Timber-concrete composite: an alternative composite floor system

by

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Abstract

The desire for sustainability has propelled innovation in structural engineering for much of the 21st century. Implement sustainable design without sacrificing the structural integrity of a building is important. The timber-concrete composite (TCC) floor system is an alternative floor system that offers superior sustainability and quick installation compared to other composite floors. TCC is comprised of a reinforced concrete slab connected to timber plate/beams by shear connectors that transfer the internal forces through the shear flow. To resist bending forces the reinforced concrete slab experiences the majority of compression stress and the timber plate/beam experience the majority of tension stress. Compared to an equivalent all-concrete section the TCC system has similar strength and stiffness as well as reduced weight.

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Acknowledgments

I'd like to thank my family, friends, classmates, and professors that have encouraged me and supported me through my college education. I would like to specifically thank Professor Kimberly Kramer and all that she has done for me from teaching me how to design a masonry column to life lessons.

Chapter 1 - Introduction

One of the biggest challenges society is facing currently is climate change. The rising temperature of the Earth is growing at an alarming rate, which can be attributed to the increase in carbon dioxide levels in the atmosphere. The manufacturing of building materials makes up 11% of the total global greenhouse gas emissions according to the latest data from the United Nations Environment Program. New sustainable ideas and materials need to be researched for the construction industry to address the growing issue of carbon emissions. A sustainable solution for buildings is to incorporate timber, a sustainable material, and reduce the amount of concrete or steel used. The timber-concrete composite floor system is a possible solution for addressing the need to reduce carbon emissions. Timber-concrete composite (TCC) floor system is a composite floor that connects a reinforced concrete slab to a timber beam/plank using different shear connectors to transfer shear flow between the materials. Under bending stresses, this floor system utilizes each material's properties by timber primarily resisting the tension force and reinforced concrete resisting the compression force.

The TCC system is used for new construction of bridges and multi-story buildings and also renovations by adding strength and stiffness to existing floors. After World War I and II the TCC floor system gained popularity due to the shortage of steel. The history and evolution of the TCC floor system is explored in Chapter 2 of this report. TCC floors have many advantages compared to a traditional timber system such as greater strength, stiffness, decreased vibration, better acoustic separation, and thermal mass, improved seismic, and fire resistance. The major drawback to some TCC floors is the additional construction time and cost associated with the shear connectors installation. However, the cost advantages are found in labor savings, less material required resulting in light floor loads, and can have quick turnaround times. The advantages and disadvantages of the floor system are expanded on in Chapter 3 of this report.

In Chapter 4 the variations in the TCC floor system are discussed. The TCC floor system has a variety of possible systems due to the variations available with timber, connections, interlayer, and concrete. The most important component of the system depends on the connector of the system because it influences the structural efficiency of the system. Chapter 5 discusses recent past projects and the different TCC systems used. There are three buildings reviewed in this report and their contributions to advancing the TCC floor system.

The results of a parametric study are presented - analyzing three different spans and two different types of shear connectors for a TCC system. Chapter 6 details the design process for a TCC system with a reinforced concrete slab and a cross-laminated timber (CLT) TCC floor plate. Currently, a lack of design guidelines is available in the United States of America for the design of TCC with semi-rigid connectors. The development of standards and design methods regarding the γ-method in Eurocode 5 (EC5), EN 1995-1-1, (DIN 1994) which is referenced in the US CLT Handbook (Karacabeyli and Douglas, 2013). Chapter 7 details the results from the parametric study utilizing the design process outlined in Chapter 6. Appendix A and B have full calculations for each parametric study for this report. Chapter 8 addresses the summary of contributions from the report while also discussing possible future research.

Chapter 2 - History and Evolution

In Europe, timber-concrete composite structures slowly appeared after World War 1 and 2 due to the shortage of steel. The development of timber-concrete composite systems started with nails and steel braces to form the connection between the two materials which was patented by Muller in 1922 (Van der Linden, 1999). A second patent from 1939 uses steel Z-profiles and I-profiles as the connection system between the timber and concrete (Van der Linden, 1999). A large gap in the development of the TCC system occurred. In the 1960s, research began to increase on the system due to the development of new joint technologies and quantifying the behavior of the system through calculations (Yeoh, 2010).

In addition to new construction, TCC systems are used for renovation projects. A few of these projects are highlighted in this paragraph. In 1997, more than 10,000 m^2 of timber floors were renovated using nails spaced throughout the beam for the connection between the timber and concrete. This method was first used in a historic building in Bratislava in 1960. At the time, the cost of this renovation technique was less than half of the cost of a new floor system (Van der Linden, 1999). Other applications of TCC used in historic building renovations occurred where steel dowels were inserted in oversized holes and then bonded to the timber with glue. Residential TCC floors were used in Switzerland were post-stressed dowels and concrete notches were used for multi-story buildings (Yeoh, 2010).

In the 1990s the increase in interest of TCC systems resulted in the construction of bridges, upgrading of timber floors, and new building construction. In Finland, a surge in TCC bridges began construction. The first TCC, road-traffic bridge was completed in 1997 with a 15 m span. The system was comprised of three glulam beams of different heights connected to a concrete slab with glued-in steel rods. Two similar bridges were built near Pori and Oulu. The first one had two side spans at 13 m and a central span of 16 m. The second bridge had two spans of 11,6 m and 10.8 m. By 2000, a fourth TCC bridge was being constructed in Finland with a span of 19m using a different structural system/application of the TCC element. This bridge was built utilizing king-post truss where the post members are TCC elements connected with glued-in steel rods (Dias, 2005). The largest of these bridges, Vihantasalmi Bridge shown in Figure 2-1, is comprised of five spans glulam king-post truss bridge with king posts in the three middle spans and the side spans having laminated timber beams (Pynnonen, 1999). The bridge has 3 spans of 42 m each in the middle and two side members of 21 m. These early timber-concrete composite bridge projects were crucial in helping to develop more research for TCC for practical applications and code development.

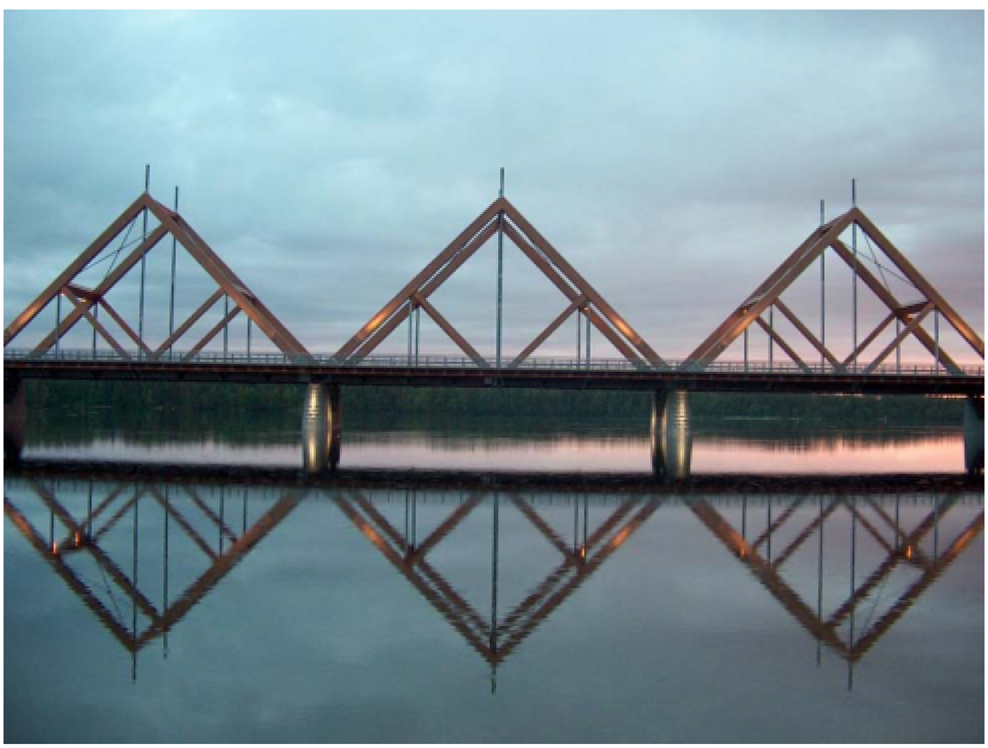

Figure 2-1: Vihantasalmi Bridge Located in Helsinki, Finland

Since the early 2000s, an interest in mass timbers and timber-concrete composites has grown due to the need to address environmental issues arising in the construction market. A growing issue in the 21st century due to the carbon footprint of the built environment, specifically the embodied energy of the materials used to construct, and the ability of the material to sequester CO_2 Timber requires a low amount of energy to produce the product and the ability to store CO_2. The development of TCC and Mass timbers, the use of large solid wood panels for wall, floor, and floor construction, in general, has been slow to adapt outside of Europe. Europe, specifically Austria, Germany, and Switzerland, started to develop mass timbers in the mid-90s which allowed more time for research and experiments (Wentzel, 2019). North America, specifically Canada, started to utilize timber construction more after 2012. An important development was made in the British Columbia Building Code in 2012 which increased the limit of light-framed wood construction from four stories to six stories. Methods to address a variety of challenges of using timber for tall buildings, taller than 10 stories, are currently being researched. The first project in North America to use a large scale application TCC system was the University of British Columbia's Earth Sciences Building (ESB) which completed construction in 2012 for a total cost of $55 million. The ESB is a five-story, 170,000 square-foot structure that effectively raised the bar for the "use of wood in large-scale, high-performance buildings" (ThinkWood, 2012).   University of Massachusetts John W. Oliver Design Building is the first large scale application TCC system in the United States which completed construction in 2017 for a total cost of $52 million. The John W. Oliver Design Building is a four-story, 87,500 square-foot structure that, "visibly demonstrates environmentally sensitive design" (WoodWorks, 2017).

These North American projects have helped propelled more research and testing of the system specifically by Skidmore, Owings & Merrill (SOM), an international design firm. SOM's ongoing Timber Tower Research Project started in May 2013 and is composed of four separate reports. The “Initial Research Report: Timber Tower Research Project” major goal was to develop a structural system for tall buildings that primarily utilizes mass timber and minimizes the embodied carbon footprint of the building compared to a prototypical building based on existing concrete. SOM’s solution utilizes the strengths of both materials, concrete, and timber, to create a concrete jointed timber frame (CJTF) system. CJTF system uses mass timber for the main structural elements with additional reinforced concrete at connection joints in the building. This system resulted in a competitive mass timber structure when compared to steel and/or reinforced concrete while reducing the carbon footprint by 60% to 75% (Skidmore, Owings & Merrill, LLP, 2013). The initial report helped to create a technically feasible building for structural engineering and other disciplines while generating interest in more research regarding mass timbers and feasible applications.

The second report of SOM’s Timber Tower Research Project, “System Report #1: Gravity Framing Development of CJTF System”, addressed the need for additional research and physical testing while providing detailed structural system information and expected behavior for a physical testing program (Skidmore, Owings & Merrill, LLP, 2014). The structural system was detailed in 5 sections of the report: gravity framing system description, design criteria, analysis model, analysis results, and design checks. The expected behavior for a physical testing program is outlined in the analysis and design of the gravity framing system along with the assumption made for design. The physical testing program was required to verify the assumption made before the system being implemented into the market. CJTF system was chosen as the first subject to expand research due to the gravity framing components represent the majority of materials used in a structure. The CJFT is the primary consideration in project cost and carbon footprint. The gravity framing system was also selected due to the untested atypical timber construction connections.

The third report, “Physical Testing Report #1: Composite Timber Floor Testing at Oregon State University”, for the Timber Tower Research Project details the results of the testing program at Oregon State University (OSU) led by SOM. The purpose of the test program was to determine how a timber-concrete composite system using cross-laminated timber (CLT) floor system improves the structural, acoustic, and fire performance of the floor system. OSU performed 20 tests on 14 full-scale specimens researching key behaviors of the TCC system, including the effectiveness of composite action, two-way bending stiffness, and continuous beam behavior. Full-scale tests were done to develop a benchmark for CLT-concrete floor panels for a design basis. The 14 full-scale composite floor panel specimens were (8) 2 ft by 10 ft 8 in with different shear connections, (3) 8 ft by 8 ft with inclined self-tapping screws, (1) 37.5 ft by 8 ft with inclined self-tapping screws, (1) 20 ft by 4 ft without connections, and (1) 13.5 ft by 4 ft without connections (Blank, 2016). The testing determined that concrete slab could be utilized to create a continuous CLT floor which enhances the two-way spanning behavior of the CLT. Overall the results show that the TCC system increased the span and two-way behavior compared to a plain CLT floor system. Some calculations were shown to support the difference between tests and calculated composite stiffness (Skidmore, Owings & Merril, LLP, 2017).

The fourth report “AISC Steel & Timber Research for High-Rise Residental Buildings”, for the Timber Tower Research Project details the investigation into structural steel frames with timber floors in high-rise residential buildings led by SOM and the American Institute of Steel Construction (AISC). The proposed structural system maximizes the advantages of each material by utilizing shallow steel framing with a composite CLT and concrete floor. The benchmark building had 8” post-tensioned concrete flat plate and with a typical 27’-6” by 32’-0” bay size allows for marketable residential units on typical floors without needing transfer column for the below-grade parking garage. The flat plate provides a flat soffit condition which is ideal for residential buildings. One critical issue was the spacing of the benchmark building having a typical bay of 27’-6” by 32’-0”. For the proposed buildings to achieve the same attributes ideal for a residential building its typical bay changed to 27’-6” by 24’-3”. The comparison between the two buildings highlighted that the proposed steel and timber structure could be viable in a high-rise residential market. This method also gave context for possible structural details and marketable bay sizes and floor openings (Skidmore, Owings & Merril, LLP, 2017).

Chapter 3 - Advantages and Disadvantages

Every structural system has advantages and disadvantages that separate itself from other systems available in the market. However, some system's disadvantages outweigh its' advantages. Certain systems are only effective for specific regions, which is often dependent on available material and skilled labor. Due to the specialty of the TCC system, early collaboration between the architectures, engineers, constructors, and owners is key.

Advantages

TCC floors are significantly light and more economical compared to reinforced concrete or steel-concrete composite floors. TCC floor systems resolve issues that are common for traditional timber floors such as deflection, vibrations, insufficient acoustics, and poor fire resistance. Three main material advantages are: an increase in stiffness, improved acoustics, and an increase in thermal mass. The increase in thermal mass reduces the energy consumption needed to heat and cool buildings. It is possible to achieve a rapid erection of the timber from prefabrication off-site and utilizing the timber as permanent formwork. Due to the lower self-weight of TCC systems, smaller foundation systems are needed. This lighter self-weight of the structure also reduces the seismic load the lateral system needs to resist.

For timber only floor systems, the TCC floor system increases the fire resistance, increased acoustical performance, and decreased vibrations. In 2002, Natterer described a variety of engineered timber structures constructed between 1974 to 2001 in Europe and determined that apart from a significant reduction, almost half, in self-weight, the fire resistance of TCC increased from 60 to 90 minutes when compared with a conventional reinforced concrete slab (Nattere, 2005). In a TCC floor system, the concrete acts as a protective cladding for the timber which reduces the effect of temperature and delays the charring of the timber. Large timber members char on the outside at a slow and predictable rate while retaining strength, slowing combustion, and allowing time to evacuate (Stone, 2013). The char protects the timber from more degradation, helping to maintain the structural integrity and reducing its fuel contribution to the fire, which in turn lessens the fire's heat and flame (Stone, 2013). However, the char from the timber provides insulation to protect the concrete and connector system against high temperatures.

The construction industry, global warming, and the economy are all intertwined. The construction industry has played an important role in the nation's economy and the global economy. In the US alone from 1999 from 2015, real, inflation-adjusted, construction investment varied from 5.1% of real gross domestic product (GDP) in 2010 and 2011 to 9.4% of GDP in 1999 (Markstein, 2017). The key building material in concrete is the cement which in turn forms the foundations and structures we live and work in. The production of cement alone accounts for approximately 5% of carbon dioxide (CO_2) emissions globally (Rubenstein, 2012). The effects of the construction industry on global warming have helped push efforts of acceptance of TCC floor systems. Timber is renewable and has a light carbon footprint unlike concrete (Stone, 2013). When compared to similar steel or concrete buildings, timber structures have the least amount of embodied energy and consume the least amount of operating energy. According to WoodWorks, embodied energy is the "energy needed to extract, process, manufacture, transport, construct and maintain a material or product" and operating energy is "energy used for heating, cooling, lighting, etc" (reThink Wood, 2015). Embodied and operating energies use nonrenewable fossil fuels, which release deleterious greenhouse gases, such as CO_2, into the environment (Clouston & Schreyer, 2008). These greenhouse gases absorb the sun's energy and redirect it back to the earth's surface which warms the earth.

Disadvantages

While many pros for the TCC system exist, some cons need to be considered when selecting the system. The primary drawback of using a TCC system is the added construction time and cost required for the shear connectors. However, depending on the project size and TCC system used, cost savings in labor can occur due to the timber being permanent forwork and speed of construction due to the prefabrication of the TCC system. Some other obstacles of using a TCC system are the lack of experience in the construction and engineering market, limited manufacturing, code limitations, and calculations.

The calculations can be very tedious due to a reiterative process needed to check all four different limit states. The design of TCC involves deflection control at the serviceability limit state (SLS), and strength control at the ultimate limit state (ULS) of the three materials: timber, concrete, and connection. Both limit states much also be check for short-term and long-term performance. The code limitations in the US are primarily created by a gap in research compared to other counties. There is no true consense among researchers for estimating the long-term performance of TCC systems (Clouston & Schreyer, 2008). The parametric study in chapter 7 compares the European Code method and the NDS 2018 method to approximate creep. The European Code method recommends creep factors developed through load duration studies to reduce the moduli of the respective material. The NDS 2018 3.5.2 addresses long-term loading and creep at the end of the design process without reducing the moduli. These obstacles will diminish with more research into design criteria and code development of mass timber buildings in general and the TCC system specifically.

The lack of experience in the market will diminish with the continued education and growth of mass timber buildings in the USA. However, a substantial problem is the lack of experienced skilled laborers regarding the TCC system construction in North America compared to overseas. One manufacturer located in German, TiComTec GmbH, worked to close the gap by sending members of the company to North America to conduct training on their TCC systems. The type of connector used in a TCC floor is a key component that drives a lot of the properties and efficiency of the floor. Currently, the majority of TCC semirigid connectors are manufactured in Europe which poses an issue of increased cost and a lack of available literature. Due to the wide variety of connectors on the market, most manufacturers in Europe provide some literature beyond the technical specification approval from the authority having jurisdiction. For example, TiComTec GmbH HBV shear connectors technical specifications are outlined in theGeneral Building Authority Approval through DIBt (Deutsches Institut fur Bautechnik) under Z-9.1-1557 (Nr. Z-9.1-1557). TiComTec also provides a technical paper on the HBV system which includes general information, different types of floor systems, previous projects, construction details, characteristics, and example calculation for acoustic, fire protection, and vibration characteristics, and technical values for the system along with examples (TiComTec GmbH, 2015).

