span of 5.46 m for 175-mm thick, Grade E1, CLT panels. The required span for the building is 6.1 m, which is greater than allowed for vibration performance; thus, 175-mm thick CLT is not sufficient when considering a single span with both ends simply supported.

However, 245-mm thick, Grade E1, seven-layer CLT panels have a maximum vibration-controlled span of 6.82 m.

Clause A.8.5.3 allows for a 20% increase in vibration-controlled spans for multi-span floors where non-structural elements are considered to provide enhanced vibration performance, provided the span is not greater than 8 m. This example floor is two-span continuous, with partitions providing enhanced vibration performance.

Thus, 5.46 m x 1.2 = 6.55 m > 6.1 m (93%), or alternatively 6.1 m / 5.46 m = 12% increase in allowable span over the code value.

The area density of the concrete topping should be verified to ensure it does not exceed twice the area density of the CLT panels. As per CSA O86, there is a 10% reduction in allowable span for floors with a concrete topping that has less than twice the CLT area density.

24 kN/m^3 x 0.038 = 0.91 kPa = less than 2 x 5.15 kN/m^3 x 0.175 = 1.80 kPa.

Thus, the 175-mm thick CLT two-span continuous panel is satisfactory for vibration control.

For further guidance on vibration performance, refer to Chapter 7 of this Handbook.

13.6.5 Glulam Roof Beam – Two-Span Continuous

Design of a two-span continuous glulam beam, 15.2 m in length, as shown on the roof plan. Beam tributary width is 6.1 m.

w_f = 44 kN/m (from mid-span support reaction of two-span continuous CLT roof panels and the self-weight of glulam).

Using pattern loading.

Trying a double 175-mm x 418-mm, two-span continuous, D. Fir-L 24f-EX glulam.

Analysing a single beam with w_f = 44 / 2 = 22 kN/m and pattern loading.

Results from analysis:

Moment:

M_f +ve = 75.8 kNm

M_f -ve = 134.8 kNm

Shear:

V_f = 96.3 kN

Deflection:

Interior

$\Delta s_{,perm}$ = 3.5 mm = L/2000 < L/360 (31%)

$\Delta s_{,live}$ = 3.6 mm = L/1944 < L/360 (52%)

$\Delta s_{,total}$ = 7.1 mm = L/986 < L/180 (42%)

Cantilever

$\Delta s_{,perm}$ = 0.9 mm = L/666 < L/180 (60%)

$\Delta s_{,live}$ = 0.9 mm = L/666 < L/180 (93%)

$\Delta s_{,total}$ = 1.8 mm = L/333 < L/90 (75%)

Bending resistance:

M_r = lesser of $M'_r K_L$ or $M'_r K_{Zbg}$

$M'_r = \Phi F_b S$

F_b = 30.6 MPa (Table 7.3, CSA O86-14)

S = (175 x 418^2) / 6 = 5096166.7 mm^2

M'_r = 0.9 x 30.6 x 5096166.7 x 10^{-6} = 140.3 kNm

K_L = 1.0 [beam restrained against rotation and lateral displacement at ends, and compressive edge supported throughout length by CLT decking]

K_D = 1.0

$$K_{Zbg} = \left(\frac{130}{b}\right)^{\frac{1}{10}}\left(\frac{610}{d}\right)^{\frac{1}{10}}\left(\frac{9100}{L}\right)^{\frac{1}{10}} = \left(\frac{130}{175}\right)^{\frac{1}{10}}\left(\frac{610}{418}\right)^{\frac{1}{10}}\left(\frac{9100}{7000}\right)^{\frac{1}{10}} = 1.03 \ \leq 1.3$$

K_L governs.

M_r = 140.3 kNm x 1.0 x 1.0 = 140.3 kNm > M_f = 134.8 kNm (93%)

Shear resistance:

7.5.7.2 (b)

$$V_r = \phi F_v \frac{2\mathrm{A}_g}{3}$$

$F_v = f_v(K_D K_H K_{Sv} K_T)$ = 2.0 x 1.0 x 1.0 x 1.0 x 1.0 = 2 MPa

A_g = 175 x 418 = 73150 mm^2

V_r = 0.9 x 2 x (2 x 73150 / 3) x 10^{-3} = 87.8 kN < V_f = 96.3 kN (110%)

Considering 175-mm x 494-mm D. Fir-L 24f-EX glulam:

A_g = 175 mm x 494 mm = 86450 mm^2

V_r = 0.9 x 2 x (2 x 86450 / 3) x 10^{-3} = 103.7 kN > V_f = 96.3 kN (93%)

7.5.7.2 (a)

$$W_r = \emptyset F_v 0.48 A_g C_v Z^{-0.18} \geq W_f$$

C_v = using 6.66 from Table 7.5.7.5D for two-span continuous beam for this example

Z = total beam volume = 175 x 494 x 15200 x 10^{-9} = 1.31 m^3 < 2 m^3 **OK**

W_r = 0.9 x 2 x 0.48 x 86450 x 6.66 x $1.11^{-0.18}$ x 10^{-3} = 488 kN > W_f = 22 x 15.2 = 334 kN (68%)

Therefore, 175-mm x 494-mm D. Fir-L 24f-EX glulam is selected.

The designer must also consider shear resistance requirements applicable to members when notched.

13.6.6 Glulam Floor Beam – Two-Span Continuous

Design of a two-span continuous glulam beam of a 15.2-m length, as shown on the floor plan. Beam tributary width is 6.1 m.

w_f = 54.5 kN/m (from mid-span support reaction of two-span continuous CLT roof panels and the self-weight of glulam).

Using pattern loading.

Considering double 175-mm x 570-mm, two-span continuous D. Fir-L 24f-EX glulam.

Analysing single beam with w_f = 54.5 / 2 = 27.25 kN/m and pattern loading.

Results from analysis:

Moment:

M_f +ve = 93.9 kNm

M_f -ve = 167.0 kNm

Shear:

V_f = 119.2 kN

Deflections:

Interior

$\Delta s,_{perm}$ = 5.1 mm = L/1372 < L/360 (26%)

$\Delta s,_{live}$ = 4.8 mm = L/1458 < L/360 (25%)

$\Delta s,_{total}$ = 9.8 mm = L/714 < L/180 (25%)

Cantilever

$\Delta s,_{perm}$ = 1.6 mm = L/375 < L/180 (48%)

$\Delta s,_{live}$ = 1.4 mm = L/429 < L/180 (42%)

$\Delta s,_{total}$ = 3.0 mm = L/200 < L/90 (45%)

Bending resistance:

M_r = lesser of $M'_r K_L$ or $M'_r K_{Zbg}$

$M'_r = \Phi F_b S$

F_b = 30.6 MPa (Table 7.3, CSA O86-14)

L_e = 1.92 x 7000 mm = 13440 mm

$$C_B = \sqrt{\frac{L_e\, d}{b^2}} = \sqrt{\frac{13440\ \times 570}{175^2}} = 15.8$$

K_L = 0.872, interpolating from Table 2.9 of Wood Design Manual (CWC, 2017)

K_L = 1.0 (beam restrained against rotation and lateral displacement at ends, and compressive edge supported throughout length by CLT decking)

Where dead load exceeds live load, applicability of CSA O86-14 Clause 5.3.2.3 for calculation of the load duration factor must be verified.

K_D = 1.0 – 0.50 log (P_L/P_s) ≥ 0.65 = 1.0 – 0.50 log (3.44/1.9) = 0.871

$$K_{Zbg} = \left(\frac{130}{b}\right)^{\frac{1}{10}}\left(\frac{610}{d}\right)^{\frac{1}{10}}\left(\frac{9100}{L}\right)^{\frac{1}{10}} = \left(\frac{130}{175}\right)^{\frac{1}{10}}\left(\frac{610}{570}\right)^{\frac{1}{10}}\left(\frac{9100}{7000}\right)^{\frac{1}{10}} = 1.00\ \le 1.3$$

K_L governs.

M_r = 261 kNm x 0.872 x 0.871 = 198 kNm > M_f = 167.0 kNm (84%)

Shear resistance:

7.5.7.2 (b)

$$V_r = \phi F_v \frac{2A_g}{3}$$

$F_v = f_v(K_D K_H K_{Sv} K_T)$ = 2.0 x 0.874 x 1.0 x 1.0 x 1.0 = 1.73 MPa

A_g = 175 mm x 570 mm = 99750 mm^2

V_r = 0.9 x 1.73 x (2 x 99750 / 3) x 10^{-3} = 103 kN > V_f = 82.3 kN (80%)

7.5.7.2 (a)

$$W_r = \emptyset F_v 0.48 A_g C_v Z^{-0.18} \ge W_f$$

C_v = using 6.66, from Table 7.5.7.5D for two-span continuous beam for this example

Z = total beam volume = 175 x 570 x 15200 x 10^{-9} = 1.516 m^3 < 2.0 m^3 **OK**

W_r = 0.9 x 1.73 x 0.48 x 99750 x 6.66 x 1.516$^{-0.18}$ x 10^{-3} = 456 kN > W_f = 27.25 x 15.2 = 414 kN (91%)

Therefore, 175-mm x 570-mm, D. Fir-L 24f-EX glulam is selected.

The designer must also consider shear resistance requirements applicable to members when notched and whether a vibration verification is required.

13.6.7 Glulam Column

Design of a second storey column at grid location D/2.

Tributary area at each floor/roof = 6.1 x 7 = 42.7 m^2

Live load reduction factor can be applied to floor live load per NBC (NRC, 2015) Clause 4.1.5.8(3), where a column supports a tributary area greater than 20 m^2 per the following equation:

$$0.3 + \sqrt{\frac{9.8}{B}} = 0.3 + \sqrt{\frac{9.8}{42.7}} = 0.78$$

Table 9 Factored column load with live load reduction

Level	B (m^2)	Dead (kPa)	Live (kPa)	Live Load Reduction Factor	Snow (kPa)	P_f (kN)
Roof	42.70	1.76			2.32	242.5
8	42.70	3.44	1.90	0.78		278.5
7	42.70	3.44	1.90	0.78		278.5
6	42.70	3.44	1.90	0.78		278.5
5	42.70	3.44	1.90	0.78		278.5
4	42.70	3.44	1.90	0.78		278.5
3	42.70	3.44	1.90	0.78		278.5
2						
Ground						
TOTAL						1913.5

Unbraced length = 3.0 m

Column effectively pinned at both ends, K_e = 1.0 [CSA O86-14 Table A.6.5.6.1.]

Concentrically loaded.

Considering a 365-mm x 342-mm, D.Fir-L 16c-E glulam column:

C_c = (3000 x 1.0) / 342 = 8.77 < 50

Φ = 0.8

f_c = 30.2 MPa

$F_c = f_c(K_D K_H K_{Sc} K_T)$ = 30.2 x 1.0 x 1.0 x 1.0 x 1.0 = 30.2 MPa

A = 365 mm x 342 mm = 124830 mm^2

Z = 365 x 342 x 3000 = 374.5x10^6 mm^3 = 0.374 m^3

$K_{Zbg} = 0.68 \times Z^{-0.13} \leq 1.0 = 0.68 \times (0.374)^{-0.13} = 0.77 \leq 1.0$

E_{05} = 0.87E = 0.87 x 12400 = 10788 MPa

$$K_c = \left[1.0 + \frac{F_c K_{Zcg} C_c^3}{35 E_{05} K_{SE} K_T}\right]^{-1} = \left[1.0 + \frac{30.2 \times 0.77 \times 8.77^3}{35 \times 10788 \times 1.0 \times 1.0}\right]^{-1} = 0.96$$

$P_r = \Phi \times F_c \times A \times K_{Zbg} \times K_C$ = 0.8 x 30.2 x 0.77 x 0.96 x 10^{-3} = 2237 kN > 1914 kN (86%)

From Wood Design Manual (CWC, 2017) Column Selection Table, a 365-mm x 342-mm D.Fir-L 16c-E glulam column has factored compressive resistance parallel to grain:

P_{rx} = 2240 kN

P_{ry} = 2250 kN

Thus, a 365-mm x 342-mm, D.Fir-L 16c-E glulam column is sufficient for strength. Design verifications for fire safety are shown in Section 13.7 of this Chapter, which may affect the required column size.

13.6.8 CLT Wall

Design of CLT wall on grid G+ (part of stair core) at first CLT storey, for accumulated loads.

The major axis of the CLT wall panel is oriented vertically.

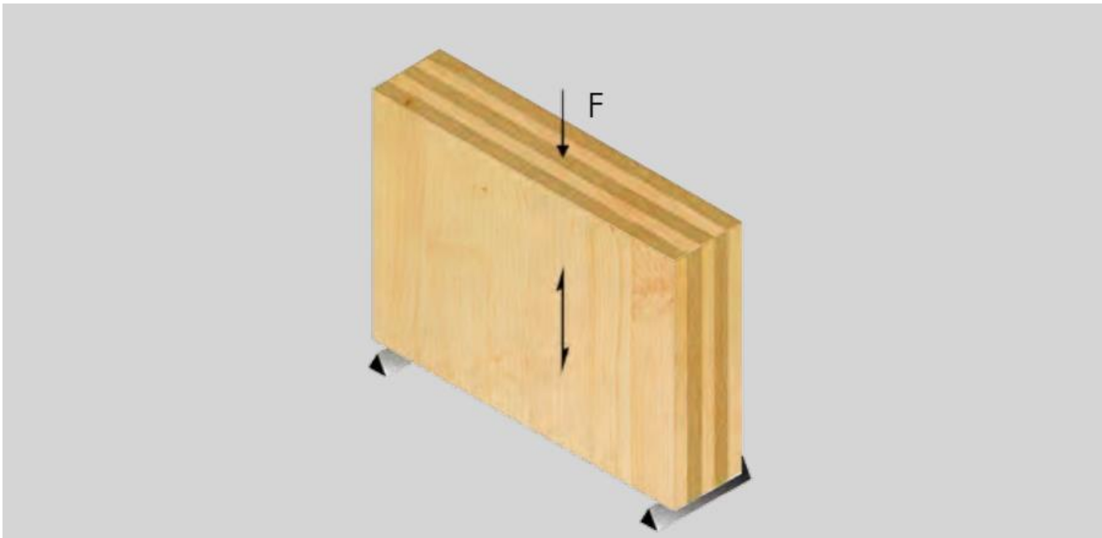


Figure 2 Diagram of panel orientation in the strong axis vertically (source: Nordic)

The unsupported length is taken as the distance between the concrete podium and the first CLT floor, equalling 3.0 m.

The simply supported CLT floor slab spans 3100 mm between grid line G and the CLT wall.

Eccentricity equal to half the wall thickness is used to calculate the moment at the top of the wall, due to the CLT.

Checking the 245-mm thick (seven-layer) Grade E1 CLT wall panel:

$K_e = 1.0$

$L_e = L = 3000$ mm

$Ø = 0.8$

$f_c = 19.3$ MPa (transverse layers)

$K_D = 1.0$

$K_H = 1.0$

$K_{Sc} = 1.0$

K_T = 1.0

I_{eff} = 872x10^6 mm^4

A_{eff} = 140x10^3 mm^2

$$r_{eff} = \sqrt{\frac{I_{eff}}{A_{eff}}} = 78.9\ mm\ for\ minor\ strength\ axis\ \text{(Table 3.10, Wood Design Manual 2017)}$$

$$K_{Zc} = 6.3\left(\sqrt{12}r_{eff}L\right)^{-0.13} = 6.3\left(\sqrt{12}\cdot 78.9\cdot 3000\right)^{-0.13} = 1.07 \leq 1.3$$

$F_c = f_c(K_D K_H K_{Sc} K_T)$ = 19.3 MPa

E_{05} = 11700 x 0.82 = 9594 MPa for 1950 Fb-1.7E SPF MSR (Table 6.3.2, CSA O86-14)

$$C_c = \frac{L_e}{\sqrt{12}r_{eff}} = \frac{3000}{\sqrt{12}\cdot 78.9} = 10.98 \leq 43$$

$$K_C = \left[1.0 + \frac{F_c K_{Zc} {C_c}^3}{35E_{05}(K_{SE}K_T)}\right]^{-1} = \left[1.0 + \frac{19.3\cdot 1.07\cdot 10.98^3}{35\cdot 9594(1.0\cdot 1.0)}\right]^{-1} = 0.92$$

P_r = Ø x F_c x A_{eff} x K_{Zc} x K_C = 0.8 x 19.3 x 140x10^3 x 1.07 x 0.92 x 10^{-3} = 2128 kN

From the Wood Design Manual (CWC, 2017) CLT Wall Panel Selection Tables, Grade E1, seven-ply (245 mm), the major axis factored compressive resistance for L = 3000 mm is equal to 2140 kN (all units are per 1-m width of wall).

