

Design of timber-concrete composite floors

by

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Abstract

Timber-concrete composite (TCC) floors use composite action to achieve higher strength and longer spans compared to timber structures. This report investigates the design and performance of TCC floors in accordance with published design guides and references. The design methods of the connections between timber and concrete are discussed. The report also compares the design of a typical bay using TCC, concrete, and timber structures. A comparative analysis is performed on the flexural strength, shear strength, deflection, total thickness, and material cost of the three. Results show that TCC floors can achieve similar strengths to the other floors while having a smaller thickness.

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Chapter 1: Introduction

Section 1A: Mass Timber Buildings

The American Wood Council defines mass timber buildings as buildings “constructed with large pre-manufactured, multilayered, solid wood panels resulting in solid timber floors and walls” (American Wood Council n.d.). Common mass timber products include cross laminated timber (CLT), glued laminated timber (glulam), laminated veneer lumber (LVL), and laminated strand lumber (LSL). CLT is made up of three to nine alternating layers of dimension lumber that are glued together to make structural panels. The alternating layers minimize the anisotropic behavior of the wood by utilizing the properties of both directions in one structural member. Buildings use the CLT panels as floors, load bearing walls, and shear walls. Glulam and LVL members are made of multiple layers of lumber stacked then glued under immense pressure. The difference is that glulam members use dimension lumber while LVL uses very thin laminas. Both types of mass timber products are made into beams and columns.

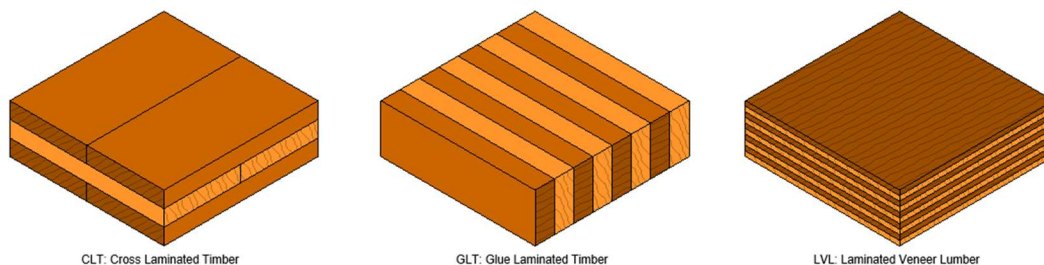


Figure 1 Types of Mass Timber

The usage of mass timber products has risen due to its sustainability and higher strength compared to traditional dimension lumber framed buildings. CLT production was found to have a reduced carbon footprint when compared to steel and concrete production. A study done by Oh et al (2023) found that CLT production produced 75% less CO₂ than reinforced concrete. The same study also stated that timber-concrete composite structures produced 50% to 70% less CO₂ than reinforced concrete structures of similar size and span.

The design of mass timber structures uses codes such as the International Building Code (IBC) and the National Design Specification for Wood Construction (NDS). The IBC gives requirements regarding occupancy, fire safety, and building height. The NDS details the design of timber structures regarding strength and connections as well as providing design values for different wood products. Design values may also be provided by the manufacturer, which are usually found through testing of the product. For CLT specifically, there is “The CLT Handbook” which is a collection of design requirements and equations for CLT that were found through testing. In Canada, CSA-O86 is used for the structural design of mass timber products. The codes mentioned will be used in this report.

Section 1B: Concrete Buildings

Concrete is one of the oldest and most used building materials in the world. Its composition of three ingredients has made it available throughout history and the world. There are many ways that concrete can be described: reinforced, lightweight, normal weight, prestressed, precast, and cast in place. These characteristics are not mutually exclusive, allowing concrete to be flexible in what it is used for. Reinforced concrete can have different types of reinforcement: steel rebar, wire mesh, and fiber. A typical concrete building uses steel rebar for

larger members like beams and columns, while wire mesh is used for thin members like slabs. These members can be cast on site or precast and assembled on site. Precast members are made in a more stable environment and are faster to install but may cost more than cast in place concrete. Concrete is also a common material used in composite structures.

A concrete building's advantages are its durability and strength. Concrete does not get affected by temperature fluctuations as much as steel and timber, it works well as an insulator and does not rust or rot. On the other hand, concrete is susceptible to freezing and thawing, members can get very heavy, and construction can take longer than other materials due to the curing time. Concrete is often used with other materials. Examples include concrete floors and shear walls in steel or timber buildings.

Section 1C: Basics of Composite Structures

A composite structure is one that utilizes the properties of two or more materials to improve the overall performance of the structure. Composite action is achieved through binding the materials together, so they act as one. As long as sufficient load transfer occurs between the materials, composite action can be achieved. One of the most common types of composite structures is steel and concrete composite beams. By pouring concrete on top of steel beam fitted with shear studs, composite action is achieved when loads are transferred between the two materials via the shear studs, where the composite action percentage is “represented by ratio of the total shear connection strength divided by the lesser of lesser of the yield strength of the steel cross section and the compressive strength of the concrete slab (AISC 360-16 Commentary pg. 16.1-410). Composite action and the connector stiffness also dictate how much slip will occur in the member. Slip is the relative separation between the layers of the composite member caused

by deformation. As the composite action of the member increases, the amount of slip between layers decreases, eventually becoming zero when the member reaches full composite action.

The focus of this report will be on timber-concrete composite (TCC) floors. TCC floors can be achieved in two ways: by connecting a concrete slab to a timber beam through shear connectors, notches or steel plates or by connecting a concrete topping to a CLT slab through shear connectors. The concrete provides the CLT improved stiffness, high resistance to bending and deflection, high load capacity, good dynamic behavior, superior sound, and thermal insulation (Siddika et al. 2021) while the CLT helps lower the weight of the floor, improves the tensile strength and lowers the environmental impact of the building. In Europe, CLT-concrete composites have been used in bridges, upgrading existing timber floors, and new floors in buildings (Yeoh et al. 2010). A few examples of structures that have used this building method include the University of British Columbia's Earth Science Building, Oregon State University's Forest Science Complex, and the Murr River Bridge in Austria (this uses LVL beams).

TCC floors can be attached through methods like mechanical connectors, adhesives, or notches as shown in figure 2. Mechanical connectors include lag screws, steel bars, steel plates, or rebar. These connections are often used in conjunction with notches, which will be elaborated on later in the report. Adhesives can also be used instead of mechanical connectors while still retaining strength. The connection method used will affect multiple aspects of design like the slip between the layers and composite action achieved.

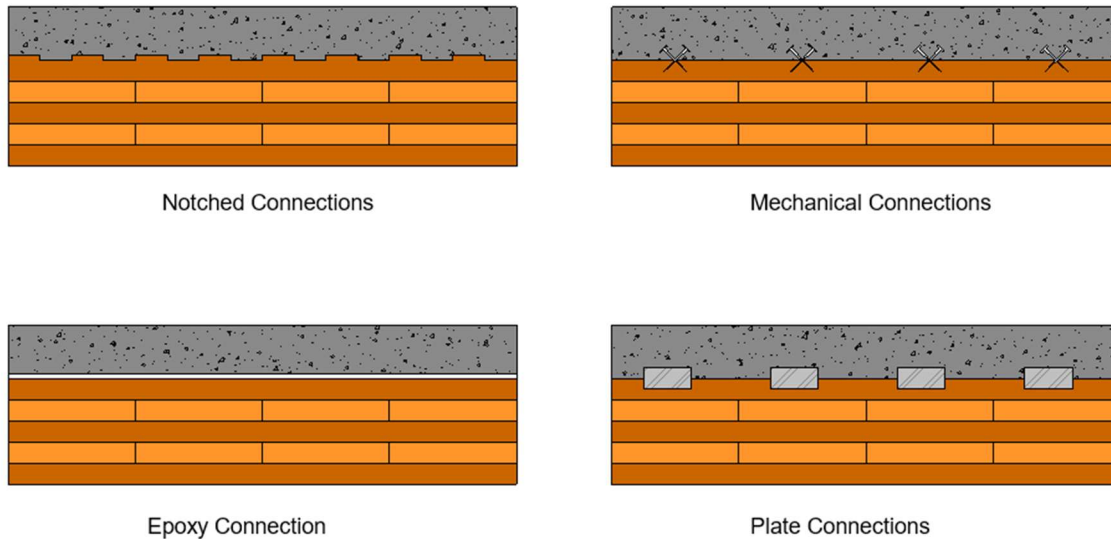


Figure 2 TCC Composite Floor Diagrams

The wood components of TCC construction can use many different engineered wood products, with glulam beams and CLT floor panels being the most common. When put under a load, the wood portion of the composite section takes the tensile force in a similar way that the reinforcing steel would usually do in a reinforced concrete beam, while the concrete section takes the compressive stress of the beam. When the connectors between the two sections are stiff enough, composite action is achieved by the transfer of shear force. Besides concrete and wood, steel reinforcing is still placed in the concrete to prevent cracking and shrinkage. Some of the benefits of using a timber-concrete composite member compared to its singular components are the following: higher span to depth ratio, greater load capacity, reduced weight compared to concrete and higher stiffness compared to timber construction.

Section 1D: Report Structure

The goal of this report is to discuss the design process for TCC floors according to the Design Guide and then compare design results with other floor types. First, the design process will be discussed, covering the design for connections, flexural strength, shear strength and deflection. Next, two case studies will cover the current use TCC floors. The paper will conclude with the design comparison between TCC floors, reinforced concrete floors, and CLT floors.

Chapter 2: Design Guidelines

In this report, one of the main sources used for TCC design is the “Design Guide For Timber-Concrete Composite Floors in Canada” (hereafter referred to as the Design Guide) by FPInnovations. This design guide outlines the theory and research behind it, provides design equations, and example problems to show the process of design. It contains design for bending, shear, deflection, vibration, and fire for composite floors made of both CLT panels and timber beams. The information about the shear connection’s strength is an estimation of the actual values. The manufacturer’s cutsheet should be used for the design of connections. However, if such data is not available, equations are provided to estimate values like stiffness.

The mass timber components will be designed using the US edition of FPInnovation’s “CLT Handbook” and the 2020 edition of “Cross-Laminated Timber Specifications Guide” provided by SmartLam North America. The prior provides design equations, general classifications of CLT, and design guidelines, while the latter is a manufacturer’s catalog of CLT panels and the design values that will be used for the equations in the “CLT Handbook”. The discussion below is largely based on these references.

Section 2A: Theory of Design

The development of the design equations used in the Design Guide are derived from Euler-Bernoulli beam differential equation. To use the Euler-Bernoulli equation the following assumptions are made: shear deformation of the layers is neglected except that of the connectors; there are no vertical separations between layers so that all layers have the same deflection, rotation and curvature; and finally, there is an assumed distributed stiffness between the two

layers of the composite floor. These assumptions allow for the floor in partial composite action to be designed as the member is assumed to have consistent properties along the member.