Chapter 4 - Components of the System

Timber Concrete Composite (TCC) floors can come in a variety of systems due to the possible variations with timber, connections, interlayer, and concrete. A section cut of a possible TCC floor is shown in Figure 4-1 calling out the four different components. The two main components that separate systems from each other are the timber and connection type. The interlayer is often formwork, insulation, or damming and can be conducted regarding the physical building requirements for flooring such as moisture protection, heat protection, and fire protection. Insulation is recommended for soundproofing and thermal mass but is not always used.

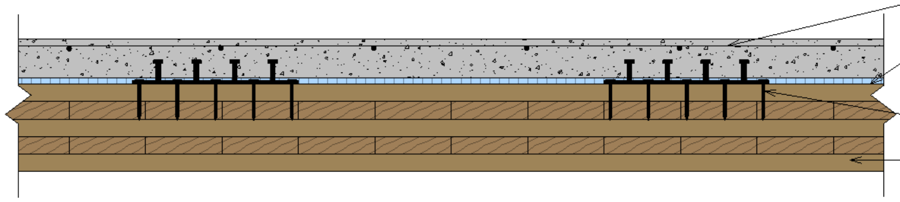

Figure 4-1: Timber-concrete composite section

Concrete

Multiple investigations have been done to address the influence of concrete properties on the timber-concrete composite connections. Push-out connection test using concrete with a density of 1.6kN/m^3 (10.3 lb/ft^3), compared to normal concrete with 2.4 kN/m^3 (15.4 lb/ft^3) density, to reduce the permanent load (Steinberg, Selle, & Faust, 2003). This report concluded that timber-lightweight concrete composite systems are affected by the modulus of elasticity of the lightweight concrete, which leads to a lower effective bending stiffness. Therefore, the connectors must be positioned at a closer spacing in lightweight concrete compared to normal weight concrete. Consequently, the design relies on the compromise between the higher cost due to lightweight concrete and the connectors being set closer, and the reduction in permanent load. In 2008 it was determined that a higher grade of lightweight concrete was recommended to fully utilize the efficiency of the concrete (Koh, Mohamad Diah, Lee, & Yeoh, 2008). This conclusion came from 12 push-out tests that were done on specimens made of lightweight foamed concrete and Malaysian hardwood connected using different types of nails done by Koh et al.

A report regarding the restoration of existing timber frame buildings using timber-concrete composites was conducted. A full-scale long-term and collapse tests were done for an SFS inclined connectors and lightweight concrete comprised of recycled sewage sludge resulting in a density of 1,760 kg/m^3 (109.8 lbs/ft^3) used on an existing timber floor (Grantham, Enjily, Fragiacomo, Nogarol, Zidaric & Amadio, 2004). The results from these tests highlighted the larger sensitivity of light-weight concrete to rheological phenomena compared to normal-weight concrete, but also highlighted the favorable lower self-weight and high strength. However, tests conducted by Fragiacomo et al. in 2007 contradicted Grantham et al. results (Fragiacomo, Amadio, & Macorini, 2007a). These tests used a head stud proprietary connector in which half of the specimens used normal-weight concrete and the remain used lightweight concrete. The variation between lightweight concrete and normal-weight concrete was found to not significantly affect the performance of the connection from both the long-term and short-term collapse tests because the governing failure for both cases was the timber.

Changing the thickness of the concrete slab which will affect the self-weight of the slab was investigated using steel-fiber-reinforced concrete with wood screws as connectors (Holschemacher, Klotz, & Weibe, 2002). The test slabs were a conventional reinforced 60 mm (2.36 inches) thick concrete slab with a minimum clear cover of 20 mm (0.79 inches) and a 48 mm (1.89 inches) with steel-fiber-reinforced concrete. The push-out tests show that the connection strength increased 1.3 times and the initial stiffness increased 2.8 times compared to the use of normal reinforced concrete.

For a notch cut in the timber case the shrinkage of concrete during the early days from the time of curing results in a gap at the outer edge of the connection. As the concrete shrinks, the notched connection pushed inward, causing an unwanted initial permanent deflection of the composite beam, specifically in a case of a very stiff connection. It was determined that to prevent this issue, the use of low-shrinkage concrete is recommended (Yeoh, Fragiacomo, Buchanan, Crews, Haskell, & Deam, 2008).

Interlayer

The interlayer is often formwork, insulation, or damming and can be conducted regarding the physical building requirements for flooring such as moisture protection, heat protection, and fire protection (TiComTec GmbH, 2015). Plywood or particleboard is used as formwork between timber beams simulating flooring which is normally used for refurbishing purposes. Insulation is recommended for soundproofing and thermal mass but is not always used. A rigid insulation layer between the timber and concrete can increase the static moment arm between the two elements without significantly increasing the weight (Hong, 2017). Due to the increase in the static moment, it is possible to increase the stiffness and the vibration performance if the shear connector connects effectively through the insulation. The use of a plastic separation layer is important to prevent moisture from the concrete and minimize the influence of friction between the two elements in calculating the stiffness properties of the connector. Protecting the timber from the moisture in concrete can also be done using concrete additives that reduce water/cement ratio which also reduces concrete shrinkage (Ceccotti, 2002).

The interlayer influences the mechanical performance of the connection in the composite system. Van der Liden and Dias looked at the influence of the interlayer qualitatively (Van der Linden, 1999; Dias, 2005). The results from different studies on both, the load-carrying capacity and stiffness of connections, show a decrease of the mechanical properties load carrying capacity and stiffness in connections comparing with or without an interlayer (Dias, Schanzlin & Dietsch, 2015). In Table 4-1 the ratios between model and experimental results regarding a decrease in load carrying capacity and connection stiffness. The results of these studies showed the high influence that the interlayer has on the mechanical performance of connections, especially stiffness. (Dias et al, 2015).

CONNECTION TYPE	LOAD CARRYING CAPACITY	CONNECTION STIFFNESS	REFERENCE
NAILS	13%	27%	Dias, 1999
INCLINED SCREWS	30%	50%	Van der Linden, 1999
NOTCHES COMBINED W/ DOWELS	30%	22%	Van der Linden, 1999
DOWELS	8%	35%	Dias, 2005
NOTCHES	16%	34%	Dias, 2005

Table 4-1: Data on the Influence of Interlayer (Dias et al, 2015)

Connections

In a TCC system, the connectors are usually positioned along the beam according to the shear force distribution so that they are concentrated near the supports where the internal shear force is high and spaced out gradually into the span as the shear force is reduced. The structural efficiency of a TCC floor system greatly relies on the stiffness of the connection. A connection that creates high composite action allows for a significant reduction to the depth of the beam while increasing the span length. Load capacity, slip modulus, ductility, and overall cost are the four main characteristics reviewed for any type of connector (Bahmer & Hock, 2015). Ceccotti presented a large number of fasteners possible for TCC and sorted them base on their stiffness or slip modulus (Ceccotti, 1995). Nails, screws, and dowels are the most flexible compared to notched timber and continuously glued connectors which are the most rigid. Figure 4-4 provides a comparison of the shear force-slip relationship for the different categories of connection for TCC systems. The shear strength and stiffness are obtained through push-out tests, per EN 26891, to characterize a connection (CEN 1991). The strength of a connection is quantified as the maximum load applied when the failure occurs during the push-out test while stiffness is quantified by the slip modulus at 40%, 60%, and 80% of the mean maximum load.

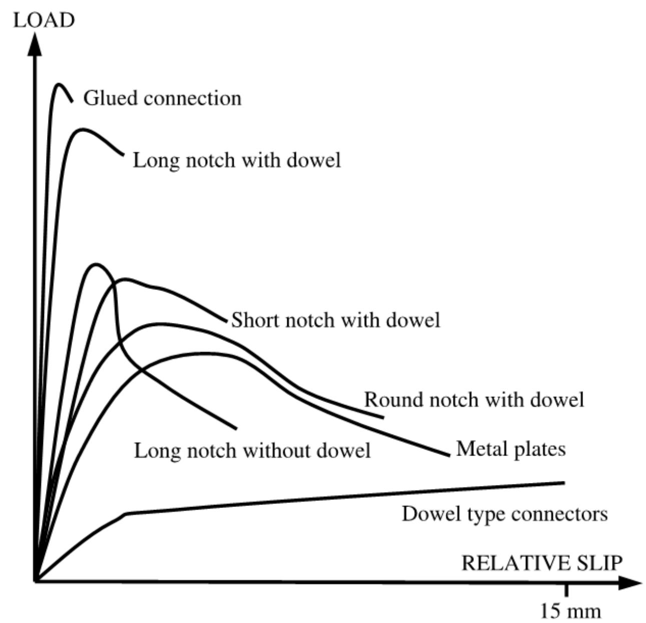

Figure 4-2: Comparisons of different categories of connection systems (Dias, 2005)

A large variety of shear connection systems are continued to be studied in different parts of the world. The connectors can be timber or metal fasteners, or notches cut in the timber and filled by concrete. Due to the large variety, the connection systems can be categorized based on installation and arrangement along the timber: discrete/continuous, vertical/inclined, glued/non-glued, and prestressed/non-prestressed (Yeoh, Fragiacomo, De Franceschi & Buchanan, 2011). Figure 4-3 is based on Ceccotti (2002) grouping fasteners based on their degree of rigidity. Groups A, B, and C have partially composite action and are ordered from low to high stiffness –

the stiffer the connection, the more composite action occurs in the system. Group D connectors are the stiffest and have full composite action. Ceccotti sorted connectors on their stiffness with the most flexible connector as (A) elements connected by nails, screws, or dowel shaped fasteners, (B) elements connected by surface connectors, (C) notches are cut from the timber which increases rigidity, or (D) continuous connectors glued into timber (Ceccotti, 1995).

Research on connection systems can be traced back to as early as the 1940s (McCullough 1943; Richart and Williams 1943) and 1970s (Pincus 1970; Pillai and Ramakrishnan 1977). Notches cut in the timber with steel screw or dowel as shown in Figure aa as category C is by far the best connection for TCC concerning strength and stiffness performance although it may not be economical if the notches have to be cut manually. The length of the notch, the presence of a lag screw, and the depth of penetration into the timber are factors that affect the performance of the connection. The notch length affects the strength stiffness and strength of the connection while the lag screw provides ductility and improved the post-break behavior (Yeoh, 2010). However mechanical connectors, such as nail plates, that do not require any cutting in the timber were found to be efficient in strength and stiffness although significantly less than a notched connection. The main difference between mechanical and notch connection is that in the first case the slip modulus largely depends about the fasteners flexibility and the timber in contact with the fastener, while notch connection the slip modulus mostly depends on the stiffness of the timber in the inclined surface of the notch and the stiffness of the concrete (Balogh and Gutkowski 2008; Kuhlmann and Michelfelder, 2006). Inclining the dowels to a 45-degree angle improves the mechanical properties because the connection is subjected primarily to axial force instead of shear. The use of glue and epoxy resin in a connection system is not entirely encouraged because of the tight quality control and complexity of an on-site application.

However, more research has been conducted regarding glue and epoxy with prefabrication (Clouston et al., 2005).

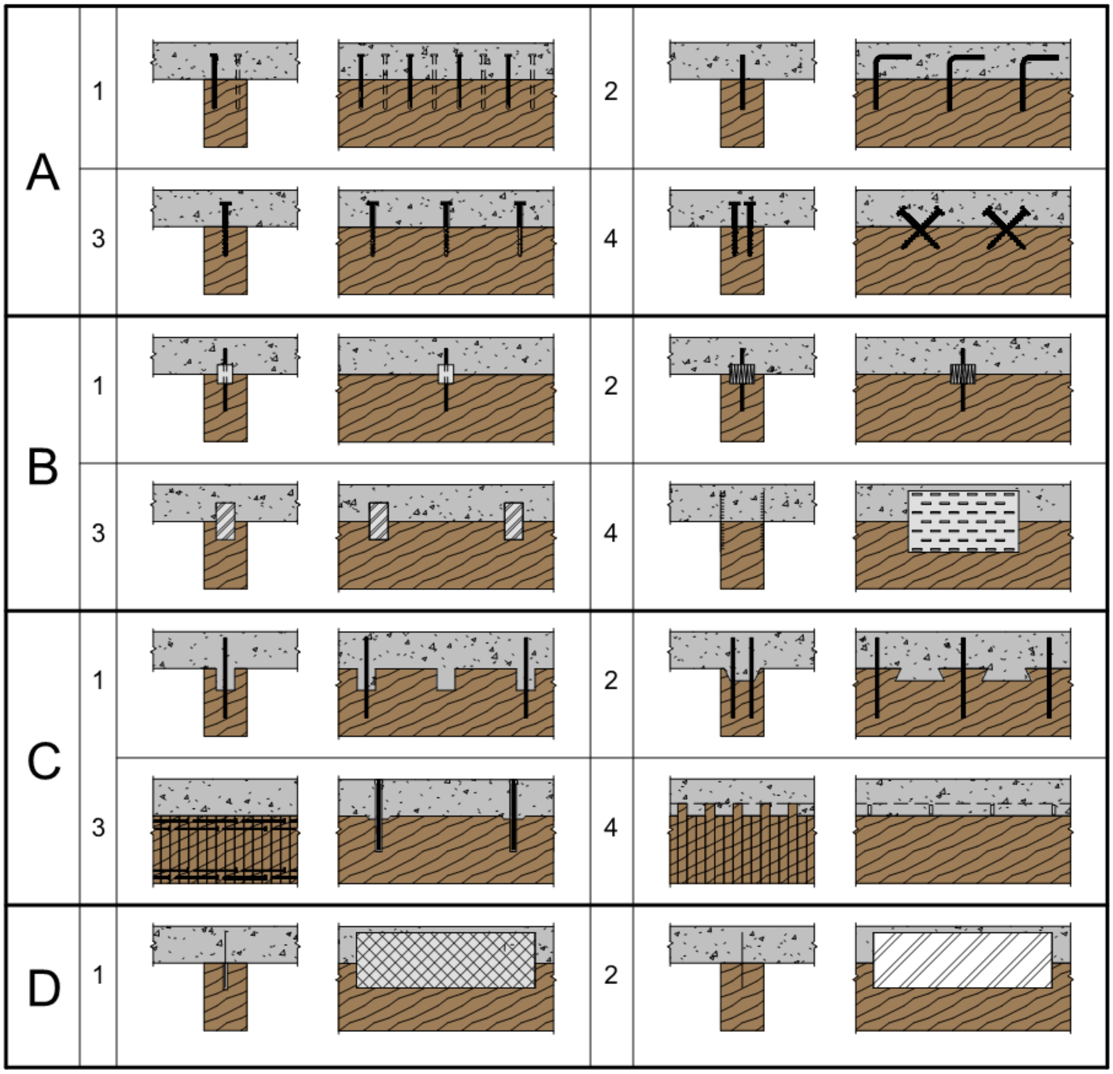

Figure 4-3: Examples of timber-concrete interlayer connections

Detailed in Figure 4-3: (A1) Nails; (A2) glued reinforced concrete steel bars; (A3/4) Screws; (B1/2) split rings and toothed plates; (B3) steel tubes; (B4) steel punched metal plates; (C1) round indentations in timber and fasteners preventing uplift; (C2) square indentation and fasteners; (C3) cup indentations and prestressed steel bars; (C4) nailed timber planks deck and steel shear plates slotted through deeper planks; (D1) steel lattice glued to timber; (D2) steel plate glued to timber (Ceccotti, 1995).

This report looks into two different connectors for a TCC system: self-taping screws (STS) and HBV steel mesh connector. The most common connectors are screws, nails, flat steel locks, and shear studs. Recently the HBV-shear connector, shown in Figure 4-3, has gained popularity because it is the only connector to create a wide area connection which makes it a standout in many connection characteristics. The STS selected for the report are ASSY plus VG screws as shown in the section cut in Figure 4-3.

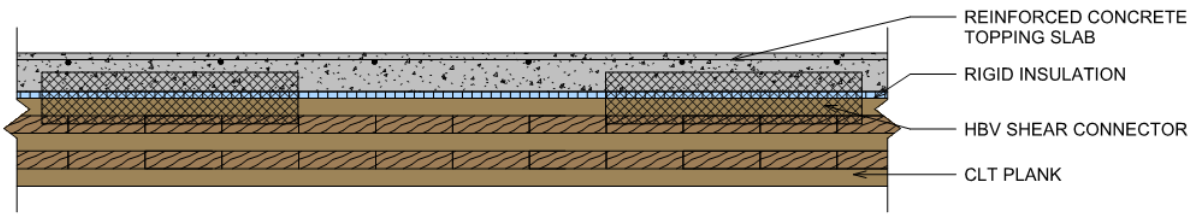

Figure 4-4: HBV Shear Plate Section

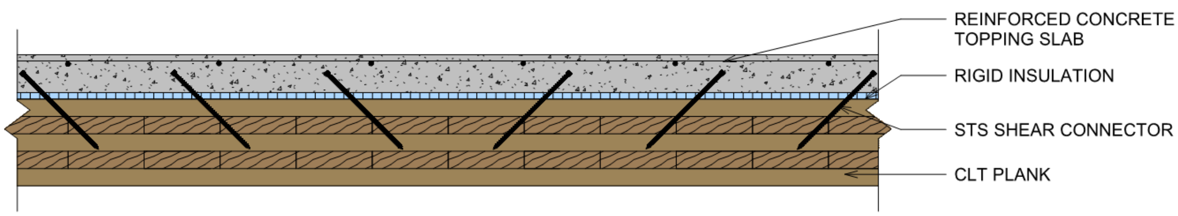

Figure 4-5: STS TCC Section

Self-taping screws (STS) are the most common connection available due to their lower cost while still being very efficient for TCC connections. STS was developed to increase the load-carrying capacity of traditional screws to advance large scale timber projects (Hong, 2017). STS is more common throughout the world than other TCC connectors due to ease of production. STS can be arranged in a variety of ways involving angles and multiple screws. STS angles typically range between 30° to 90° to the wood grain. There is also a popular arrangement where two screws are placed between 30° to 45° crosswise which helps to transfer tension and compression forces and ultimately provides a slightly higher capacity than single STS (Closen, 2012).

The HBV shear connector from TiComTec GmbH creates a wide area connection which corresponds to slip modulus, which describes the efficiency of a connector, which is high compared to other connectors (Bahmer & Hock, 2015). In 1992, Leander Bathon started what would evolve into the HBV System by embedding hollow cambered steel parts into the wood to bond the timber to other building materials. By 1999 Bathon was able to implement the system in its first pilot project in a floor renovation. Many trials with different variations of steel plates and adhesives for the system were tested in the years 2000 to 2002. These tests highlighted the optimum tuning of the connectors in elastic-plasticity for the load-bearing of the composite system along with the applicability of the adhesives (TiComTec GmbH, 2015). One of the main concerns with the HBV shear connector is the special adhesive which requires a professional to carry out. however, this requirement accelerates the construction because the timber and shear components are made in a manufacturing plant with a high level of pre-fabrication (TiComTec GmbH, 2015).

Timber

The timber used for TCC floors is often sawn lumber, glue-laminated timber, composite lumber, such as nail laminated timber (NLT), laminated veneer lumber (LVL), or cross-laminated timber (CLT). Sawn lumber and glulam timber are often used in beam floor style TCC where plywood spans between the beams. The beam floor system is primarily used for restoring and upgrading historical structures. While NLT, LVL, and CLT are used for plate floor style TCC which allows for large size building elements to be installed quickly in panel form.