The interaction equation for combined loads, where moment is applied at the top of the wall is:

$$\frac{P_f}{P_r} + \frac{1}{M_r}\left[M_f + \frac{P_f\,\Delta}{1 - \frac{P_f}{P_{E,\nu}}}\right] \leq 1$$

M_r = 155 kNm in minor axis (CLT Panel Strength Selection Table, Wood Design Manual (CWC, 2017)

Eccentricity = 245/2 = 123 mm

M_f = 0 kNm (no side load)

P_E = $\frac{\pi^2 E_{05} K_{SE} K_T I}{L_e^2}$ = π^2 x 9594 x 1.0 x 1.0 x 872x10^6 / 3000^2 x 10^{-3} = 9174 kN

$P_{E,\nu}$ = $\frac{P_E}{1+\frac{P_E}{(GA)_{eff,f}}}$ = 9174 / (1 + 9174/22.2x10^3) = 6491 kN

Table 10 Wall loads

Level	Dead (kPa)	Live (kPa)	Snow (kPa)	Tributary Width (m)	Dead (kN/m)	Live (kN/m)	Snow (kN/m)
Roof	1.76		2.32	1.55	2.73		3.60
7	3.44	1.9		1.55	5.33	2.95	
6	3.44	1.9		1.55	5.33	2.95	
5	3.44	1.9		1.55	5.33	2.95	
4	3.44	1.9		1.55	5.33	2.95	
3	3.44	1.9		1.55	5.33	2.95	
2	3.44	1.9		1.55	5.33	2.95	
					37.44	17.7	3.6

P_f = (37.44 x 1.25) + (17.7 x 1.5) + 3.6 = 77 kN

$$\frac{77}{2140} + \frac{1}{155}\left(0 + \frac{77 \times 0.123}{1 - \frac{77}{6491}}\right) = 0.04 + 0.06 = 0.10$$

Therefore, a Grade E1, 245-mm thick (seven-layer) CLT wall is acceptable.

Note: a 105-mm thick (three-ply) grade V2 CLT wall has a major axis factored compressive resistance C_r = 423 kN for L = 3000 mm and a major axis moment resistance M_r = 16 kNm and would be acceptable for gravity design only.

13.7 DESIGN FOR FIRE RESISTANCE

13.7.1 Introduction

The fire resistance of exposed wood members can be calculated per the method in CSA O86-14 Annex B, based on the charring rate. Wood members can also be protected from fire using encapsulation materials, such as Type X gypsum board, to achieve higher fire-resistance ratings.

CSA O86-14 Annex B is used to calculate the structural fire resistance for wood elements with a large cross-section. This annex is considered an alternative solution and is not yet directly referenced in the NBC (NRC, 2015). The method is valid for large cross-section wood elements including solid sawn timber, SCL, CLT, and glulam members. The method is based on the reduced cross-section approach, and accounts for section loss due to charring as well as a portion of the heated zone beyond the char layer. The Wood Design Manual (CWC, 2017) has selection tables for solid sawn and glulam beams and columns, and CLT floor, roof and wall assemblies.

For glulam wood members, another alternative fire design methodology is available in Appendix D-2.11. of the NBC (NRC, 2015).

The following fire safety design examples are based on the required 2-hour fire resistance for buildings having more than six storeys; they assume no protective membrane (such as Type X gypsum board) is present on the fire side(s) of the member and that fire resistance is achieved only through charring.

Refer to Chapter 8 of this Handbook for a detailed explanation of fire performance of CLT assemblies.

13.7.2 Fire Design – Glulam Beam

Consider the two-span continuous glulam floor beam designed previously (13.6.6) for this fire-resistance beam design example.

The following conditions are considered:

Species and grade: D.Fir-L, 24f-EX

Beam dimensions: 175 mm x 570 mm

Pattern loading

Exposed to fire on three sides (top side protected by CLT floor deck designed to provide at least two hours of fire resistance)

Load combination: 1.0**D** + 1.0**L**

Lateral support: top continuous, bottom at all supports

E = 12800 MPa

f_b = 30.6 MPa

From analysis:

M_f +ve = 80.09 kNm

M_f -ve = 127.46 kNm

V_f = 72.39 kN

W_f = 310.69 kN

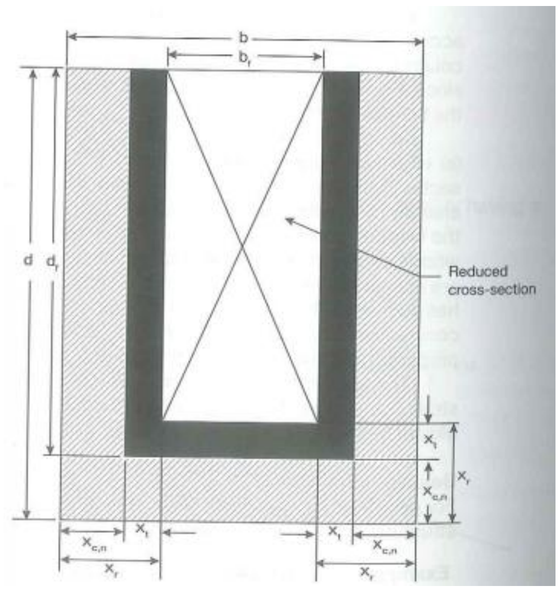


Figure 3 Reduced cross-section due to charring
(source: CWC Wood Design Manual (CWC, 2017), p. 776)

Char depth after 120 minutes (2 hours) of standard fire exposure:

$X_{c,n} = \beta_n t = (0.70)\ (120) = 84$ mm

Zero-strength zone depth:

$x_t = 7$ mm where $t \geq 20$ minutes

Therefore, the resulting loss of cross-section on each side of exposure is:

$x_r = 84 + 7 = 91$ mm

$b_f = 175 – [(2)\ (91)] = -7$ mm

Therefore, 175-mm wide glulam beam fully exposed to fire is not capable of providing a 2-hour fire resistance.

From the Beam Selection Tables for Fire Resistance in the Wood Design Manual (CWC, 2017), the minimum glulam beam width to resist a 2-hour fire exposure is 265 mm.

315 mm x 646 mm:

b_f = 315 – [(2) (91)] = 133 mm

d_f = 646 – [(1) (91)] = 555 mm

Size factor based on original beam size:

$$K_{Zbg} = \left(\frac{130}{b}\right)^{\frac{1}{10}}\left(\frac{610}{d}\right)^{\frac{1}{10}}\left(\frac{9100}{L}\right)^{\frac{1}{10}} = \left(\frac{130}{315}\right)^{\frac{1}{10}}\left(\frac{610}{646}\right)^{\frac{1}{10}}\left(\frac{9100}{7000}\right)^{\frac{1}{10}} = 0.93 \quad Governs\ over\ K_L = 1.0$$

K_L = 1.0 for calculating moment resistance to positive bending moment in beam (compressive edge fully supported along length by CLT floor decking designed to provide at least a 2-hour fire resistance). The stability factor K_L should be calculated using the reduced cross-section if the compressive edges are not fully restrained for the entire fire exposure.

K_L = 0.42 for calculating moment resistance to negative bending moment in beam (calculated per CSA O86-14 Clause 7.5.6.4 using net cross-section).

Section modulus based on reduced cross-section = S_f = (133 x 555^2) / 6 = 6827888 mm^3

F_b = 30.6 MPa x K_D x K_{fi} = 30.6 x 1.15 x 1.35 = 47.51 MPa

M_r = Φ x F_b x S_f x K_x x K_{zbg} = 1.0 x 47.51 x 6827888 x 1.0 x 0.93 x 10^{-6} = 301 kNm > 80.09 kNm (27%)

M_r = Φ x F_b x S_f x K_x x K_L = 1.0 x 47.51 x 6827888 x 1.0 x 0.42 x 10^{-6} = 135 kNm > 127.46 kNm (94%)

Beam volume = 315 x 646 x 15200 x 10^{-9} = 3.1 m^3

Using the Selection Tables for Fire Resistance, W_r = ($W_rL^{0.18}$) x $L^{-0.18}$ = 649 x $(7)^{-0.18}$ = 457 kN > W_f = 310.69 kN

13.7.3 Fire Design – Glulam Column

Consider the glulam column designed previously (13.6.7) for this fire resistance column design example.

The following conditions are considered:

Species and grade: D.Fir-L, 16c-E

Column dimensions: 365 mm x 342 mm

Column height is 3.0 m

Exposed to fire on four sides

Load combination: 1.0**D** + 1.0**L** + 1.0**S**

Effectively pinned at both ends (K_e = 1.0)

Total specified load = 1467.23 kN

Char depth after 120 minutes of standard fire exposure:

$X_{c,n} = \beta_n t = (0.70)\ (120) = 84$ mm

Zero-strength zone depth:

$x_t = 7$ mm where $t \geq 20$ minutes

Therefore, the resulting loss of cross-section on each side of exposure is:

$x_r = 84 + 7 = 91$ mm

$b_f = 342 - [(2)\ (91)] = 160$ mm

$d_f = 365 - [(2)\ (91)] = 183$ mm

The modified compressive strength parallel to grain is:

$F_c = 30.2$ MPa

$K_{fi} = 1.35$ (specified strength adjustment factor for fire design)

$K_D = 1.15$ for short-term loading

$F_c = K_{fi} \times f_c \times K_D \times K_H \times K_{SC} \times K_T = 1.35 \times 30.2 \times 1.15 \times 1.0 \times 1.0 \times 1.0 = 46.9$ MPa

Slenderness ratios calculated using reduced cross-section are:

$C_{cb} = 3000 / 160 = 18.75$

C_{cd} = 3000 / 183 = 16.39

Size factors, based on original beam size are:

K_{Zcb} = 6.3 x $(342 \text{ x } 3000)^{-0.13}$ = 1.04 ≤ 1.3

K_{Zcd} = 6.3 x $(365 \text{ x } 3000)^{-0.13}$ = 1.03 ≤ 1.3

Modulus of elasticity, E = 12,400 MPa

$$K_{cb} = \left[1.0 + \frac{46.9 \times 1.04 \times 18.75^3}{35 \times 12400 \times 1.0 \times 1.0}\right]^{-1} = 0.57$$

$$K_{cd} = \left[1.0 + \frac{46.9 \times 1.03 \times 16.39^3}{35 \times 12400 \times 1.0 \times 1.0}\right]^{-1} = 0.67$$

P_{rb} = Φ x F_c x A x K_{Zcb} x K_c = 1.0 x 46.9 x 160 x 183 x 1.04 x 0.57 x 10^{-3} = 814 kN < 1467.23 kN

P_{rd} = Φ x F_c x A x K_{Zcd} x K_c = 1.0 x 46.9 x 160 x 183 x 1.03 x 0.67 x 10^{-3} = 847 kN < 1467.23 kN

Previously designed column size fails fire resistance check under 2-hour fire exposure.

Using the Column Selection Tables for Fire Resistance, a 365-mm x 456-mm, 16c-E D.Fir-L glulam section with 3.0 m effective length has:

P_{rx} = 1580 kN > 1467.23 kN

P_{ry} = 1290 kN < 1467.23 kN   NG

The largest standard glulam column size provided in the Wood Design Manual (CWC, 2017) does not meet the requirements for a 2-hour fire exposure. A custom column size may be designed to meet the 2-hour fire exposure but is not shown here in detail. For example, a 365-mm x 532-mm, 16c-E D.Fir-L glulam section would meet the requirements for a 2-hour standard fire exposure. In addition, the designer must refer to the product manufacturers' section sizes, which may differ from the standard sizes in CSA O86-14.

13.7.4 Fire Design – CLT Floor

Consider the simply supported span floor CLT panel designed previously (13.6.3) for this fire resistance floor panel design example.

The following conditions are considered:

Stress grade: E1

Five-layer CLT panels, 35-mm thick plies (175-mm total thickness)

Simply supported span of 6100 mm

Exposed to fire on underside

Load combination: 1.0**D** + 1.0**L**

Specified dead load = 3.24 kPa

Specified live load = 1.9 kPa

Total specified load = 3.24 + 1.9 = 5.14 kPa x 1 m = 5.14 kN/m

Specified bending moment:

M_f = 1/8 x w_f x L^2 = 1/8 x 5.14 x 6.1^2 = 23.9 kNm/m

Reduced cross-section dimensions based on notional charring rate:

$X_{c,n} = \beta_n t$ = (0.80) (120) = 96 mm

Zero-strength zone depth:

x_t = 7 mm where t ≥ 20 minutes

Therefore, the resulting loss of cross-section on each side of exposure is:

x_r = 96 + 7 = 103 mm

Neutral axis after a 2-hour standard fire exposure:

$$\bar{y} = \frac{\left(\frac{35}{2}\right)(35) + \left(35 + 35 + {}^{2}/_{2}\right)(2)}{35 + 2} = 20.4\ mm$$

Moment of inertia for a 1-m width of panel:

$$I = \frac{(1000)(35)^3}{12} + \frac{(1000)(2)^3}{12} + (1000)(35)\left(20.4 - {}^{35}/_{2}\right)^2 + (1000)(2)\left(35 + 35 + {}^{2}/_{2} - 20.4\right)^2 = 9 \times 10^6\ mm^4$$

$$S_{eff} = \frac{I}{c} = \frac{9 \times 10^6}{\left((35 + 35 + 2) - 20.4\right)} = 0.17 \times 10^6\ mm^3$$

$F_b = f_b \times K_D \times K_H \times K_{sb} \times K_T \times K_{fi}$ = 28.2 x 1.15 x 1.0 x 1.0 x 1.25 = 40.54 MPa

K_{rb} = adjustment factor for bending = 0.85

$M_r = \Phi \times F_b \times S_{eff} \times K_{rb}$ = 1.0 x 40.54 x 0.17×10^6 x 0.85 x 10^{-6} = 5.9 kNm/m < 24.61 kNm/m

Therefore, a five-layer, 175-mm thick CLT panel requires a protective membrane on the underside to resist a 2-hour standard fire exposure.

Considering a 175-mm thick, five-layer CLT floor panel with one layer of 15.9-mm Type X gypsum board directly applied on the underside of the panel:

One layer of 15.9-mm Type X gypsum board provides a 30-min fire-resistance duration (per Clause B.8.1 of CSA O86-14); therefore, the CLT panel must have a 90-minute fire resistance rating on its own.

$X_{c,n} = \beta_n t = (0.80)\ (90) = 72$ mm

Zero-strength zone depth:

$x_t = 7$ mm, where $t \geq 20$ minutes

Therefore, the resulting loss of cross-section on each side of exposure is:

$x_r = 72 + 7 = 79$ mm

Neutral axis after a 90-min standard fire exposure:

$$\bar{y} = \frac{\left(\frac{35}{2}\right)(35) + \left(35 + 35 + {}^{26}/_{2}\right)(26)}{35 + 26} = 45.4\ mm$$

Moment of inertia for a 1-m width of panel:

$$I = \frac{(1000)(35)^3}{12} + \frac{(1000)(26)^3}{12} + (1000)(35)\left(45.4 - {}^{35}/_{2}\right)^2 + (1000)(2)\left(35 + 35 + {}^{26}/_{2} - 45.4\right)^2 = 69.0 \times 10^6\ mm^4$$

$$S_{eff} = \frac{I}{c} = \frac{69 \times 10^6}{\left((35 + 35 + 26) - 45.4\right)} = 1.36 \times 10^6\ mm^3$$

$F_b = f_b \times K_D \times K_H \times K_{sb} \times K_T \times K_{fi} = 28.2 \times 1.15 \times 1.0 \times 1.0 \times 1.25 = 40.54$ MPa

K_{rb} = adjustment factor for bending = 0.85

$M_r = \Phi \times F_b \times S_{eff} \times K_{rb} = 1.0 \times 40.54 \times 1.36 \times 10^6 \times 0.85 \times 10^{-6} = 46.9$ kNm/m > 24.61 kNm/m

Therefore, a five-layer, 175 mm-thick CLT panel with one layer of Type X gypsum board directly applied to the underside of the panel can resist a 2-hour standard fire exposure.

From the Solid Floor and Roof Panel Selection Tables for Fire Resistance in the Wood Design Manual (CWC, 2017), a seven-ply (245-mm thick), Grade E1 CLT panel with a 2-hour exposure has M_r = 42.8 kNm/m > 24.61 kNm/m.

Thus, a seven-layer, 245-mm thick CLT panel would be required for an exposed CLT ceiling.

13.7.5 Fire Design – CLT Wall

Consider the CLT wall previously designed (Section 13.6.8) for this fire safety wall panel design example.

This example assumes that the vertical loads on the wall panel are concentric; its transient eccentricity due to charring is not considered, i.e. the designer is to consider whether the fire design of the CLT wall panels is to include combined compression and bending from eccentric loads and/or P-Δ effects. Refer to Chapter 8 of this Handbook for further details and explanations for applicable charring rates and considerations for combined axial compression and bending due to charring.