The effective bending stiffness of the composite member is a value to estimate a constant stiffness throughout the composite member and is found using the gamma method. The gamma method is a way to solve the beam differential equation by first taking a sinusoidal load on the beam and an assumed sinusoidal deflection and axial force function. The solutions found for the differential equation are then used to find the effective bending stiffness (EI_{eff}) in terms of the stiffnesses of timber and concrete (EI_t and EI_c) as well as the gamma factor. Gamma factors represent the amount of composite action where $\gamma_c=1$ and γ_t is calculated using equation 1, a_t and a_c are the distances between the neutral axis and centroid of timber layer and concrete layer respectively (Figure 3). The gap, t , exists when a layer of insulation is placed between the concrete and timber. The insulation is used for sound and thermal reasons as well as reducing the amount of moisture transferred from the wet concrete to timber. The value $h_{c,\text{eff}}$ is the height of concrete in compression and $t_{c,\text{eff}}$ is the height of the gap plus the height of concrete in tension.

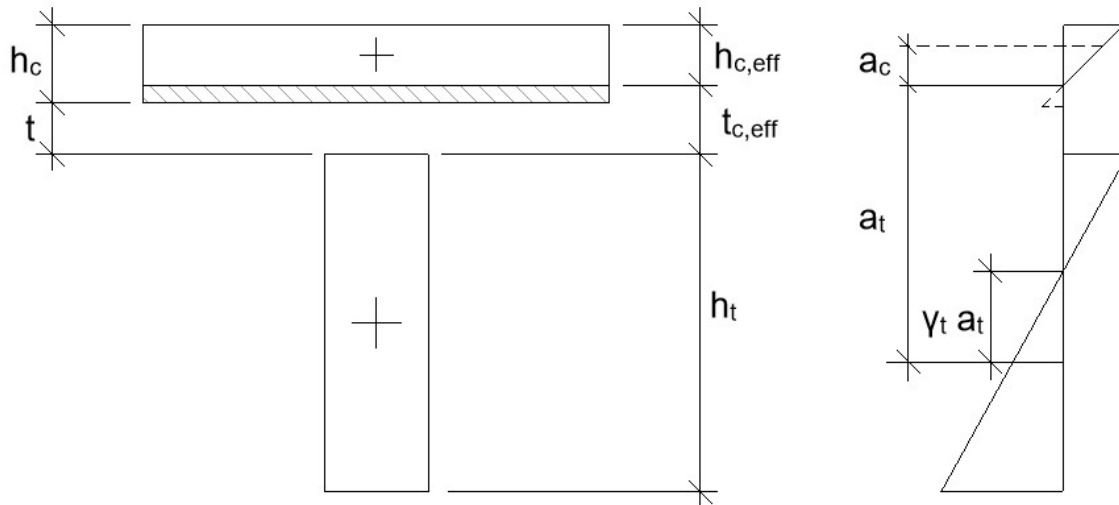


Figure 3 TCC Cross Section and Neutral Axis

$$\gamma_t = \frac{1}{1 + \frac{\pi^2(EA)_t}{KL^2}}$$

(1)

Where:

K is the distributed connection stiffness of the member, calculated with equation (8)

$(EA)_t$ is the timber area multiplied by the timber modulus of elasticity

L is the length of the member

$$a_c = \frac{\gamma_t(EA)_t * r}{\gamma_c(EA)_c + \gamma_t(EA)_t}$$

(2)

$$a_t = \frac{\gamma_c(EA)_c * r}{\gamma_c(EA)_c + \gamma_t(EA)_t}$$

(3)

Where:

$$r = \frac{h_t}{2} + h_c - 0.5h_{c,eff} + t$$

$$A_c = h_{c,eff} * b_c$$

b_c is the width of the concrete

The equation for effective bending stiffness is the following:

$$(EI)_{eff} = (EI)_c + (EI)_t + \gamma_c(EA)_c a_c^2 + \gamma_t(EA)_t a_t^2$$

(4)

When a timber beam is used, the composite section will be analyzed as a T-beam. An effective width will need to be determined for the concrete slab. The value of $b_{c,1}$ is used for edge beams while the value of $b_{c,2}$ is used for internal beams.

$$b_{c,1} = \min (0.1 * L, 12h_c, b_1) \quad (5)$$

$$b_{c,2} = \min (0.25 * L, 24h_c, b_2) \quad (6)$$

Where:

L is the span length

h_c is height of the concrete slab

b_1 is the distance between the center of an edge beam and halfway to the next beam

b_2 is the distance between the middle of an internal beam and halfway to the next beam

While the gamma method allows for a generalized approach to determining design values for the composite members, the method does have flaws. Section 2.3.3 of the Design Guide states that when the spacing of the connections is too large, the gamma method may become less accurate. This is due to the inverse relationship of the distributed connection stiffness, K , and the spacing of the connectors, s_{eff} , that will be discussed further in the next section.

Section 2B: Shear Connectors and Connection Systems

Timber-concrete composite members gain composite action through the connections between the wood and concrete layers. The connections can come in many forms like mechanical connections (self-tapping screws, lag bolts, nails), adhesives, or notches in the wood member. Behavior of these connections depends on the type of connection as well as the orientation of installation, timber properties, and concrete properties. This section will discuss how the selection of connection types influences the design and capacity of the TCC members.

Chapter 3 in the Design Guide discusses the evaluation of shear connectors and gives methods to test the properties if they are not provided by the manufacturer. To obtain the stiffness of the connector, shear tests shall be performed according to ASTM D1761. Results from the tests will be used to create a load-slip curve. The slope of the load-slip curve between 10% and 40% of the max shear load will be used as the serviceability stiffness, also called the slip modulus, k_s . The slope of the curve between 0% and 60% will be used as the stiffness for strength design, k_u (Cuerrier-Auclair 2020). The design guide states that k_u can be estimated as two thirds of k_s . If k_u is greater than k_s , then k_u is assumed to be equal to k_s . If k_u is underestimated, bending capacity will be overestimated, while shear will be underestimated.

The serviceability stiffness can be determined from either manufacturer data or using the equation below that was adapted from the Eurocode 5. The equation is as follows:

$$k_s = 2 \frac{\rho_m^{1.5} * d}{23} \quad (7)$$

Where:

ρ_m is the density of the timber member

d is the diameter of the dowel-type connector.

The distributed connection stiffness, K , is defined as the following:

$$K = \frac{nk}{s_{eff}} \quad (8)$$

Where:

n is the number of rows of connections

k is the individual connector stiffness for either strength or serviceability

s_{eff} is the effective spacing of the connectors

$$s_{eff} = 0.75s_{min} + 0.25s_{max}$$

(9)

s_{min} is the minimum connection spacing

s_{max} is the maximum connection spacing

The above stiffness equations do not account for the angle of the connection from vertical. Pullout tests done by Tomasi et al. (2010) found discrepancies between the theoretical and experimental connection stiffness when angled. The test used two timber members connected by screws in different patterns. Experimental stiffness values for the vertical screws were almost identical to the values from the equation. Screws placed at an angle had a much larger stiffness than what was predicted. Screws placed at an angle also resulted in a larger shear capacity. For both stiffness and strength, screws placed in an X pattern provided the largest values.

Two studies were conducted by Mirdad et al (2022a and 2022b) on how the composite action is affected by factors such as embedment length, and connection angle. One study found that “embedment stiffness increases whereas withdrawal stiffness decreases with increasing insertion angle” (Mirdad et al. 2022b). Another study concluded that the composite action of a TCC system is most vulnerable to changes in connection spacing and timber thickness. The embedment length and angle of the connections are the most influential to composite action (Mirdad et al. 2022a).

Another connection method is making notches in the wood to create keys between two materials. Rasmussen et al (2023) showed that notched connections exhibited improved stiffness,

lower cost, and more efficient construction. Mechanical connectors can be installed in the middle of the notch to add additional stiffness and reduce separation between layers.

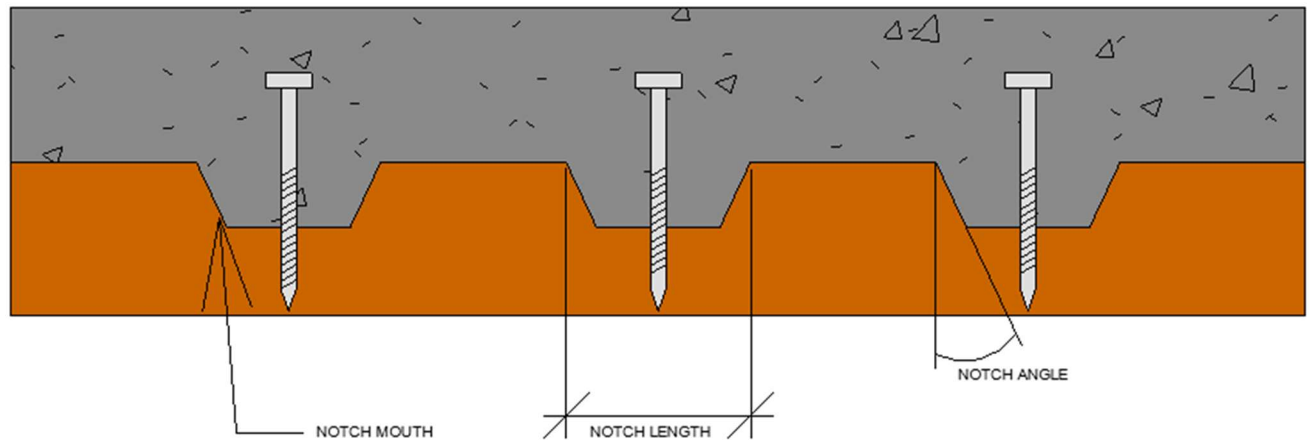


Figure 4 Diagram of Notched Connection

There are five common failure modes at the interface of TCC: concrete compressive failure, timber compressive failure, concrete shear failure, timber shear failure, and concrete rotation failure. The data collected by Rasmussen et al. (2023) showed that timber shear failure is the most common type of failure, occurring in 38% of specimens. Timber compression failure is the second most common mode, occurring in 16% of specimens. Concrete shear failure came next, being found in 15% of the specimens. Figure 5 gives a visual representation of timber and concrete shear failures.

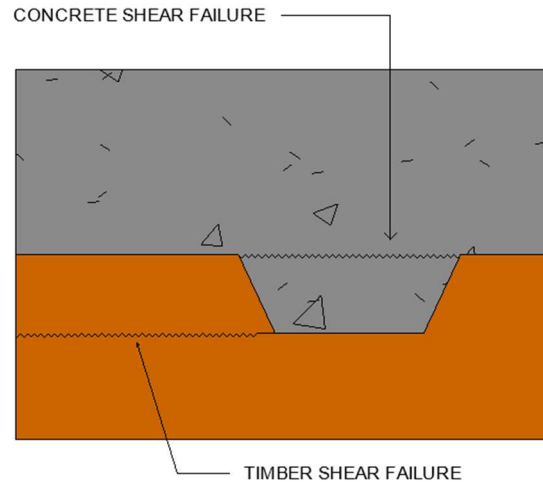


Figure 5 Diagram of Notched Shear Failures

Placing mechanical fasteners in the notches, Zhang et al. (2019) found that self-tapping screws inserted into the notches in a vertical orientation added little stiffness in the longitudinal direction. They concluded that the notches had a larger stiffness than the mechanical connectors. Two solutions to prevent shear failure in the timber layer include using a higher strength timber member and increasing the spacing between the notches (needs to be greater than the length of the notch).