The beam floor system was one of the first TCC to enter the market because it resembled characteristics in a steel-concrete composite floor with small vibration sensitivity and good sound insulation (TiComTec, 2015). The beam floor typically spans 6.5 m (21.3 feet) with 140 mm (5.5 inches) timber and 120 mm (4.72 inches) concrete (Dias et al, 2015). Plywood with or without insulation is typically used for the interlayer and becomes permanent formwork. Temporary shoring is generally used to support the weight of uncured concrete until composite action is achieved (Clouston & Schreyer, 2008). Figure 4-6 shows one possible configuration of the components for a simple beam floor system.

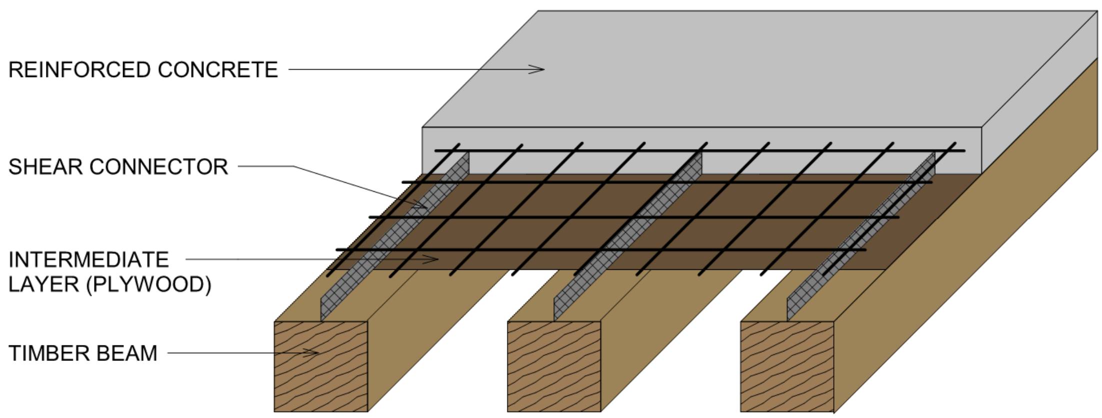

Figure 4-6: Beam floor system

The Plate floor system has become popular for new construction due to its ability to carry higher static loads, easy constructability with pre-fabrication, beautiful finish due to the finished underside, good vibration, and sound properties (TiComTec GmbH, 2015). The plate system accelerates the on-site construction by increasing the prefabricated panel size with shop-fabricated connectors provided before shipment to the site. Compared to the beam floor that typically spans 6.5 m (21 feet) a plate floor can span 15 m (49 feet) or more (TiComTec GmbH, 2015). Figure 4-7 shows one possible configuration for a plate floor system with engineered timber products for the plate, referring to the large-sized timber element.

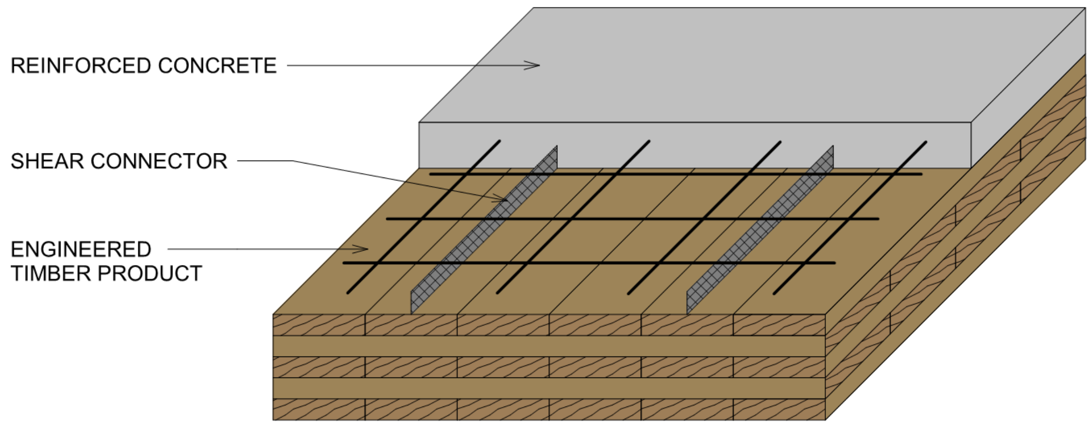

Figure 4-7: Plate floor system

Chapter 5 - Past Projects

The timber concrete composite floor system originated in Europe and gained increasing popularity in many countries before it was accepted in North America. The TCC system developed primarily from the lack of steel available from World War I and II. In the early 1930s, TCC systems were commonly used for bridge decks in North America. Eventually, TCC fell out of favor for steel and concrete composite flooring, until recent research and development efforts started in the early 1990s. The first large scale TCC project in the United States completed construction in 2017 in Massachusetts.

University of Massachusetts

The John W. Oliver Design Building at the University of Massachusetts Amherst was the first project in the USA to utilize a wood-concrete composite floor system. Construction for the project was completed in January 2017 by construction manager Suffolk based in Boston, Massachusetts. The building is a four-story, 87,500 square-foot structure with a glued-laminated timber (glulam) columns and a beam frame, glulam braced frame, cross-laminated timber (CLT) shear walls, timber-concrete composite floor system, and unconventional cantilevered forms. According to WoodWorks Carbon Calculator, the advanced mass timber building has the total potential carbon benefit of 2,532 metric tons of CO_2 which is equivalent to 535 cars off the road for a year or energy to operate 267 homes for a year (Lathe, 2017).

Leers Weinzapfel Associates were the architecture firm responsible for the building. Equilibrium Consulting and Simpson, Gumpertz, & Heger (SGH) were the two structural engineering firms that worked together for the project. Equilibrium Consulting is well known for their innovative timber design and were responsible for structural calculations and drawings for the project. As the structural engineer of record, SGH reviewed and sealed all construction documents for the project and served an administrative role with quality control and installation review. The building was designed for both steel and mass timber but two professors, Peggi Closton and Alex Schreyer, lead the initiative for mass timber over the steel design. The gravity framing system spans 24 feet with common glulam floor beam sizes of 14-1/4”x15” or 16-1/2” deep. Glulam columns ranged from 14-1/4”x22-1/2” to 25-1/2” deep. The TCC floor system used 5-ply CLT panels with 1 inch of rigid insulation by approximately 4 inches of reinforced concrete (Lathe, 2017). The design building is the largest installation of TCC in North America. In figure 5-1 below the TCC CLT floor panel with the shear connectors already in place are shown being installed.

Figure 5-1: Installation of CLT Panels (Lathe, 2017)

Earth Science Building University of British Columbia

The Earth Sciences Building at the University of British Columbia (UBC) is a five-story building located in Vancouver, British Columbia completed in 2012 by general contractor Bird Construction. The academic wing of the UBC Earth Science Building was the first building in Canada to specify such a large solid timber panel system. The laboratory wing of the building has a larger amount of exposed concrete due to the requirements for laboratories and their machinery. Tieing the two wings together with different timber elements helped to provide warmth that complemented the urban surrounding. Several milestones were reached during the project, including the application of the laminated veneer lumber (LVL) with HBV shear connector composite floor system, CLT, and a variety of novel connection details (Think Wood, 2012).

The architect for this project was Perkins+Will headquartered out of Chicago, Illinois. This project was another Equilibrium Consulting that preceded the University of Massachusetts project. The HBV System composite floor consists of 89 mm thick LSL panel, foamed board insulation, and 100 mm of reinforced concrete was designed to resist a live load of 65 psf to 100 psf in some areas. This floor assembly is over 50% lighter than a solid concrete floor structure. The average panels used were 22 feet long and were supported by steel beams on the first floor and glulam beams on the second through the fourth floor. Glulamanated beams were used in most of the building. The lateral system is comprised of a concrete core and glulam chevron braced frames shown in figure 5-2. the full story transfer truss The project is estimated that by selection timber for the academic wing approximately 1,094 metric tons of CO_2 were stored in the timber which is equivalent to 200 passenger vehicles in one year off the road. The Earth Science Building showcased collaborative design and a commitment to environmental efforts using natural resources in Canada.

Figure 5-2: Installation of CLT floor with HBV shear connectors (ThinkWood, 2012)

LifeCycle Tower One

The LifeCycle Tower One (LCT ONE) is an eight-story wood-concrete office building located in Austria. The development and project management for the building built in 2012 was Cree GmbH which has a partner company located in North America of Cree Buildings. The structural engineer and architect for the project was Arup. LCT ONE came about primarily through researching options for the future of the industry. Ultimately a high-performance hybrid prefabricated wood/concrete building system emerged because it is CO_2 neutral and has a low impact on finite resources. The construction process was very quick lasting only nine months start to finish due to a large number of prefabricated members. The hybrid concrete-timber slabs were produced in a precast concrete shop after the glulam beams were cut and their shear connectors attached. While the foundation and core of the building were being built on-site, the wall components and composite slab were being built off-site. Five carpenters took a total of eight days to assemble eight floors once the core was finished (Tahan, 2013). Figure 5-3 below shows the carpenters installing the prefabricated members to their water- and airtight compliance.

Figure 5-3: Installation of prefabricated TCC floor

LCT building system was developed using a core and shell as the structural system along with enclosing tall timber building. Nabih Tahan best described the building as the "intel inside of a computer" because of its hidden operating system due to the core and shell of the LCT system (Tahan, 2014). The floor system is a wood-concrete composite slab which is approximately 30 feet long and 10 feet wide. The glulam posts support the exterior end of the slabs. The interior side of the slab is supported by the core (Tahan, 2013). Hinged connections are used to transfer lateral forces from the posts to the slabs. The glulam posts are exposed on the interior and required an increase in size for fire protection (Tahan, 2014). The composite slabs were tested and passed a two-hour fire rating test also with providing built-in fire separation between floors because there is no wood-to-wood contact between each floor (Tahan, 2014).

Chapter 6 - Design Method

Standards and Design Methods

The analysis for TCC slabs follows the γ-method which is for semirigid connectors prescribed in the European Standard for Timber Design, Eurocode 5 Appendix B (DIN 1994). No design guidelines are currently available in the USA for TCC with semirigid connectors, but some information using transformed sections is available. Some suggest using the method of transformed sections (Stalnaker and Harris 1997; Gurfinkel 1973) which is done for steel-concrete composites, but this method is only valid for TCC because the interlayer shear connection is not usually fully rigid (Clouston & Schreyer, 2008). Therefore the transformed sections method is nonconservative for partially composite sections. Design equations for semirigid connectors are in the European Standard for Timber Design, Eurocode 5 Appendix B (DIN 1994). The γ-method assumes that in the derivation of these equations that for linear elastic behavior, Bernoulli's theory of elementary mechanics does not apply to the cross-section as a whole, but is valid for each separate component (Clouston& Schreyer, 2008). Detailed in figure 6- 1 is the stress field which is the algebraic summation of internal normal and bending stresses for each component.

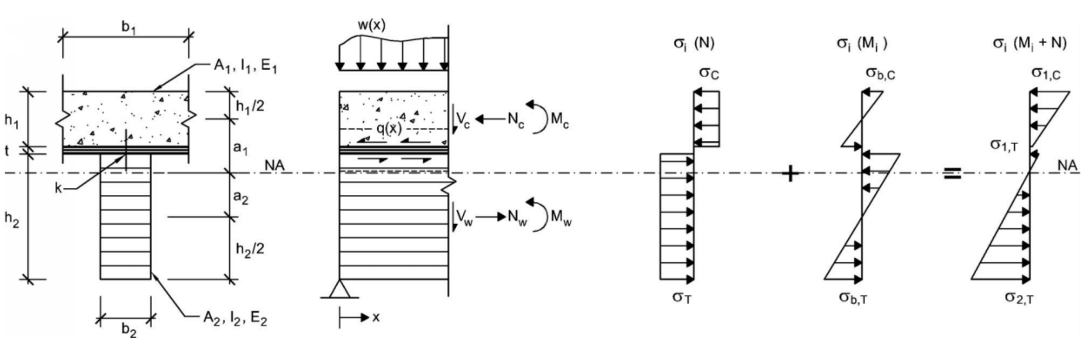

Figure 6-1: Stress field of TCC with semirigid connectors (Clouston et al., 2008)

Four calculations are performed for a section for two different limit states, Serviceability Limit State (SLS) and Ultimate Limit State (ULS). For each limit state, short-term and long-term analysis is performed. ULS is used for strength analysis while SLS is used for serviceability analysis.

Analysis Method for TCC System with CLT

Determine Timber and Concrete Properties

For the concrete properties, the modulus of elasticity (E_1) and specified compression strength (f_c') are required for the design. For the different timbers required properties are modulus of elasticity (E_2), tension parallel to grain (F_t), bending strength (F_b), and shear strength (F_v). Depending on the type of timber used in a project the required properties in both directions will be needed.

Calculate CLT Properties

For CLT floors using USA CLT Handbook, in Chapter 3, the basic engineering mechanics are given for the calculations. The width of each panel, b_i, is assumed to be 12 inches. the height of each layer, h_i, is given in Structurlam product documents. Using PRG 320 tables, the transverse lamination modulus of elasticity, *E*, are assumed to be E/30. The change in transverse layers modus is to adjust for bending perpendicular to the strong axis, The parallel axis theorem is used for the calculation where z_i is the distance between the centroid of each layer and the neutral axis.

$$EI_{eff,CLT} = \sum_{i=1}^{n} E_i b_i \frac{{h_i}^3}{12} + \sum_{i=1}^{n} E_i A_i {z_i}^2 \qquad (Equation\ 6-1)$$

The effective shear stiffness is calculated using many of the same values the effective shear stiffness can be calculated. The distance *a* is the distance from the centroid of the top layer of wood to the centroid to the bottom layer of wood. From PRG 320, the longitudinal shear stiffness, G, of the lamination is assumed to be E/16 and the transverse shear stiffness, G, is assumed to be longitudinal G/10. The change in the transverse layers shear stiffness is due to rolling shear.

$$GA_{eff,CLT} = \frac{a^2}{\left[\left(\frac{h_1}{2G_1 b}\right) + \left(\sum_{i=2}^{n-1} \frac{h_i}{G_i b_i}\right) + \left(\frac{h_n}{2G_n b}\right)\right]} \qquad (Equation\ 6-2)$$

The apparent bending stiffness, *EI*, is found by reducing the effective bending stiffness. The shear deformation adjustment factor K_s can be found in NDS 2018 Table C10.4.1.1, which is based on specific loading and end-fixity.

$$EI_{app,CLT} = \frac{EI_{eff,cLT}}{1 + \frac{K_s EI_{eff,CLT}}{GA_{eff,CLT} L^2}} \qquad (Equation\ 6-3)$$

Determine the Composite Fastener Properties

Two different composite fasteners are presented. For each of the shear connector types, determine the equivalent slip modulus, K_{ser}, and shear strength, *Tk,* or *Frk* from the manufacturer's literature or technical approvals.

HBV Composite Fastener

HBV shear connectors come in three heights: 90 mm (3.54 inches), 105 mm (4.13 inches), and 120mm (4.72 inches). From TiComTec the calculations for the HBV-System are detailed in the "Technical Dossier HBV-Systems 2014-05". For the shifting modulus per mm from HBV guide. Where d_{zs} is the thickness of the interlayer in mm, d_0 is equal to 1mm and *Zul* means acceptable. The slip modulus, K_{ser}, is calculated as

$$K_{ser} = 825 - 250(d_{zs}/d_0)^{0.2} \qquad (Equation\ 6-4)$$

The acceptable shear load is calculated as

$$zul,T = 90 - 4.5(d_{zs}/d_0)^{0.5} \qquad (Equation\ 6-5)$$

The characteristic value of the shear load bearing is calculated as

$$T_k = 160 - 8(d_{zs}/d_0)^{0.5} \qquad (Equation\ 6-6)$$

The rated value of the load-bearing capacity is calculated as

$$T_d = T_k/1.25 \approx 1.42\ zul.T \qquad (Equation\ 6-7)$$

ASSY plus VG Screw

The ASSY plus VG screw approach is based on a similar approach to the HBV system. European Technical Assessment no. ETA-13/0029 is used for ASSY plus VG screw from Wurth Gmbh & Co (ETA-13/0029).

For a screw placed between 30° and 45°, characteristic load-carrying capacity, F_{Rk}, for ASSY plug VG screw compares tensile capacity, $f_{tens,k}$, and characteristic withdrawal capacity, $F_{ax,\alpha,Rk}$, for ASSY plus VG screw is calculated as

$$F_{Rk} = (\cos(\alpha) + \mu\sin(\alpha)) * min\begin{cases} F_{ax,\alpha.Rk} \\ f_{tens,k} \end{cases} \qquad (Equation\ 6-8)$$

Where α is the angle of a screw, μ is the coefficient of friction, $f_{tens,k}$ is the tensile capacity parameter from Table 2.4, and the characteristic withdrawal capacity, $F_{ax,\alpha,Rk}$, is calculated as

$$F_{ax,\alpha,Rk} = \frac{f_{ax,k} d l_{ef}}{1.2\cos(\alpha)^2 + \sin(\alpha)^2}\left(\frac{\rho_k}{350}\right)^{0.8} \qquad (Equation\ 6-9)$$

Where $f_{ax,k}$ is the withdrawal parameter in Table 2.4, d is the diameter of the screw, l_{ef} is the effective penetration depth into the timber member, α angle of the screw, and ρk is the characteristic timber member density. l_{ef} is limited to 110 mm for 8 mm screws and 170 mm for 10 mm screws. The coefficient of friction, μ, is 0.25 for direct contact between timber and concrete otherwise μ is 0.

For the slip modulus, K_{ser}, is calculated using Table 2.2 in ETA-13/0029 where l_{ef} is the effective penetration depth of the screw into the timber. For ultimate service load the ultimate slip modulus, K_u, is calculated as (2/3)K_{ser}.

Calculate the Effective Bending Stiffness

The effective bending stiffness can be calculated using the γ-method found in European Standard for Timber Design, Eurocode 5, Part 1, Annex B (DIN, 1994). Effective bending stiffness EI_{eff} is calculated as

$$EI_{eff} = \sum_{i=1}^{3} (E_i I_i + \gamma_i E_i A_i a_i^2) \qquad (Equation\ 6-10)$$

Where subscripts i =1 and i = 2 refer to the respective components of CLT or concrete, E is the modulus of elasticity, I is the moment of inertia, A is the cross-sectional area, a is the distance from the centroid of the respective component to the neutral axis, and γ is a dimension-less shear connection reduction factor.

The value of γ_1 ranges between 0, no composite action, and 1, full composite action and is calculated as

$$\gamma_i = \left[\frac{1 + \pi^2 E_i A_i s_i}{K_i {L_0}^2}\right]^{-1} \qquad (Equation\ 6-11)$$

Where s is the spacing of connectors, K is slip modulus and L is the beam span. The slip modulus, K, is determined as the slope of the load/slip curve from experimental tests from manufactures literature. The value for γ_2 is always 1.