The following conditions are considered:

Stress grade: E1

Seven-layer CLT panels, 35-mm thick plies (245-mm total thickness)

Effective height of 3000 mm

Wall is effectively pinned at both ends (K_e = 1.0)

Exposed to fire on one side without gypsum wall board encapsulation

Load combination: 1.0**D** + 1.0**L**

Specified dead load = 37.4 kN/m

Specified live load = 17.7 kN/m

Specified snow load = 3.6 kN/m

Total specified load = 37.4 + 17.7 + 3.6 = 58.7 kN/m

Reduced cross-section dimensions based on notional charring rate:

$X_{c,n} = \beta_n t$ = (0.80) (120) = 96 mm

Zero-strength zone depth:

x_t = 7 mm where t ≥ 20 minutes

Therefore, the resulting loss of cross-section on each side of exposure is:

x_r = 96 + 7 = 103 mm

Neutral axis after a 2-hour standard fire exposure:

$$\bar{y} = \frac{\left(\frac{35}{2}\right)(35) + \left(35 + 35 + \frac{35}{2}\right)(35) + \left(35 + 35 + 35 + 35 + \frac{2}{2}\right)(2)}{35 + 35 + 2} = 55\ mm$$

Moment of inertia for 1-meter width of panel:

$$I = \frac{(1000)(35)^3}{12} + \frac{(1000)(35)^3}{12} + \frac{(1000)(2)^3}{12} + (1000)(35)\left(55 - {}^{35}/_{2}\right)^2 + (1000)(35)\left(35 + 35 + \frac{35}{2} - 55\right)^2 + (1000)(2)\left(35 + 35 + 35 + 35 + \frac{2}{2} - 55\right)^2 = 108.1 \times 10^6\ mm^4$$

$$A_{eff} = (1000)(35) + (1000)(35) + (1000)(2) = 72000\ mm^2/m$$

$$C_c = \frac{L_e}{\sqrt{12}\sqrt{I/A_{eff}}} = \frac{3000}{\sqrt{12}\sqrt{108.1 \times 10^6/72000}} = 22.4$$

K_D = duration of load factor = 1.15 (per B.3.3 of CSA O86-14)

K_H = system factor = 1.0

K_{Sc} = service condition factor in bending = 1.0

K_T = treatment factor = 1.0

K_{fi} = strength adjustment factor for fire design = 1.25 (per B.6.3 of CSA O86-14)

f_c = 19.3 MPa (Table 8.2.4 in CSA O86-14)

F_c = 19.3 x 1.15 x 1.0 x 1.0 x 1.0 x 1.25 = 27.7 MPa

Size factor is based on the original CLT panel dimensions using only the longitudinal plies.

$$A_{eff} = (1000)(35) + (1000)(35) + (1000)(35) + (1000)(35) = 140000\ mm^2/m$$

$$I_{eff} = 4 \times \frac{(1000)(35)^3}{12} + 2 \times \left((1000)(35)\left(122.5 - \frac{35}{2}\right)^2 + (1000)(35)\left(122.5 - 35 - 35 - \frac{35}{2}\right)^2\right) = 443.1 \times 10^6 mm^4$$

$$K_{Zc} = 6.3\left(\sqrt{12}\left(\sqrt{\frac{443.1 \times 10^6}{140000}}\right)(3000)\right)^{-0.13} = 1.12 \leq 1.3$$

K_{SE} = service condition factor for modulus of elasticity = 1.0

E = 11,700 MPa

$$K_C = \left[1.0 + \frac{F_c\, K_{Zc}\, C_c{}^3}{35\, E\, K_{SE}\, K_T}\right]^{-1} = \left[1.0 + \frac{27.7 \times 1.12 \times 22.4^3}{35 \times 11700 \times 1.0 \times 1.0}\right]^{-1} = 0.54$$

Note that for fire design, Ø is taken as 1.0 (per B.3.2 of CSA O86-14).

$$P_r = \emptyset\, F_c\, A\, K_{Zc}\, K_C = 1.0 \times 27.7 \times 72000 \times 1.12 \times 0.54 = 1206.2\,\mathrm{kN/m}\ \text{of panel width}$$

From the Solid Wall Panel Selection Tables for Fire Resistance in the Wood Design Manual (CWC, 2017), seven-ply (245-mm thick) Grade E1 CLT panel with a 2-hour fire exposure has P_r = 1180 kN. Using the same tables, it can be shown that neither a three-ply (105-mm thick) or five-ply (175-mm thick) Grade E1 CLT wall panel can resist any compression force for a 3000-mm effective height.

Thus, a seven-layer, 245-mm thick CLT panel is required for an exposed CLT wall with fire damage on one side.

13.7.6 Fire Design – Discussion

The fire resistance design examples above demonstrate the significant impact charring has on the section size required to resist a 2-hour standard fire exposure for a tall wood building, namely where no further fire protection measures are undertaken.

Also, as per the NBC (NRC, 2015), support elements must be provided with the same fire-resistance rating as the element being considered.

Connections that are critical for the support of gravity loads acting on the structure must be designed to have at least the same fire-resistance rating as the elements they support. Connections where the steel is located within the reduced cross-section of the wood element are considered appropriately protected (see CSA B.9 of O86-14). It is strongly recommended to consult with the engineered wood product manufacturer for proper design and detailing of connections in mass timber buildings.

Furthermore, it is important to note that CSA O86-14 Clause B.2.2 describes modifications to the glulam layup required for the Annex B calculation method to be valid. Fundamentally, core lamination(s) are removed, the tension zone moves inward, and outer tension lamination(s) are added. It is recommended practice to note this clause on drawings and/or specifications, to ensure that the glulam manufacturer is aware of the correct layup to use; the designer may wish to review layup shop drawings prior to manufacture, to ensure conformity.

The economical design of mass timber elements with practical tributary widths and areas will likely result in the need for a protective membrane, such as Type X gypsum board, to help achieve the required fire resistance without the requirement for a significant increase in the cross-sectional area of the members. Moreover, encapsulation materials are most likely to be required, as per NBC (NRC, 2015), to limit ignition and contribution of mass timber to fire growth, intensity, and duration. Ultimately, these encapsulation materials enhance the inherent fire resistance of mass timber elements. A good building design could include some exposed wood elements, but only in controlled locations.

13.8 LATERAL DESIGN

13.8.1 Introduction

The design of CLT shear walls and diaphragms is defined in Clause 11.9 of CSA O86-14 Update 2 (CSA, 2017), in combination with the CWC Commentary on CSA O86-14. Refer to Chapter 4 of this Handbook for a detailed explanation of lateral design for CLT structures.

All CLT structures should be designed using capacity design principles with, at minimum, moderately ductile connections at specified locations, and all other connections protected using an over-strength factor. CSA O86-14 Update 2 (CSA, 2017) states that the appropriate force modification factors for CLT structures are $R_d \leq 2.0$ and $R_o = 1.5$ for platform-type construction.

The designer must check bearing perpendicular to grain, elastic shortening, and shrinkage for platform construction.

The type of lateral analysis for earthquake loads depends on the level of seismic hazard. For structures where $I_E\ F_a\ S_a(0.2) \leq 0.35$ or those meeting either of the other criteria in Clause 4.1.8.7 of the NBC (NRC, 2015), the equivalent static force procedure (ESFP) may be used. Where these criteria do not apply, a dynamic analysis is the default approach. The dynamic analysis may be linear, using the modal response spectrum method, the time history method, or a non-linear dynamic analysis. Refer to Clause 4.1.8.12 of NBC 2015 and the structural commentaries for guidance on the dynamic analysis procedure.

Podium structures, where mass timber is used above one or two storeys of above-grade reinforced concrete construction, typically have many of the following irregularities:

- Type 1 – Vertical stiffness
- Type 2 – Weight
- Type 3 – Vertical Geometry
- Type 4 – In-plane
- Type 5 – Out-of-plane

Type 4 and 5 irregularities can still exist between the concrete and mass timber portions of the structure, where the lateral systems for each are typically independent, while neither of these irregularities are allowed within the mass timber portion only.

This eight-storey example has irregularity types 1 and 2, where the concrete podium is much stiffer and heavier at the first storey compared to the timber storey above.

Where irregularities are present in a structure, a dynamic analysis should be the default procedure for earthquake design.

Alternatively, a two-stage approach may be considered by the designer for podium structures. This enables the wood and concrete portions of the structure to be analysed independently using the appropriate ductility and over-strength factors for each material, where forces are transferred from the mass timber above to the podium below. Current design provisions for using a two-stage approach can be found in the 2015 NBC and Commentary J, and in ASCE 7-16. The advantage of the two-stage approach is that it may allow the simplified elastic static force procedure to be used for both the concrete podium and the mass timber structure above.

From Clause 12.2.3.3 in ASCE 7-16, it can be determined that a two-stage analysis procedure is applicable to the example building. It has been shown in Section 13.4.3 that the stiffness of the concrete podium is at least ten times the stiffness of the upper wood portion.

The period of the entire building per ASCE 7-16 is:

$$T_a = \sqrt{(0.05 \times 21^{0.75})^2 + (0.05 \times 3.6^{0.75})^2} = 0.51\ s$$

The period of the upper wood portion is:

$$T_a = 0.05 \times 21^{0.75} = 0.49\ s$$

The ratio of periods between the entire building and the upper wood portion is 0.51 / 0.49 = 1.04 < 1.10, and thus complies with the clause.

Therefore, it is appropriate to analyse the upper wood portion using the modal response spectrum procedure; the single storey concrete podium can be analysed with the equivalent static force procedure.

The upper wood portion will be designed as a separate structure using the higher R_d and R_o for CLT shear walls. The lower concrete podium can be designed as a separate structure using R_d and R_o for conventional concrete shear walls, plus the forces related to the lateral capacity of the upper part of the structure. The design forces need not exceed those calculated using an R_dR_o value of 1.3.

13.8.2 State-of-the-Art Earthquake Design Considerations

The following guidance is based on the latest research on CLT lateral design and goes beyond the requirements and guidance currently provided in CSA O86-14 Update 2 (CSA, 2017) and the CWC Commentary (CWC, 2017), which applies primarily to platform-type construction. The following recommendations have been implemented in the 2019 Edition of the CSA Standard O86-19 (CSA, 2019).

CSA O86-14 Update 2 (CSA, 2017) allows seismic energy to be dissipated by wall panels acting in rocking, to allow R_dR_o = 3.0. It is assumed that rocking behaviour is the desired energy dissipation method and that sliding is to be avoided, where the designer wishes to use R_dR_o = 3.0.

To ensure rocking behaviour, test results indicate that the aspect ratio of each shear wall or shear wall segment (where a segment refers to a wall made from one or more adjacent CLT panels) should be between 2:1 and 4:1. Therefore, connections between shear wall segments and shear wall panels to the floor below need to be designed to yield, thereby allowing the segments to rotate. Structures with wall segments with an aspect ratio less than 2:1 must be designed with forces calculated using R_dR_o = 1.3 (refer to Chapter 4).

It is recommended that CLT shear wall panels have a thickness of no less than 87 mm. CLT panels can be of any size or aspect ratio. A shear wall segment constructed from one or more CLT panels, if rigidly connected, will behave as one segment when determining the aspect ratio of shear wall segments, in accordance with the limits discussed above.

CSA O86-14 Update 2 (CSA, 2017) states that Type 4 or 5 irregularities, as defined in the NBC (NRC, 2015), shall not be allowed. It is also recommended that Type 6, 8 or 9 irregularities, in addition to Type 4 or 5 irregularities, shall not be allowed. Type 1, 2, 3 or 7 irregularities are permissible, but structures with these irregularities must be analysed using dynamic analysis. This has implications for the design approach to CLT structures on concrete podiums, where a two-step design process cannot be justified.

CSA O86-14 Update 2 (CSA, 2017) does not consider the compressive resistance at the ends of CLT shear walls due to overturning forces. The compressive resistance should be greater than or equal to the overturning forces.

CLT diaphragms are to be capacity-protected with non-dissipative connections to walls beneath the diaphragm level and between adjacent CLT diaphragm panels. Compressive resistance perpendicular to the face of the diaphragm where overturning forces are transferred from the walls is to be greater than or equal to the overturning forces. Diaphragm chords, struts, and collectors, including those around openings, are to be capacity-protected, where the seismic design force need not exceed the force determined using R_dR_o = 1.3.

Where $I_EF_aS_a(0.2)$ is greater than 0.35, gravity-resisting elements that are not part of the seismic-force-resisting system are to have satisfactory resistance and displacement capability, to support their loads while undergoing seismically induced deformations.

13.8.3 Structural Modelling

This Section gives a general description of the structural finite element model developed as part of the first iteration for the lateral design of the example structure, as well as additional recommendations.

Glulam beams and columns are modelled as 'beam' elements with material properties given in Section 13.3. Beams are continuous and columns are hinged from floor to floor.

Shear walls and floors/roof are modelled as 'surface' elements with multi-ply CLT panel properties, per Section 13.3. The structural model developed is a first iteration and does not include connection stiffnesses. Note that the model will underestimate displacements and further iterations would be necessary to complete the design.

The diaphragms are modelled as semi-rigid elements using the in-plane stiffness properties of the CLT panels, for each direction. CLT floor and roof panels are defined individually, with moment releases at the joints between them.

The structural model should account for P-Δ effects and any effects that influence the lateral stiffness of the building, per Sentence 8 of Clause 4.1.8.3. of NBC 2015, such as the connection stiffness between elements in a CLT structure (the latter has not been included for the example in this Chapter).

13.8.4 Linear Dynamic Analysis (Seismic)

The design example of the eight-storey building was analysed using a three-dimensional finite element model. The model was used to determine the fundamental natural period of the structure and then analysed using linear dynamic analysis, per Clause 4.1.8.12 of NBC 2015. The modal response spectrum method was used per the NBC and the guidance provided in Commentary J of NBC 2015.

The modal response spectrum analysis was performed for the wood portion of the building only, using R_dR_o equal to 3.0 for the two-stage approach discussed in Section 13.8.1. This analysis was conducted to confirm the behaviour of the building while accounting for torsional sensitivity (a structural irregularity) in one direction (see Section 13.4.3).

The linear dynamic analysis uses the spectral acceleration values, S(T), based on the site-specific accelerations and site class (see Section 13.4.3 of this Chapter). Accidental torsional moments are accounted for in accordance with NBC (NRC, 2015). Separate design cases are calculated to determine the elastic base shear, V_e, and design base shear, V_d. The design base shear is calculated by multiplying the elastic base shear by the Importance Factor and then dividing by R_dR_o. Since the example building is irregular according to Article 4.1.8.7. of the NBC, the design base shear is to be taken as the maximum of the calculated V_d and 100% of V, as determined using the ESFP. To determine V, the fundamental natural period, T_a, may be based on the frequency analysis output of the structural model.

The fundamental natural period of the building, T_a, was previously estimated as 0.55 seconds in Section 13.4.3, using the NBC (NRC, 2015) equations. Using frequency analysis, the period of the finite element model was calculated, as noted in Table 13.11, to be 0.88 s and 0.80 s in the North-South and East-West directions, respectively. NBC Article 4.1.8.11 states that the fundamental natural period determined from the model shall not be greater than twice that determined by the code equation for shear walls. The discrepancy in natural fundamental frequencies likely results from the NBC equation being based on measurements taken from concrete structures rather than wood structures, which are lighter and typically more flexible.

Therefore, V =1039 kN, where T_a = 0.8 seconds (compared to 1327 kN where T_a = 0.55 seconds) (refer to Section 13.4.3).

From the structural model, V_e = 15529 kN in the North-South direction, V_e = 13406 kN in the East-West direction, V_d = 5177 kN in the North-South direction, and V_d = 4469 kN in the East-West direction.

In the example, V_d is the larger of V_d and V and therefore V_d governs, in accordance with NBC 4.1.8.12.(9).

Elastic storey shears, storey forces, member forces, and deflections from the linear dynamic analysis are multiplied by the ratio of V_d/V_e to determine design values per NBC 4.1.8.12.(10).

For the example building, the ratio of V_d/V_e in both directions is 0.33, the inverse of which is 3, the same as the value for R_dR_o.

The lateral deflection of each storey derived by modal response spectrum analysis for the North-South direction (perpendicular to long face) is shown in Table 11. Since deflection is obtained from the linear dynamic analysis, deflections are multiplied by V_d/V_e. The inter-storey drift, x_i, is less than 2.5%h_s = 0.025 x 3000 mm = 75 mm. Therefore, the seismic design of this building is adequate. Lateral deflections determined from the model for the East-West direction due to seismic load are less than those for the North-South direction.

Table 11 Lateral deflection (North-South direction) at centre of gravity due to seismic load

Storey	X_i, mm	$X_i(V_d/V_e)$, mm	x_i, mm
8	40.3	13.4	1.8
7	34.7	11.6	2.0
6	28.7	9.6	2.1
5	22.4	7.5	2.1
4	16.1	5.4	2.0
3	10.1	3.4	1.8
2	4.9	1.6	1.6

The periods of the example design finite element model were derived by running a frequency analysis using the Lanczos method of eigenvalue extraction. The results are shown in Table 12 for the first 10 modes and also indicate significant modal participation. See Figures 4 to 6 for mode shapes.