Timber compression failure is caused by the buckling or crushing of the timber surrounding the notch instead of shearing off. The failure itself is ductile but can occur alongside brittle failures like timber shear failure. A study done by Thai et al (2020) shows that parts of the timber notch may be crushed while others may partially shear off due to the geometry of notch and wood. The study concluded that when the length of the wood behind the notch (heel) increases, the probability of timber compression failure increases.

Concrete shear failure is a brittle failure when the concrete slab shears away from the notched area. The goal of an experiment by Djoubissie et al (2018) was to find a connection that reduced the probability of brittle shear failures. Pushout tests were conducted on TCC members

with notched connections and mechanical connectors. The two types of connectors were lag screws and steel rebar that was threaded at the bottom, allowing it to be inserted into the wood. The purpose of the steel bar was to improve the ductility of the concrete in the notch. Both connections are used to avoid the concrete's brittle failure in shear. The study concluded that while the lag screws worked, the rebar connections had a higher shear strength, preventing the concrete from failing in shear. The downside of the rebar connection is that it has a lower stiffness than the lag screws, which could decrease capacity in other areas (Djoubissie et al. 2018).

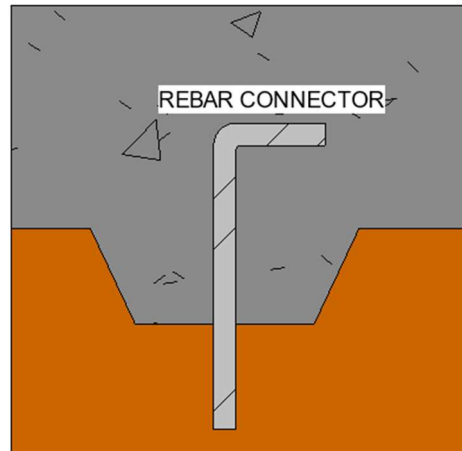


Figure 6 REBAR CONNECTION

Compression failure in concrete was the least common failure mode according to Rasmussen et al. Like the timber compression failure, concrete within the notch is crushed under the compression forces and stresses. Design for concrete compression failure

Using the data collected from previous experiments, Rasmussen et al. conducted an experiment to test notched connection between the concrete and timber members that can be deconstructed and reused. They tested fourteen specimens; each specimen being composed of one glulam member with supports at third points and two concrete slabs and a small gap in

between them. There are six total notches, three notches on each side being spaced closer to the end supports instead of the middle support.

The results of the bending tests showed that reduced notch widths caused the composite section to have a higher chance of experiencing concrete compression failure. The concrete compression failures were observed to be ductile due to “the forces transferred by the notches did not drop much in the post-failure load-carrying mode” (Rasmussen et al. 2024). The mechanical connections in the notches also contributed to the ductility and helped transfer forces after the failure (Rasmussen et al. 2024). In the specimens without fasteners, failures were brittle, with composite action being lost after the notched connection failed. As for general observations regarding the deck, the deck was easier to deconstruct after failure and allowed for easier reuse of material (Rasmussen et al. 2024). Separating the concrete portion of the composite section also reduced the amount of shrinkage cracking.

Adhesive connections are another possibility for achieving composite action. One concern with an adhesive connection is the degradation of the adhesive over time. The long-term performance of an adhesively bonded TCC member was looked at in an experiment done by Tannert et al. (2016). The TCC members had half circle voids made in the concrete either perpendicular or parallel to the span to run conduit through the floor. The members were subjected to 30% of their maximum load over a length of 4.5 years and then loaded to failure. When the members were loaded to failure, the behavior of the members was linear until they experienced a brittle failure. At failure, the value of γ_t was measured to be close to 1. The experiment concluded that there was very little degradation of the adhesive connection as shown by the high connection efficiency percentage, that the adhesive connection can be considered a rigid connection and has a brittle failure.

The next concern for adhesive connections is the constructability. A study (Brunner et al. 2006) looked at the application of applying a wet adhesive to the timber portion and then pouring wet concrete over the adhesive. Findings from the experiment showed that the adhesive was displaced at the point where the concrete was initially poured and increased with an increase in the flowrate and volume of concrete. The study concluded that when constructing a TCC member with adhesive connections, care must be taken to not disrupt the adhesive layer, which makes this method of connection difficult to use for on-site construction.

Even though the connection types offer different advantages, they are all necessary for establishing composite action. The stiffness of the connection is needed to calculate strength capacities like bending. When the stiffness of the connection is higher, the bending capacity increases.

Section 2C: Flexural Strength

The Design Guide dedicates section 6.3 to the design of the flexural strength of the composite member. The nominal bending capacity of the TCC section is governed by the minimum strength of each component. The timber component has a tensile force and a moment while the concrete component has a compressive force and a moment.

As seen in Figure 3, the tension portion of the concrete is assumed to be part of the gap. The depth of the gap, $t_{c,eff}$, is the combination of the physical gap and the tension depth of the concrete, and used in calculating the section modulus and moment capacity. The section modulus of the concrete is found using the equation below, with variables previously established:

$$S_c = \frac{(EI)_{eff}}{E_c(0.5h_{c,eff} + \gamma_c a_c)}$$

(10)

To get the moment capacity of the concrete portion the following equation will be used:

$$M_{r,\gamma,c} = 0.9\phi f'_c S_c \quad (11)$$

Where:

f'_c is the compressive strength of concrete

ϕ is equal to 0.65

The maximum stress of the concrete section under compression forces must meet the following requirement:

$$\sigma_{c,actual} = \frac{N_c}{A_{c,eff}} < 0.9\phi f'_c \quad (12)$$

Where:

N_c is the axial compression force in the concrete

$A_{c,eff}$ is the effective area of concrete in compression

In a CLT-concrete composite floor, the tension, $T_{r,t}$ and moment resistance, $M_{r,t}$, will be calculated using the design bending and tensile stress, F_b and F_t respectively, from the CLT's product data.

$$M_{r,t} = F_b * S_T \quad (13)$$

Where:

S_T is the section modulus of the timber component

$$T_{r,t} = F_t * A_{parallel} \quad (14)$$

Where:

A_{parallel} is the area of lamina's parallel to the tension load

The moment capacity of the timber portion is based on the equation for combined loading per the NDS. The tension force and bending moment in the timber component are transformed using equations (16) and (17).

$$\frac{T_{f,t}}{T_{r,t}} + \frac{M_{f,t}}{M_{r,t}} \leq 1.0 \quad (15)$$

$$T_{f,t} = \frac{\gamma_t(EA)_t a_t}{(EI)_{eff}} M \quad (16)$$

$$M_{f,t} = \frac{(EI)_t}{(EI)_{eff}} M \quad (17)$$

By setting the moment, M , to be larger than the applied moment, the capacity be determined with the following:

$$M_{r,\gamma,T} = \frac{(EI)_{eff}(T_{r,t} * M_{r,t})}{\gamma_t(EA)_t a_t M_{r,t} + (EI)_t T_{r,t}} \quad (18)$$

The gamma method assumes linear elastic behavior for the shear connections between the timber and concrete, the equation below can be used to check if the connections will yield first.

$$mV_{r,conn} > \min (T_{r,t}; 0.9\phi_c f'_c A_c) \quad (19)$$

Where:

m is the number of connectors located between the point of zero moment and the critical cross section

$V_{r,conn}$ is the design shear strength of the connection

If the strength of the is not adequate, moment capacities will need to be calculated using the elasto-plastic model. This model considers that while there is no composite action due to yielding of the connections, they still contribute to the overall strength of the section. The value of N is set to the total shear capacity of the connections to account for this. The moment capacity is determined as the following:

$$M_{r,EP} = N \left(\frac{h_t}{2} + t + h_c - \frac{h_{c,eff}}{2} \right) + \sigma_{b,c} \frac{b_c h_{c,eff}^2}{6} + \sigma_{b,t} \frac{b_t h_t^2}{6} \quad (20)$$

Where:

$\sigma_{b,c}$ is the stress in the concrete layer after the connection yields

$\sigma_{b,t}$ is the stress in the timber layer after the connection yields

N is the axial force at the interface between the timber and concrete

The moment capacity of the entire composite section is equal to the minimum of the concrete moment capacity, $M_{r,\gamma,c}$, the timber moment capacity, $M_{r,\gamma,t}$, or the elasto-plastic moment capacity, $M_{r,EP}$.

The flexural strength of the composite member is dependent on the connection used. For the effect that notched connections have on bending capacity, Rasmussen et al (2024) conducted an experiment that concluded that reduced notch widths caused a lower bending capacity and a higher chance of experiencing concrete compression failure. The notches also help with the

strength of the composite section after connections yield or fail. For adhesive connections, the elasto-plastic model is not needed due to their brittle nature. Values that are dependent on the connection type like γ_t will need to be determined through experimental data. In the bending tests done by Tannert et al (2016) on TCC floors with adhesive connections, the actual flexural strength at failure was slightly larger than the theoretical flexural strength. The increase in strength is likely due to the rigid adhesive connection not experiencing plastic deformation.

Section 2D: Shear Strength

The shear strength of the composite section is discussed in section 6.2 of the Design Guide. The shear strength in the composite section is the timber and concrete layers uses their initial shear strengths multiplied by the ratio of the effective bending stiffness to the bending stiffness of that layer. The calculation of the non-composite shear capacities of the concrete and timber follows equations from their respective code books. The timber and concrete shear capacities with no composite action are the following:

$$V_{r,t} = \phi F'_v \frac{2}{3} A_t \quad (21)$$

$$V_{r,c} = 0.21 \phi_c \lambda \sqrt{f'_c} b_c h_c \quad (22)$$

By summing the moments of the composite section, including the moment due to axial force, then taking derivatives of each moment component, shear forces can be found.

$$\Sigma M = M = \frac{(EI)_c}{(EI)_{eff}} M_c + \frac{(EI)_t}{(EI)_{eff}} M_t + N * r \quad (23)$$

$$V = \frac{dM}{dx} \quad (24)$$

Where:

$$r = \frac{h_t}{2} + t + h_c - \frac{h_{c,eff}}{2} \quad (25)$$

$$N = \frac{\gamma_i(EA)_i a_i}{(EI)_{eff}} M \quad (26)$$

$$V = \frac{(EI)_c}{(EI)_{eff}} V_c + \frac{(EI)_t}{(EI)_{eff}} V_t + \frac{\gamma_i(EA)_i a_i}{(EI)_{eff}} V_{r,i} * \left(\frac{h_{c,eff}}{2} + \frac{h_t}{2} + t_{eff} \right) \quad (27)$$