The distance between the centroid of the timber members and the overall natural axis, a_2, is dependent on the shear reduction factor, γ_1 and is calculated as

$$a_2 = \frac{\gamma_1 E_1 A_1 (h_1 + h_2)}{2 \sum_{i=1}^{2} \gamma_i E_i A_i} \qquad (Equation\ 6-12)$$

Where h_1 and h_2 are the height of their respective component. The distance a1 is determined from the simple geometry of the composite beam.

Calculate the Apparent Stiffness of the Floor System

Calculate the apparent stiffness of the floor system per USA CLT Handbook, Chapter 3, or develop a rolling shear model to consider the stiffness losses due to the rolling shear behavior. The apparent bending stiffness is found by reducing the effective bending stiffness. The shear deformation adjustment factor K_s can be found in NDS 2018 table C10.4.1.1, which is based on specific loading and end-fixity.

$$EI_{app,comp} = \frac{EI_{eff,comp}}{1 + \frac{K_s EI_{eff,comp}}{GA_{eff,CLT} L^2}} \qquad (Equation\ 6-13)$$

Calculate the Normal Stresses

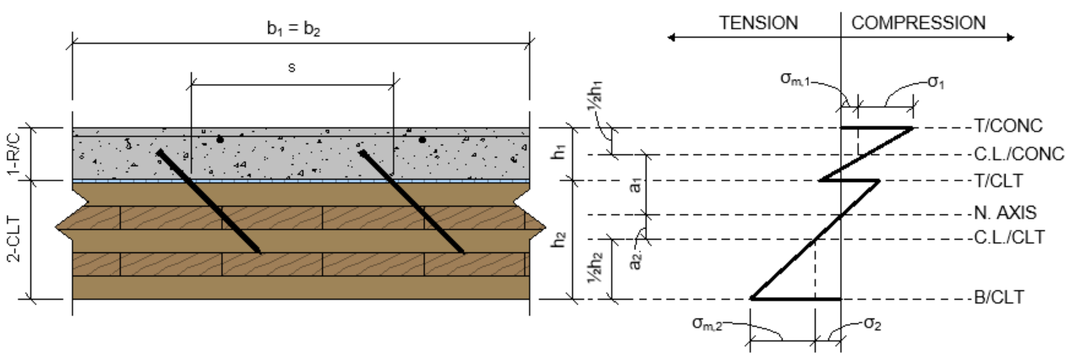

Figure 6-2: Cross-Section and Stress Diagram

The two equations below make up the various stresses distributed within the TCC section. Where σ_i is the normal stress for the respective component and $\sigma_{m,i}$ is the maximum stress for the respective component.

$$\sigma_i = \frac{\gamma_i E_i a_i M}{EI_{eff}} \qquad (Equation\ 6-14)$$

$$\sigma_{m,i} = \frac{E_i h_i M}{2EI_{eff}} \qquad (Equation\ 6-15)$$

For stress at top of concrete: $\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M}{EI_{eff}} + \frac{E_1 h_1 M}{2EI_{eff}}$ $\quad (Equation\ 6-16)$

For stress at bottom of concrete: $\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M}{EI_{eff}} - \frac{E_1 h_1 M}{2EI_{eff}}$ $\quad (Equation\ 6-17)$

For stress at top of timber: $\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M}{EI_{eff}} + \frac{E_2 h_2 M}{2EI_{eff}}$ $\quad (Equation\ 6-18)$

For stress at bottom of timber: $\sigma_{b,2} = -\sigma_2 - \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M}{EI_{eff}} - \frac{E_2 h_2 M}{2EI_{eff}}$ $(Equation\ 6-19)$

Compare Calculated Stresses to Allowable Code Limits

Wood Tensile Failure

For combined bending and tension a linear interaction formula is utilized:

$$\frac{\sigma_2}{F_t'} + \frac{\sigma_{m,2}}{F_b'} \leq 1.0 \qquad (Equation\ 6-20)$$

Where F'_t is the allowable parallel-to-grain tensile stress, F'_b is allowable bending tensile stress, σ_2 is tensile stress in timber due to the force couple in the composite section, and $\sigma_{b,t}$ is the maximum tensile stress in the timber due to the force couple about the wood section.

Wood Shear Failure

The maximum beam shear stress, $\tau_{2,max}$ occurs at the neutral axis of the timber component where, h = a_2 +(h_2/2), V is the applied shear force, and can be calculated as

$$\tau_{2,max} = \frac{E_2 h^2 V}{EI_{eff}} \qquad (Equation\ 6-21)$$

The maximum shear stress in the timber $\tau_{2,max}$ is compared to the allowable shear stress, F'_v with appropriate adjustment factors applied.

$$\tau_{2,max} \leq F_v' \qquad (Equation\ 6-22)$$

Concrete Compressive Failure

The maximum compressive stress in the concrete, $\sigma_{t,1}$ is compared against the allowable concrete compressive strength, F_c is assumed to be one half the specified compressive strength f'_c (Nilson et al,2004).

$$\sigma_{t,1} \leq F_c \qquad (Equation\ 6-23)$$

Concrete Tensile Failure

The maximum stress in the concrete is calculated using normal stress equations where compression governs. Therefore, the concrete slab is subjected to compression stresses exclusively and tension failure is not considered.

Connector Shear Failure

The maximum shear flow, F_q, in the connector can be computed as

$$F_q = \frac{\gamma_1 E_1 A_1 a_1 s V}{EI_{eff}} \qquad (Equation\ 6-24)$$

The maximum shear force in the connector, F_q, is checked against the allowable shear capacity of the connector, T_k or F_{Rk} or F_{ult}, depending on the manufacture.

$$F_q \leq F_{ult} \qquad (Equation\ 6-25)$$

Evaluate the Floor System for Deflections

For the deflections, initial and long-term deflections were considered. There is no information on TCC long-term modifiers in United States code requirements but there is some in the European Code on long-term modifiers for TCC systems. The European Code has the specific modification factor applied early on for the timber, concrete, and connections while in the NDS a long term modifier is applied later on. The short-term serviceability for deflection can be calculated by

$$\Delta_{LL} = \frac{5w_{LL}L^4}{384EI_{eff}} \qquad (Equation\ 6-26)$$

Long-term deflection is calculated as

$$\Delta_{TL} = K_{cr}\Delta_{LT} + \Delta_{ST} \qquad (Equation\ 6-27)$$

Where Δ_{LT} is immediate deflection due to the long-term component of the design, Δ_{ST} is deflection due to the short-term or normal component of the design load, and K_{cr} is a time-dependent deformation (creep) factor with different cases described in Chapter 3.5 of the NDS.

Calculate Connector Efficiency

The structural efficiency of a TCC system depends on the connection. A highly composite connection allows for a significant reduction of beam depth and longer span length. The equation below estimates the efficiency of the composite structure utilizing the live load deflection at mid-span where NC, PC, and FC refer to no, partial, and fully composite action.

$$Eff = \frac{\Delta_{NC} - \Delta_{PC}}{\Delta_{NC} - \Delta_{FC}} \qquad (Equation\ 6-28)$$

Chapter 7 - Parametric Study

A parametric study was conducted to compare the effectiveness of two different connectors for three different span lengths for a timber-concrete composite slab. This parametric study compared spans of 22 feet, 24 feet, and 26 feet. These calculations were done do address the ultimate limit state and serviceability limit state for the short-term and long-term. The detail of calculations can be used to expand the study to create a finite element model and then physical testing if desired. There are limited detailed calculations for the various cases needed for design.

Design Criteria

The Figure above shows a TCC system with 2.75” concrete topping slab with #3 steel reinforcement spaced at 12” o.c., 0.35” rigid insulation, and 5-ply CLT, Grade E1M4, 139 E (6.90”). Two different connectors will be looked at for this parametric study: 8 mm diameter ASSY plus VG screw and 90 mm HBV shear plate.

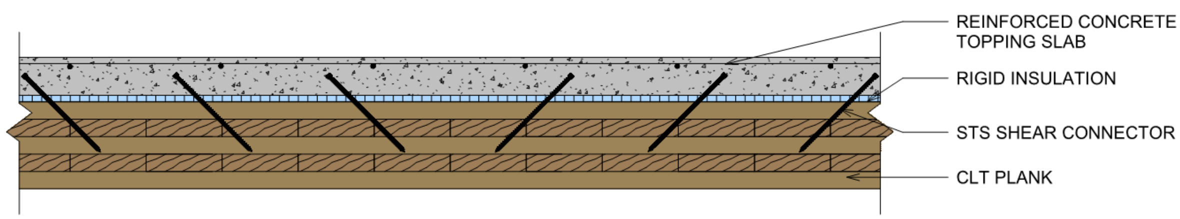

Figure 7-1: ASSY plus VG Screw TCC Section

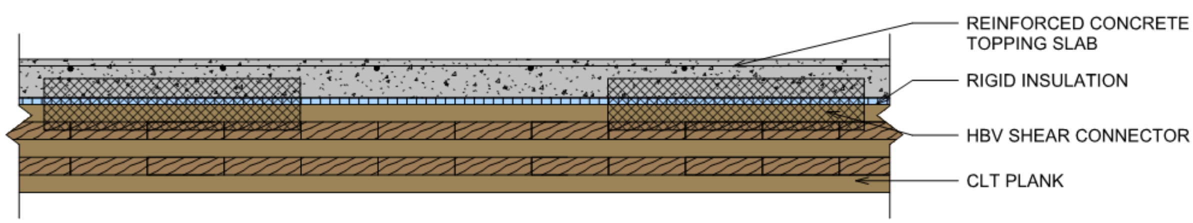

Figure 7-2: HBV Shear Plate TCC Section

Loading Criteria

The gravity framing system is analyzed and design for the following loads.

Dead Load: calculated based on design dimensions

- Density of mass-timber: 35 lbs/ft^3
- Density of concrete: 150 lbs/ft^3
- Sustained/Expected Load: 100%

Superimposed Dead Load: Partitions + CMEP

- Total Floor Load: 20 psf
- Sustained/Expected Load: 78.2%

Live Load: Office Building

- Total: 50 psf
- Reduction: Possible for columns and spandrels, but not considered
- Sustained/Expected Load: 21.8% (10.9 psf mean load per ASCE Table C4.3-2)

Deflection Criteria

The gravity framing system is designed to meet the following deflections:

Live Load: Deflection < span/360

- Span is measured from center-to-center of support

Total Load: Deflection < span/240

- Span is measured from center-to-center of supports
- Long term deflection due to creep in timber and concrete included
- Long term modifier is taken as 2.0 based on NDS 2018 and ACI 318-16

Strength Criteria

The structural system was designed to satisfy the strength requirements of ACI-318 and NDS-2018 where applicable. Components outside the scope of these codes are designed using information from Eurocode and previous design examples with testing.

Results

The parametric study highlighted the impact of connectors on the overall TCC system while also detailing the steps for TCC calculations. All six of the detailed calculations are located in appendix A and B. Table 7-1 shows the arrangement for each of the six calculations. The HBV system has different spacing due to the length of the connector and the panel.

SPECIFICATION	SPAN (FT)	CONNECTOR TYPE	SPACING (IN)		
			ENDS s_e	MIDDLE s_e	EFF. s_{eff}
STS-22	22	8Ø SCREW	6	12	7.5
STS-24	24	8Ø SCREW	6	12	7.5
STS-26	26	8Ø SCREW	6	12	7.5
HBV-22	22	90 HBV	54	54	54
HBV-24	24	90 HBV	60	60	60
HBV-26	26	90 HBV	60	72	63

Table 7-1: Specification Spacing Description

A major highlight of the study is looking at the deflection from the NDS 2018 compared to Eurocode 5. The main difference between the two codes is where the deformation/creep factor is applied in the calculation. The US method applies the deformation factor at the end of the calculations so it only applied to the deflection. The European method is applied at the beginning of calculation to the modulus of elasticity for long term calculation which leads to a more conservative design. Table 7-1 compares live load deflection, Δ_{LL}, total load deflection, Δ_{TL}, and long term total load deflection, $\Delta_{TL\infty}$.

SPECIFICATION	DEFLECTION		
	Δ_{LL}	Δ_{TL}	$\Delta_{TL\infty}$
STS-22	0.201	0.763	0.766
STS-24	0.274	1.042	1.053
STS-26	0.366	1.391	1.415
HBV-22	0.164	0.622	0.655
HBV-24	0.23	0.876	0.923
HBV-26	0.314	1.195	1.263

Table 7-2: Deflection Results

The two long term deflection shows the difference between the European and US method of calculations. The European method for long term total load deflection is greater and more conservative than the US method. The STS fails in serviceability for the 26 feet span. Since the connector is such a key component of the success of the floor system it is important to look at the efficiency. Table 7-2 shows the calculations for the efficiency of each of the six calculations. The efficiency levels out around 80% for these calculations. The HBV shear connector is almost 10% higher for the 22 feet specification.

SPECIFICATION	DEFLECTION			EFFIENCY %
	Δ_{NC}	Δ_{PC}	Δ_{FC}	
STS-22	0.478	0.201	0.085	70.60%
STS-24	0.677	0.274	0.12	72.40%
STS-26	0.933	0.366	0.165	73.94%
HBV-22	0.478	0.164	0.085	79.86%
HBV-24	0.677	0.23	0.12	80.30%
HBV-26	0.933	0.313	0.166	80.80%

Table 7-3: Connector Efficiency

Before each of the calculations, four cross-sectional stress distribution figures are shown for each of the different limit states. These graphics help to highlight how much a connector can impact the design of a TCC floor system. In figure 7-3 shows the ultimate limit state cross-sectional stress distribution for the HBV-24 and figure 7-4 for STS-24. The slip modulus from the connector has a direct effect on the increased stress and the efficiency of a system. The smaller difference between the values at the bottom of the concrete and top of timber leads to a better system will less stress.

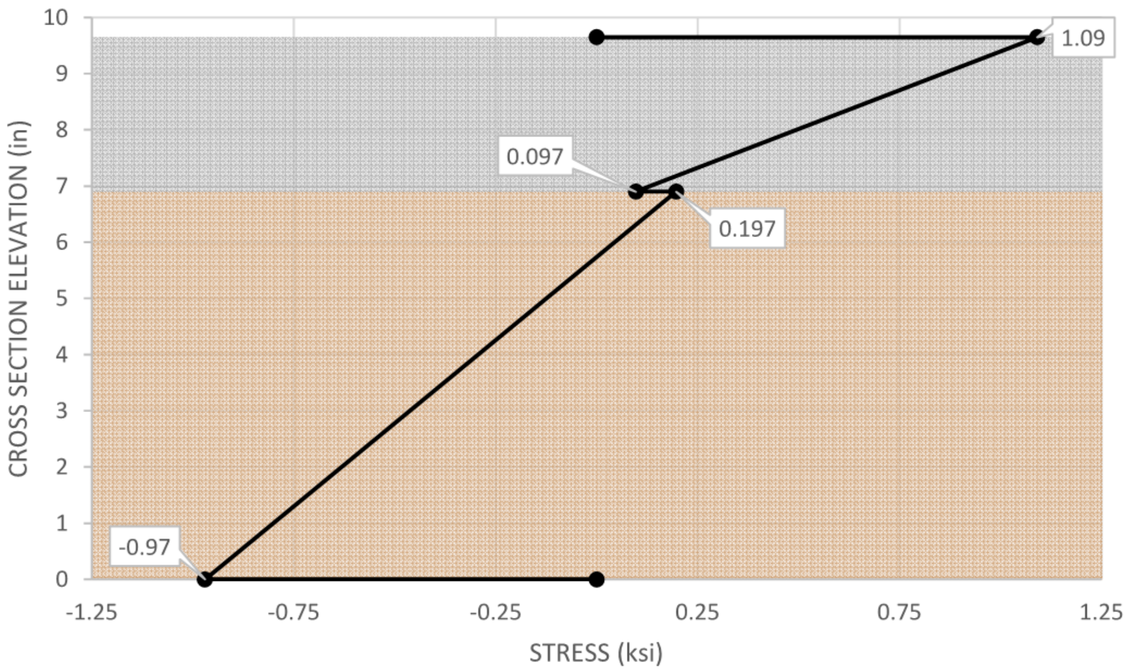

Figure 7-3: Ultimate limit state cross-sectional distribution for HBV-24

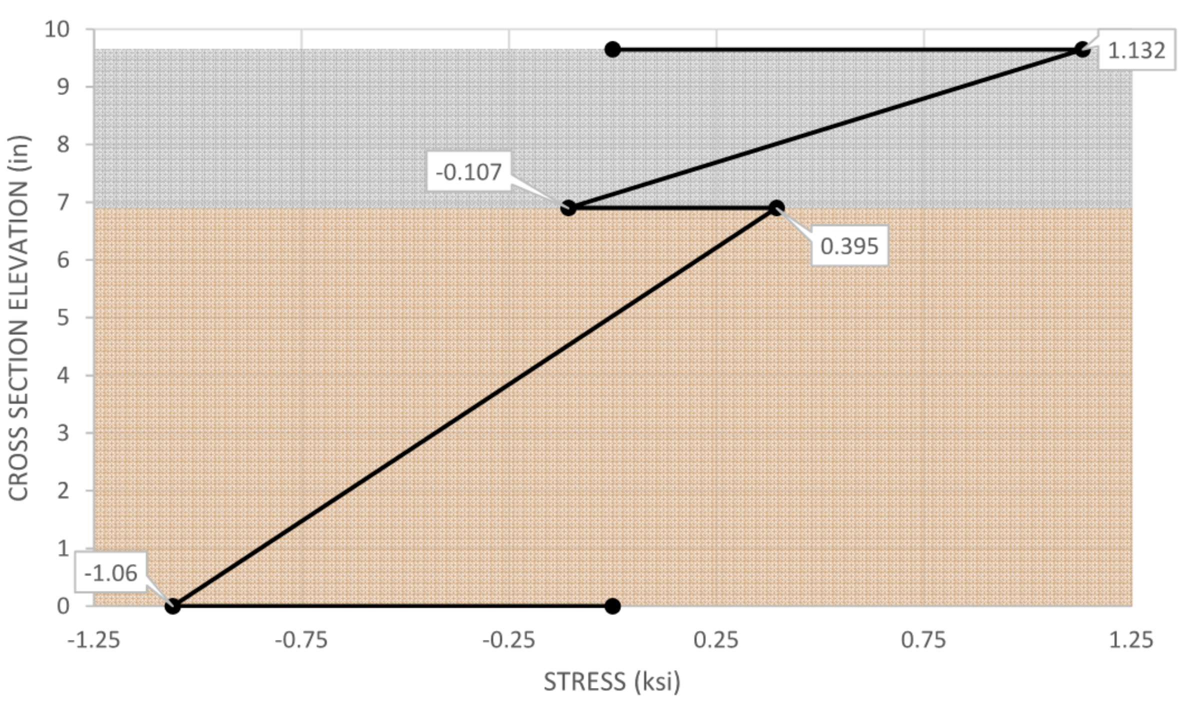

Figure 7-4: Ultimate limit state cross-sectional distribution for STS-24

Chapter 8 - Conclusion

Timber-concrete composite is a high-performance floor system that consists of a cast-in-place concrete slab integrally connected to timber beams/planks. The TCC floor is up to four times stiffer and two times stronger than a typical floor where timber and concrete are not joined together. When compared to a sole timber floor sound and vibration performance improve along with fire RES. Restoration and new construction can benefit from TCC floor systems in a variety of ways. Although TCC is not a new concept in the past twenty years there has been more research in the system and its contribution to the built environment.