Table 12 First ten modes in East-West and North-South directions

	East-West Direction			North-South Direction		
Mode	F (Hz)	T (s)	Mass Participation (%)	F (Hz)	T (s)	Mass Participation (%)
1	1.252	0.799	76	0.957	1.045	0.2
2	2.695	0.371	0.1	1.130	0.885	75
3	4.437	0.225	19	3.962	0.252	0.4
4	7.936	0.126	4.0	4.317	0.232	18
5	10.552	0.095	1.0	7.449	0.134	0.5
6	11.160	0.090	0.0	8.102	0.123	4.0
7	12.498	0.080	0.3	9.832	0.102	0.1
8	13.670	0.073	0.1	10.719	0.093	1.0
9	14.650	0.068	0.4	11.655	0.086	0.1
10	21.142	0.047	0.0	12.562	0.080	0.1
TOTALS			100			99

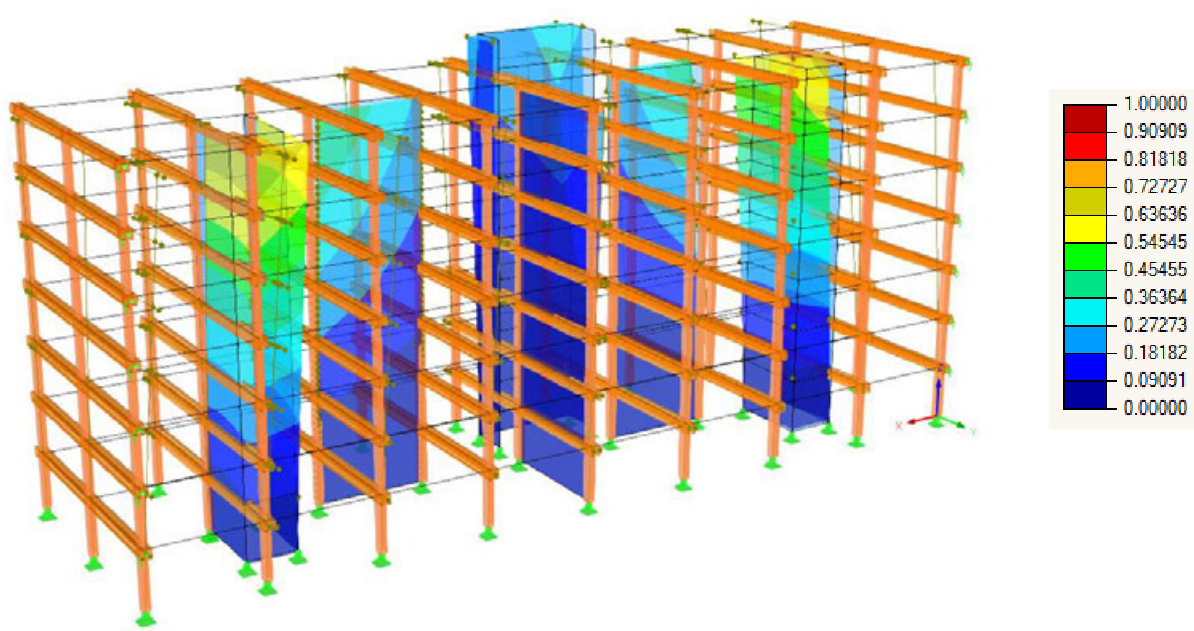


Figure 4 Structure first mode shape

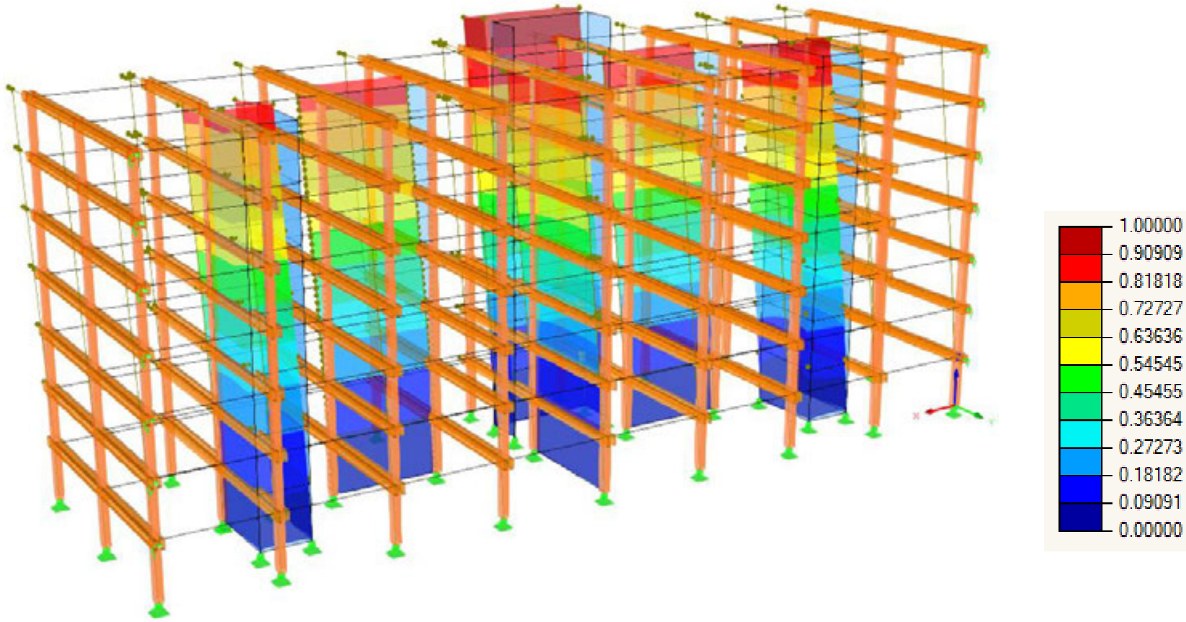


Figure 5 Structure second mode shape

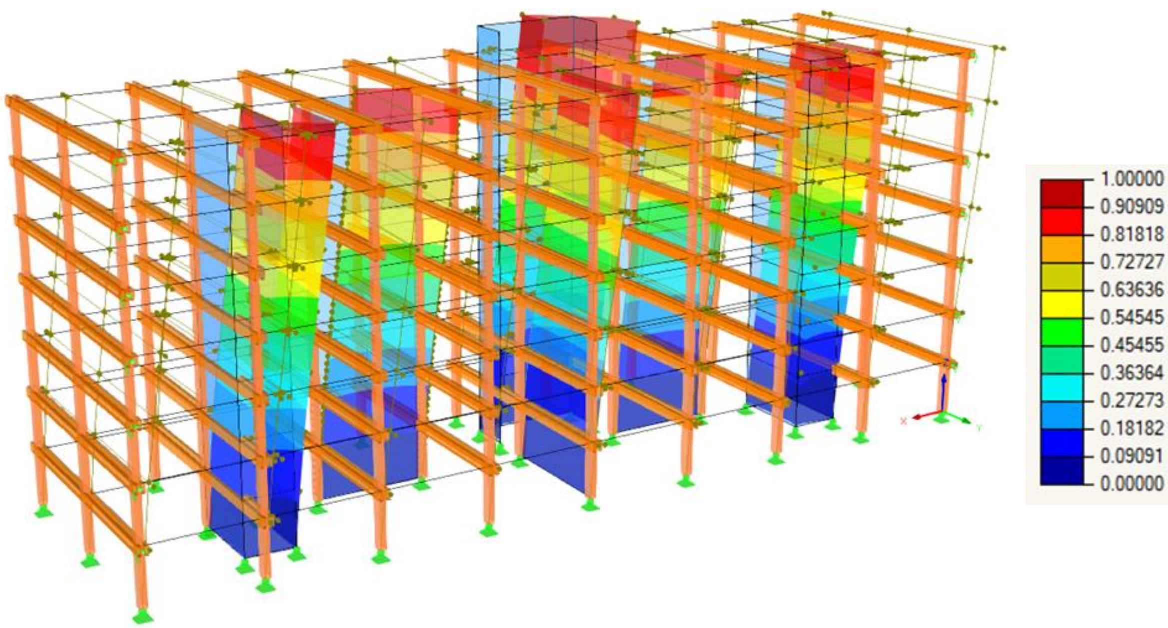


Figure 6 Structure third mode shape

The stiffness of the floor and roof diaphragms are assessed by comparing the diaphragm deflection between supports, to the average storey drift at the shear walls located at the supports of the diaphragm span in the N-S direction, per C12.3.1.3 of ASCE 7-16. For this model, the ratio was determined to be less than 2.0, therefore defining the diaphragms as rigid or semi-rigid rather than flexible.

13.8.5 Shear Wall Lateral Design

Design of the CLT shear wall at the elevator core, as shown on plan in Figure 7.

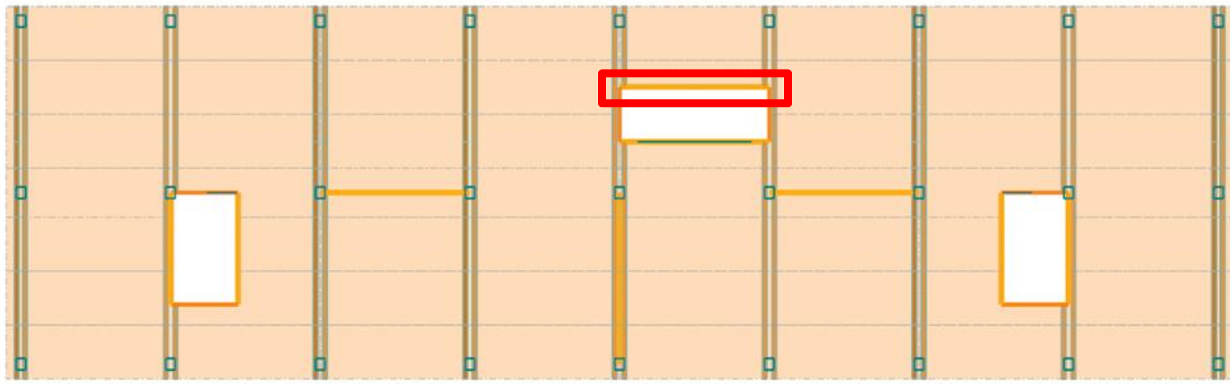


Figure 7 Example shear wall location

This shear wall resists lateral loads in the East-West direction. Earthquake loads control the strength design of this wall, but lateral drift must be verified for earthquake and wind loads.

The storey drifts and inter-storey drifts for this shear wall under seismic loading are listed in Table 13; similarly, shear wall forces are listed in Table 14. As mentioned previously, the stiffness of the connections has not been taken into account in the model. Consequently, the storey drifts may be underestimated.

Table 13 Shear wall displacement due to seismic loading (load case 1.0E)

Storey	X_i, (mm)	$X_iR_dR_o/I_E$, (mm)	x_i, (mm)	$X_i/H_s0.025$
Penthouse	19.1	57.3	4.5	0.06
Roof	17.6	52.8	7.5	0.1
8	15.1	45.3	8.1	0.11
7	12.4	37.2	8.4	0.11
6	9.6	28.8	8.4	0.11
5	6.8	20.4	8.1	0.11
4	4.1	12.3	6.3	0.08
3	2	6	6	0.08
2				

Table 14 Shear wall forces due to seismic loading (load combination 1.0D + 1.0E)

Level	Storey Shear (kN)	Accumulated Shear (kN)	Overturning T_f=C_f (kN) (see S400)	P_f from Dead Load (kN)	Resisting T_f=C_f (kN)	Net T_f=C_f (kN)
Penthouse	22	22	7	10	5	2
Roof	70	92	52	36	18	34
8	79	171	137	114	57	80
7	36	207	238	187	94	145
6	20	227	350	251	126	225
5	24	251	473	304	152	321
4	34	285	614	336	168	446
3	35	320	771	343	172	599
Base shear		358				

The storey drifts and inter-storey drifts for this shear wall resulting from wind loads are listed in Table 15. Similarly, shear wall forces are listed in Table 16.

Table 15 Shear wall displacement due to wind (load case 0.75W)

Level	X_i, (mm)	x_i, (mm)	% h/500
Penthouse	3.2	0.2	5
Roof	3	0.5	8.3
8	2.5	0.4	6.7
7	2.1	0.5	8.3
6	1.6	0.5	8.3
5	1.1	0.5	8.3
4	0.6	0.3	5
3	0.3	0.3	5
2			

Table 16 Shear wall forces from wind loading (load combination 0.9D + 1.4W)

Level	Storey Shear (kN)	Accumulated Shear (kN)	Overturning T_f=C_f (kN)	P_f from Dead Load (kN)	Resisting T_f=C_f (kN)	Net T_f=C_f (kN)
Penthouse	3	3	1	9	4	-3
Roof	6	8	5	32	16	-11
8	13	21	15	103	51	-36
7	13	34	32	168	84	-52
6	11	45	54	226	113	-59
5	10	55	81	274	137	-56
4	11	66	113	302	151	-38
3	10	76	150	309	154	-4
Base shear		80				

Comparing values listed in Tables 13 through 16, shows that the seismic loads govern the lateral design of the building. Under wind loading, the shear wall does not go into net tension at the hold-down locations.

The panel design force need not exceed the force determined using R_dR_o = 1.3, where seismic loads govern, thus:

Maximum V_f = (358 x 1.5 x 2)/1.3 = 826 kN

Wall length = 6.1 m

v_f = 826 kN / 6.1 m = 135 kN/m

Using 175-mm thick, grade E1, five-layer CLT panel.

There are no current criteria for in-plane shear resistance for CLT in CSA O86-14. Based on manufacturer guidelines, V_r = 190 kN/m > v_f =135 kN/m

Note that to meet the aspect ratio limits in CSA O86-14 Update 2 (CSA, 2017) Clause 11.9.2.5, the wall will be divided into four sub-segments of 1.525 m in width, to achieve an aspect ratio of 2:1, within the code bounds.

To allow ductility to occur in connections, it is important that the CLT panels have a much higher in-plane shear resistance than the connections.

The designer must also verify CLT floor and roof slabs for diaphragm in-plane shear resistance and detail panel-to-panel connections as non-dissipative (capacity-protected).

13.9 WIND-INDUCED VIBRATION

The determination of building vibration is based on the procedure found in Commentary I of the NBC Commentary.

While the maximum lateral wind loading and deflection are generally in the direction parallel to the wind (i.e. the along-wind direction), the maximum acceleration of a building leading to possible human perception of motion or even discomfort may occur in the direction perpendicular to the wind (i.e. the across-wind direction). Across-wind accelerations are likely to exceed along-wind accelerations if the building is slender about both axes, that is if $\sqrt{wd}/H$ is less than one-third, where *w* and *d* are the across-wind effective width and along-wind effective depth, respectively, and *H* is the height of the building. The across- and along-wind accelerations, a_w and a_d, in m/s^2, are calculated from the equations below:

$$a_w = f_{n^2} w g_p \sqrt{wd}\left(\frac{a_r}{\varrho_B g \sqrt{\beta_w}}\right)$$

$$a_D = 4\pi^2 f_{nD^2} g_p \sqrt{\frac{K_s F}{E_{eH}\beta_D}\frac{\Delta}{C_g}}$$

where, $a_r = 78.5 \times 10^{-3}\left[V_H/\left(f_{nW}\sqrt{wd}\right)\right]^{3.3}$, in N/m^3, ρ_B is the average density of the building in kg/m^3, β_w and β_D are the fractions of critical damping in across- and along-wind directions and are taken as 0.015, f_{nW} and f_{nD} are the fundamental natural frequencies in across- and along-wind directions in Hz, Δ is the maximum wind-induced lateral deflection at the top of the structure in metres obtained from the finite element model, and *g* is the acceleration due to gravity (9.81 m/s^2).

Substituting these values into the equations above, a_W and a_D are 0.3% and 0.5% of *g*, respectively, in the North-South direction, and 0.1% and 0.3% of *g* respectively, in the East-West direction. These values are outside the range of 0.5% to 1.5% of *g,* where movement of a building becomes perceptible to most people (sentence 76 of Commentary I).

13.10 CONNECTIONS

Refer to Chapter 5 of this Handbook for an in-depth review of connections and connection design in CLT structures.

The design of connections uses capacity-based design principles. Connections are divided between those that allow non-linear deformations and energy dissipation (ductile) and those which are non-dissipative with sufficient over-strength to remain linear elastic (non-ductile).

The following connection types should be energy dissipative, enabling the ductile behaviour of the CLT structure:

a) Discrete hold-downs and tension ties for resisting overturning (applying an over-strength such that vertical joints between shear wall segments yield before the hold-downs) Please note that this requirement was modified in CSA O86-2019.

b) Vertical joints between shear wall segments enabling rotation.

c) Shear connectors between shear walls and foundations, and shear walls to floors beneath, in uplift only.

Examples of non-dissipative (capacity-protected) connections are:

a) Connectors between floor or roof panels to resist horizontal diaphragm shears, or that are acting as chords or collectors.

b) Connectors between perpendicular walls.

c) Connectors between floor or roof panels and walls below, to transfer diaphragm shears into shear walls.

Examples of these are shown in elevation on drawing sheet S401, included in Annex A to this Chapter.

To ensure sufficient over-strength, non-dissipative connectors should not yield when the dissipative connectors are at their maximum resistance or should be designed to withstand the predicted/selected displacement of the structure. The majority of displacement in a CLT structure is predicted to be from the non-linear deformation of ductile connectors. An iterative process may be required to achieve a connection design that provides compatibility of displacement demands at different design force demands, while ensuring correct yielding behaviour.