The shear strength of the timber under composite action can be determined by setting the shear force in the concrete, V_c , to zero. The timber shear force, V_t , is then set to the timber shear strength under composite action, $V_{r,\gamma,t}$, and total shear force, V , will be set to the non-composite shear strength of the timber, $V_{r,t}$. The composite shear strength, $V_{r,\gamma,t}$, is then solved for as seen below.

$$V_{r,t} = \frac{(EI)_c}{(EI)_{eff}} * 0 + \frac{(EI)_t}{(EI)_{eff}} V_{r,\gamma,t} + \frac{\gamma_t(EA)_t a_t}{(EI)_{eff}} V_{r,\gamma,t} * \left(\frac{0}{2} + \frac{h_t}{2} + \frac{t}{2} \right)$$

$$V_{r,t} = \frac{(EI)_t + \gamma_t(EA)_t a_t}{(EI)_{eff}} V_{r,\gamma,t} * \left(\frac{h_t}{2} + \frac{t}{2} \right)$$

$$V_{r,t} = \frac{(EI)_t + 0.5\gamma_t(EA)_t a_t (h_t + t)}{(EI)_{eff}} V_{r,\gamma,t}$$

$$V_{r,\gamma,t} = \frac{(EI)_{eff}}{(EI)_t + 0.5\gamma(EA)_t (h_t + t) a_t} V_{r,t}$$

(28)

When solving for concrete shear capacity:

$$V_{r,\gamma,c} = \frac{(EI)_{eff}}{(EI)_c + 0.5\gamma_c(EA)_c (2h_c - h_{c,eff} + t) a_c} V_{r,c}$$

(29)

When a gap is present, the shear strength of the connections must be checked. This is done using equation (27), the shear strength specified by the manufacturer and setting the shear force in both the timber and concrete to zero.

$$V_{r,\gamma,conn} = \frac{n(EI)_{eff}}{\gamma_t(EA)_t a_t s_{eff}} V_{r,conn}$$

(30)

Additionally, the Design Guide states that the shear strength of the connections must be greater than the unfactored total shear force in the member. This is done to limit the probability of fatigue in the connections, making the long-term deflections of the member more accurate.

The shear strength of the composite section is equal to the minimum shear strength of the timber, concrete and connection. If the connection were to govern and be less than the actual shear force, the elasto-plastic model detailed in Section 2C will be used. The timber and concrete

shear strength will need to be calculated with the assumption that there is no composite action, that the connections are ductile, and they have yielded. Similar to flexural strength, the connections still contribute to the overall strength of the section without achieving composite action. TCC members with adhesive connections will not use the elasto-plastic model.

$$V_{r,EP.t} = \left(V_{r,t} - \frac{mV_{r,conn}}{L_m} \frac{h_t + t}{2} \right) \frac{(EI)_0}{(EI)_t} + \frac{mV_{r,conn}}{L_m} r \quad (31)$$

$$V_{r,EP.c} = \left(V_{r,c} - \frac{mV_{r,conn}}{L_m} \frac{2h_c - h_{c,eff} + t}{2} \right) \frac{(EI)_0}{(EI)_t} + \frac{mV_{r,conn}}{L_m} r \quad (32)$$

Where:

m is the number of connections between zero moment and the critical section

L_m is the length of the member between the point of zero moment and the critical section

$(EI)_0$ is the bending stiffness without composite action

Section 2E: Deflection and Slip

Both short and long-term deflections need to be considered for design. Short-term deflection, also called instantaneous deflection, is the members immediate response when subjected to a load. The effective stiffness discussed in Section 2A shall be used in the short-term deflection calculation. The limits for short-term deflection are established in the building code, for example, Table 1604.3 in the IBC.

Analyzing the long-term deflections of timber-concrete composite requires more than the effective stiffness of the section. Both materials are subject to deformations caused by

temperature and humidity fluctuations, sustained loading and creeping, and for concrete, the curing process. Creeping is present in both timber and concrete structures, mostly seen as sagging floors and beams. Shrinkage is another form of long-term deformation found in both concrete and timber, occurring when moisture leaves the member. While both materials experience creeping and shrinkage, the rates are different, making it more complicated.

The Design Guide's analysis of long-term deflection uses equations found in the Canadian concrete code, CSA A23.3-14, to calculate a creep coefficient. The equation uses the percentage of compression reinforcing, ρ' , and a time-dependent factor, S , to give a coefficient that will be used to estimate the long-term deflection when multiplied by the short-term deflection as shown in equation (29) below. The time-dependent factor ranges between 1.0 for three months and 2.0 for five or more years. Equation (34) is the ACI's equation to calculate the creep factor where ξ is the same as S , and ρ' is the compression reinforcement percentage. Values for ξ can be found in table 24.2.4.1.3 in ACI 318-19.

$$K_{creep} = 1 + \frac{S}{1 + 50\rho'} \quad (33)$$

$$\lambda_{\Delta} = \frac{\xi}{1 + 50\rho'} \quad (34)$$

The Canadian equation gives a larger creep deflection than that of the ACI. The equations for timber creeping in both the US and Canadian codes are similar. A creep factor $K_{creep,t}$ for the Canadian code and K_{cr} in the NDS is applied to the long term deflection, then added to the short term deflection. The creep factor from both codes is 1.5 and 2.0 for glue laminated timber and cross laminated timber respectively while in dry conditions. The short-term deflection is

calculated at the normal short-term loading while the long-term loading is calculated under the specified long-term loads.

For TCC sections, the effective long-term modulus of elasticity is determined using the long-term deflection and creep coefficients of the layers. The modulus of elasticity of each component, E_i , is divided by the corresponding creep factor, $K_{creep,i}$, while the connection stiffness, K , is divided by two times the timber creep factor, $K_{creep,t}$ (Cuerrier-Auclair 2020). The new long-term values for the moduli of elasticity and connection stiffness are used to find the long-term effective bending stiffness $(EI)_{eff,LT}$, of the section. Using the effective long-term stiffness and the long-term loading, a long-term deflection can be. The total deflection, Δ_{max} , of the section is the sum of the short-term and long-term deflection.

$$E_{c,LT} = \frac{E_c}{K_{creep,c}} \quad (35)$$

$$E_{t,LT} = \frac{E_t}{K_{creep,t}} \quad (36)$$

$$K_{LT} = \frac{K}{2K_{creep,t}} \quad (37)$$

$$\Delta_{max} = K_{cr}\Delta_{LT} + \Delta_{ST} \quad (38)$$

Slip is the relative displacement between layers of the composite section when the section bends. The amount of slip found in the composite section correlates to the amount of composite action present in the composite section, with no slip meaning full composite action and large slip

meaning little to no composite action. The type of connection used for the section affects the amount of slip that will occur. Estimating the slip between layers can be done using more advanced calculations like those established in a 2009 study by U.A. Girhammer. The study uses the words “cumbersome and not well suited” when describing the exact procedure for interlayer slip for design purposes. Their suggestion for the approximate calculation of interlayer slip is to use the effective internal shear force at the interface between the layers and divide it by the connection stiffness.

Section 2F: Vibrations

Humans and equipment may be sensitive to the vibrations in the structure, causing discomfort for the occupants and possible damage to equipment. Low frequency floors are vulnerable to vibration problems. Codes set minimum acceptable values for floor natural frequencies. The American Institute of Steel Construction (AISC) gives acceptable natural frequencies of “7, 9, and 11 Hz for solid concrete, steel joist-concrete slab, and wood structures, respectively” (Murray 1991). The minimum natural frequency requirements necessitate the minimum stiffness of the composite section.

The calculations of natural frequency and deflection of a TCC floor for vibrations is based on a one-meter-wide section of floor (Cuerrier-Auclair 2020). The natural frequency and the span limit are as follows:

$$f_1 = \frac{\pi}{2L^2} \sqrt{\frac{(EI)_{eff}^{1m}}{m_L}} \quad (39)$$

$$L_v \leq 0.329 \frac{((EI)_{eff}^{1m})^{0.264}}{m_L^{0.264}} \quad (40)$$

Where:

$(EI)_{eff}^{1m}$ is the effective bending stiffness based on a one-meter-wide section

m_L is the mass per unit length of the section

L is the span length

L_v is the span length limited by vibration

Further research has been conducted on TCC floors to determine limitations for vibration. A study done by Lei Zhang et al. (2022) found that concrete layer improved the static and dynamic performance of the timber panels due to the composite action increasing the stiffness of the section.

Section 2G: Fire

Fire safety is a concern for any building, especially in a building with a timber structure. Chapter seven in the Canadian TCC design guide is dedicated to calculating the strength of a TCC floor in the event of a fire. The design of the TCC for fire starts with the concrete section and whether the concrete portion of the composite member will be directly exposed to fire. For timber, the Design Guide references CSA O86-14 for calculating the char sections depending on if the member is exposed to fire from one side or two sides for a timber slab and beam respectively. The CSA O86 equations can be found below where β_0 is a charring rate in mm/min, $x_{c,0}$ is the depth of the char in mm, t is the exposure time to fire in minutes, h_t is the initial slab thickness and h_{fire} is the remaining full-strength depth of the slab. In the US CLT

Handbook, the char rate is given as 1.5 in/hour or 0.635 mm/min. Additionally, the United States does not use a fixed zero-strength layer, x_t , and instead uses twenty percent of the charring height as well as the time being to the power of 0.813 to mimic the non-linear behavior of charring.

These discrepancies cause the height of charring to be larger when using the Canadian equations.

$$x_{c,0} = \beta_0 t + x_t \quad (41)$$

$$h_{fire} = h_t - x_{c,0} \quad (42)$$

The shear connections in the composite section also need to be considered when designing for fire. Provisions from the design guide specify a minimum cover distance between the shear connector and the bottom of the wood.

Fire design is an important part of the composite section design due to the timber layers vulnerability to it. It is important to check the reduced section of TCC section under fire to verify that it will have enough capacity. The concrete would also need to meet the minimum thickness requirements as stated above. The shear connections were not affected until the fire reached them.

Section 2H: Carbon Emissions and Environmental Impact

Embodied carbon is defined as the total carbon emissions released during the life cycle of a building. It encapsulates emissions produced during the production phase of the materials to the construction phase to everyday use and finally to the demolition of the building (Weir et al. 2023). In 2023, the United Nations Environment Programme (UNEP) made a report stating that the building and construction industry accounts for 37% of the global greenhouse gas emissions.

Studies on the environmental impact of TCC members use metrics such as global warming potential (GWP) and human toxicity potential (HTP). A study was conducted in South Korea (Oh et al. 2022) that compared the emissions and strengths of the floors constructed of three different materials, including a reinforced concrete floor, a CLT floor, and a CLT-concrete composite floor. These environmental impact values published by the Environmental Product Declaration database were used in the study, as shown in Table 1 below.