Summary of Contributions

This report was intended to research TCC systems and their applications while focusing on detailed calculations. In chapter 6 the development of thorough TCC calculations specifically relating to the use of the CLT plate system using two different connectors. While there are some example calculations for a TCC system utilizing North American standards none walk through multiple nuances that the Eurocode presents. The multiple worked-out examples in appendixes A and B help to educate professionals on hand calculations for a TCC calculation using a CLT plate which makes it an even more difficult calculation. Reviewing the differences between the Eurocode and North American codes, such as NDS 2018, highlight the discrepancy to be addressed and researched. The large variety of TCC connector information and research is predominantly out of Europe and has a variety of factors that apply to the Eurocode.

Future Research

There is currently more research being conducted in the United States for the TCC floor system from its applications to cost. There is a variety of research that can be expanded from this report base calculations specifically with different connectors, finite element software analysis, and physical testing. The possibility to do calculations with a variety of different connectors would be advantageous for manufactures in the united states which lag behind their European competitors. Software analysis with different finite element software to compare the hand calculations to the software. European finite element software lends itself to the Eurocode making the gamma method easier to apply, while north American software requires more steps and modifications to achieve similar results. The most important future research would be the physical testing of full-scale models to provide more information on TCC systems. More research on the topic of the TCC floor system in North America will help to accelerate the implementation of this system into codebooks and a variety of buildings.

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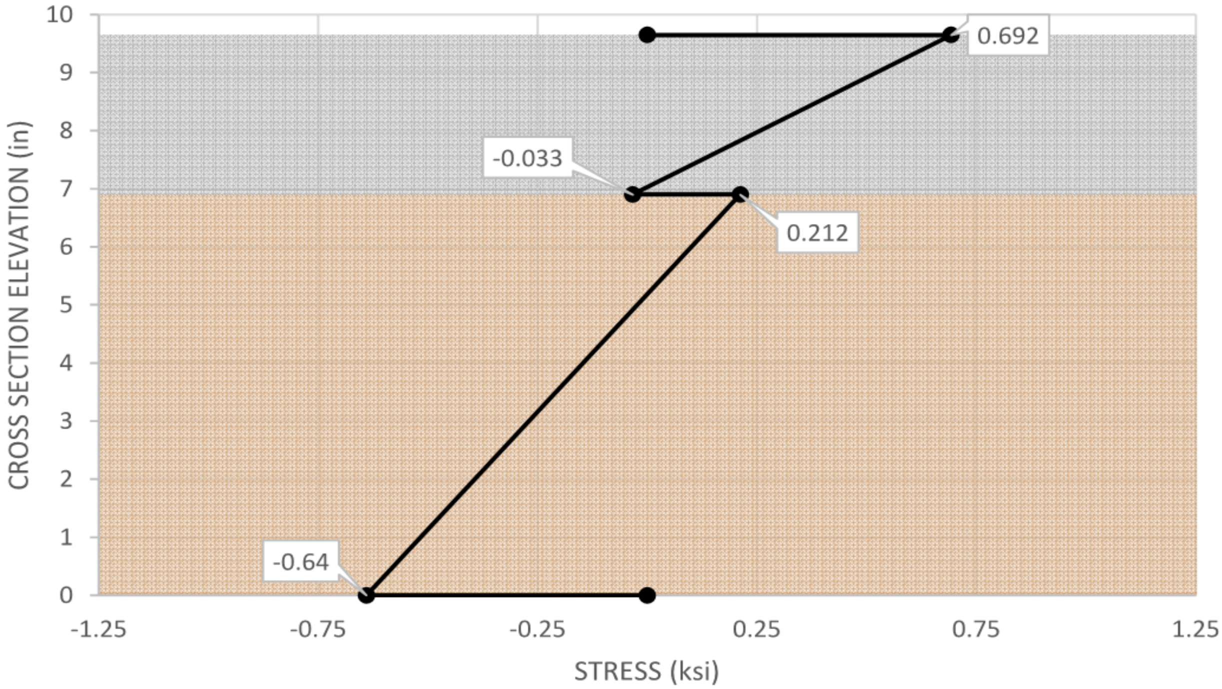
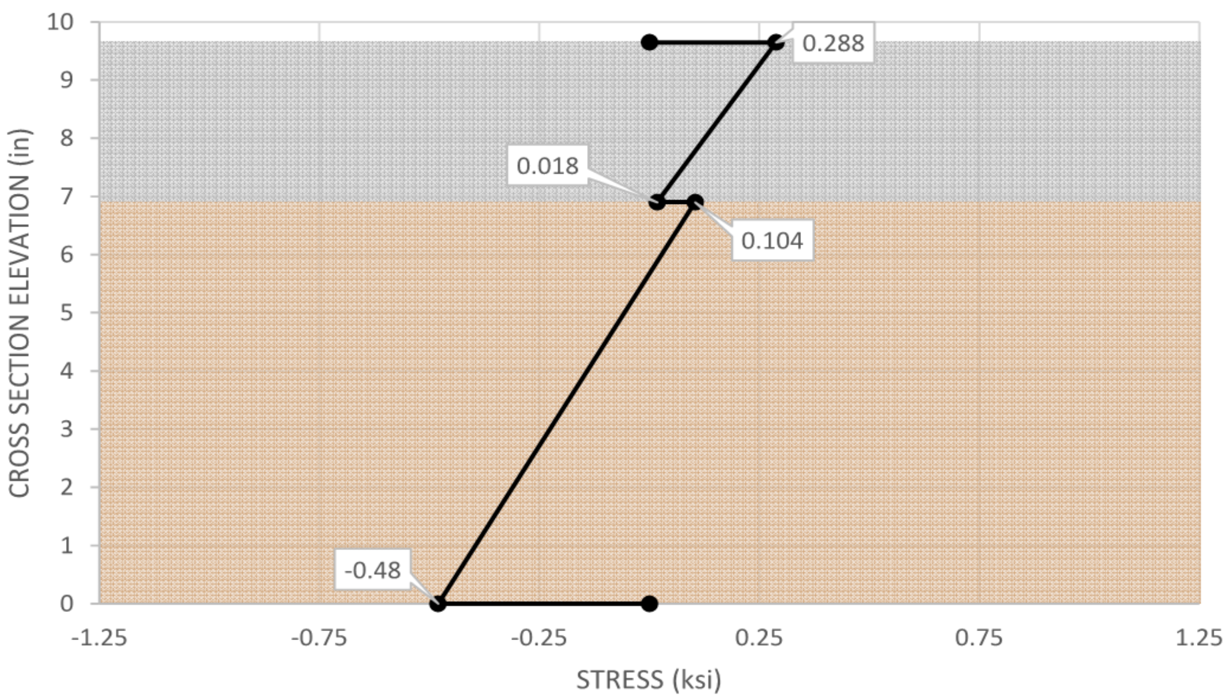
Appendix A - STS Calculation

8 mm dia. ASSY plus VG screw 22 FT SPAN

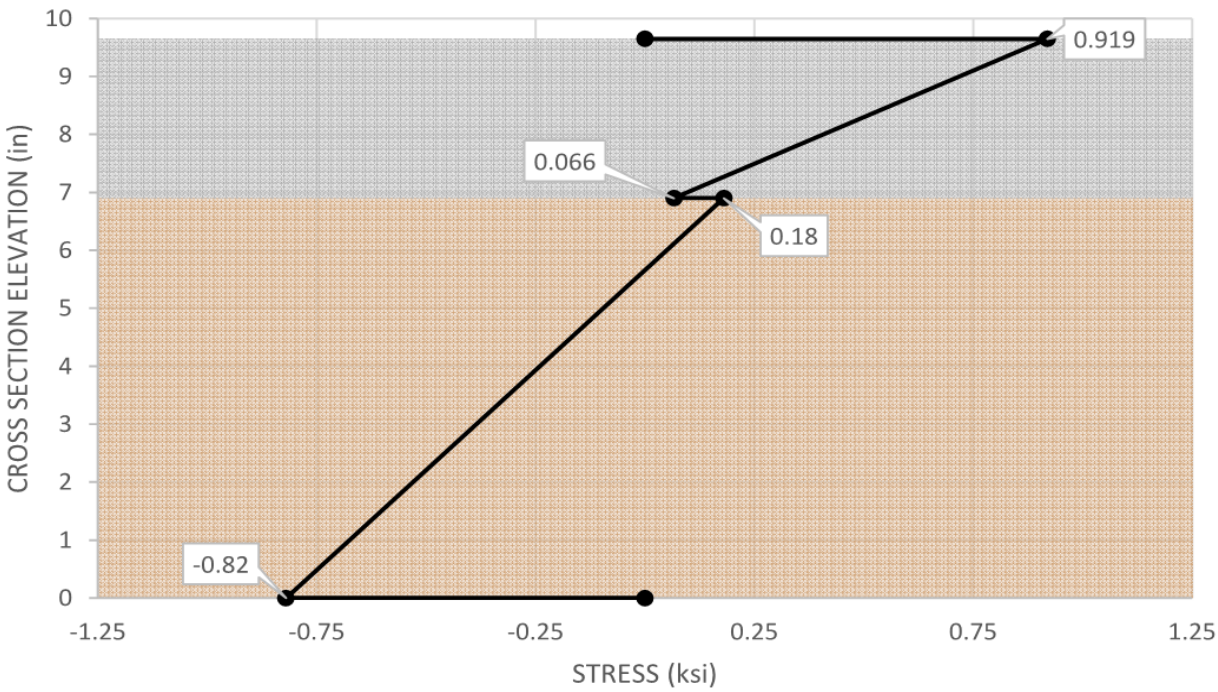


SERVICABILITY LIMIT STATE CROSS-SECTIONAL STRESS DISTRIBUTION

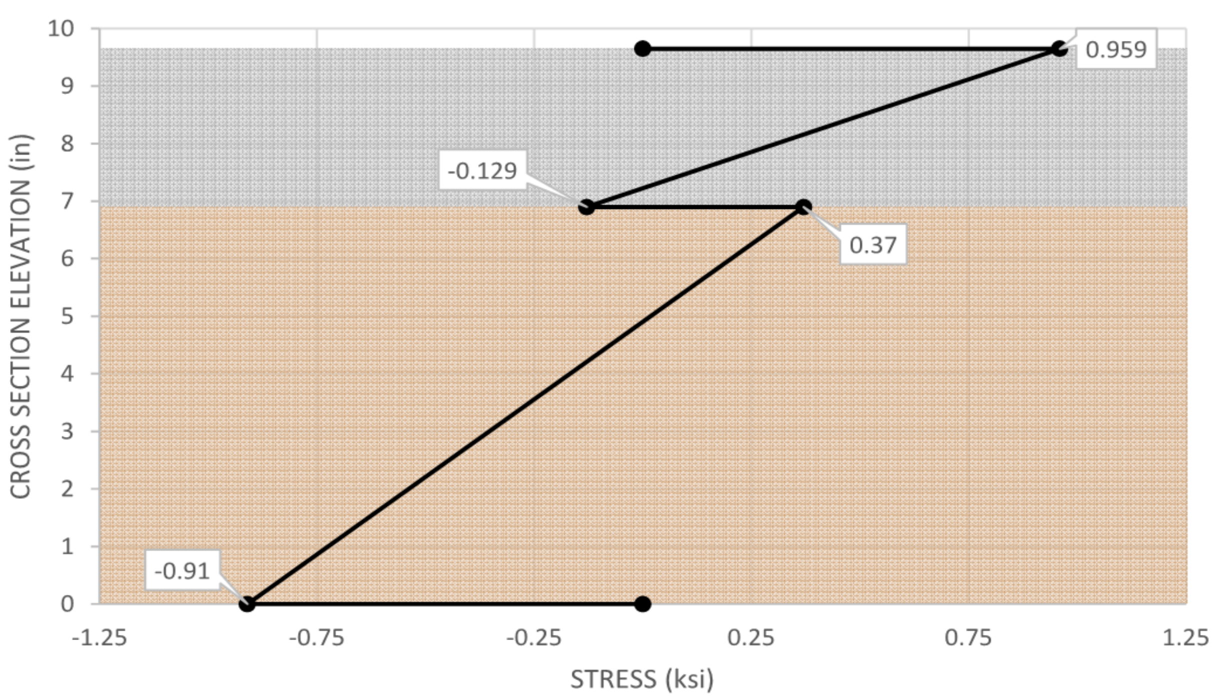


SERVICABILITY LIMIT STATE LONG-TERM CROSS-SECTIONAL STRESS DISTRIBUTION

8 mm dia. ASSY plus VG screw 22 FT SPAN

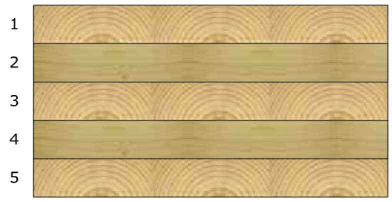


ULTIMATE LIMIT STATE CROSS-SECTIONAL STRESS DISTRIBUTION



ULTIMATE LIMIT STATE LONG-TERM CROSS-SECTIONAL STRESS DISTRIBUTION

STEP DESCRIPTION	COMPUTATION	REFERNCE
CLT Properties	CLT CALCULATIONS	Structurlam Crosslam



Type: 5-Ply CLT, Grade E1M4, 139 E

h_i = 1.375 in

b = 12 in

h_t = 6.90 in

a = 5.52 in

L = 22.0 ft

K_s = 11.5

Major Strength Axis

$F_{b,0}$ = 2100 psi

E_0 = 1800 ksi

$F_{t,0}$ = 1575 psi

$F_{c,0}$ = 1875 psi

$F_{v,0}$ = 160 psi

$F_{s,0}$ = 50 psi

Minor Strength Axis

$F_{b,90}$ = 875 psi

E_{90} = 1400 ksi

$F_{v,90}$ = 135 psi

$F_{s,90}$ = 45 psi

CLT Handbook, Chapter 3

CLT Calculations									
				GA_{eff}		EA_{eff}	EI_{eff}		
Layer	E (ksi)	h (in)	z (in)	G (ksi)	h/G/b (in^2/kip)	EA (kip)	$Ebh^3/12$ (kip-in^2)	EAz^2 (kip-in^2)	Sum of Layers (kip-in^2)
1	1800	1.380	2.760	112.5	0.00102	29808	4730.53	227065	231796
2	46.6667	1.380	1.375	8.75	0.01314	772.8	122.643	1461.08	1583.72
3	1800	1.380	0.000	112.5	0.00102	29808	4730.53	0	4730.53
4	46.6667	1.380	1.375	8.75	0.01314	772.8	122.643	1461.08	1583.72
5	1800	1.380	2.760	112.5	0.00102	29808	4730.53	227065	231796
						90969.6			471490

CLT Handbook Chapter 3 EQ 24

$$EA_{eff} = \sum_{i=1}^{n} E_i b_i h_i$$

$$EI_{eff} = \sum_{i=1}^{n} E_i b_i \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i A_i z_i^2$$

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

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

Cross Sectional Properties (Per foot of width)

SLS/ULS Short-Term			SLS Long-Term			ULS Long-Term		
EA_{eff} =	90969.6	kip	EA_{eff} =	47878.7	kip	EA_{eff} =	71629.6	kip
EI_{eff} =	471490	kip-in^2	EI_{eff} =	248153	kip-in^2	EI_{eff} =	371252	kip-in^2
GA_{eff} =	1075.5	kip	GA_{eff} =	566.1	kip	GA_{eff} =	846.9	kip
EI_{app} =	439686.3	kip	EI_{app} =	235933.9	kip	EI_{app} =	352972.1	kip