As described in Chapter 5 of this Handbook, the maximum predicted resistance of a ductile connector is to be taken as the 95th percentile of the ultimate resistance obtained statistically, if the strength distribution is known, or via testing from reversible cyclic loading.

13.10.1 CLT Floor-Panel-to-Beam Connection

Design of the connection between a CLT floor panel and a glulam beam; refer to drawing S200 in Annex A for an example of this connection.

Connection to be **capacity-protected**, i.e. non-dissipative, with design force at R_dR_o=1.3.

Per metre: N_f = (4.8 x 2.0 x 1.5)/1.3 = 11.1 kN

Factored load per beam = 11.1/2 = 5.6 kN (double beam)

Considering 8 Φ x 330 self-tapping screws at 1000 mm o/c at 45° in alternating directions:

Note: screws must meet minimum penetration length of $5d_F$

G = 0.42 SPF – EI CLT (Governs)

G = 0.49 D.Fir-L - GL

According to the supplier's fastener resistance table:

$N_r'_{45°}$ = 5912 N

Factored lateral strength resistance:

$N_r = N_r'\_n_F\, n_R\, J'\, K'$

$J' = J\, J_E\, J_G\, J_{PL} = 0.9 \times 1 \times 1 \times 1 = 0.9$

$K' = K_D\, K_{SF}\, K_T = 1.15 \times 1 \times 1 = 1.15$

K_D = 1.15 for short load duration

$N_r = 5912 \times 1^{0.9} \times 0.9 \times 1.15 = 6.1$ kN > 5.6 kN (92%)

Therefore, 8 Φ x 330 self-tapping screws @ 1000 mm o/c at 45° in alternating directions can be used.

13.10.2 Glulam-Beam-to-Column and Column-to-Column Connection

Refer to drawing S202 of Annex A for an example of a double glulam-beam-to-column connection, along with a column-to-column connection.

13.10.3 Glulam-Column-to-Concrete Connection

Design of connection between concrete podium and glulam column; refer to drawing S201 of Annex A for example of this connection type.

Checking HSS: 254 x 254 x 13, 180 LG

C_r = 3350 kN (from S016 Green Table) > C_f = 2624.6 kN

Checking bearing plate (top and bottom):

$$B_r = \frac{0.85\, \Phi_c\, f'_c}{A_b}$$

Letting $C_f = B_r$

$$A_b = \frac{C_f}{0.85\, \Phi_c\, f'_c}$$

$$q_f = \frac{C_f}{A_b} = \frac{C_f}{l_p\, w_p}$$

Considering taking the plate cantilever along the length:

$$M_f = \frac{(q_f\, w_p)\, m^2}{2}$$

$$M_r = \Phi_s Z F_y = \Phi_s \left(\frac{w_p\, t_p^{\,2}}{4}\right) F_y$$

Letting $M_f = M_r$

$$\frac{(q_f\, w_p)\, m^2}{2} = \Phi_S \left(\frac{w_p\, t_p^{\,2}}{4}\right) F_y$$

$$t_p = \sqrt{\frac{4\frac{(q_f x\, m^2)}{2}}{\Phi_S F_y}} = \sqrt{\frac{2\, q_f m^2}{\Phi_S F_y}} = \sqrt{\frac{2\, C_g m^2}{l_p w_p\, \Phi_S\, F_y}}$$

Similarly, taking the plate cantilever along the width:

$$t_p = \sqrt{\frac{2\, C_f\, n^2}{l_p w_p\, \Phi_S\, F_y}}$$

Therefore:

$$t_p = \max\left(\sqrt{\frac{2\, C_f m^2}{l_p w_p\, \Phi_S\, F_y}}\, ,\, \sqrt{\frac{2\, C_f n^2}{l_p w_p\, \Phi_S\, F_y}}\right)$$

If l_p = 355 mm

w_p = 332 mm

Φ_S = 0.9

F_y = 300 MPa

m = (355 – 254)/2 = 50.5 mm

n = (332 – 254)/2 = 39 mm

$$t_p = \max\left(\sqrt{\frac{2 \times 2624.6 \times 50.5^2}{355 \times 382 \times 0.9 \times 300 / 10^3}}, \sqrt{\frac{2 \times 2624.6 \times 39^2}{355 \times 382 \times 0.9 \times 300 / 10^3}}\right) = \max(20.5\ ,\ 15.8) = 20.5 \text{ mm} \rightarrow 25.4 \text{ mm}$$

Verifying deflection of plate: t_p = 25.4 mm ≥ min (m/5, n/5) ≥ 10.1 mm

Therefore, a 355 mm x 332 mm x 25.4 mm baseplate can be used.

13.10.4 Shear Wall Hold-Down and Shear Connectors

Hold-downs may be discrete or continuous. Discrete hold-downs are to remain moderately ductile while continuous rod type hold-down systems are to remain linearly elastic. Moderately ductile connections have a ductility ratio of 3.0 or more as determined by testing, or are connections using steel brackets or steel side plates that fail in fastener yielding modes (d), (e), or (g), per CSA O86 for nails or screws driven into the face of the CLT and loaded parallel- or perpendicular-to-grain (refer to Chapters 4 and 5 of this Handbook).

Discrete Hold-Downs (moderately ductile)

Design forces for discrete hold-downs shall be multiplied by a 1.20 over-strength factor so that their yield resistance is greater than the forces developed in them when the vertical CLT segment connections reach their yielding resistance.

Refer to Chapters 4 and 5 of this Handbook for further information on yielding versus capacity-protected connections in a structural system and when utilising 95% of connection resistance.

Maximum net uplift force is 599 kN (see Table 14 above and elevation on S400) from overturning at the base of the shear wall. Connection to be moderately ductile.

The maximum capacity of a single 6.4-mm diameter lag screw in single shear with a mild steel side plate in grade E1 CLT is 2.06 kN. Multiplying by K’ = 1.15 for short term loading = 2.06 x 1.15 = 2.37 kN/lag screw.

Therefore, (599 kN x 1.20)/ 2.37 = 304 screws are required. Hold-down at each end and each side of the shear wall can be provided with 152 6.4-mm diameter lag screws with a minimum penetration into the CLT of 71 mm. Design of the concrete anchors and steel plate are not shown in this example.

Connector displacement is not considered in this example.

Deformation in the steel plate is not considered in this example.

Higher up the wall, at elevation = 15.6 m, the uplift force is 350 – 238 = 112 kN.

T_f = 112 kN x 1.2 = 134 kN

The maximum capacity of a single 6.4-mm diameter lag screw in single shear with a mild steel side plate in grade E1 CLT is 2.06 kN. Multiplying by K’ = 1.15 for short term loading = 2.06 x 1.15 = 2.37 kN/lag screw.

Therefore, 134 / 2.37 = 56 6.4-mm diameter lag screws are required. A vertical tension tie plate is provided at each end and on each side of the shear wall with 14 6.4-mm diameter lag screws on each side of the joint, with a minimum penetration into the CLT of 71 mm. Design of the steel hold-down is not shown in the example.

Continuous Rod Hold-Down (linearly elastic)

For continuous rod type hold-downs, the factored design net tension force is 599 kN (from the combination of earthquake plus dead loads); this is multiplied by R_dR_o=3 and then divided by 1.3 to give 1382 kN in the steel rod at R_dR_o=1.3. All floors should be tied to increase the efficiency of the system and help limit deflections (drift).

Using high strength steel (F_y = 896 MPa = 130 ksi), a rod with a diameter of 57 mm (2.25") is required at the lowest wood storey, with a resistance of 1583 kN > 1382 kN in tension (calculated from the lower of 13.2.(a)(i) and (iii) and 13.12.1.3 of CSA S16-14). The rod diameter can be reduced going up the wall between restraints at each floor, relative to the accumulated and between-floor tension forces.

Compression is resisted by the CLT shear wall panels in direct bearing and must also be checked. Shear wall lateral deflection is to be checked incorporating rod elongation displacement.

Rod elongation displacement is not considered in this example.

Elongation of a continuous rod hold-down connected to the concrete podium slab must also be considered. Calculation not included here.

Where angle brackets are installed along the base of the CLT shear wall to resist shear (shear keys may also be used to resist shear), they may be used in combination with hold-downs to resist overturning. Experimental tests have shown that these brackets may contribute significantly in the vertical (uplift) direction as well. Therefore, when assuming panel rocking and considering that the angle brackets resist both shear and uplift forces, consideration of the shear-uplift interaction may be required. Refer to Clause 11.9.3 in the Wood Design Manual (CWC, 2017) for a suggested interaction equation. For this design example, it is assumed that there is no interaction.

The designer must ensure that screws fail in yielding modes (d), (e) or (g) per CSA O86, to achieve the minimum moderate ductility ratio.

The deformation of the hold-down or rod design should be checked, since the hold-down deflection or rod elongation should be designed to allow for the wall segments to rock while the hold-down or rod remains in the elastic range. The example in this Handbook shows the first iteration, for illustrative purposes. Additional iterations may be necessary to achieve this design objective.

Shear Connection

The maximum capacity of a single 12-gauge screw in single shear with a mild steel side plate in grade E1 CLT is 2.02 kN (see screw selection tables in Wood Design Manual (CWC, 2017)). Multiplying by K' = 1.15 for short term loading = 2.02 x 1.15 = 2.32 kN/screw.

Base shear of the shear wall is 358 kN (see Table 14) at the top of the concrete.

V_f = 358 kN x 1.20 over-strength factor = 430 kN

Number of screws = 430 kN / 2.32 kN = 186 screws required.

Two angle bracket shear connectors should be provided on each side of the CLT wall, each with 48 #12 screws with a minimum penetration into the CLT of 71 mm. Design of the concrete anchors and angle brackets are not shown in this example.

From Table 14, the maximum shear force between the roof or floor and the shear wall is 79 kN. Connection to be capacity-protected per Clause 11.9.2.4 of CSA Standard O86-14 Update 2 (CSA, 2017).

V_f = (79 kN x 2.0 x 1.5) / 1.3 = 182 kN (29.9 kN/m) at R_dR_o = 1.3

The maximum capacity of a single 8-gauge screw in single shear with a mild steel side plate in grade E1 CLT is 1.25 kN (see screw selection tables in Wood Design Manual (CWC, 2017)). Multiplying by K' = 1.15 for short term loading = 1.25 x 1.15 = 1.44 kN/screw.

Number of screws = 182 / 1.44 = 126 screws required.

Two angle bracket shear connectors each with 64 #8 screws into wall and floor with minimum penetration into the CLT of 71 mm should be provided. Design of the steel angle bracket is not shown in the example.

Displacement of shear connectors is not considered in this example.

13.10.5 Floor-Panel-to Shear-Wall Panel below Connection

Connection is designed for shear force at 5^{th} floor = 251 kN (see Table 14). Steel angle brackets at the bottom of the CLT floor panels are provided to transfer diaphragm shear plus accumulated wall shear. Connection is non-dissipative and protected at R_dR_o design force.

V_f = (251 x 2.0 x 1.5)/1.3 = 579 kN

The maximum capacity of a single 12-gauge screw in single shear with a mild steel side plate in grade E1 CLT is 2.02 kN (see screw selection tables in Wood Design Manual (CWC, 2017)). Multiplying by K' = 1.15 for short term loading = 2.02 x 1.15 = 2.32 kN/screw.

Therefore, 579 kN / 2.32 kN = 250 screws required.

Three angle bracket connectors are provided at the bottom of the floor panels with 250 / 3 = 83 #12 screws each, into the wall and the floor, with a minimum penetration into the CLT of 71 mm.

13.10.6 Floor-Panel-to-Shear-Wall Panel Above Connection

Connection is designed for shear force at 5^{th} floor = 251 kN (see Table 14). Steel angle brackets are provided at the top of the CLT floor panels to transfer accumulated wall shear. Connection is dissipative.

V_f = 251 kN

The maximum capacity of a single 12-gauge screw in single shear with a mild steel side plate in grade E1 CLT is 2.02 kN (see screw selection tables in Wood Design Manual (CWC, 2017)). Multiplying by K' = 1.15 for short term loading = 2.02 x 1.15 = 2.32 kN/screw.

Therefore, 251 kN / 2.32 kN = 108 screws required.

Three angle bracket connectors are provided at the top of the floor panels with 108 / 3 = 36 #12 screws each, into the wall and the floor, with a minimum penetration into the CLT of 71 mm.

13.10.7 Shear-Wall-Panel-to-Panel Vertical Connection

Two flat metal plate connectors are provided on each side of the shear wall to resist tension forces. Connection should be moderately ductile. Design is for forces listed in Table 14.

At first CLT storey, tension force = 771 kN – 614 kN = 157 kN

The maximum capacity of a single 10-gauge screw in single shear with a mild steel side plate in grade E1 CLT is 1.57 kN (see screw selection tables in Wood Design Manual (CWC, 2017)). Multiplying by K' = 1.15 for short term loading = 1.57 x 1.15 = 1.81 kN/screw.

Therefore, 157 kN / 1.81 kN = 87 screws required, or 87/ 4 = 22 screws on each side of the vertical joint.

Two connectors are provided on each side of the panel at the first CLT storey with 22 #10 screws on each side of the vertical joint, with minimum penetration into the CLT of 71 mm.

Displacement in panel-to-panel connectors is not considered in this example.

Deformation in the steel plate is not considered in this example.

13.10.8 Floor-Panel-to-Panel Connection

Design of the connection between floor panels parallel to the span to resist diaphragm forces; refer to drawing S200 in the Annex for an example of this connection. Connection is non-dissipative, and capacity-protected at R_dR_o=1.3 design force.

N_f = (4.8 kN x 2.0 x 1.5)/1.3 = 11.1 kN (per metre)

Considering ¼" Φ x 120 lag screws @ 150 mm o/c:

d_f = 6.4 mm

K_D = 1.15 for short load duration

K_{SF} = 1.0

K_T = 1.0

J_E = 1.0 for installed side grain

$K' = K_D K_{SF} K_T$ = 1.15 x 1 x 1 = 1.15

J_X = 0.9 for CLT

G = 0.42 – SPF CLT & Ply

t_1 = 38 mm

t_2 = 82 mm

$f_1 = 50\ G\ (1 - 0.01 d_F)\ J_X$ = 17.7 MPa (perpendicular to the grain)

f_2 = 17.7 MPa (perpendicular to the grain)

f_y = 310 MPa for lag screws meeting SAE J429 Grade 1

n_u, is the unit lateral yielding resistance and is taken as the minimum of the following:

a) $f_1\, d_F\, t_1$ = 4.3 kN

b) $f_1\, d_F\, t_2$ = 9.28 kN

c) *N/A*

d) $f_1\, d_F^{\,2}\ (\sqrt{\frac{1}{6} \times \frac{f_3}{(f_1+f_3)} \times \frac{f_y}{f_1}} + \frac{1}{5}\left(\frac{t_1}{d_F}\right))$ =1.76 kN

e) $f_1\, d_F^{\,2}\ (\sqrt{\frac{1}{6} \times \frac{f_3}{(f_1+f_3)} \times \frac{f_y}{f_1}} + \frac{1}{5}\left(\frac{t_2}{d_F}\right))$ =1.76 kN

f) $f_1\, d_F^{\,2} \times \frac{1}{5} \times \left(\frac{t_1}{d_F} + \frac{f_2}{f_1} \times \frac{t_2}{d_F}\right)$=2.75 kN

g) $f_1\, d_F^{\,2}\ (\sqrt{\frac{2}{3} \times \frac{f_3}{(f_1+f_3)} \times \frac{f_y}{f_1}}\ )$=2.72 kN

Using n_u = 1.76 kN,

$N_u = n_u \times K'$ = 2.02 kN

n_F = 1000/150 = 6.7 screws per metre

$n_S = 1$

$J_A = 1$

$N_r = \Phi\ N_n\ n_F\ n_s\ J_A\ J_E = 0.8 \times 2.02 \times 7.7 \times 1 \times 1 \times 1 = 12.4$ kN/m > $N_F = 11.1$ kN/m (92%)

Therefore, ¼" Φ x 120 lag screws @ 150 mm o/c can be used.

Panel-to-panel connector displacement is not considered in this example.

Floor-panel-to-beam connection should also be designed for diaphragm shear force in the perpendicular direction, with fasteners acting perpendicular to the grain in the CLT and parallel to the grain in the beam (not included in this example).