Table 1 Carbon Emission Factors

Material Type	CO ₂ Emission Factor (kg CO ₂ -eq/m ³)
Rebar	3,562.6
Concrete	258.58
CLT	87.64

An expected result of when the span length of the member increased, the CO₂ emissions or equivalent emissions increased was found. The concrete floor had the largest emissions out of the three floors, with an almost linear relationship between span and emissions. The CLT floor and the TCC floor however acted differently. Looking at only one of the three loading conditions, when live load equals 6 kN/m², the TCC floor has a larger emission amount than the CLT floor up until the span reaches five meters. As the span increases, the strength requirements increase, meaning a larger member size. However, for the loading stated above, the TCC member's CO₂ emissions steady out and match the CLT member's emissions. The reasoning for this is because the concrete layer adds additional strength, making the CLT layer of the TCC smaller than the pure CLT floor.

The emissions caused by the smaller CLT section with the additional concrete equal the emissions from the thicker CLT section. At shorter spans, the TCC floor performs worse than the CLT floor in terms of emissions. The additional concrete topping causes an initial spike in emissions that may outweigh the additional stiffness provided. Additionally, the strength of the floor when in composite action is not necessary as the CLT floor can accomplish the same strength and serviceability requirements that the TCC floor can (Oh et al. 2022).

When comparing the emissions of the CLT and TCC floors to the concrete floor, the CLT and TCC floors average 26.46% and 34.81%, respectively, of the emissions that a concrete floor of the same strength and span produce. When selecting between a CLT and TCC floor when basing a structure's design off sustainability, CLT would be the one to select.

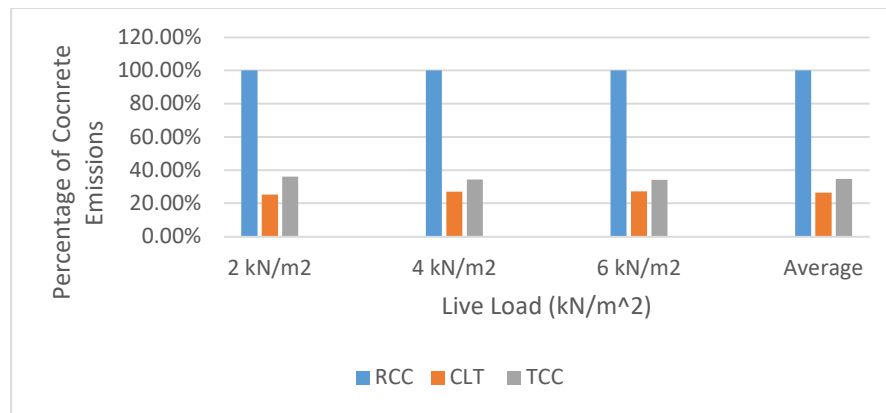


Figure 7: CO2 Emissions of CLT and TCC Compared to RCC for Different Loads (Oh et al. 2022)

Table 2: Percentages used in Comparison Graph

2 kN/m ²		4 kN/m ²		6 kN/m ²		Average	
RCC	100.00%	RCC	100.00%	RCC	100.00%	RCC	100.00%
CLT	25.21%	CLT	26.92%	CLT	27.24%	CLT	26.46%
TCC	36.00%	TCC	34.24%	TCC	34.19%	TCC	34.81%

The study found that the concrete floor had the largest emissions, with an almost linear relationship between span and emissions. Under certain live load (6 kN/m^2), the TCC floor has a larger emission amount than the CLT floor for span less than five meters, and similar emission for longer spans. The study also revealed that an increase in the compressive strength and CLT strength by 10% and 16% respectively had little effect on the emissions.

Overall, the study found that the TCC floor “emitted 50% to 70% less CO_2 ” compared to the reinforced concrete floors, with the percentage increasing with longer spans. The researchers then adjusted values to account for carbon storage of the wood which further increased the emission reduction to 77%. It clearly showed the benefits of utilizing TCC floors over reinforced concrete floors to help reduce the overall footprint of the building.

Section 2I: Constructability

Besides design, construction issues exist for TCC members. The time of construction is a big concern. Installing hundreds of shear connections can be very time consuming and cause labor costs to increase. One way to avoid this is the prefabrication of the members. Prefabricating members can be done in two different ways: making the entire composite section or only installing the shear connectors. This construction process for preinstalled shear connections would be identical to installing a CLT floor and pouring a non-composite, non-structural concrete topping. While preinstalled connections do save time and labor, there are still issues, like the care needed when transporting the CLT panels. If a connector is bent or damaged during shipping, it would cause a weak point in the composite member.

If the concrete is poured during the prefabrication process, a second concrete topping will need to be poured on site to join the panels together. Additionally, pre-pouring the concrete

increases weight for shipping and installation. It may be preferred to pour the concrete on site to avoid these situations. No matter when concrete is poured, reinforcement will need to be installed to meet minimum shrinkage requirements per ACI 318.

Connections between the CLT panel to beams, columns and other panels need to be done before the concrete is poured. When connecting two CLT panels together, standard connections can be used such as self-tapping screws at 45-degree angles that penetrate both panels. These connections will need to meet vertical and lateral load requirements. Connections from the panel to a beam will vary depending on the material of the beam. Additionally, if the width of the beam can accommodate them, shear connectors may be installed between CLT panels to allow for further connection to the concrete slab. This type of configuration can be found in figure 8.



Figure 8 Olvern Design Building Before Concrete

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Chapter 3: Case Studies

Though a new structural system, TCC floor is gaining popularity around the world.

Below are the two newly completed projects where TCC floors are used.

Earth Science Building, University of British Columbia

The University of British Columbia's Earth Science Building expansion is a five-story, 158,000 SF educational building that houses the lecture halls, offices, labs, and workspaces for the students and faculty of the Department of Earth, Ocean and Atmospheric Sciences.

Dimensions of the building's first floor can be found in Figure 9.

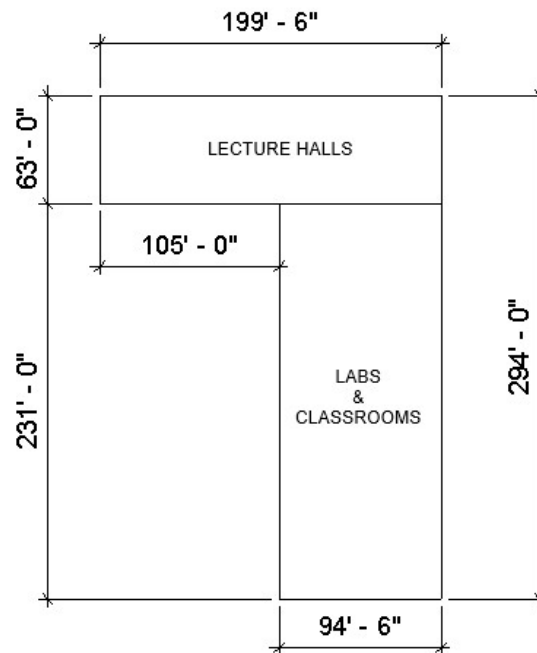


Figure 9 Dimensions of Earth Science Building

The building needed to be primarily wood due to the 2009 Wood First Act that was passed in British Columbia, Canada, promoting the usage of wood as the primary structural material for any provincially funded project (British Columbia Wood First Act 2009). A

document written by the Canadian Wood Council (2012) outlines the design and construction process of this building. The main design criteria for the building were to include a live load of 65 psf for office areas and 100 psf for lab areas; acoustics and vibrations control to reduce noise coming in and out of lecture halls, offices and study spaces; fire resistance corresponding to the number of floors in the building; and availability of materials. When considering the spans needed, vibration, acoustic, and fire protection requirements led the design team to selection of a timber-concrete composite floor for its favorable properties. The addition of concrete to a solid wood floor improves the floor's performance.



Figure 10 UBC Earth Sciences Building

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The final design for the floor was composed of a 3.5” (89 mm) thick laminated strand lumber (LSL) panel, 0.8” (20 mm) of foam insulation board, and a roughly 4” (100 mm) thick layer of reinforced concrete. The composite panels span 21’ (6.4 m) between steel girders in the first-floor lecture halls and between glulam beams on the floors above (Canadian Wood Council 2012). Data on the width of each LSL panel is not given. LSL floors panels were chosen for the shorter lead time and wider availability. To achieve composite action, a European system, known as the Holtz-Beton-Verbund System (HBV System) was used. HBV Systems uses a steel plate as a shear connector. The steel plates are inserted into slots in the LSL panels, extending through the insulation layer and into the concrete right underneath the reinforcement. Information on the sizing, spacing and strength of the connections is not provided. According to the publication, this combination of LSL, concrete, connections, and insulation resulted in a floor that was 50% lighter than if the floor was made of only reinforced concrete.

University of Massachusetts Amherst Olver Design Building

The university of Massachusetts’ Olver Design Building was discussed in the Constructability section of this paper. This four-story building houses the college’s Department of Architecture, the Building and Construction Technology program, and the Department of Landscape Architecture and Regional Planning. The original design was for a steel framed building but when funding for a mass timber building was secured, the was redesigned for mass timber after construction had already begun (WoodWorks 2017).

The switch to mass timber caused changes to all aspects of design. Regarding structural, the span lengths and clearances for MEP design needed to be kept similar. Using pure CLT floors would cause thicker floors than the original steel design. The solution for this was to use

CLT-concrete composite floors. Tom Chung, the lead architect, stated that using CLT-concrete composite floors allowed the floors to have a smaller thickness as well as reducing the number of beams (WoodWorks 2017). The spans of the floors ranged between 20 and 26 feet with connection sizes and spacings varying depending on the span and load requirement. The floors are composed of 5-ply CLT that is 6-7/8" thick, 1" of rigid insulation, and 4" of reinforced concrete topping for a total thickness of 11-7/8" (WoodWorks 2017). The lateral force resisting system of the building used CLT shear walls.

Construction started on the building in August of 2015 and ended in January of 2017. The total cost of construction was \$36 million (WoodWorks 2017). The construction process used CLT panels with pre-installed steel plate connections to speed up the installation. The CLT floor was connected to both steel and glulam beams. Both the steel and glulam beams used shear studs to achieve composite with the concrete.



Figure 11 Olver Design Building During Construction

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The WoodWorks article on this building has a life cycle environmental impact assessment comparison between the initial steel design and the TCC design. The assessment covers global warming potential (GWP), stratospheric ozone depletion, acidification of land and water, eutrophication, tropospheric ozone depletion and depletion of non-renewable energy and resources. The mass timber design reduced five out of the six measurements. The only category that did not change was the eutrophication, which stayed the same. The percentage reduction of each category is as follows: 13.1% for GWP, 10.1% for stratospheric ozone depletion, 8.9% for acidification, 0.0% for eutrophication, 3.6% for tropospheric ozone depletion, and 14.8% for depletion of nonrenewable resources.

Overall, the Olver Design Building illustrates the design and construction process using CLT-concrete composites and its environmental benefits. On the other hand, the cost of the building increased from the steel design. No original number is given but additional funding was needed when the design went to mass timber. Using TCC over modular CLT also increased the construction cost.

Chapter 4: Design Comparisons

Design comparisons were conducted among three types of floor systems, a TCC floor, a timber (CLT) floor, and a reinforced concrete floor. The design process for the TCC floor used the process and equations discussed in Chapter 2 of this report. The timber floor was designed using the CLT Handbook, NDS 2018 and the manufacturer's data. The reinforced concrete floor used ACI 318-19. Structural members such as girders and columns were designed in accordance with the corresponding materials codes.