STEP DESCRIPTION	COMPUTATION	REFERNCE
	CONNECTOR CALCULATION	
Screw Properties	Diameter d = 8 mm = 0.315 in	Adolf Wurth GmbH & Co. KG
	Total Length l = 240 mm = 9.449 in	
	Penetration Depth l_{ef} = 160 mm = 6.299 in	
	Interlayer Thickness t = 8.89 mm = 0.35 in	
	Angle α = 45	ETA-13/0029 Table 2.3
	Timber Density ρ_k = 480 kg/m[3]	
	Coefficient of Friction μ = 0	
	Yield Moment $M_{y,k}$ = 20 Nm	ETA-13/0029 Table 2.4
	Tensile Capacity $f_{tens,k}$ = 17 kN	
	Withdrawal Parameter $f_{ax,k}$ = 11 N/mm[2]	
Characteristic Withdrawal Capacity	$F_{ax,\alpha,Rk} = \frac{f_{ax,k} d l_{ef}}{1.2\cos(\alpha)^2 + \sin(\alpha)^2}\left(\frac{\rho_k}{350}\right)^{0.8}$ $F_{ax,\alpha,Rk} = \frac{(11)(8)(160)}{1.2\cos(45)^2 + \sin(45)^2}\left(\frac{480}{350}\right)^{0.8}$ = 17179.5 N	ETA-13/0029 Table 2.3
Characteristic Load-Carrying Capacity	$F_{Rk} = (\cos(\alpha) + \mu\sin(\alpha))min\begin{cases} F_{ax,\alpha,Rk} \\ f_{tens,k} \end{cases}$ $F_{Rk} = (\cos(45) + 0\sin(45))(17kN) =$ 8.93 kN	
Slip Modulus	$K_{ser} = 100 l_{ef}$ $K_{ser} = 100(170\ mm) = 16000$ N/mm	ETA-13/0029 Table 2.3
Slip Modulus Ultimate	$K_u = (2/3)K_{ser}$ $K_u = (2/3)(17000\ N/mm) =$ 10667 N/mm Conversions F_{Rk} = 2.0077 kips K_{ser} = 91.3624 k/in K_u = 60.9082 k/in	
	Spacing between connectors	
	in rows at the ends s_e = 152.4 mm = 6 in	
	in rows in the middle s_m = 304.8 mm = 12 in	
	Effective Spacing between s_{eff} = 190.5 mm = 7.5 in	
	Number of rows of connectors n_r = 1	
	Deformation factor for long term	
	loading for concrete k_{def_c} = 2.5	ETA-13/0029 Table 2.1
	loading for timber k_{def_t} = 0.9	
	loading for STS k_{def_sts} = 0.6	
	Stiffness reduction for ULS ψ_2 = 0.3	
Connector Stiffness Adjustment for Long Term Loading	$K_{SLS\_LT} = \frac{K_{SER}}{1 + k_{def\_sts}} = \frac{17000\ N/mm}{1 + 0.6} =$ 10000 N/mm $K_{ULS\_LT} = \frac{K_u}{1 + \psi_2 k_{def\_sts}} = \frac{11333\ N/mm}{1 + (0.3 * 0.6)} =$ 9039.55 N/mm	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	EFFECTIVE STIFFNESS	
General Values	Gamma Span Length $L = L_0 =$ 22 ft Connector slip Modulus $K_{ser} =$ 16 kN/mm = 91.36 kips/in Connector Spacing $s_{eff} =$ 7.5 in	Annex B of EN 1995-1-1
Concrete Properties	Slab Depth $h_1 =$ 2.75 in Compressive Strength $f_c' =$ 4000 psi Weight of Concrete $w_c =$ 150 pcf	
CLT Properties	Clt Height $h_2 =$ 6.90 in $EA_{eff,CLT} =$ 90969.6 kip $EI_{eff,CLT} =$ 471490 kip-in^2	CLT Handbook EQ 24 & 25
Modulus of Elasticity	$E_{1\_SLS} = {w_c}^{1.5} 33\sqrt{f_c'}$ $E_{1\_SLS} = (150\ pcf)^{1.5} 33\sqrt{4000\ psi} =$ 3834.25 ksi	ACI 318, 19.2.2.1.a
Area of Concrete	$A_{c\_SLS} = h_c b_c$ $A_{c\_SLS} = (2.75\ in)(12\ in) =$ 33 in^2	EN 1995.1-1 Annex B EQ 3.2
Inertia of Concrete	$I_{1\_SLS} = b_1 {h_1}^2/12$ $I_{1\_SLS} = (12\ in)(2.75\ in)^2/12 =$ 20.8 in^4	EN 1995.1-1 Annex B EQ 3.3
	$E_c A_{c\_SLS} = (3834.3\ ksi)(33\ in^2) =$ 126530 kip $E_c I_{c\_SLS} = (38343\ ksi)(20.8\ in^4) =$ 79740.5 kip-in^2	
Gamma Factor	$\gamma_{1\_SLS} = \left[1 + \frac{\pi^2 E_1 A_1 s_1}{K_1 {L_0}^2}\right]^{-1}$ $\gamma_{1\_SLS} = \left[1 + \frac{\pi^2 (3834.3\ ksi)(33\ in^2)(7.5\ in)}{(91.4\ \ kip/in)(22\ ft)^2}\right]^{-1} =$ 0.40471	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2\_SLS} = \frac{\gamma_1 E_1 A_1 (h_1 + h_2)}{2\gamma_1 E_1 A_1 + 2\gamma_2 EA_{eff}}$ $a_{2\_SLS} = \frac{(0.4047)(3834\ ksi)(33\ in^2)(2.75\ in + 6.90\ in)}{2(0.4047)(3834\ ksi)(33\ in^2) + 2(1.0)(90970\ kip)} =$ 1.73783 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1\_SLS} = \frac{h_1 + h_2}{2} - a_{2\_SLS}$ $a_{1\_SLS} = \frac{6.90\ in + 2.75\ in}{2} - 1.74 =$ 3.08717 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_SLS} = (E_1 I_1 + \gamma_1 E_1 A_1 {a_1}^2) + \left(EI_{eff} + \gamma_2 EA_{eff} {a_2}^2\right)$ $(EI)_{eff,comp\_SLS} =$ 567789.7 kip-in^2 + 746221 kip-in^2 = 1314011 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Composite & CLT Effective	$Ratio_{CLT} = \frac{EI_{eff,comp\_SLS}}{EI_{eff,CLT}}$ $Ratio_{CLT} = \frac{1314011\ kip \cdot in^2}{471489.9\ kip \cdot in^2} =$ 2.787	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	APPARENT STIFFNESS	
Bare CLT Stiffness	Length L = 22 ft	
	K Value K_s = 11.5	
	$EI_{eff,clt}$ = 471490 kip-in^2	
	$GA_{eff,clt}$ = 1075.5 kips	
	$EI_{app,CLT\_SLS} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{471489.9\ kip \cdot in^2}{1+\frac{(11.5)(471489.9\ kip \cdot in^2)}{(1075.5\ kip)(22\ ft)^2}} =$ 439686 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{439686.3\ kip \cdot in^2}{471489.9\ kip \cdot in^2} =$ 93.3%	
Composite Section Stiffness	L = 22 ft	
	K_s = 11.5	
	$EI_{eff,comp}$ = 1314011 kip-in^2	
	$GA_{eff,clt}$ = 1075.5 kips	
	$EI_{app,comp\_SLS} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{1314011\ kip \cdot in^2}{1+\frac{(11.5)(1314011\ kip \cdot in^2)}{(1075.5\ kip)(22\ ft)^2}} =$ 1093564 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{1093564\ kip \cdot in^2}{1314011\ kip \cdot in^2} =$ 83.2%	
	$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{1093564\ kip \cdot in^2}{439686.3\ kip \cdot in^2} =$ 2.487	
Apparent Floor Stiffness	L = 22 ft	
	$EI_{app,CLT}$ = 439686 kip-in^2	
	$EI_{app,comp}$ = 1093564 kip-in^2	
	$K_{app,CLT\_SLS} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$	
	$K_{app,CLT\_SLS} = \frac{48(439686.3\ kip \cdot in^2)}{(22\ ft)^3} =$ 1.14702 kip/in	
	$K_{app,comp\_SLS} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$	
	$K_{app,comp\_SLS} = \frac{48(1093564\ kip \cdot in^2)}{(22\ ft)^3} =$ 2.85282 kip/in	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length $L = 22$ ft Shear $V_u = 1.37$ kip Moment $M_u = 90.39$ kip-in	
	Concrete: Modulus of Elasticity $E_1 = 3834.25$ ksi; Height $h_1 = 2.75$ in; Centroid $a_1 = 3.08717$ in; Gamma Factor $\gamma_1 = 0.40471$ CLT: $E_2 = 1800$ ksi; $h_2 = 6.90$ in; $a_2 = 1.73783$ in; $\gamma_2 = 1$ $EI_{eff,comp\_SLS} = 1314011$ kip-in^2	
	Bending Stress Calculations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.405)(3834\ ksi)(3.09\ in)(90.4\ kip \cdot in)}{1314011\ kip \cdot in^2} =$ 0.330 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(3834\ ksi)(2.75\ in)(90.4\ kip \cdot in)}{1314001\ kip \cdot in^2} =$ 0.363 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.330 + 0.363 =$ 0.692 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.330 - 0.363 =$ ##### ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(1800\ ksi)(1.738\ in)(90.4\ kip \cdot in)}{1314011\ kip \cdot in^2} =$ 0.215 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(1800\ ksi)(6.90\ in)(90.4\ kip \cdot in)}{1314011\ kip \cdot in^2} =$ 0.427 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} + \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.215 + 0.427 =$ 0.212 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.215 - 0.43 =$ -0.64 ksi	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	DEFLECTION CALCUATIONS	
	Short-Term / Long-Term	
Loading	Dead Load w_{DL} = 54.50 plf; w_{DL} = 54.50 plf	
	Superimposed Dead Load w_{SDL} = 20.00 plf; w_{SDL} = 15.64 plf	
	Live Load w_{LL} = 50.00 plf; w_{LL} = 10.9 plf	
	Span Length L = 22 ft; L = 22 ft	
	Stiffness $EI_{eff,comp\_SLS}$ = 1314011 kip-in^2	
	Short-Term Deflections	
Dead Load	$\Delta_{DL} = \frac{5w_{DL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(54.5\,plf)(22\,ft)^4}{384(1314011.0\,kip\cdot in^2)} =$ 0.2186 in	
Superimposed Dead Load	$\Delta_{SDL} = \frac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(20\,plf)(22\,ft)^4}{384(1314011.0\,kip\cdot in^2)} =$ 0.0802 in	
Live Load	$\Delta_{LL} = \frac{5w_{LL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(50\,plf)(22\,ft)^4}{384(1314011.0\,kip - in^2)} =$ 0.2006 in	
	Serviceability Check	
Allowable LL Deflection	$\Delta_{allow,LL} = \frac{L}{360} = \frac{(22ft)(12\,in)}{360(1\,ft)} =$ 0.73 in	IBC Table 1604.3
Check	$\Delta_{LL} \leq \Delta_{allow,LL} \rightarrow \Delta_{LL} = 0.201\,in < \Delta_{allow,LL} = 0.73\,in$ ∴ ACCEPTABLE	
	Long-Term Deflections	
Dead Load	$\Delta_{DL\_LT} = \frac{5w_{DL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(54.5\,plf)(22\,ft)^4}{384(1314011\,kip\cdot in^2)} =$ 0.21861 in	
Superimposed Dead Load	$\Delta_{SDL\_LT} = \frac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(15.6\,plf)(22\,ft)^4}{384(1314011\,kip\cdot in^2)} =$ 0.06274 in	
Live Load	$\Delta_{LL\_LT} = \frac{5w_{LL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(10.9\,plf)(22\,ft)^4}{384(1314011\,kip\cdot in^2)} =$ 0.04372 in	
	Serviceability Check	
	Time Dependent Creep Factor K_{cr} = 2.0	NDS 2018 Section 3.5.1
	Deflection due to short-term Δ_{ST} = 0.2006 in	
	Deflection due to long-term Δ_{LT} = 0.28135 in; $\Delta_{LT} = \Delta_{DL\_LT} + \Delta_{SDL\_LT}$	
	$\Delta_{TL} = K_{cr}\Delta_{LT} + \Delta_{ST}$	NDS EQ 3.5-1
	$\Delta_{TL} =$ 2.0(0.2813 in)+(0.2006 in) = 0.763 in	
	$\Delta_{allow,TL} = \frac{L}{240} = \frac{(22ft)(12\,in)}{240(1\,ft)} =$ 1.10 in	IBC Table 1604.3
	$\Delta_{TL} \leq \Delta_{allow,TL} \rightarrow \Delta_{TL} = 0.763\,in < \Delta_{allow,TL} = 1.10\,in$ ∴ ACCEPTABLE	

Serviceability Limit State
Long-Term Loading

STEP DESCRIPTION	COMPUTATION	REFERNCE
	EFFECTIVE STIFFNESS	
	Deformation factor for long term	
	loading for concrete k_{def_c} = 2.5	EN 1995.1-1
	loading for timber k_{def_t} = 0.9	
	loading for STS k_{def_sts} = 0.6	
	Stiffness reduction for ULS ψ_2 = 0.3	
	Effective Spacing between s_{eff} = 190.5 mm = 7.5 in	ETA-13/0029 Table 2.1
	Gamma Span Length L_0 = 22 ft	
	Connector slip Modulus K_{SLS_LT} = 10 kN/mm = 57.10 kips/in	
	Concrete / CLT	
	Modulus of Elasticity E_1 = 3834.25 ksi; E_2 = 1800 ksi	
	Area A_1 = 33 in^2; A_1 = 26.60 in^2	
	Momemnt of Inertia I_1 = 20.7969 in^4; I_1 = 137.863 in^4	
	Height h_1 = 2.75 in; h_2 = 6.90 in	
	EA_{eff,CLT_SLS_LT} = 47878.74 kip EI_{eff,CLT_SLS_LT} = 248153 kip-in^2	CLT Handbook EQ 24 & 25
Modulus of Adjustment for long term loading	$E_{1\_SLS\_LT} = \frac{E_1}{1+k_{def\_c}} =$ 1095.5 ksi $E_{2\_SLS\_LT} = \frac{E_2}{1+k_{def\_t}} =$ 947.368 ksi	EN 1995.1-1
Gamma Factor	$\gamma_{1,SLS\_LT} = \left[1+\frac{\pi^2 E_{1\_SLS\_LT} A_1 s_{eff}}{K_{ser\_LT} {L_0}^2}\right]^{-1}$ $\gamma_{1,SLS\_LT} = \left[1+\frac{\pi^2 (1095\ ksi)(33\ in^2)(7.5\ in)}{(57.1\ kip/in)(22\ ft)^2}\right]^{-1} =$ 0.598	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2,SLS\_LT} = \frac{\gamma_{1\_SER} E_1 A_1 (h_1+h_2)}{2\gamma_{1\_SER} E_1 A_1 + 2\gamma_2 EA_{eff}}$ $a_{2,SLS\_LT} = \frac{(0.598)(1095\ ksi)(33\ in^2)(2.75\ in + 6.90\ in)}{2(0.598)(1095\ ksi)(33\ in^2) + 2(1.0)(47878.7\ kip)} =$ 2.22787 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1,SLS\_LT} = \frac{h_1+h_2}{2} - a_{2,SLS\_LT}$ $a_{1,SLS\_LT} = \frac{6.90\ in + 2.75\ in}{2} - 2.23 =$ 2.597 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_SLS\_LT} = \left(E_1 I_1 + \gamma_{1\_SER} E_1 A_1 {a_{1\_SER}}^2\right) + \left(EI_{eff} + \gamma_2 EA_{eff} {a_{2\_SER}}^2\right)$ $(EI)_{eff,comp\_SLS\_LT} =$ 168588 kip-in^2 + 485794 kip-in^2 $(EI)_{eff,comp\_SLT\_LT} =$ 654381.8 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Compsite & CLT Effective	$Ratio_{CLT\_SER} = \frac{EI_{eff,comp\_SER}}{EI_{eff,CLT}}$ $Ratio_{CLT\_SER} = \frac{654381.8\ kip \cdot in^2}{248152.6\ kip \cdot in^2} =$ 13.67	

<table>
<tr><th>STEP DESCRIPTION</th><th>COMPUTATION</th><th>REFERNCE</th></tr>
<tr><td></td><td>APPARENT STIFFNESS</td><td></td></tr>
<tr><td>Bare CLT Stiffness</td><td>Length L = 22 ft
K Value K_s = 11.5
$EI_{eff,clt}$ = 248153 kip-in^2
$GA_{eff,clt}$ = 566.1 kips

$$EI_{app,CLT\_SLS\_LT} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{248152.6\ kip \cdot in^2}{1+\frac{(11.5)(248152.6\ kip \cdot in^2)}{(566.1\ kip)(22\ ft)^2}} = \quad 231414 \text{ kip-in}^2$$
$$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{231413.8\ kip \cdot in^2}{248152.6\ kip \cdot in^2} = \quad 93.3\%$$</td><td>CLT Handbook EQ 35</td></tr>
<tr><td>Composite Section Stiffness</td><td>L = 22 ft
K_s = 11.5
$EI_{eff,comp}$ = 654382 kip-in^2
$GA_{eff,clt}$ = 566.1 kips

$$EI_{app,comp\_SLS\_LT} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{654381.8\ kip \cdot in^2}{1+\frac{(11.5)(654382\ kip \cdot in^2)}{(566.1\ kip)(22\ ft)^2}} = \quad 549558 \text{ kip-in}^2$$
$$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{549558.1\ kip \cdot in^2}{654381.8\ kip \cdot in^2} = \quad 84.0\%$$
$$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{549558.1\ kip \cdot in^2}{231413.8\ kip \cdot in^2} = \quad 2.375$$</td><td>CLT Handbook EQ 35</td></tr>
<tr><td>Apparent Floor Stiffness</td><td>L = 22 ft
$EI_{app,CLT}$ = 231414 kip-in^2
$EI_{app,comp}$ = 549558 kip-in^2

$$K_{app,CLT\_SLS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$$
$$K_{app,CLT\_SLS\_LT} = \frac{48(231413.8\ kip \cdot in^2)}{(22\ ft)^3} = \quad 0.6037 \text{ kip/in}$$
$$K_{app,comp\_SLS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$$
$$K_{app,comp\_SLS\_LT} = \frac{48(549558.1\ kip \cdot in^2)}{(22\ ft)^3} = \quad 1.43365 \text{ kip/in}$$</td><td></td></tr>
</table>

STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length $L = 22$ ft Shear $V_u = 0.89$ kip Moment $M_u = 58.84$ kip-in	
	Concrete: Modulus of Elasticity $E_1 = 1095.5$ ksi; Height $h_1 = 2.75$ in; Centroid $a_1 = 2.59713$ in; Gamma Factor $\gamma_1 = 0.59794$ CLT: $E_2 = 947.368$ ksi; $h_2 = 6.90$ in; $a_2 = 2.22787$ in; $\gamma_2 = 1$ $EI_{eff,comp\_SLS\_LT} = 654382$ kip-in^2	
	Bending Stess Calulations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.598)(1096\ ksi)(2.60\ in)(58.8\ kip \cdot in)}{654381.8\ kip \cdot in^2} =$ 0.153 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(1096\ ksi)(2.75\ in)(58.8\ kip \cdot in)}{654381.8\ kip \cdot in^2} =$ 0.135 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.153 + 0.135 =$ 0.288 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.153 - 0.135 =$ 0.018 ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(947.4\ ksi)(2.23\ in)(58.8\ kip \cdot in)}{654381.8\ kip \cdot in^2} =$ 0.19 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(947.5\ ksi)(6.90\ in)(58.8\ kip \cdot in)}{654381.8\ kip \cdot in^2} =$ 0.294 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.19 + 0.294 =$ 0.104 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.19 - 0.294 =$ -0.48 ksi	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	DEFLECTION CALCUATIONS	
Loading	Short-Term: Dead Load w_{DL} = 54.50 plf; Superimposed Dead Load w_{SDL} = 20.00 plf; Live Load w_{LL} = 50.00 plf; Span Length L = 22 ft; Stiffness $EI_{eff,comp\_SLS}$ = 1314011 kip-in^2 Long-Term: w_{DL} = 54.50 plf; w_{SDL} = 15.64 plf; w_{LL} = 10.9 plf; L = 22 ft; $EI_{eff,comp\_SLS\_LT}$ = 654381.8 kip-in^2	
	Short-Term Deflections	
Dead Load	$\Delta_{DL} = \frac{5w_{DL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(54.5\,plf)(22\,ft)^4}{384(1314011.0\,kip \cdot in^2)} =$ 0.2186 in	
Superimposed Dead Load	$\Delta_{SDL} = \frac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(20\,plf)(22\,ft)^4}{384(1314011.0\,kip \cdot in^2)} =$ 0.0802 in	
Live Load	$\Delta_{LL} = \frac{5w_{LL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(50\,plf)(22\,ft)^4}{384(1314011.0\,kip - in^2)} =$ 0.2006 in	
	Servicablility Check	
Allowable LL Deflection	$\Delta_{allow,LL} = \frac{L}{360} = \frac{(22ft)(12\,in)}{360(1\,ft)} =$ 0.73 in	IBC Table 1604.3
Check	$\Delta_{LL} \leq \Delta_{allow,LL} \rightarrow \Delta_{LL} = 0.2006\,in < \Delta_{allow,LL} = 0.73\,in$ $\therefore$ ACCEPTABLE	
	Long-Term Deflections	
Dead Load	$\Delta_{DL\_LT} = \frac{5w_{DL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(54.5\,plf)(22\,ft)^4}{384(654381.8\,kip \cdot in^2)} =$ 0.43897 in	
Superimposed Dead Load	$\Delta_{SDL\_LT} = \frac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(15.6\,plf)(22\,ft)^4}{384(654381.8\,kip \cdot in^2)} =$ 0.12597 in	
Live Load	$\Delta_{LL\_LT} = \frac{5w_{LL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(10.9\,plf)(22\,ft)^4}{384(654381.8\,kip \cdot in^2)} =$ 0.08779 in	
	Servicablility Check Deflection due to short-term Δ_{ST} = 0.2006 in Deflection due to long-term Δ_{LT} = 0.56495 in $\Delta_{LT} = \Delta_{DL\_LT} + \Delta_{SDL\_LT}$ Deflection due to Total Load Δ_{TL} = 0.7655 in $\Delta_{TL} = \Delta_{LT} + \Delta_{ST}$	
	$\Delta_{allow,TL} = \frac{L}{240} = \frac{(22ft)(12\,in)}{240(1\,ft)} =$ 1.10 in	IBC Table 1604.3
	$\Delta_{TL} \leq \Delta_{allow,TL} \rightarrow \Delta_{TL} = 0.766\,in < \Delta_{allow,TL} = 1.10\,in$ $\therefore$ ACCEPTABLE	