13.11 SYMBOLS

A	=	cross-sectional area of a member or the bearing area, mm^2
A_{eff}	=	effective cross-sectional area, mm^2
A_g	=	gross cross-sectional area, mm^2
a_D	=	along-wind peak acceleration, m/s^2
a_W	=	across-wind peak acceleration, m/s^2
ASTC	=	apparent sound transmission class
B_x	=	ratio at level x used to determine torsional sensitivity
B	=	maximum value of B_x
b	=	width of a member or lamination, mm
b_f	=	resultant member width after fire exposure, mm
C_a	=	accumulation factor for snow
C_b	=	basic roof snow load factor
C_B	=	slenderness ratio for bending members
C_c	=	slenderness ratio for compression members
C_e	=	wind exposure coefficient
C_f	=	compression force
C_g	=	wind gust effect coefficient
C_p	=	wind external pressure coefficient
C_s	=	roof slope factor
C_t	=	wind topographic coefficient
C_v	=	shear load coefficient
C_w	=	wind exposure factor
CCMC	=	Canadian Construction Materials Centre
CLT	=	cross-laminated timber

d	=	depth of a member or lamination, mm
d_f	=	resultant member depth after fire exposure, mm
D	=	width of building parallel to wind direction
D	=	dead load
E	=	modulus of elasticity, MPa
E	=	earthquake load
E_{05}	=	5th percentile of the modulus of elasticity, MPa
EI_{eff}	=	effective bending stiffness of CLT panels, Nmm^2
e	=	eccentricity, mm
ESFP	=	Equivalent Static Force Procedure
F_a	=	seismic site coefficient
F_c	=	modified compressive strength parallel to grain, MPa
F(PGA)	=	seismic site coefficient for PGA
F(PGV)	=	seismic site coefficient for PGV
F_j	=	static lateral load applied at n^{th} floor to produce static deformation, N
F_s	=	seismic site coefficient
F_s	=	modified strength in rolling shear of laminations in the transverse layers, MPa
F(T)	=	seismic site coefficient for spectral acceleration
F_t	=	portion of V to be concentrated at top of the structure
F_v	=	seismic site coefficient
F_x	=	lateral force applied to level x
f_b	=	specific bending strength, MPa
f_c	=	specified compression strength parallel to grain, MPa
f_{cp}	=	specified compression strength perpendicular to grain, MPa
f_n	=	fundamental natural frequency, Hz

f_s	=	specified strength in rolling shear of laminations in the transverse layers, MPa
f_t	=	specified tensile strength parallel to the grain, MPa
f_v	=	specified shear strength, MPa
g	=	acceleration of gravity, 9.81 m/s^2
G	=	shear modulus, MPa
GA_{eff}	=	effective in-plane (planar) shear rigidity of CLT panels, N
GL	=	glued-laminated timber
h_s	=	inter-storey height
H	=	height of the building
I	=	moment of inertia of a section, mm^4
I_{eff}	=	effective out-of-plane moment of inertia of CLT panels, mm^4
I_E	=	earthquake importance factor of the structure
I_S	=	snow importance factor of the structure
I_W	=	wind importance factor of the structure
J	=	base overturning reduction factor
J_g	=	group action factor
K	=	form factor (Kappa)
K’	=	load duration modification factor
K_C	=	slenderness factor for compression members
K_{creep}	=	creep adjustment factor
K_D	=	modification factor for duration of load
K_{fi}	=	modification factor for fire
K_H	=	system factor
K_L	=	modification factor for lateral stability
K_{rb}	=	adjustment factor for bending moment resistance of CLT panels

K_{Sb} = modification factor for service condition for bending

K_{Sc} = modification factor for service condition for compression parallel to the grain

K_{Scp} = modification factor for service condition for compression perpendicular to the grain

K_{SE} = modification factor for service condition for modulus of elasticity

K_{SF} = modification factor for service condition for connections

K_{St} = modification factor for service condition for tension parallel to the grain

K_{Stp} = modification factor for service condition for tension perpendicular to the grain

K_{Sv} = modification factor for service condition for longitudinal shear

K_T = modification factor for treatment

K_{Zbg} = modification factor for the size effect for flexure for glued-laminated timber

K_{Zc} = modification factor for the size effect for compression parallel to the grain

K_{Zcp} = modification factor for the length of bearing for compression perpendicular to the grain

K_{Zv} = modification factor for the size effect for shear

L = length of a component, mm

L = span length, m

L = live load

L_e = effective length, mm

L_u = laterally unsupported length of a component, mm

L_v = vibration-controlled span limit, mm

M_r = factored resistance of a member in flexure, kNm

M_v = factor to account for higher mode effect on base shear

N_u = unit lateral strength resistance, N

PGA = Peak Ground Acceleration expressed as a ratio to gravitational acceleration

PGA_{ref} = reference PGA for determining F(T), F(PGA) and F(PGV)

PGV	=	Peak Ground Velocity, m/s
P_r	=	compression resistance of member, N
p	=	specified external wind pressure
q	=	hourly mean reference wind pressure for the design return period, kPa
R_d	=	ductility-related force modification factor reflecting the capability of a structure to dissipate energy through cyclic inelastic behaviour
R_o	=	over-strength-related force modification factor accounting for the dependable portion of reserve strength in a structure
r_{eff}	=	effective radius of gyration, mm
S(T)	=	design spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period of T
S_a(T)	=	5% damped spectral response acceleration, expressed as ratio to gravitational acceleration, for a period of T
SFRS	=	Seismic Force Resisting System(s) is that part of the structural system that has been considered in the design to provide the required resistance to the earthquake forces and effects
S	=	section modulus, mm
S_{eff}	=	effective out-of-plane section modulus of CLT panels, mm^3
S_f	=	section modulus based on reduced cross-section, mm^3
S_r	=	associated rain load, kPa
S_s	=	ground snow load with 1/50-year probability of exceedance, kPa
SLS	=	Serviceability Limit State
STC	=	Sound Transmission Class
T_a	=	fundamental lateral period of vibration of the building or structure, in s, in the direction under consideration
t	=	fire exposure duration, min
ULS	=	Ultimate Limit State
V	=	lateral earthquake design force at the base of the structure, kN

V_d = lateral earthquake design force at the base of the structure, kN

V_f = factored shear load on a member, kN

V_H = mean wind speed at the top of the structure, m/s

V_r = factored shear resistance, N

W = seismic weight of structure, N

W = width of building perpendicular to wind direction

W = wind load

W_f = factored total load, N

W_j = dead weight at each floor for vibration check, N

W_r = total factored shear resistance, N

$X_{c,n}$ = char depth for notional charring, mm

$X_{c,o}$ = char depth for one-dimensional charring, mm

$\bar{y}$ = distance to neutral axis, mm

Z = total beam volume, mm^3

B_d = fraction of critical damping in along-wind direction

B_n = notional charring rate, mm/min

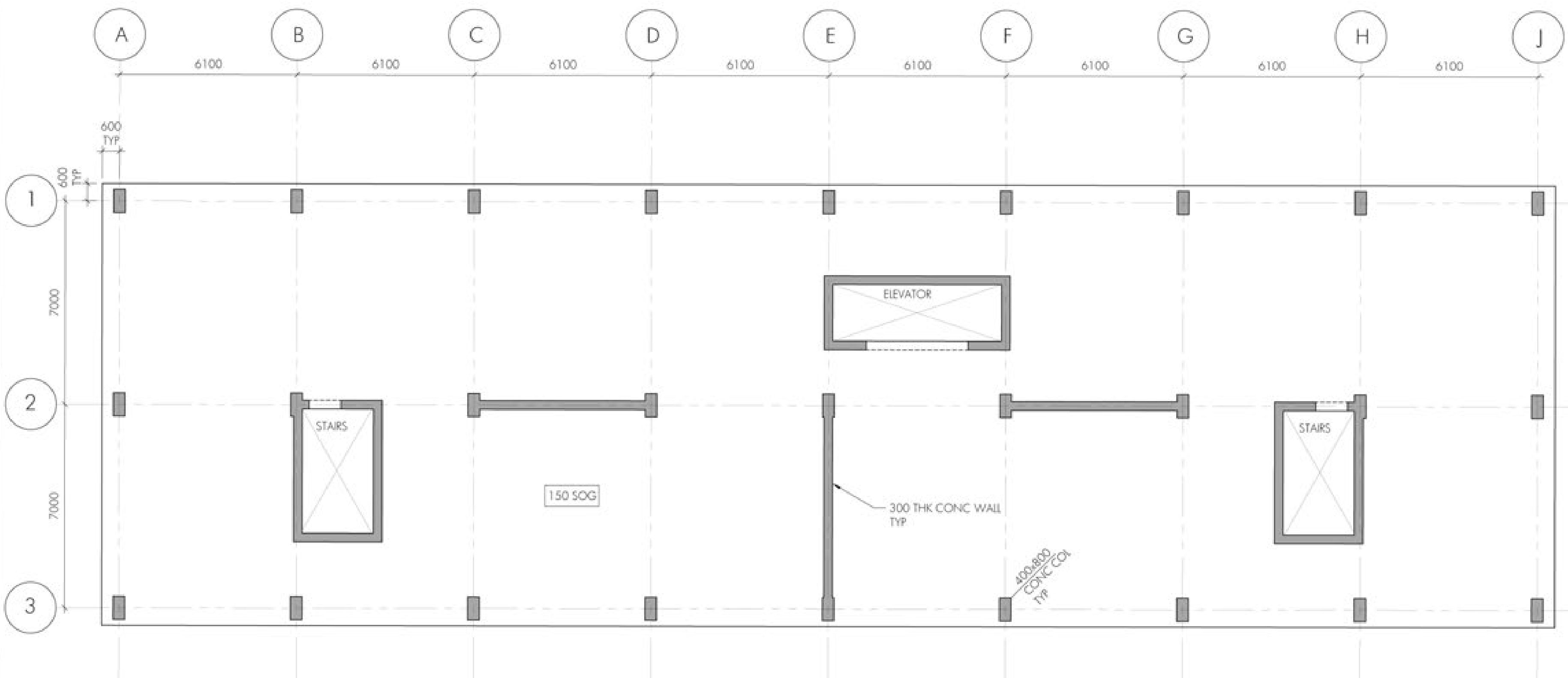
B_o = one-dimensional charring rate, mm/min

B_w = fraction of critical damping in across-wind direction

ρ_B = average density of building, kg/m^3

Φ = resistance factor

ANNEX A



1 / S100 GROUND FLOOR PLAN
1:150

PRELIMINARY

moses structural engineers
366 Adelaide St. West, Suite LL04
Toronto, Ontario, M5V 1R7
Tel: 416 255 3337
info@mosesstructures.com

Project:
TALL WOOD EXAMPLE
CANADA

Project No:
18.011

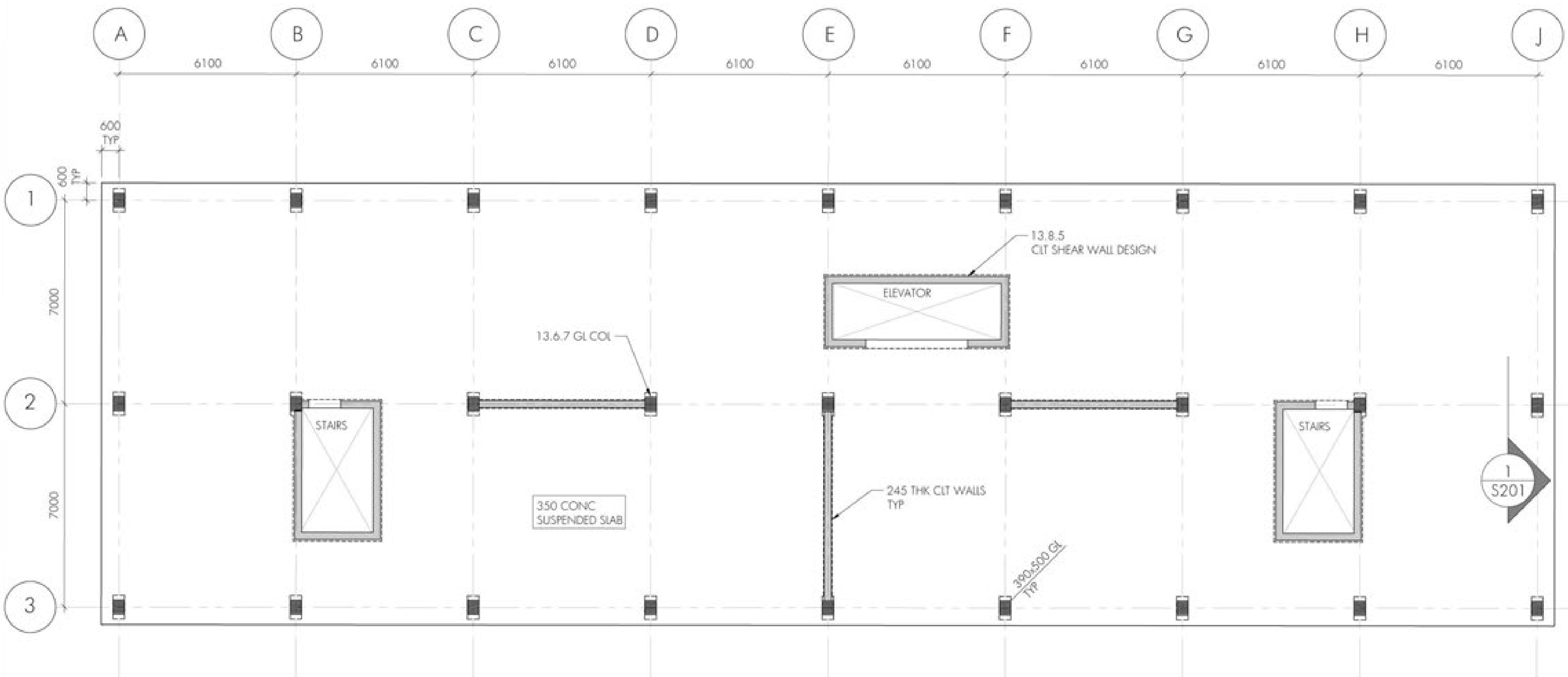
Date:	FEBRUARY 02, 2018
Scale:	AS NOTED
Drawn By:	KM
Checked By:	DM

REVISIONS AND DISTRIBUTION LOG

Rev	Date	Note

Drawing Title:
GROUND FLOOR PLAN

Drawing No:
S100



1 S101 SECOND FLOOR PLAN 1:150

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moses structural engineers
366 Adelaide St. West, Suite LL04
Toronto, Ontario, M5V 1R7
Tel: 416 255 3337
info@mosesstructures.com

Project:
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CANADA

Project No:
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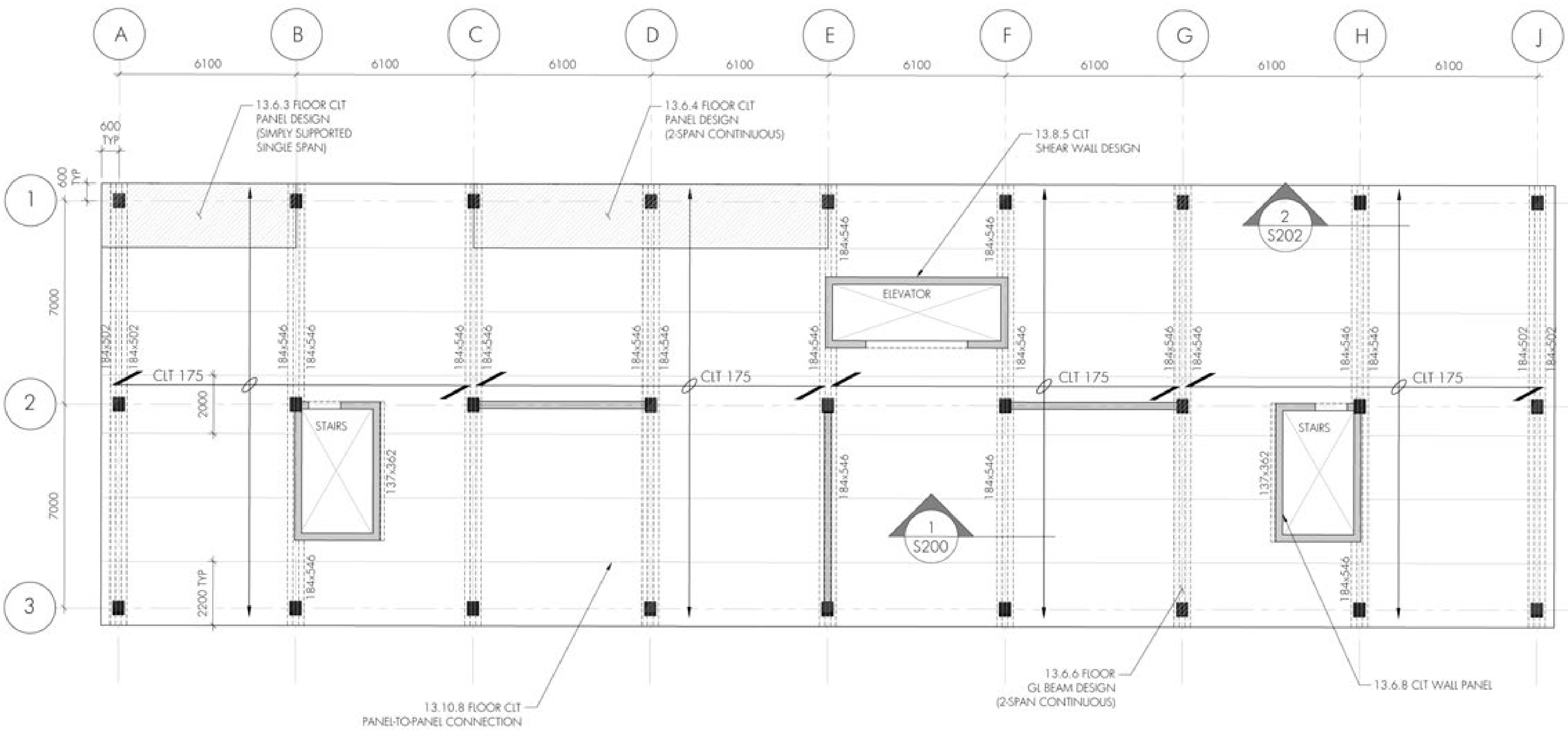
Drawing Title:
SECOND FLOOR PLAN