The layout of the structure is based on the maximum span limits of the CLT. The structure is a 20' by 20' bay with a one-way slab supported by columns in each corner and girders running in the horizontal direction (Figure 12). Shoring is assumed during construction of the TCC floor before concrete gains strength and composite action is achieved.

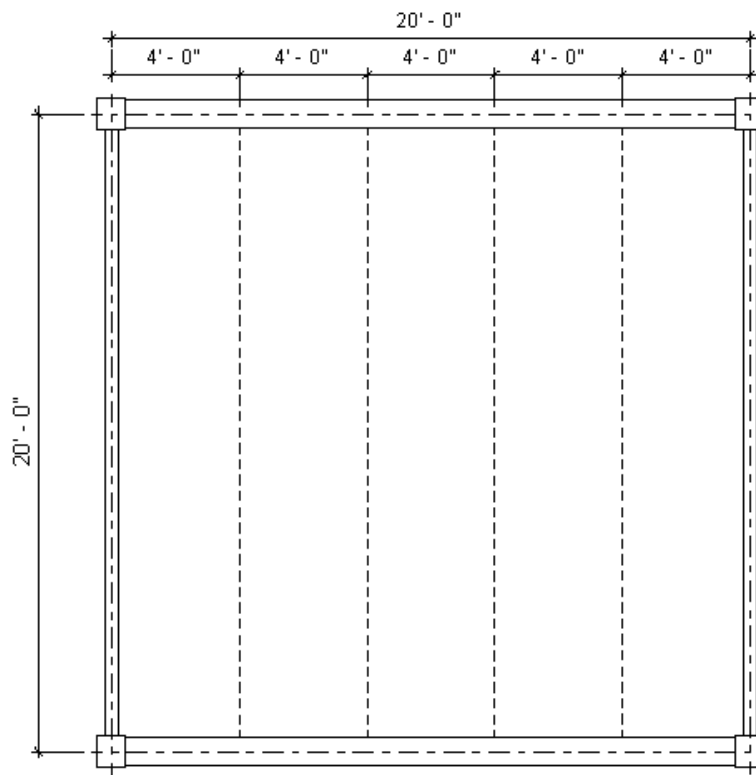


Figure 12 Structural Bay for Design

The design follows the loads of an office building. The live load is set at the ASCE 7 recommendation of 50 psf with an additional 15 psf for partitions (65 psf or 3.11 kN/m^2 total). The superimposed dead load is 25 psf (1.20 kN/m^2) for the TCC and concrete floors. The CLT manufacturer recommended using a dead load of 20 psf (0.96 kN/m^2) for CLT floors not in composite action. The strength design will use the load combinations 1a. (1.4D) and 2a. (1.2D+1.6L) from the ASCE 7-22. Instantaneous deflections are checked under the service dead load plus live load. The long-term deflections used the full dead load plus thirty percent of the live load (D+0.3L). The total deflection will be the short-term deflection plus the long-term deflection times the creep factor. The deflection limits are set by IBC table 1604.3. For limits are the following: L/360 for live load deflection, L/240 for instantaneous dead and live load, and L/180 for long term deflection.

The TCC floor's design consisted of using various iterations of thickness and connection strengths to achieve the necessary values for moment and shear. The selected product for the timber section was a CLT floor, using design values taken from the "Cross-Laminated Timber Specification Guide 2020" by SmartLam. The selected CLT product was a 3-alt SL-V5M2. This CLT floor is made of 3-layers of select structural hem-fir and is 4-1/8" (~105mm) thick. While other panels may have had one design value be greater than that of the selected panel, it was more important to select a member that would be efficient in all design aspects. The "Cross-Laminated Timber Specification Guide 2020" also gave the max span length for the CLT panel as 18 feet for office settings and design values. The design values are presented in Table 3 below:

Table 3 Design Strengths for CLT

Major Strength Direction				Minor Strength Direction			
$F_{b,0}$ (psi)	E_0 (10^6 psi)	$F_{t,0}$ (psi)	$F_{v,0}$ (psi)	$F_{b,0}$ (psi)	E_0 (10^6 psi)	$F_{t,0}$ (psi)	$F_{v,0}$ (psi)
1400	1.6	925	150	1400	1.6	925	150

These values were multiplied by their respective adjustment factors found in the NDS 2018. The adjustment factors are formatted in Table 4 below

Table 4 List of Adjustment Factors for CLT Floor

Adjustment Factor	Bending, F_b	Tension, F_t	Shear, F_v
C_D	0.9	0.9	0.9
C_M	1	1	1
C_t	1	1	1
C_L	1	-	-
K_f	2.54	2.70	2.88
ϕ	0.85	0.80	0.75
λ	0.80	0.80	0.80
F' (psi)	2,418.08	1,598.40	259.20

The concrete section of the composite section was 3" (75 mm) thick with a specified compressive strength, f'_c , of 4000 psi (24.1 MPa). Steel rebar is added to the concrete to meet the minimum amount of shrinkage reinforcement. Section 24.4.3.2 in the ACI 318-19 gives a minimum reinforcement ratio of 0.0018 for shrinkage and temperature reinforcement. Using this ratio, the minimum area needed is 0.0648 in²/ft, which can be satisfied by using #3 bars at 20" on center. The reinforcement, being in the effective compression concrete area, would not add bending strength to the composite section.

The connection system for the composite section consists of 3 rows of 5/8" (16 mm) lag bolts with a minimum spacing of 8" (203.2 mm) and a maximum spacing of 18" (457.2 mm). The effective spacing is 10.5" (266.7 mm). These bolts are placed at a 45-degree angle. The connections are 4.5" (114.3 mm) long with a 2.125" (53.975 mm) embedment depth in the timber, 2" (50.8 mm) embedment in the concrete and 0.375" (9.525 mm) is taken up by the gap. The serviceability stiffness of an individual connection used Equation 3 in Chapter 2. The serviceability stiffness of one connection is 75.83 k/in (13.28 kN/mm). The distributed stiffness was equal to 23.92 kips/in² (0.84 kN/mm²) and γ_t equal to 0.797. The shear strength of the connections was established in research done by Khorsandria et al (2016) where multiple types of TCC connections were investigated and pull-out tests were used to determine connection shear strength and stiffness. The shear strength established in the study for a 5/8" (16mm) diameter bolt with 3.93" (100mm) of embedment was 8.54 kips (38 kN) $\pm$ 6%. The calculations use the lower bound of this value, 8.03 kips (35.72 kN), as the shear strength. The total number of connections used is roughly 350 bolts (70 bolts for a 4' wide panel). Total pricing for the bolts ranges from \$650 to \$800 depending on manufacturer and supplier.

The design of the columns and girders supporting the floor were designed to be glued laminated timber and used the design tables from the 2018 NDS Supplement. The columns were designed for compression only and a height of 12'. The final design gave 10.75"x12" 16F-1.3E glulam columns. The girders were designed for bending, shear and deflection. The girder size is 10.75"x22.5" and have a stress class of 24F-1.7E. Had concrete girders been used in the TCC structure, the girders would be 12"x20" with (3) #9 bars for bottom reinforcement and #4 stirrups at 16" on center for shear reinforcement. To meet strength needs before the concrete dries,

shoring running vertically will need to be provided in the form of 5.125"x15" glulam beams with a stress class of 20F-1.5E at 4' on center. This layout is depicted in Figure 11 as the dashed lines.

The concrete structure was designed as a one-way slab using ACI 318-19. The thickness of the slab must meet the thickness requirements in Table 7.3.1.1 or meet the deflection requirements in Table 24.2.2. The design thickness of the slab is 10" reinforced with #6 rebar at 12" on center parallel to the span. Section 24.4.3.2 requires shrinkage reinforcement in the horizontal direction. Shrinkage reinforcement is #6 rebar at 24" on center. Girders were designed to be 16"x24" with (4) #9 bars used for bottom reinforcement and #4 stirrups at 12" on center for shear reinforcement. Calculations for the column showed that only a 7"x7" column was needed but due to the size of the beams, the column size needed to be at least 16"x16".

The SmartLam CLT specification guide that was used for the CLT section gave a 7-alt SL-V5M2. This panel is 9-5/8" with an additional 1.5" of concrete topping. The superimposed dead load limit for the CLT floor is specified by the manufacturer as 20 psf accounting for a 1.5" concrete topping. The sizes of the girders and columns for the CLT floor matched the girders and columns of the TCC floor.

Out of the three floors designed, the TCC floor was the most tedious. The selection of shear connections, CLT panel and concrete thickness created multiple variables that needing to be accounted for. To meet certain limits like moment capacity or deflection, each component had to be looked at to see what would provide the most strength for the smallest change. For example, if the section had too much deflection, adjustments in the concrete made a larger difference than adjustments in the CLT. Meanwhile, the CLT structure had design values provided by a manufacturer and the concrete structure was straightforward using design equations from the ACI.

A summary of the final designs can be found in Table 5 below. The self-weights listed in the table are for the floors/slabs. The ultimate loads set by the limit states above are provided in the table and include the self-weight.

Table 5 Member Sizes and Design Strengths

	TCC		Concrete		CLT	
Timber Thickness	4.125	in	0	in	9.625	in
Gap Thickness	0.375	in	0	in	0	in
Conc. Thickness	3	in	10.00	in	1.5	in
Total Thickness	7.5	in	10.00	in	11.125	in
SW, timber	9.68	psf	0	psf	22.54	psf
SW, concrete	36.88	psf	124.74	psf	18.75	psf
SW, total	46.56	psf	124.74	psf	41.29	psf
Girder Depth	22.5	in	24	in	22.5	in
Girder Width	10.75	in	16	in	10.75	in
Strength Class	24F-1.7E		4000	psi	24F-1.7E	
Column Size	10.75" x 12"		16" x 16" w/ (4) #8		10.75" x 12"	
Limit State	Ultimate	Capacity	Ultimate	Capacity	Ultimate	Capacity
Moment (k-ft)	37.97	62.59	56.79	64.76	33.86	91.54
Shear (k)	7.60	15.99	11.36	25.21	6.77	15.40
ΔLL (in)	0.67	0.294	0.67	0.127	0.67	0.227
$\Delta D+L$ (in)	1.00	0.618	1.00	0.419	1.00	0.46
ΔT (in)	1.33	1.14	1.33	0.788	1.33	0.793

The table shows that by using a TCC floor in place of a concrete or CLT floor, the total slab thickness is reduced by 2.5" and 3.625". For the CLT member, deflection governed the design, leading to an increased thickness, even though the strength capacities were adequate. The increase in stiffness of the TCC member allows it to reach the spans that the CLT cannot. For long-term deflections, the TCC member has the advantage over reinforced concrete structure over cracking. The cracking of concrete reduces the moment of inertia of the concrete, creating larger long-term deflections. With a similar thickness to the other floors, the TCC floor would be

stronger and have less deflection regardless of what material increases in thickness. The TCC floor would also be able to meet spans of 25' to 30' while still meeting deflection criteria.