STEP DESCRIPTION	COMPUTATION	REFERNCE
General Values	EFFECTIVE STIFFNESS Gamma Span Length $L = L_0 =$ 22 ft Connector slip Modulus $K_U =$ 10.6667 kN/mm = 60.91 kips/in Connector Spacing $s_{eff} =$ 7.5 in	Annex B of EN 1995-1-1
Concrete Properties	Slab Depth $h_1 =$ 2.75 in Compressive Strength $f'_c =$ 4000 psi Weight of Concete $w_c =$ 150 pcf	
CLT Properties	Clt Height $h_2 =$ 6.90 in $EA_{eff,CLT} =$ 90969.6 kip $EI_{eff,CLT} =$ 471490 kip-in^2	CLT Handbook EQ 24 & 25
Modulus of Elasticity	$E_{1\_ULS} = w_c^{1.5} 33\sqrt{f'_c}$ $E_{1\_ULS} = (150\ pcf)^{1.5} 33\sqrt{4000\ psi} =$ 3834.25 ksi	ACI 318, 19.2.2.1.a
Area of Concrete	$A_{c\_ULS} = h_c b_c$ $A_{c\_ULS} = (2.75\ in)(12\ in) =$ 33 in^2	EN 1995.1-1 Annex B EQ 3.2
Inhertia of Concrete	$I_{1\_ULS} = b_1 {h_1}^2/12$ $I_{1\_ULS} = (12\ in)(2.75\ in)^2/12 =$ 20.8 in^4	EN 1995.1-1 Annex B EQ 3.3
	$E_c A_{c\_ULS} = (3834.3\ ksi)(33\ in^2) =$ 126530 kip $E_c I_{c\_ULS} = (38343\ ksi)(20.8\ in^4) =$ 79740.5 kip-in^2	
Gamma Factor	$\gamma_{1\_ULS} = \left[1 + \frac{\pi^2 E_1 A_1 s_1}{K_1 {L_0}^2}\right]^{-1}$ $\gamma_{1\_ULS} = \left[1 + \frac{\pi^2 (3834.3\ ksi)(33\ in^2)(7.5\ in)}{(60.9\ kip/in)(22\ ft)^2}\right]^{-1} =$ 0.31188	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2\_ULS} = \frac{\gamma_1 E_1 A_1 (h_1 + h_2)}{2\gamma_1 E_1 A_1 + 2\gamma_2 EA_{eff}}$ $a_{2\_ULS} = \frac{(0.3119)(3834\ ksi)(33\ in^2)(2.75\ in + 6.90\ in)}{2(0.3119)(3834\ ksi)(33\ in^2) + 2(1.0)(90970\ kip)} =$ 1.45982 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1\_ULS} = \frac{h_1 + h_2}{2} - a_{2\_ULS}$ $a_{1\_ULS} = \frac{6.90\ in + 2.75\ in}{2} - 1.46 =$ 3.36518 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_ULS} = (E_1 I_1 + \gamma_1 E_1 A_1 {a_1}^2) + \left(EI_{eff} + \gamma_2 EA_{eff} {a_2}^2\right)$ $(EI)_{eff,comp\_ULS} =$ 526633.1 kip-in^2 + 665352 kip-in^2 = 1191985 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Compsite & CLT Effective	$Ratio_{CLT} = \frac{EI_{eff,comp\_ULS}}{EI_{eff,CLT\_ULS}}$ $Ratio_{CLT} = \frac{1191985\ kip \cdot in^2}{471489.9\ kip \cdot in^2} =$ 2.528	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	APPARENT STIFFNESS	
Bare CLT Stiffness	Length $L = 22$ ft K Value $K_s = 11.5$ $EI_{eff,clt} = 471490$ kip-in^2 $GA_{eff,clt} = 1075.5$ kips	
	$EI_{app,CLT\_ULS} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{471489.9\ kip\cdot in^2}{1+\frac{(11.5)(471489.9\ kip\cdot in^2)}{(1075.5\ kip)(22\ ft)^2}} = 439686$ kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{439686.3\ kip\cdot in^2}{471489.9\ kip\cdot in^2} = 93.3\%$	
Composite Section Stiffness	$L = 22$ ft $K_s = 11.5$ $EI_{eff,comp} = 1191985$ kip-in^2 $GA_{eff,clt} = 1075.5$ kips	
	$EI_{app,comp\_ULS} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{1191985\ kip\cdot in^2}{1+\frac{(11.5)(1191985\ kip\cdot in^2)}{(1075.5\ kip)(22ft)^2}} = 1007710$ kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{1007710\ kip\cdot in^2}{1191985\ kip\cdot in^2} = 84.5\%$	
	$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{1007710\ kip\cdot in^2}{439686.3\ kip\cdot in^2} = 2.292$	
Apparent Floor Stiffness	$L = 22$ ft $EI_{app,CLT} = 439686$ kip-in^2 $EI_{app,comp} = 1007710$ kip-in^2	
	$K_{app,CLT\_ULS} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$	
	$K_{app,CLT\_ULS} = \frac{48(439686.3\ kip\cdot in^2)}{(22\ ft)^3} = 1.14702$ kip/in	
	$K_{app,comp\_ULS} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$	
	$K_{app,comp\_ULS} = \frac{48(1007710\ kip\cdot in^2)}{(22\ ft)^3} = 2.62884$ kip/in	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length L = 22 ft Shear V_u = 1.86 kip Moment M_u = 122.98 kip-in	
	Concrete / CLT Modulus of Elasticity E_1 = 3834.25 ksi; E_2 = 1800 ksi Height h_1 = 2.75 in; h_2 = 6.90 in Centroid a_1 = 3.36518 in; a_2 = 1.45982 in Gamma Factor γ_1 = 0.31188; γ_2 = 1 $EI_{eff,comp}$ = 1191985 kip-in^2	
	Bending Stress Calculations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.312)(3834\ ksi)(3.365\ in)(123\ kip \cdot in)}{1191985\ kip \cdot in^2} =$ 0.415 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(3834\ ksi)(2.75\ in)(123\ kip \cdot in)}{1191985\ kip \cdot in^2} =$ 0.544 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.415 + 0.544 =$ 0.959 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{+\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.415 - 0.544 =$ ##### ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(1800\ ksi)(1.46\ in)(123\ kip \cdot in)}{1191985\ kip \cdot in^2} =$ 0.271 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(1800\ ksi)(6.90\ in)(123\ kip \cdot in)}{1191985\ kip \cdot in^2} =$ 0.641 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} + \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.271 + 0.641 =$ 0.37 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.27 - 0.641 =$ -0.91 ksi	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	STRENGTH ANALYSIS	
Concrete	Normal Stress in Concrete σ_1 = 415.2 psi	
	Allow Comp Strength of Conc F_c = 2000 psi $F_c = f_c'/2$	
	$\sigma_1 < F_c \rightarrow \sigma_1 = 415.2\,psi < F_c = 2000\,psi$ ∴ ACCEPTABLE	
Timber	Bending Strength F_b = 2100 psi	
	Format Conversion Factor K_F = 2.54	
	Resistance Factor Φ = 0.85	
	Design Bending Strength	
	$F_b' = \Phi F_b K_F = 0.85(2100\,psi)(2.54) =$ 4533.9 psi	
	Tension Strength F_t = 1575 psi	
	Format Conversion Factor K_F = 2.70	
	Resistance Factor Φ = 0.80	
	Design Tension Strength	
	$F_t' = \Phi F_t K_F = 0.80(1575\,psi)(2.70) =$ 3402.0 psi	
	Average CLT Stress σ_2 = 271.1 psi	
	Extreme Fiber Stress $\sigma_{m,2}$ = 640.7 psi	
	$\frac{\sigma_2}{F_t'} + \frac{\sigma_{m,2}}{F_b'} \leq 1 \rightarrow \frac{271.1\,psi}{3402\,psi} + \frac{640.7\,psi}{4534\,psi} =$ 0.221 < 1.0 ∴ ACCEPTABLE	
	Shear Strength F_v = 160 psi	
	Format Conversion Factor K_F = 2.88	
	Resistance Factor Φ = 0.75	
	Design Shear Strength	
	$F_v' = \phi F_v K_F = 0.75(160\,psi)(2.88) =$ 345.6 psi	
	CLT Modulus of Elasticity E_2 = 1800 ksi	
	NA of timber h = 4.91 in	
	Shear V = 1.86 kip	
	$EI_{eff,comp}$ = 1191985 kip-in^2	
	$\tau_{2,max} = \frac{E_2 h^2 V_u}{2(EI)_{eff,comp}} = \frac{(1800\,ks)(4.91\,in)^2(1.86\,kip)}{2(1191985\,kip \cdot in^2)} =$ 33.92 psi	EN 1995.1-1 Annex B EQ 3.9
	$\tau_{2,max} < F_v' \rightarrow \tau_{2,max} = 33.92\,psi < F_v' = 345.6\,psi$ ∴ ACCEPTABLE	

STEP DESCRIPTION	COMPUTATION	REFERNCE
Fastener Load Calculation	STRENGTH ANALYSIS F_{ULT} = 2.00765 kips Gamma Factor γ_1 = 0.31188 ksi Concrete Modulus of Elasticity E_1 = 3834.25 in Concrete Area A_1 = 33.00 in^2 Composite Centroid a_1 = 3.37 in Fastener Spacing s = 7.5 in Ultimate Shear Loading V = 1.8634 kip $EI_{eff,comp\_ULS}$ = 1191985 kip-in^2 $F_q = \frac{\gamma_1 E_1 A_1 a_1 s V}{EI_{eff,comp}}$ $F_q = \frac{(0.312)(3834\ ksi)(33 in^2)(3.24\ in)(7.5\ in)(1.86\ kip)}{1191985\ kip \cdot in^2} =$ 1.557 kip $F_q < F_{ULT} \rightarrow F_q = 1.557\ kip < F_{ULT} = 2.01\ kip$ ∴ ACCEPTABLE	EN 1995.1-1 Annex B EQ 3.10

<table>
<tr><th>STEP DESCRIPTION</th><th>COMPUTATION</th><th>REFERNCE</th></tr>
<tr><td></td><td>EFFECTIVE STIFFNESS
Deformation factor for long term
loading for concrete k_{def_c} = 2.5
loading for timber k_{def_t} = 0.9
loading for STS k_{def_sts} = 0.6
Stiffness reduction for ULS ψ_2 = 0.3
Effective Spacing between s_{eff} = 190.5 mm = 7.5 in
Gamma Span Length L_0 = 22 ft
Connector slip Modulus K_{U_LT} = 9.03955 kN/mm = 51.62 kips/in</td><td>EN 1995.1-1

ETA-13/0029 Table 2.1</td></tr>
<tr><td></td><td>Concrete: Modulus of Elasticity E_1 = 3834.25 ksi; Area A_1 = 33 in^2; Momemnt of Inertia I_1 = 20.7969 in^4; Height h_1 = 2.75 in
CLT: E_2 = 1800 ksi; A_1 = 50.5387 in^2; I_1 = 261.939 in^4; h_2 = 6.90 in
EA_{eff,CLT_ULT_LT} = 71629.6 in
EI_{eff,CLT_ULT_LT} = 371252 kip-in^2</td><td>CLT Handbook EQ 24 & 25</td></tr>
<tr><td>Modulus of Adjustment for long term loading</td><td>$E_{1,ULT\_LT} = \frac{E_1}{1+\psi_2 k_{def\_c}} =$ 2191 ksi
$E_{2,ULT\_LT} = \frac{E_2}{1+\psi_2 k_{def\_t}} =$ 1417.32 ksi</td><td>EN 1995.1-1</td></tr>
<tr><td>Gamma Factor</td><td>$\gamma_{1_{ULT\_LT}} = \left[1+\frac{\pi^2 E_{1,ULT\_LT} A_1 s_{eff}}{K_{ULT\_LT} {L_0}^2}\right]^{-1}$
$\gamma_{1,ULT\_LT} = \left[1+\frac{\pi^2 (2191\ ksi)(33\ in^2)(7.5\ in)}{(51.6\ kip/in)(24\ ft)^2}\right]^{-1} =$ 0.402</td><td>EN 1995.1-1 Annex B EQ 3.5</td></tr>
<tr><td>Timber to Composite Centroid</td><td>$a_{2,ULT\_LT} = \frac{\gamma_{1\_ULT\_LT} E_{1,ULT\_LT} A_1 (h_1+h_2)}{2\gamma_{1,ULT\_LT} E_{1,ULT\_LT} A_1 + 2\gamma_2 EA_{eff,CLT\_ULT\_LT}}$
$a_{2,ULT\_LT} = \frac{(0.402)(2191.0\ ksi)(33\ in^2)(2.75\ in+6.90\ in)}{2(0.40)(2191.0 ksi)(33\ in^2)+2(1.0)(71629.6\ kip)} =$ 1.393 in</td><td>EN 1995.1-1 Annex B EQ 3.6</td></tr>
<tr><td>Concrete to Composite Centroid</td><td>$a_{1,ULT\_LT} = \frac{h_1+h_2}{2} - a_{2\_ULT}$
$a_{1,ULT\_LT} = \frac{6.90\ in+2.75\ in}{2} - 1.39 =$ 3.432 in</td><td></td></tr>
<tr><td>Effective Comp Stiffness</td><td>$(EI)_{eff,comp\_ULT\_LT} = \left(E_{1,ULT\_LT} I_1 + \gamma_{1,ULT\_LT} E_{1,ULT\_LT} A_1 {a_{1,ULT\_LT}}^2\right) + \left(EI_{eff} + \gamma_2 EA_{eff} {a_{2,ULT}}^2\right)$
$(EI)_{eff,comp\_ULT\_LT} =$ 387965 kip-in^2 + 510183 kip-in^2
$(EI)_{eff,comp\_ULT\_LT} =$ 898148 kip-in^2</td><td>EN 1995.1-1 Annex B EQ 3.1</td></tr>
<tr><td>Ratio of Compsite & CLT Effective</td><td>$Ratio_{CLT\_ULT} = \frac{EI_{eff,comp\_ULT\_LT}}{EI_{eff,CLT\_ULT\_LT}}$
$Ratio_{CLT\_ULT} = \frac{898147.8\ kip\cdot in^2}{371251.9\ kip\cdot in^2} =$ 12.54</td><td></td></tr>
</table>

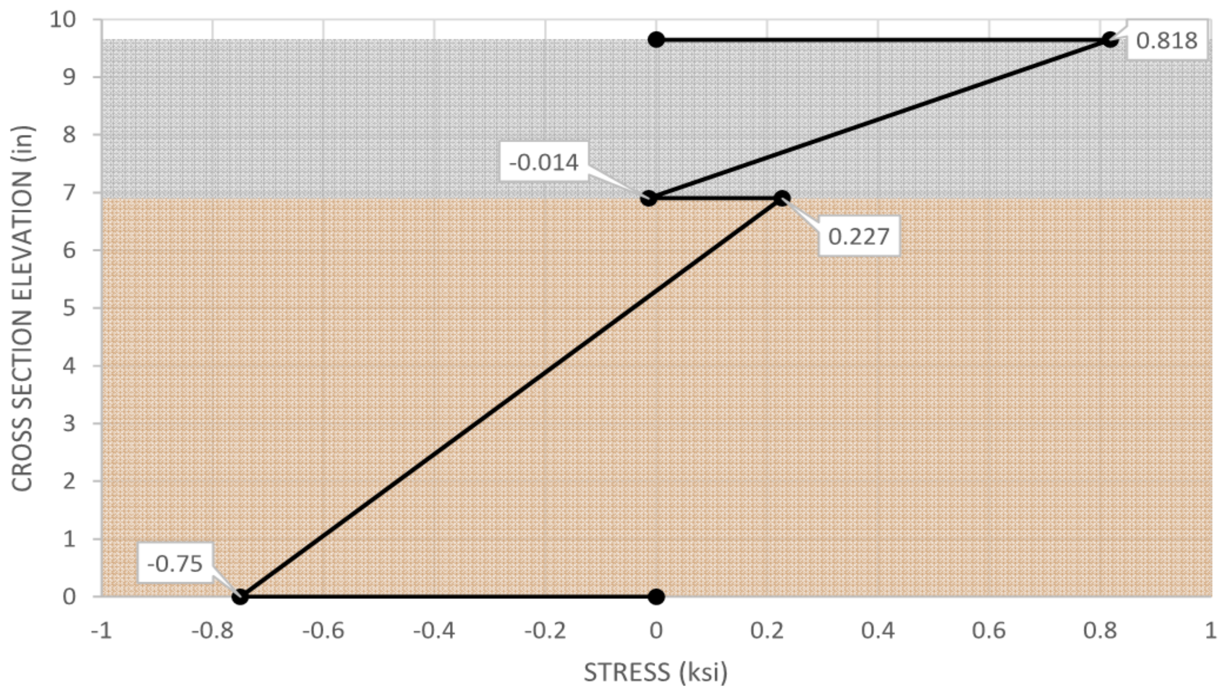
STEP DESCRIPTION	COMPUTATION	REFERNCE
	APPARENT STIFFNESS	
Bare CLT Stiffness	Length L = 22 ft K Value K_s = 11.5 EI_{eff,CLT_ULT_LT} = 371252 kip-in^2 GA_{eff,CLT_ULT_LT} = 846.9 kips	
	$EI_{app,CLT\_ULS\_LT} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{371251.9\ kip \cdot in^2}{1+\frac{(11.5)(371252 kip \cdot in^2)}{(846.9\ kip)(22\ ft)^2}} =$ 346210 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{346209.7\ kip \cdot in^2}{371251.9\ kip \cdot in^2} =$ 93.3%	
Composite Section Stiffness	L = 22 ft K_s = 11.5 $EI_{eff,comp}$ = 898148 kip-in^2 $GA_{eff,clt}$ = 846.9 kips	
	$EI_{app,comp\_ULS\_LT} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{898147.8\ kip \cdot in^2}{1+\frac{(11.5)(898148\ kip \cdot in^2)}{(846.9\ kip)(22\ ft)^2}} =$ 764388 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{764387.9\ kip \cdot in^2}{898147.8\ kip \cdot in^2} =$ 85.1%	
	$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{764387.9\ kip \cdot in^2}{346209.7\ kip \cdot in^2} =$ 2.208	
Apparent Floor Stiffness	L = 22 ft $EI_{app,CLT}$ = 346210 kip-in^2 $EI_{app,comp}$ = 764388 kip-in^2	
	$K_{app,CLT\_ULS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$	
	$K_{app,CLT\_ULS\_LT} = \frac{48(346209.7\ kip \cdot in^2)}{(22\ ft)^3} =$ 0.90317 kip/in	
	$K_{app,comp\_ULS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$	
	$K_{app,comp\_ULS\_LT} = \frac{48(764387.9\ kip \cdot in^2)}{(22\ ft)^3} =$ 1.99408 kip/in	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length L = 22 ft	
	Shear V_u = 1.86 kip	
	Moment M_u = 122.98 kip-in	
	Concrete: Modulus of Elasticity E_1 = 2191 ksi; CLT: E_2 = 1417.32 ksi	
	Concrete: Height h_1 = 2.75 in; CLT: h_2 = 6.90 in	
	Concrete: Centroid a_1 = 3.43231 in; CLT: a_2 = 1.39269 in	
	Concrete: Gamma Factor γ_1 = 0.40198; CLT: γ_2 = 1	
	$EI_{eff,comp}$ = 898148 kip-in^2	
	Bending Stess Calulations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.402)(2191\ ksi)(3.43\ in)(123\ kip \cdot in)}{898147.8\ kip \cdot in^2} =$ 0.414 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(2191\ ksi)(2.75\ in)(123\ kip \cdot in)}{898147.8\ kip \cdot in^2} =$ 0.413 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.414 + 0.413 =$ 0.826 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.414 - 0.413 =$ 0.001 ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(1417.3\ ksi)(1.39\ in)(123\ kip \cdot in)}{898147.8\ kip \cdot in^2} =$ 0.27 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(1417\ ksi)(6.90\ in)(123\ kip \cdot in)}{898147.8\ kip \cdot in^2} =$ 0.67 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} + \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.27 + 0.670 =$ 0.399 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.27 - 0.670 =$ -0.9 ksi	