Drawing No:
S101

TYPICAL COLUMN & WALL SCHEDULE		
FLOOR	GL COLUMNS	CLT WALLS
3	390x450	245
4	390x450	245
5	390x350	175
6	390x350	175
7	390x350	175
8	390x350	175



1 / S102 3rd TO 8th FLOOR PLAN
1:150

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moses structural
engineers
366 Adelaide St. West, Suite LL04
Toronto, Ontario, M5V 1R7
Tel: 416 255 3337
info@mosesstructures.com

Project:
TALL WOOD EXAMPLE
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Project No:
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Date: FEBRUARY 02, 2018
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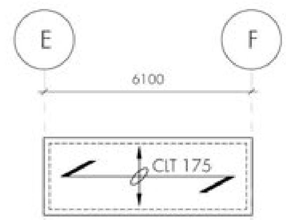
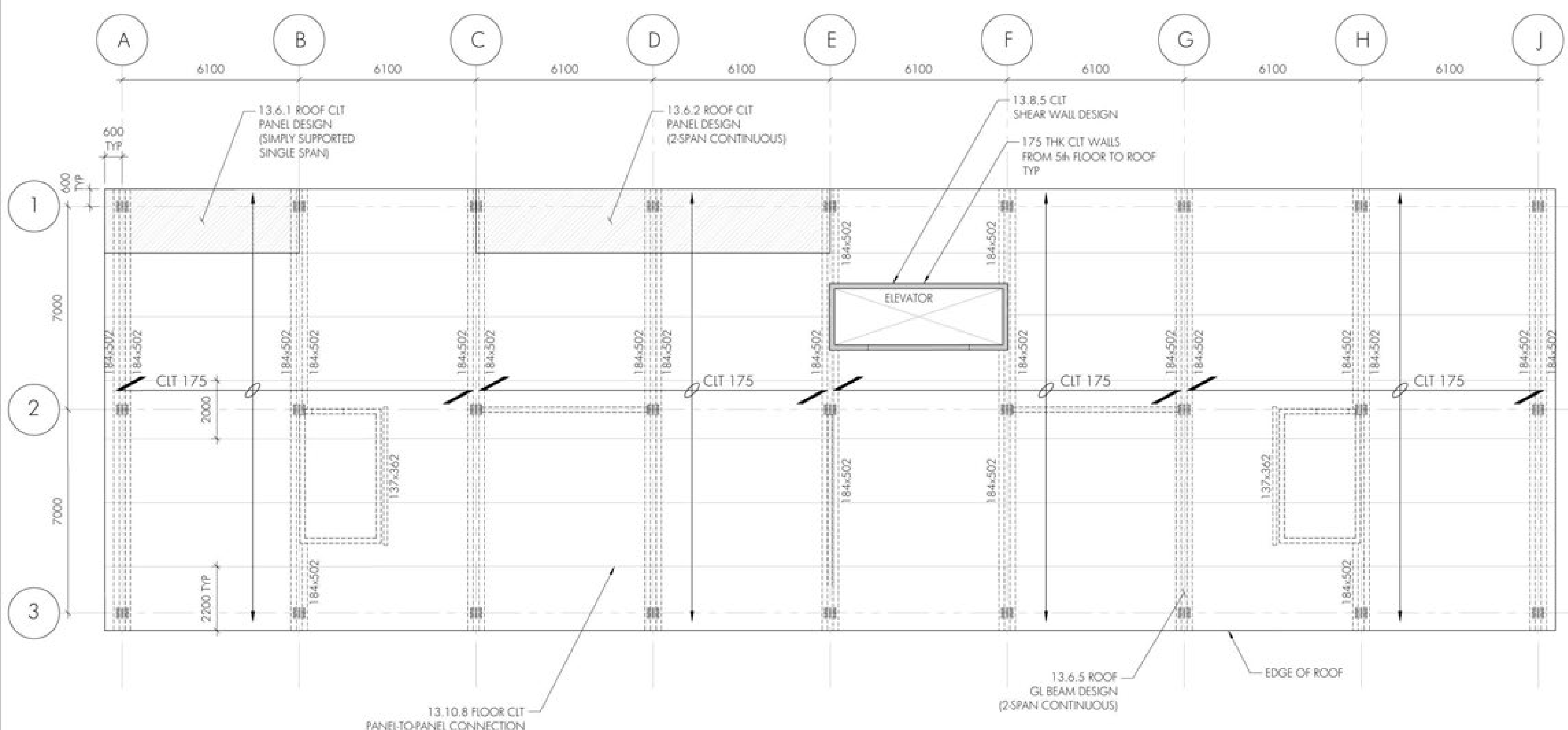
Drawing Title:
3rd TO 8th FLOOR PLANS

Drawing No:
S102



2 S103 HIGH ROOF PLAN 1:150



1 S103 ROOF PLAN 1:150

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moses structural engineers
366 Adelaide St. West, Suite LL04
Toronto, Ontario, M5V 1R7
Tel: 416 255 3337
info@mosesstructures.com

Project:
TALL WOOD EXAMPLE
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Project No:
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Date:	FEBRUARY 02, 2018
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Checked By:	DM

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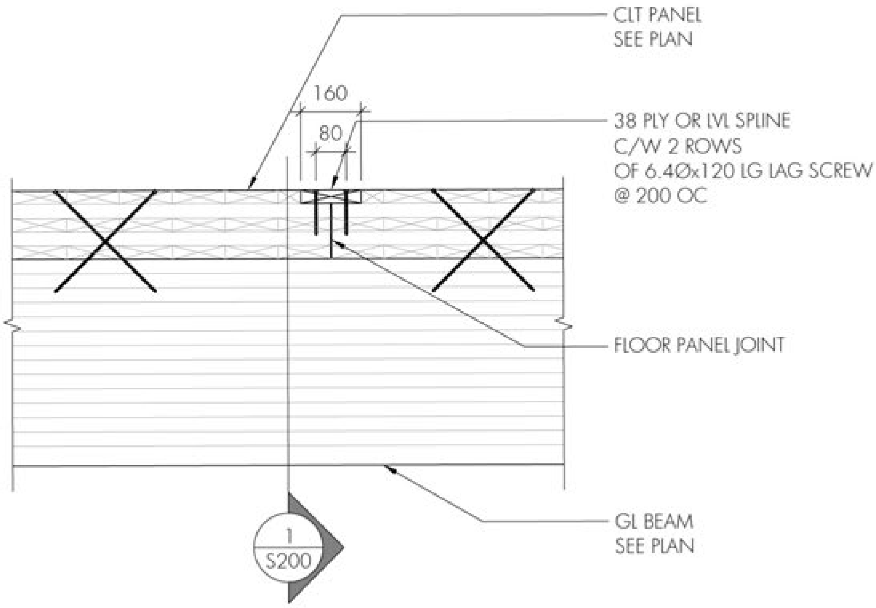
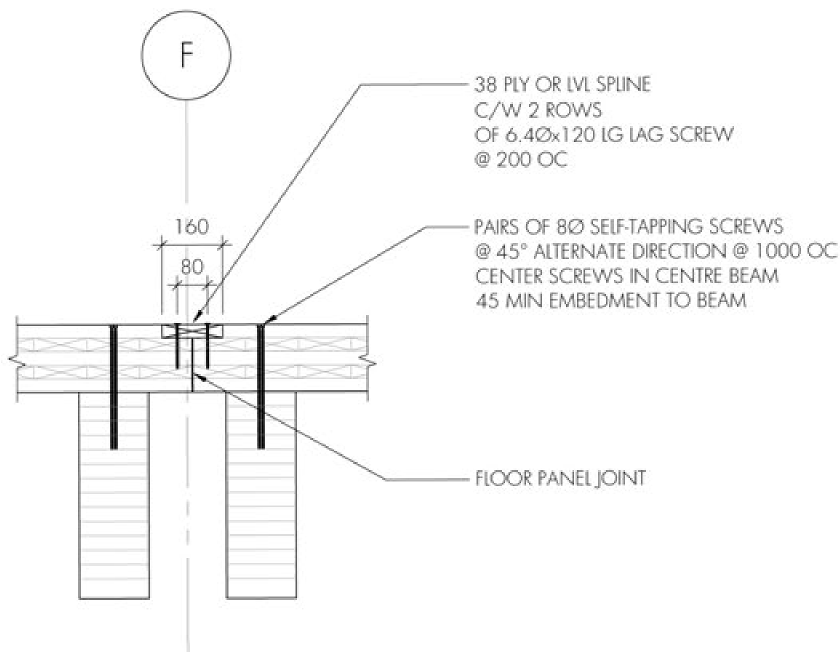
Drawing Title:
ROOF PLAN

Drawing No:
S103



2 / S200 SECTION
1:20



1 / S200 SECTION
1:20

Project:
TALL WOOD EXAMPLE
CANADA

Project No:
18.011

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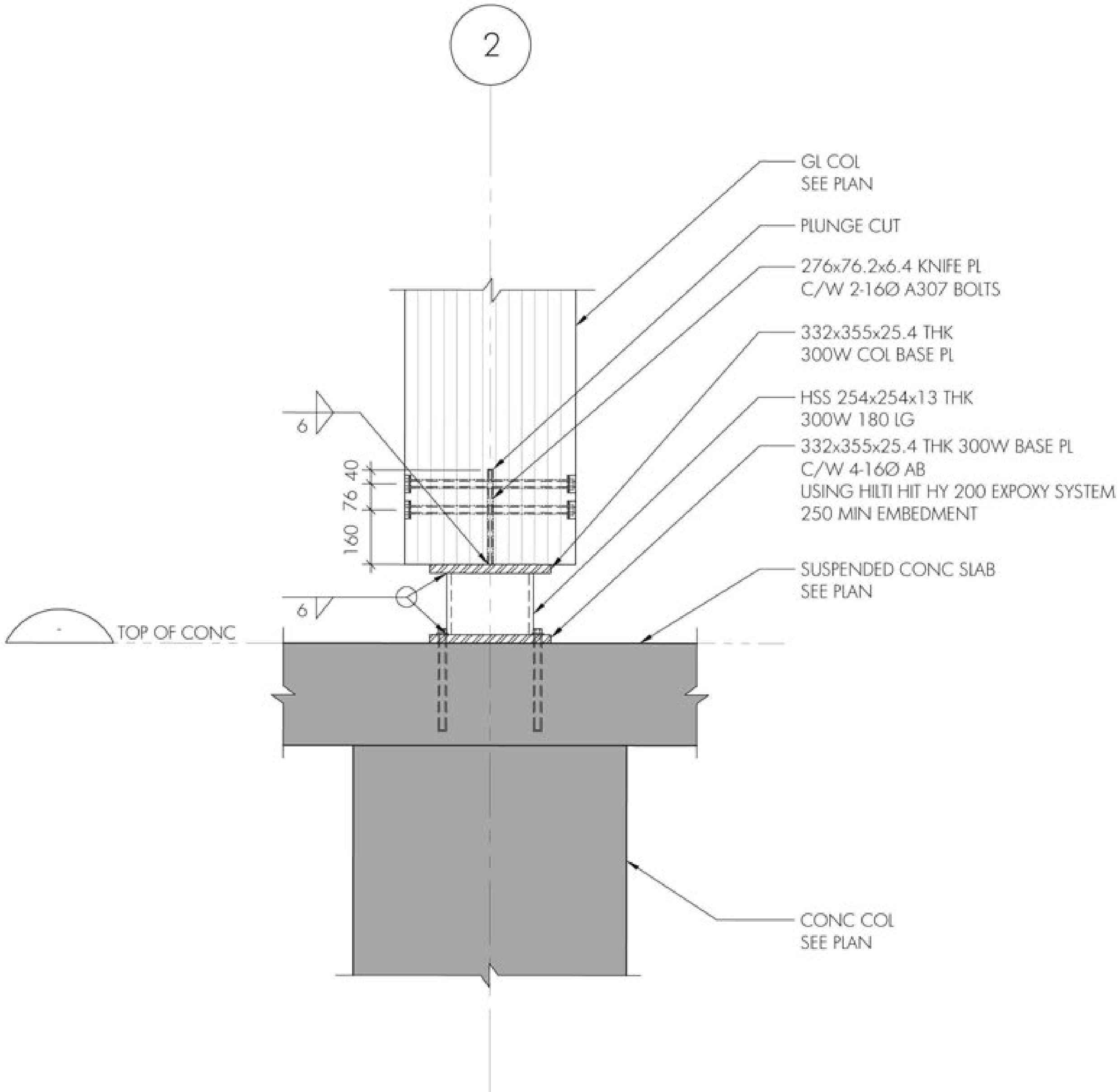
Drawing Title:
CONNECTION DETAIL - CLT FLOOR SPLICE

Drawing No:
S200

Date: FEBRUARY 27, 2019 Scale: AS NOTED Drawn By: MH Checked By: MP/DM Revision: - -



1 / S201 SECTION
1:20

Project:
TALL WOOD EXAMPLE
CANADA

Project No:
18.011

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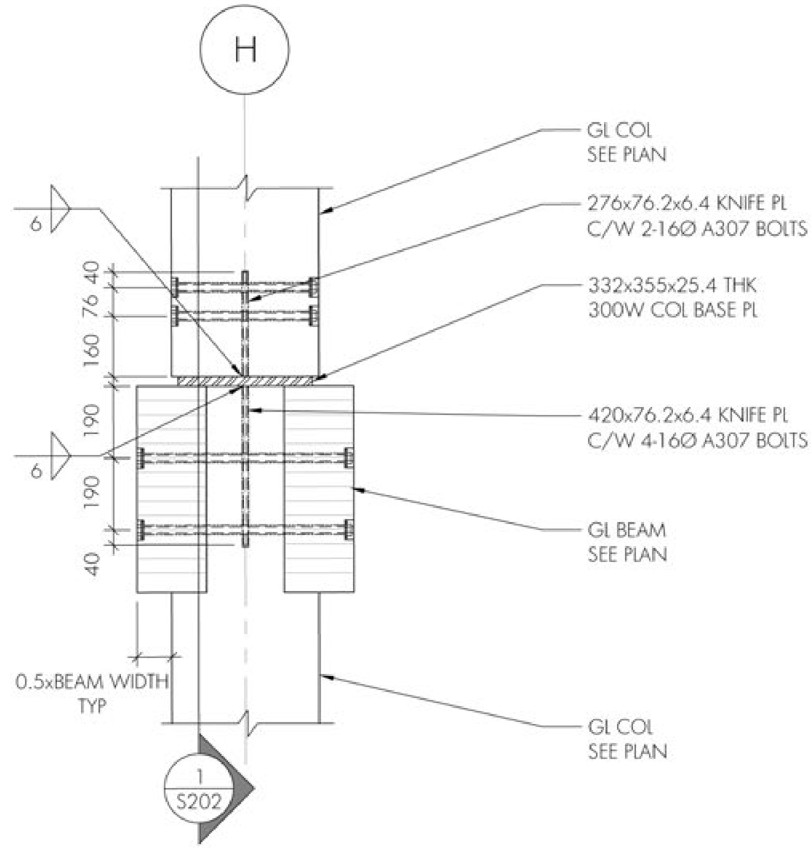
Drawing Title:
CONCRETE SLAB-TO-GLULAM COLUMN BASE CONNECTION SECTION

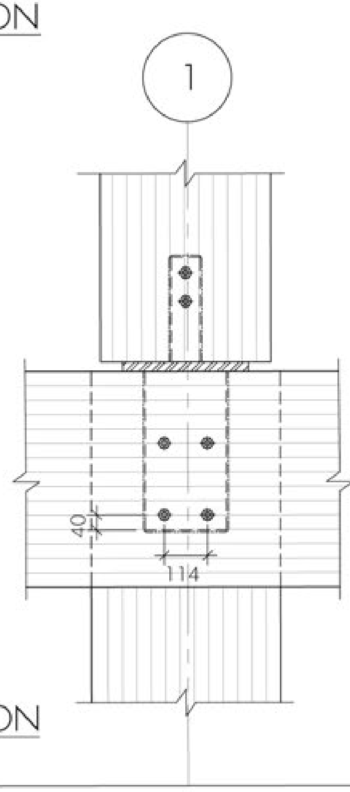
Drawing No:
S201

Date: FEBRUARY 27, 2019 Scale: AS NOTED Drawn By: MH Checked By: MP/DM Revision: - -