As for material cost, the CLT structure is the most expensive while the concrete structure is the least expensive. The price breakdown can be found in Table 6 below. The prices for the CLT came from the Mass Timber Price Index report from 2024, the concrete and rebar prices use the recent national averages, and the cost of the shear connections for the TCC are the average of multiple suppliers. The TCC structure's price primarily comes from the CLT panel. The costs of girders, columns, formwork, labor and transportation were not considered.

Table 6 Cost of Structures

	TCC	Concrete	CLT
CLT Vol. (ft ³)	137.5	0	320.83
CLT Cost (\$/cf)	40	40	40
CLT Total Cost (\$)	\$ 5,500.00	\$ -	\$ 12,833.20
Conc. Vol. (yd ³)	3.7	12.35	1.85
	166.39	166.39	166
Conc. Total Cost (\$)	\$ 615.64	\$ 2,054.92	\$ 307.10
Amount of Rebar	24	30	0
Rebar Cost (\$/bar)	6.49	24.47	0
Rebar Total Cost (\$)	\$ 155.76	\$ 734.10	\$ -
Amount of Connections	350	0	0
Connection Total Cost (\$)	\$ 742.00	\$ -	\$ -
Total Cost (\$)	\$ 7,013.40	\$ 3,033.72	\$ 13,140.30

The design and cost comparisons of the three floors give a better understanding of the advantages and disadvantages of the materials. While TCC floors can reduce the depth and weight when used in place of a concrete floor, the cost increase makes it unfit for buildings wanting reduced costs. On the other hand, the TCC floor allows for a cheaper, shallower floor when used in place of a CLT floor at the cost of being slightly heavier.

Chapter 5: Conclusion

TCC floors are a recent development in North American construction. With its growing popularity, multiple design guides have been established but there is no official standard like the Eurocode 5. Of the currently published design guides, FPInnovations' *Design Guide For Timber-Concrete Composite Floors in Canada* (2020) (referred to as the Design Guide throughout this report) covers the full design of a TCC floor. This report gave a comprehensive introduction to the Design Guide and compared TCC floors to other types of floor structures.

The strength and stiffness of TCC section depends on the property of the timber and concrete components, as well as the connection between the two materials. The Design Guide uses the gamma method in which the parameter, gamma, representing the amount of composite action is used in determining the effective stiffness of the composite section. Various connection methods and their strength and stiffness calculations were discussed.

The flexural strength of a TCC floor is dependent on strengths of the timber, concrete and connections. Equations for flexural strength design combine both local bending (bending in each material) and global bending (compression in concrete and tension in timber). If the connections were to yield before the timber or concrete, the elasto-plastic model is needed. Depending on the relative thickness of the two material, the composite section bending strength can be governed by either timber or concrete.

Shear strength of the TCC floor is calculated by taking the derivative of the sum of the moments in the section. The connections will also need to be checked in strength to ensure that they do not fail. If the connections were to govern, the elasto-plastic model will be used to ensure that when the connections yield, the floor still has the necessary shear strength. The shear

strength is usually governed by the timber or the connections due to concrete's inherently high shear strength.

Further research is needed on fire and vibration design. For the current practice, the Design Guide and research illustrated that a TCC floor acts like a timber structure with a concrete topping when subject to fire and vibration.

Overall, TCC floors provide a better solution utilizing materials' strengths and avoiding their weaknesses. TCC floors have an improved bending and shear strength and lower deflection over CLT and concrete floors, without cracking problems compared to concrete slabs. While the TCC floor can outperform the other two floors, the disadvantage is the cost and construction issues. For the material cost, the TCC floor is over double the cost of the concrete floor while being half the cost of the CLT floor. The shear connections needed for composite action will increase the cost as well as lead to more construction problems. This can be resolved by pre-installing connections in the shop but that, in turn, creates the risk of damage during transportation.

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Appendix A - Permission to Use Figures



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Appendix B - Calculations

The following section contains the calculations used in Chapter 4 of this paper.

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Report	George Luetje		
Step / Variable	Calculations		References
EI for Concrete Section	Concrete Information		ACI 318-19 Eq. 19.2.2.1.b
	Assume that:		
	f'c = 3500 psi		

Report	George Luetje			
Step / Variable	Calculations		References	
	Find distance between the centroid of concrete and neutral axis $a_c = \frac{\gamma_t(EA)_t r}{\gamma_c(EA)_c + \gamma_t(EA)_t} = 1.46 \text{ in}$ $\gamma_c = 1$ $\gamma_t = 0.75$ $r = 4.16 \text{ in}$ $(EA)_t = 3.17\text{E}+08 \text{ lb}$ $(EA)_c = 4.42\text{E}+08 \text{ lb}$ (using $h_{c,eff}$) $h_{c,eff} = 2.732 \text{ in}$		"Design Guide of TCC Floors in Canada" Eq. 2.18	
	Find distance between the centroid of timber and neutral axis $a_t = \frac{\gamma_c(EA)_c r}{\gamma_c(EA)_c + \gamma_t(EA)_t} = 2.70 \text{ in}$		"Design Guide of TCC Floors in Canada" Eq. 2.19	
Find (EI)eff	$(EI)_{eff} = (EI)_c + (EI)_t + \gamma_c(EA)_c a_c^2 + \gamma_t(EA)_t a_t^2$ $(EI)_c = 3.64\text{E}+08 \text{ lb-in}^2$ $(EI)_t = 1.5\text{E}+08 \text{ lb-in}^2$ $\gamma_c = 1$ $\gamma_t = 0.753713$ $(EA)_t = 3.17\text{E}+08 \text{ lb}$ $(EA)_c = 4.42\text{E}+08 \text{ lb}$ $a_c = 1.46 \text{ in}$ $a_t = 2.70 \text{ in}$ $(EI)_{eff} = 3.19\text{E}+09 \text{ lb-in}^2$		"Design Guide of TCC Floors in Canada" Eq. 2.15	
Calculate bending strength and Tension Strength of CLT Section	Design Values for CLT Panel			
	Adjustment Factor	Bending, F_b	Tension, F_t	Shear, F_v
	C_D	0.9	0.9	0.9
	C_M	1	1	1
	C_t	1	1	1
	C_L	1	-	-
	K_f	2.54	2.7	2.88
	ϕ	0.85	0.8	0.75
	λ	0.8	0.8	0.8
	F' (psi)	2,418.08	1,598.40	259.20
Not used due to LRFD				
$F'_b * S_{eff} = 22453.6 \text{ lb-ft}$				
$T_{r,t} = F'_t * A_{parallel} = 211429 \text{ lb} = 211.4 \text{ kips}$ $A = 132.28 \text{ in}^2$				

Report	George Luetje		
Step / Variable	Calculations		References
Calculate bending strength of CLT in TCC section	$M_{r,y,t} = \frac{(EI)_{eff} T_{r,t} M_{r,t}}{\gamma_t (EA)_t a_t M_{r,t} + (EI)_t T_{r,t}}$ $M_{r,t} = 22453.6 \text{ lb-ft} = 269443.2 \text{ lb-in}$ $T_{r,t} = 211429 \text{ lb} = 211 \text{ kips}$ $(EI)_{eff} = 3.19\text{E}+09 \text{ lb-in}^2$ $\gamma_c = 1$ $\gamma_t = 0.753713$ $a_t = 2.69987 \text{ in}$ $(EA)_t = 3.17\text{E}+08 \text{ lb}$ $(EA)_c = 4.42\text{E}+08 \text{ lb}$ $(EI)_t = 1.5\text{E}+08 \text{ lb-in}^2$ $M_{r,y,t} = 73.84 \text{ k-ft}$		"Design Guide of TCC Floors in Canada" Eq. 6.29
Calculate bending strength of Concrete in TCC section	$S_c = \frac{(EI)_{eff}}{E_c (0.5 h_{c,eff} + \gamma_c a_c)}$ $(EI)_{eff} = 3.19\text{E}+09 \text{ lb-in}^2$ $E_c = 3.37\text{E}+06 \text{ psi}$ $h_{c,eff} = 2.732 \text{ in}$ $\gamma_c = 1$ $a_c = 1.46 \text{ in}$ $S_c = 3.35\text{E}+02$ $M_{r,y,c} = 0.9 \phi_c f'_c S_c$ $f'_c = 3500 \text{ psi}$ $\phi_c = 0.65$ $M_{r,y,c} = 6.87\text{E}+05 \text{ lb-in} = 57.23 \text{ k-ft}$		"Design Guide of TCC Floors in Canada" Eq. 6.30
Check Conn. In bending and Elasto-Plastic Model	$m V_{r,conn} > \min (T_{r,t}; 0.9 \phi_c f'_c A_c)$ $m = 34.5 \quad (\# \text{ of connection between 0 moment and crit. Sect.})$ $V_{r,conn} = 8.031 \text{ kips}$ $m V_r = 277.05 \text{ kips}$ $T_{r,t} = 211 \text{ kips}$ $0.9 \phi_c f'_c A_c = 268.5 \text{ kips}$ Connections yield after timber and concrete, do not need to check Elasto-Plastic Model		"Design Guide of TCC Floors in Canada" Eq. 6.38
	Moment capacity of the TCC member $M_r = \min(M_{r,y,t}; M_{r,y,c})$ $M_{r,y,t} = 73.84 \text{ k-ft}$ $M_{r,y,c} = 57.23 \text{ k-ft}$ $M_r = 57.23 \text{ k-ft}$		

Report	George Luetje	
Step / Variable	Calculations	References
Shear calcs	$V_{r,t} = 11.40$ kips $V_{r,c} = 0.21\phi_c\lambda\sqrt{f'_c}b_ch_c = 14.74$ kips $V_{r,\gamma,t} = \frac{(EI)_{eff}}{(EI)_t + 0.5\gamma(EA)_t(h_t + t)a_t}V_{r,t} = 22.68$ kips $(EI)_c = 3.6E+08$ lb-in² $(EI)_t = 1.5E+08$ lb-in² $\gamma_c = 1$ $\gamma_t = 0.75$ $(EA)_t = 3.2E+08$ lb $(EA)_c = 4.4E+08$ lb $a_c = 1.46$ in $a_t = 2.70$ in $(EI)_{eff} = 3.2E+09$ lb-in² $h_t = 4.125$ in $t = 0.4$ in $V_{r,\gamma,c} = \frac{(EI)_{eff}}{(EI)_c + 0.5\gamma_c(EA)_c(2h_c - h_{c,eff} + t)a_c}V_{r,c} = 30.48$ kips $h_c = 3$ in $t = 0.4$ in $h_{c,eff} = 2.732$ in Elasto-Plastic Shear $V_{r,EP,t} = \left(V_{r,t} - \frac{mV_{r,conn}}{L_m}\frac{h_t + t}{2}\right)\frac{(EI)_0}{(EI)_t} + \frac{mV_{r,conn}}{L_m}r = 19.84$ kips $mV_r = 277.0549$ kips $L_m = 120.0$ in $r = 4.16$ in $V_{r,EP,c} = \left(V_{r,c} - \frac{mV_{r,conn}}{L_m}\frac{2h_c - h_{c,eff} + t}{2}\right)\frac{(EI)_0}{(EI)_t} + \frac{mV_{r,conn}}{L_m}r$ $V_{r,EP,c} = 25.2$ kips $V_r = 19.8$ kips	SmartLam CLT Spec