2
S202
SECTION
1:20

NOTE:
CLT FLOOR NOT SHOWN FOR CLARITY



1
S202
SECTION
1:20

Project:
TALL WOOD EXAMPLE
CANADA

Project No:
18.011

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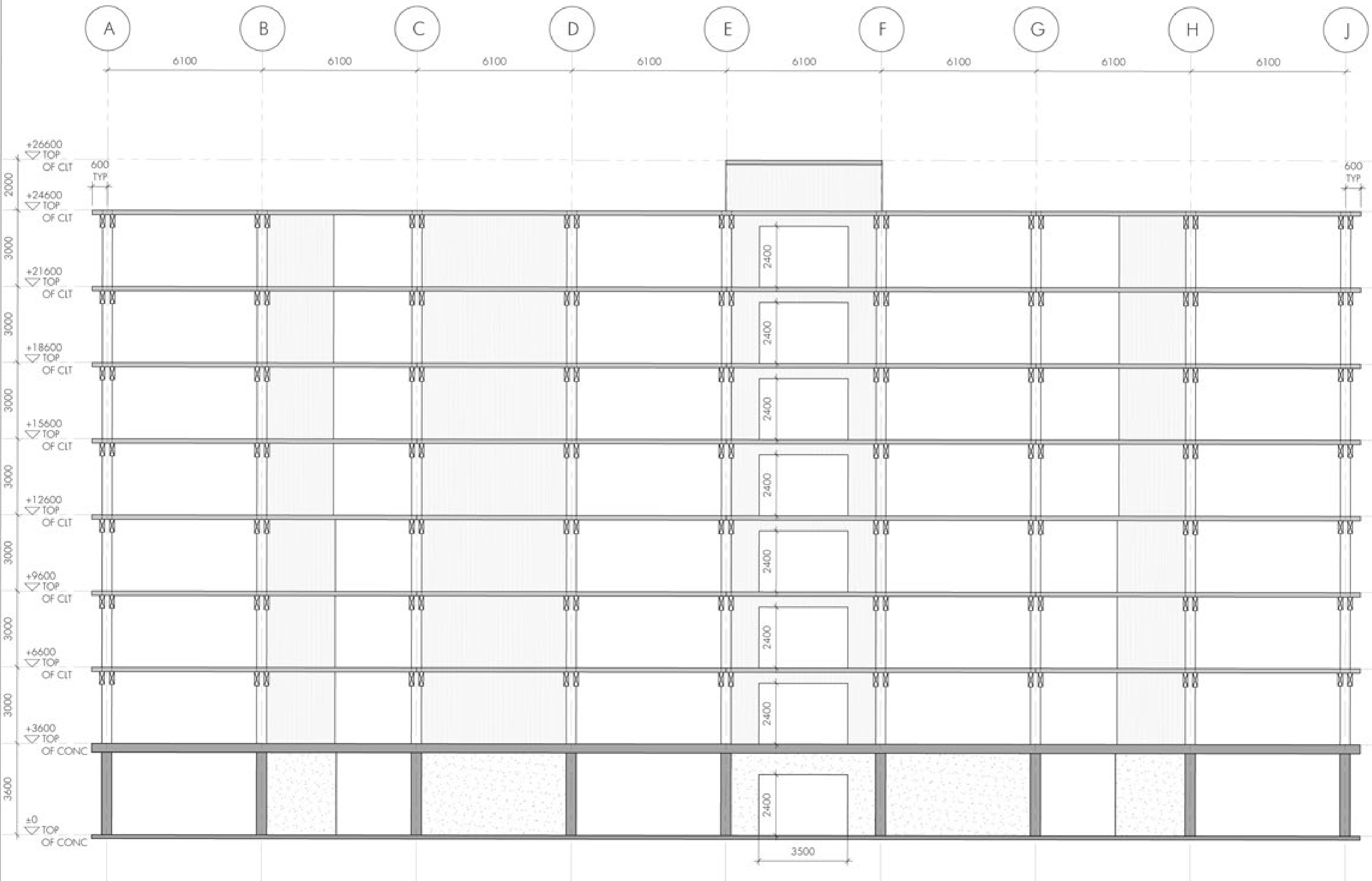
Drawing Title:
BEAM-COLUMN CONNECTIONS

Drawing No:
S202

Date: FEBRUARY 27, 2019 Scale: AS NOTED Drawn By: MH Checked By: MP/DM Revision: - -



1 S300 SOUTH ELEVATION 1:150

PRELIMINARY

moses structural engineers
366 Adelaide St. West, Suite LL04
Toronto, Ontario, M5V 1R7
Tel: 416 255 3337
info@mosesstructures.com

Project:

TALL WOOD EXAMPLE

CANADA

Project No:

18.011

Date:	FEBRUARY 02, 2018
Scale:	AS NOTED
Drawn By:	KM
Checked By:	DM

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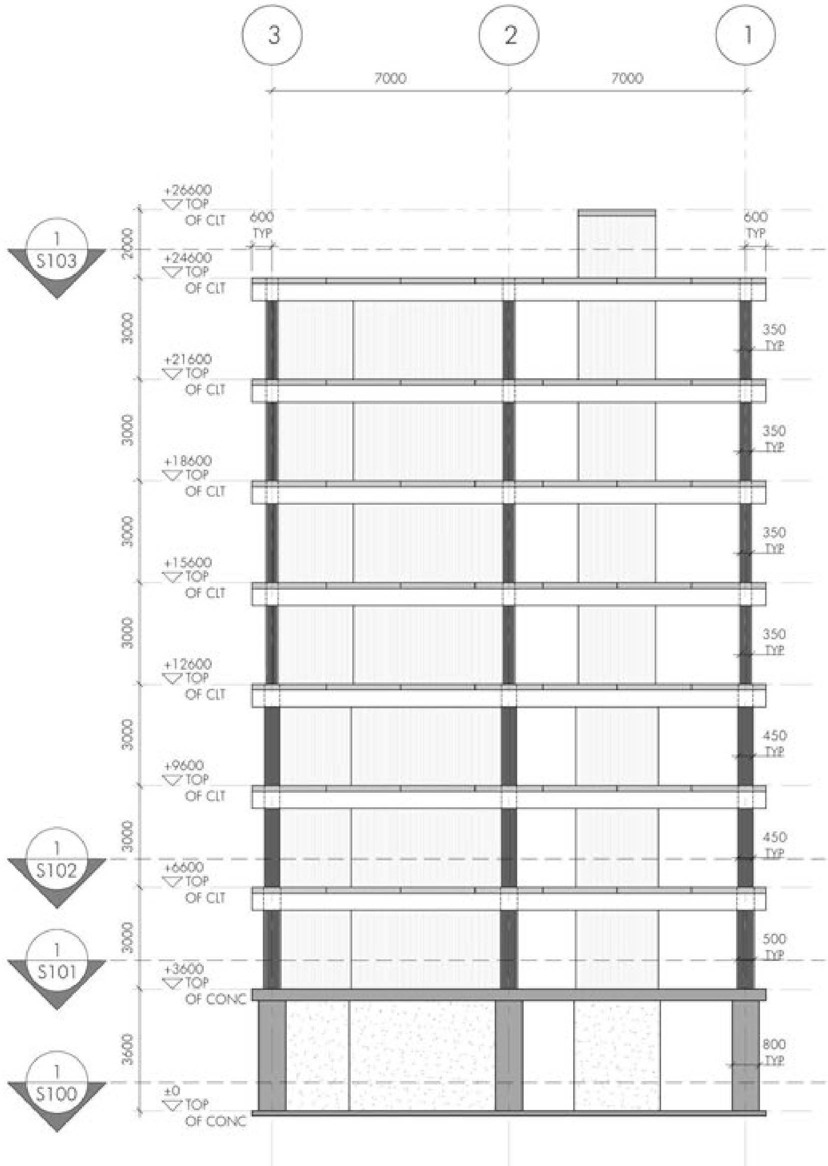
Rev	Date	Note

Drawing Title:

SOUTH ELEVATION

Drawing No:

S300



1 S301 EAST ELEVATION
1:150

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moses structural engineers
366 Adelaide St. West, Suite LL04
Toronto, Ontario, M5V 1R7
Tel: 416 255 3337
info@mosesstructures.com

Project:

TALL WOOD EXAMPLE
CANADA

Project No:
18.011

Date: FEBRUARY 02, 2018
Scale: AS NOTED
Drawn By: KM
Checked By: DM

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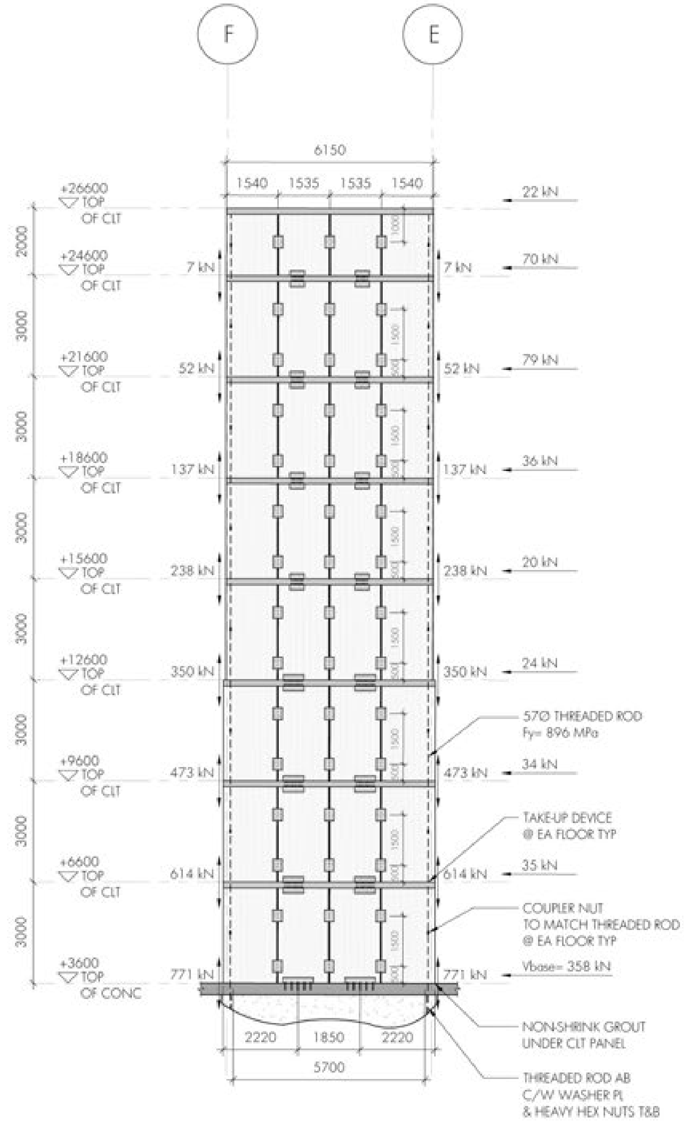
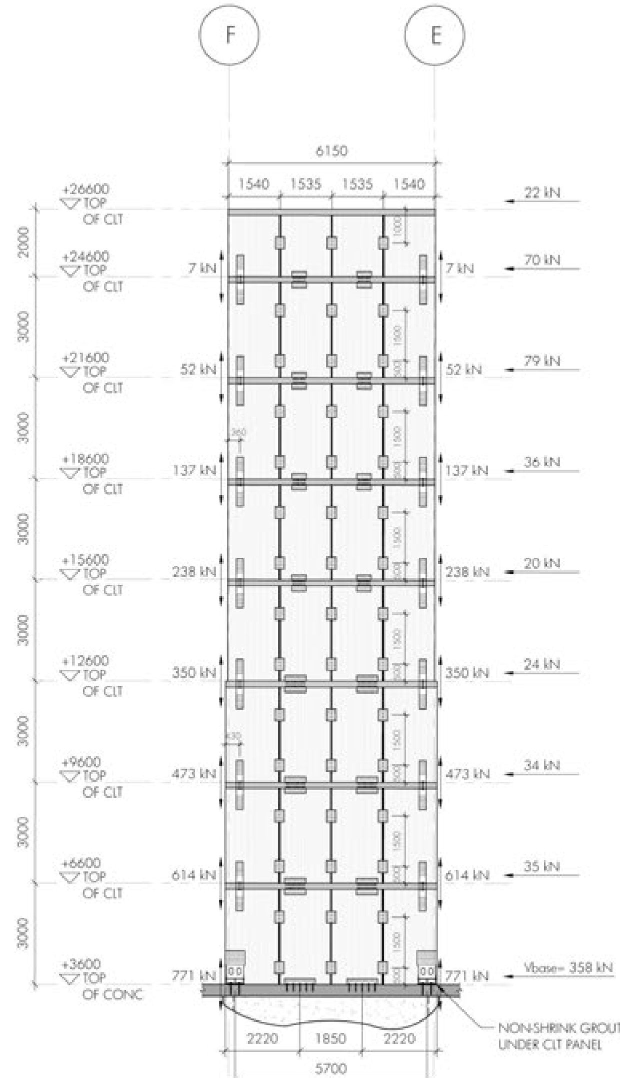
Drawing Title:
EAST ELEVATION

Drawing No:
S301



OPTION 2
W/ CONTINUOUS HOLDOWNS



OPTION 1
W/ DISCRETE HOLDOWNS

1 / S400 CLT SHEAR WALL ELEVATION @ ELEVATOR
1:150

PRELIMINARY

moses structural engineers
366 Adelaide St. West, Suite LL04
Toronto, Ontario, M5V 1R7
Tel: 416 255 3337
info@mosesstructures.com

Project:
TALL WOOD EXAMPLE
CANADA

Project No:
18.011

Date: FEBRUARY 02, 2018
Scale: AS NOTED
Drawn By: KM
Checked By: DM

REVISIONS AND DISTRIBUTION LOG

Rev	Date	Note

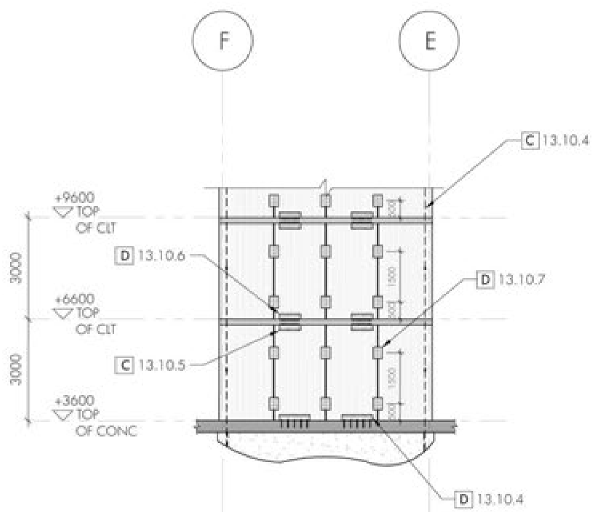
Drawing Title:
CLT SHEAR WALL ELEVATION AT ELEVATOR

Drawing No:
S400

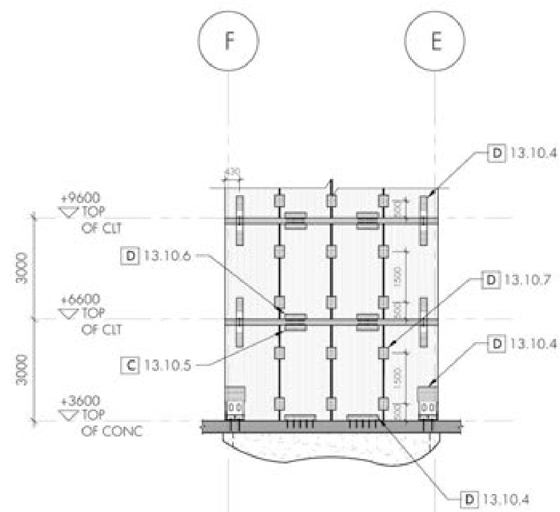
KEY: CONNECTION CLASSIFICATION

C	OVER STRENGTH / CAPACITY PROTECTED (NON-DISSIPATIVE)
D	DUCTILE (DISSIPATIVE)

NOTE: SECTION NUMBERS ARE NOTED CORRESPONDING TO THE WORKED SOLUTION FOR EACH CONNECTOR.



OPTION 2
W/ CONTINUOUS HOLDOWNS



OPTION 1
W/ DISCRETE HOLDOWNS

1/S401 CONNECTION CLASSIFICATION
1:150

PRELIMINARY

moses structural engineers
366 Adelaide St. West, Suite LL04
Toronto, Ontario, M5V 1R7
Tel: 416 255 3337
info@mosesstructures.com

Project:
TALL WOOD EXAMPLE
CANADA

Project No: 18.011

Date: FEBRUARY 02, 2018
Scale: AS NOTED
Drawn By: KM
Checked By: DM

REVISIONS AND DISTRIBUTION LOG

Rev	Date	Note

Drawing Title:
CONNECTION CLASSIFICATION

Drawing No:
S401

FUNDING PARTNERS

Forestry Innovation Investment

Ontario

Natural Resources Canada / Ressources naturelles Canada

Canada

Québec

Alberta Government

NORDIC STRUCTURES

STRUCTURLAM MASS TIMBER CORPORATION
Intelligence In Wood

CRIBE centre for research & innovation in the bio-economy

COLLABORATORS

Canadian Wood Council / Conseil canadien du bois

APA

FPInnovations