Report	George Luetje					
Step / Variable	Calculations					References
	Load Calcs					
	DL =	25 psf		SW =	46.56 psf	
	LL =	65 psf				
	Load Combinations				(psf)	
	Combo 1	1.4 DL	+	0 LL	= 100.2	
	Combo 2	1.2 DL	+	1.6 LL	= 189.9	
	Combo 3	1 DL	+	1 LL	= 136.6	
	Combo 4	0 DL	+	1 LL	= 65	
	Combo 5	1 DL	+	0.3 LL	= 91.1	
	Distributed Loads					
	Combo 1	w =	0.40 k/ft	=	33.4 lb/in	
	Combo 2	w =	0.76 k/ft	=	63.3 lb/in	
	Combo 3	w =	0.55 k/ft	=	45.5 lb/in	
	Combo 4	w =	0.26 k/ft	=	21.7 lb/in	
	Combo 5	w =	0.36 k/ft	=	30.4 lb/in	
	Moments L = 20 ft					
	Combo 1	M =	20.04 k-ft			
	Combo 2	M =	37.97 k-ft			
	Combo 3	M =	27.31 k-ft			
	Combo 4	M =	13.00 k-ft			
	Combo 5	M =	18.21 k-ft			
	Shear					
	Combo 1	V =	4.01 kips			
	Combo 2	V =	7.59 kips			
	Combo 3	V =	5.46 kips			
	Combo 4	V =	2.60 kips			
	Combo 5	V =	3.64 kips			

Report	George Luetje		
Step / Variable	Calculations		References
	Deflection $\Delta = \frac{5wL^4}{384(EI)_{eff}}$ w(D+L) = 45.5 lb/in LL = 21.7 lb/in wLT = 30.4 lb/in L = 20 ft = 240 in (EI)eff = 3.2E+09 lb-in^2 K,creep,t = 2 S,time,c = 3 (EI)c,LT = 1.2E+08 (EI)t = 7.5E+07 (EI)eff,LT = 1.6E+09 γt,LT = 0.19 Δ D + L 0.616 in LL 0.293 in LT 0.83 in Total = 1.13 in		

Report	George Luetje			
Step / Variable	Calculations		References	
TCC Girder Calc	Stress Class: 24F-1.7E		NDS Table 5.3.1	
	TCC Girder			
	L = 240 in = 20 ft			
	L (SI) = 6096 mm = 6.096			
	L (floor) = 6095.703 mm = 6.095703			
	S = 907.0313 in^3 = 2118847 mm^3			
	d = 22.5 in = 571.5 mm			
	b = 10.75 in = 273.05 mm			
	LRFD			
	Mmax = 267923.6 kN-mm = 2369.52 k-in			
	fb = 126.4478 kN/mm^2 = 2612.39 psi			
	CD	0.9		-
	CM	1		1
	Ct	1		1
	CL	1		1
	CV	0.876		0.876
	Cfu	1		1
	Cc	1		1
	Ci	1		1
	Cvr	1		1
	K	-		2.54
	φ	-		0.85
	λ	-		0.8
	F'b,req	2397.2		1726.0
	Fb =	2400		2400
	Fb' =	1892.784		3632.463
	CD	0.9		-
	CM	1		1
	Ct	1		1
	CL	1		1
	CV	0.876289		0.876289
	Cfu	1		1
	Cc	1		1
Ci	1	1		
Cvr	1	1		
K	-	2.88		
φ	-	0.75		
λ	-	0.8		
Fv =	210	210		
F'v =	189	362.88		
fv =	177.3687	245.0883		
Deflections				
E =	1700000 psi	= 1700000		
I =	10204.1 in^4			
w =	2.860151 k/ft	= 238.346		
L =	240			
Δ =	0.593564 in			
L/240 =	1 in			

Report	George Luetje																								
Step / Variable	Calculations	References																							
	TCC Column Column 10.75 in 12 in Stress Class 1.6F-1.3E Fc <table><tr><td>Values</td><td>925</td></tr><tr><td>CD</td><td>0.9</td></tr><tr><td>CM</td><td>1</td></tr><tr><td>Ct</td><td>1</td></tr><tr><td>CL</td><td>-</td></tr><tr><td>CV</td><td>1</td></tr><tr><td>Cfu</td><td>-</td></tr><tr><td>Cc</td><td>1</td></tr><tr><td>CI</td><td>-</td></tr><tr><td>CVR</td><td>-</td></tr><tr><td>Cp</td><td>0.97637</td></tr><tr><td>Cb</td><td>0</td></tr></table> F'c = 812.8 psi	Values	925	CD	0.9	CM	1	Ct	1	CL	-	CV	1	Cfu	-	Cc	1	CI	-	CVR	-	Cp	0.97637	Cb	0
Values	925																								
CD	0.9																								
CM	1																								
Ct	1																								
CL	-																								
CV	1																								
Cfu	-																								
Cc	1																								
CI	-																								
CVR	-																								
Cp	0.97637																								
Cb	0																								

Report	George Luetje			
Step / Variable	Calculations			References
	Concrete Floor Calculations			
	B(in)	48 in		
	H(in)	10 in		
	d(in)	8.5 in		
	Density	150 pcf		
	f'c	4000 psi		
	L (ft)	20.00 ft		
	λ	1		
	As	3 in2		
	d'	0 in		
	A's	0 in2		
	n	8		
	wL	0.26 k/ft		
	wD	0.1 k/ft		
	fy	60 ksi		
	Es	29000 ksi		
	Ec	3605.0 ksi		
	SW	0.5 k/ft		
	Wu	1.136 k/ft		
	Mu	56.79 k-ft		
	Rebar Size	6	0	
	Rebar Area	0.44 in2		
	# of Bars	4 bars		
	As	1.76 in2		
	Theor. As	1.670 in2		
	"a"	0.647 in		
	c	0.761 in		
	Beta 1	0.85		
	εt	0.030498		
	Strain Good?	YES		
	φMn	64.76 k-ft	$M_n = A_s F_y * \left(d - \frac{a}{2}\right)$	
	φMn>Mu?	YES		
	fr=	474.3 psi	ΔD+L=	0.419 in
	yt=	5 in	ΔLL=	0.127 in
	Ig=	4000 in^4	L/360 =	0.667 in
	Mcr=	31.62 k-ft	λΔ =	2
	Ma=	29.997 k-ft	EC/λΔ =	1802.5
	Ie=	4000 in^4	ΔLT =	0.66 in
	Ec=	3605.0 ksi	ΔT =	0.79 in
	ΔD=	0.150 in	L/180 =	1.33 in
	ΔD=	3.804 mm		

Report	George Luetje			
Step / Variable	Calculations		References	
1.4D 1.2D+1.6L D+L LL DL+.3LL	Values			
	Width = 48 in = 1219.2 mm			
	Length = 20 ft = 6095.703 mm			
	Topping = 1.5 in = 38.1 mm			
	Loads			
		Wu (kN/m)	Mu (kN-m)	Vu (kN)
	Combo 1	4.04	18.79	12.33
	Combo 2	9.88	45.90	30.12
	Combo 3	7.66	35.58	23.35
	Combo 4	3.79	17.62	11.57
	Combo 5	4.72	21.91	14.37
	Mmax = 35.58 kN-m = 26.24 k-ft			
	M/ft = 6.56 k-ft/ft per foot of width			
	M/ft = 6560.56 lb-ft/ft what is necessary from Fb*S			
	Slab Selection: SL-V5M2, 5-alt			
	d = 9.625 in 244.475 mm			
	FbSeff = 13250 lb-ft/ft			
	Adjustment Factor	Fb		
	CD	0.9	From NDS table 10.3.1	
	CM	1	From NDS table 10.3.1	
	Ct	1	From NDS table 10.3.1	
	CL	1	No CL calc needed because d<b	
	K	2.54	From the CLT Catalogue	
	φ	0.85	From the CLT Catalogue	
	λ	0.8	From NDS table N3.3	
Fb', ASD 11925				
Fb', LRFD 22885.4				
ASD: Fb'Seff*b = 47700 lb-ft = 47.7 k-ft				
ASD: Mmax = 35.58 kN-m = 26.24 k-ft				
LRFD: Fb'Seff*b = 91541.6 lb-ft = 91.5416 k-ft				
LRFD: Mmax = 45.90 kN-m = 33.86 k-ft				
Shear				
LRFD: Vmax = 30.12 kN = 6.772 kips				
ASD: Vmax = 23.35 kN = 5.249 kips				
Vs = 3850 lb/ft				
Adjustment Factor Vs				
CD	1	From the CLT Catalogue		
CM	1	From NDS table 10.3.1		
Ct	1	From NDS table 10.3.1		
Kf	2	From the CLT Catalogue		
φ	0.75	From the CLT Catalogue		
λ	0.8	From NDS table N3.3		
Vs', ASD 3850		* width = 15.4 kips		
Vs', LRFD 4620		* width = 18.48 kips		

Report	George Luetje		
Step / Variable	Calculations		References
	Deflection		
	(EI)eff = 1.03E+09 lb-in^2/ft		
	(EI)eff = 4.12E+09 lb-in^2 = 1.18E+13 N-mm^2		
	SW = 28.10 pcf	* d =	90.15 plf
	SW = 150.00 pcf	* d =	75.00 plf
	w = 7.66 N/mm		
	Δinstant = 11.66 mm	=	0.46 in (D+L Case)
	ΔLL = 5.78 mm	=	0.23 in (LL Case)
	L/360 = 16.93 mm	=	0.67 in
	Kcr = 2.00		
	wLT = 4.72 N/mm		
	ΔLT = 7.18 mm	=	0.28 in
	ΔT = 20.13 mm	=	0.79 in
	L/180 = 33.87 mm	=	1.33 in
	CLT Girder		
	L = 240 in	=	20 ft
	L (SI) = 6096 mm		6.096
	L (floor) = 6095.7 mm		6.10
	S = 907.0 in^3	=	2118847 mm^3
	d = 22.5 in	=	571.5 mm
	b = 10.75 in	=	273.05 mm
	LRFD		
	Mmax = 248.85 kN-mm	=	2200.815 k-in
	fb = 0.12 kN/mm^2	=	2426.4 psi
	ASD LRFD		
	CD	0.9	-
	CM	1	1
	Ct	1	1
	CL	1	1
	CV	0.876289	0.876289
	Cfu	1	1
	Cc	1	1
	Ci	1	1
	Cvr	1	1
	K	-	2.54
	φ	-	0.85
λ	-	0.8	
F'b,req	2200.7	1603.139	
Actual Values from NDS			
Fb = 2400	2400		
Fb' = 1892.784	3632.463		