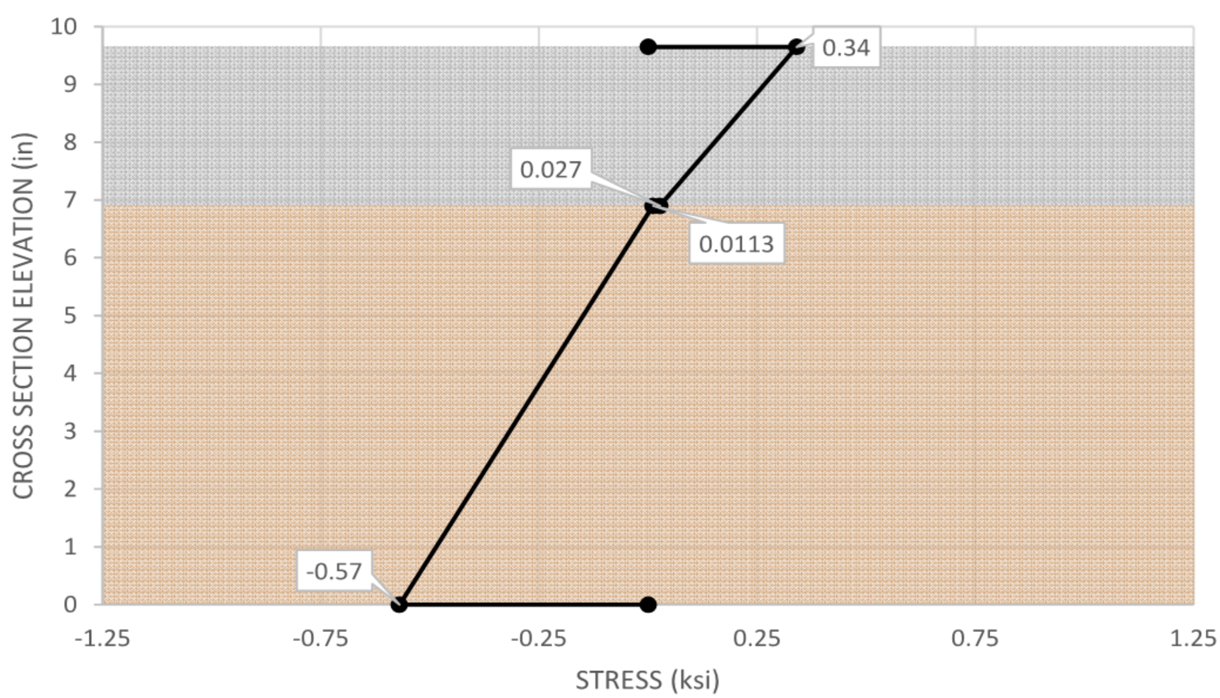
STEP DESCRIPTION	COMPUTATION	REFERNCE
Fastener Load Calculation	STRENGTH ANALYSIS F_{ULT} = 2.00765 kips Gamma Factor γ_1 = 0.40198 ksi Concrete Modulus of Elasticity E_1 = 2191.00 in Concrete Area A_1 = 33.00 in^2 Composite Centroid a_1 = 3.43 in Fastener Spacing s = 7.5 in Ultimate Shear Loading V = 1.8634 kip $EI_{eff,comp}$ = 898148 $kip\text{-}in^2$ $$F_q = \frac{\gamma_1 E_1 A_1 a_1 s V}{EI_{eff,comp}}$$ $$F_q = \frac{(0.402)(2191\ ksi)(33 in^2)(3.43\ in)(7.5\ in)(1.86\ kip)}{898147.8\ kip \cdot in^2} =$$ 1.552 kip $F_q < F_{ULT} \rightarrow F_q = 1.552\ kip < F_{ULT} = 2.01\ kip$ ∴ ACCEPTABLE	EN 1995.1-1 Annex B EQ 3.10

STEP DESCRIPTION	COMPUTATION	REFERNCE
	CONNECTOR EFFICIENCY	
Partial Composite Section	Effective Bending Stiffness EI_{eff_SLS} = 1314011 in Deflection Δ_{PC} = 0.20056 in	
Non-Composite Section	Effective Bending Stiffness EI_{eff_SLS} = 551231 in (set K_{ser} close to zero) Deflection Δ_{NC} = 0.47809 in	
Fully Composite	Concrete MOE E_1 = 3834.25 psi Timber MOE E_2 = 1800 psi Width b' = 12 in Width of Transformed Concrete b' = 25.56 in $b' = b(E_1/E_2)$ Height of Timber h_1 = 2.75 in Height of Concrete h_2 = 6.90 in Bottom to Centroid of Transformed Concrete y'_1 = 8.275 in Timber Section y_2 = 3.45 in Area of Transformed Concrete A'_1 = 70.29 in^2 Area of Timber Section A_2 = 82.8 in^2 Moment of Inertia of Transformed Concrete I'_1 = 44.30 in^4 Timber Section I_2 = 328.51 in^4 Centroid of Transformed Section $y_c = \frac{y'_1 A'_1 + y_2 A_2}{A'_1 + A_2}$ = 5.665 in Distance to Concrete Centroid d_1 = 4.29 in Distance to Timber Centroid d_2 = 0.84 in $d = \left\lvert y_c - \frac{h}{2} \right\rvert$ Transformed Section Moment of Inertia about NA I_{NA} = 1725.27 in^4 $I_{NA} = \sum I_i + A_i d_i^2$ Bending Stiffness $E_2 I_{NA}$ = 3105482 $k\text{-}in^2$ Deflection of Fully Composite Section $\Delta_{FC} = \frac{5wL^4}{384 E_2 I_{NA}} = \frac{5(50\,plf)(22ft)^4}{384(3105482\,k \cdot in^2)} =$ 0.085 in	
Shear Connector Efficiency	$Efficiency = \frac{\Delta_{NC} - \Delta_{PC}}{\Delta_{NC} - \Delta_{FC}} = \frac{0.4781" - 0.2056"}{0.4781" - 0.085"} =$ 70.6%	

8 mm dia. ASSY plus VG screw 24 FT SPAN

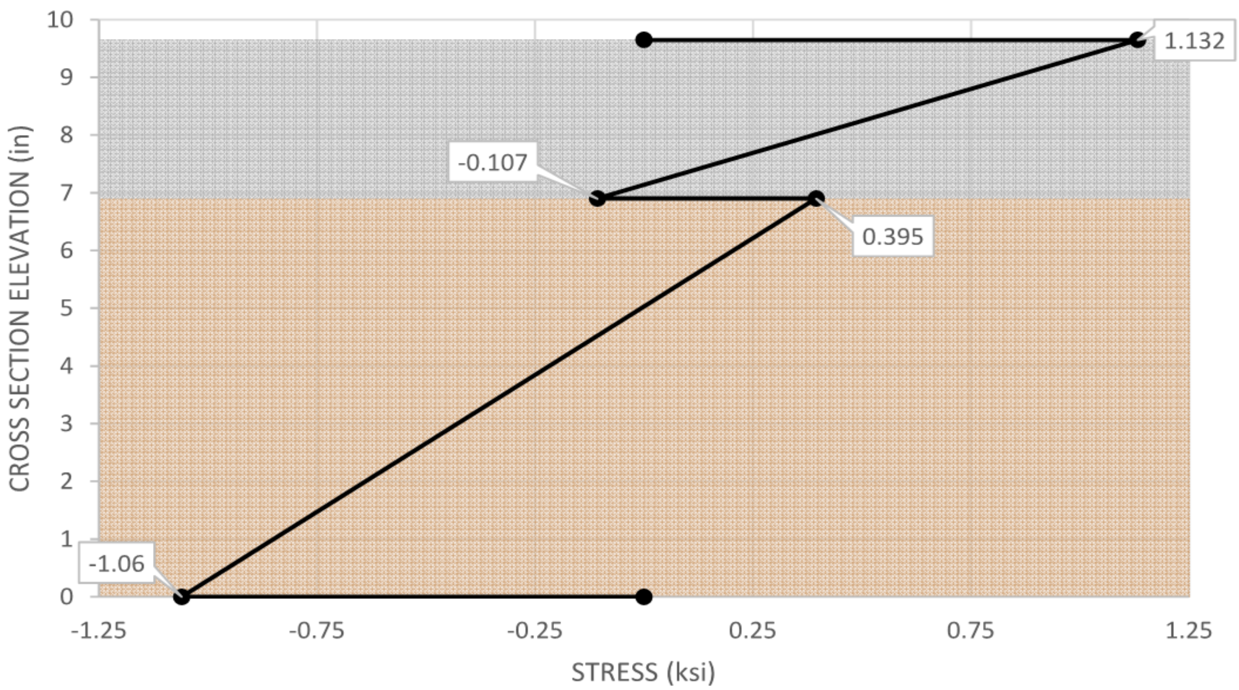


SERVICABILITY LIMIT STATE CROSS-SECTIONAL STRESS DISTRIBUTION

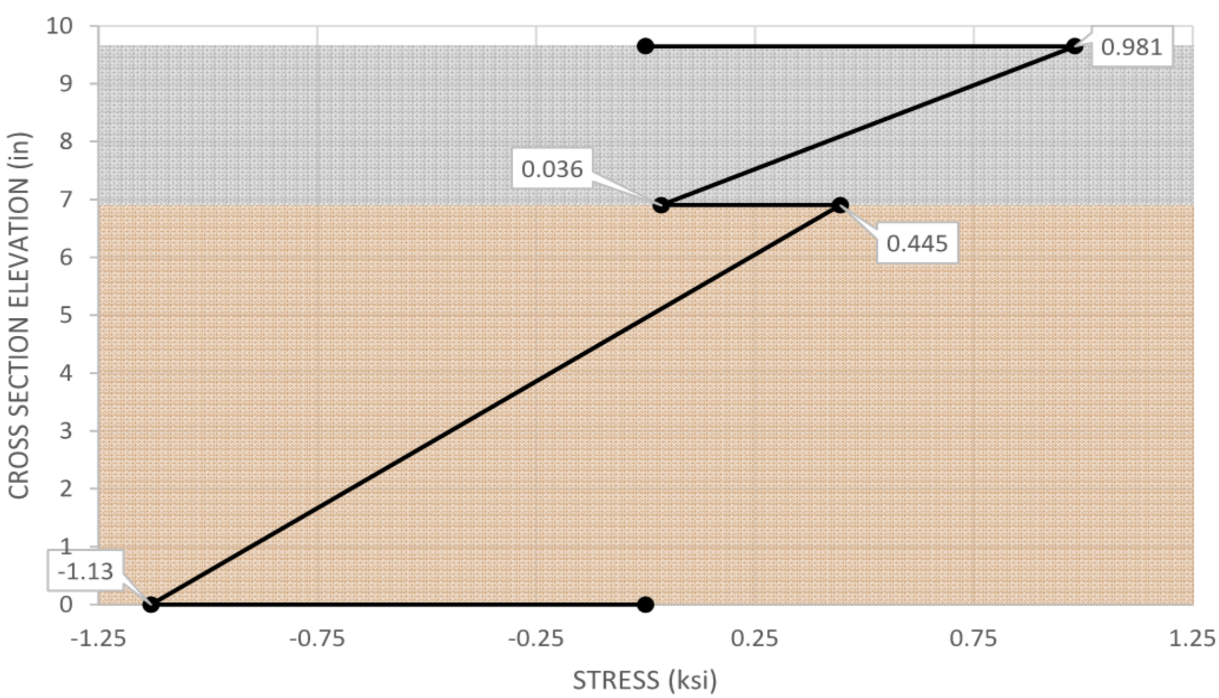


SERVICABILITY LIMIT STATE LONG-TERM CROSS-SECTIONAL STRESS DISTRIBUTION

8 mm dia. ASSY plus VG screw 22 FT SPAN



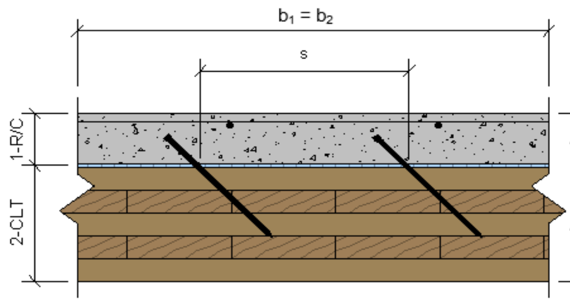
ULTIMATE LIMIT STATE CROSS-SECTIONAL STRESS DISTRIBUTION



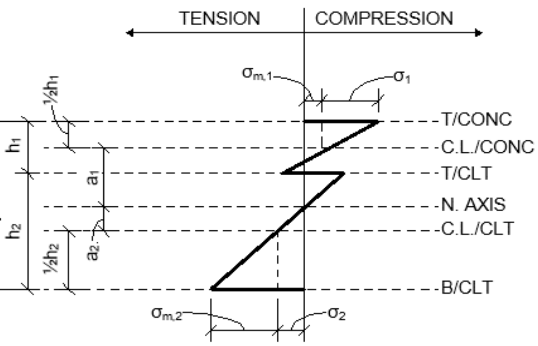
ULTIMATE LIMIT STATE LONG-TERM CROSS-SECTIONAL STRESS DISTRIBUTION

STEP DESCRIPTION	COMPUTATION	REFERNCE

8 mm dia. ASSY plus VG screw 24 FT SPAN



Cross-section



Stress Diagram

Dead Load Calculation

		Density				Height					
Mass Timber	ρ_t	=	35	pcf	h_t	=	6.90	in	L =	24	ft
Concrete	ρ_c	=	150	pcf	h_c	=	2.75	in			

$$w_{DL} = \rho_t h_t + \rho_c h_c$$
$$w_{DL} = (35\ pcf)(6.90\ in) + (150\ pcf)(2.75\ in) = 54.50 \text{ psf}$$

Loading Criteria

	Loading			Sustained/Expected			Reference
Dead Load	w_{DL} =	54.50	psf	m_{DL} =	100%		ASCE 7-16
Superimposed Dead Load	w_{SDL} =	20	psf	m_{SDL} =	78.2%		
Live Load	w_{LL} =	50	psf	m_{LL} =	21.8%	(10.9 psf mean load)	Table C4.3-2

LRFD / **Service**

Loading Cases

Load Case Factors				Equations	DL	SDL	LL	w_u
Combo	DL	SDL	LL		PSF	PSF	PSF	PLF
G01	1.4	1.4	0.0	$1.4(w_{DL} + w_{SDL})$	76.3	28	0	104.3
G02	1.2	1.2	1.6	$1.2(w_{DL} + w_{SDL}) + 1.6w_{LL}$	65.4	24	80	169.4
S01	1.0	1.0	1.0	$1.0w_{DL} + 1.0w_{SDL} + 1.0w_{LL}$	54.5	20	50	124.5
S02	0.0	1.0	1.0	$1.0w_{SDL} + 1.0w_{LL}$	0	20	50	70
S03	1.0	0.782	0.218	$1.0w_{DL} + 0.782w_{SDL} + 0.218w_{LL}$	54.5	15.64	10.9	81.04

Combo	w_u	V_u	M_u
	PLF	kips	kip-in
G01	104.3	1.252	90.12
G02	169.4	2.033	146.4
S01	124.5	1.494	107.6
S02	70	0.84	60.48
S03	81.04	0.972	70.02

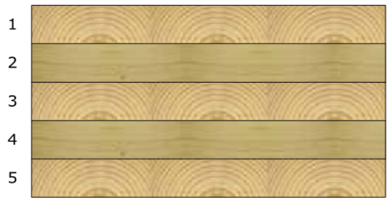
$$V_u = \frac{w_u L}{2}$$

$$M_u = \frac{w_u L^2}{8}$$

STEP DESCRIPTION	COMPUTATION	REFERNCE

CLT Properties

CLT CALCULATIONS



Type: 5-Ply CLT, Grade E1M4, 139 E

h_i = 1.375 in

b = 12 in

h_t = 6.90 in

a = 5.52 in

L = 24.0 ft

K_s = 11.5

Structurlam Crosslam

Major Strength Axis

$F_{b,0}$ = 2100 psi

E_0 = 1800 ksi

$F_{t,0}$ = 1575 psi

$F_{c,0}$ = 1875 psi

$F_{v,0}$ = 160 psi

$F_{s,0}$ = 50 psi

Minor Strength Axis

$F_{b,90}$ = 875 psi

E_{90} = 1400 ksi

$F_{v,90}$ = 135 psi

$F_{s,90}$ = 45 psi

CLT Calculations

CLT Handbook, Chapter 3

Layer	E (ksi)	h (in)	z (in)	GA_{eff}: G (ksi)	GA_{eff}: h/G/b (in^2/kip)	EA_{eff}: EA (kip)	EI_{eff}: $Ebh^3/12$ (kip-in^2)	EI_{eff}: EAz^2 (kip-in^2)	EI_{eff}: Sum of Layers (kip-in^2)
1	1800	1.380	2.760	112.5	0.00102	29808	4730.53	227065	231796
2	46.6667	1.380	1.375	8.75	0.01314	772.8	122.643	1461.08	1583.72
3	1800	1.380	0.000	112.5	0.00102	29808	4730.53	0	4730.53
4	46.6667	1.380	1.375	8.75	0.01314	772.8	122.643	1461.08	1583.72
5	1800	1.380	2.760	112.5	0.00102	29808	4730.53	227065	231796
						90969.6			471490

CLT Handbook Chapter 3 EQ 24

$$EA_{eff} = \sum_{i=1}^{n} E_i b_i h_i$$

$$EI_{eff} = \sum_{i=1}^{n} E_i b_i \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i A_i z_i^2$$

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

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

Cross Sectional Properties (Per foot of width)

SLS/ULS Short-Term			SLS Long-Term			ULS Long-Term		
EA_{eff} =	90969.6	kip	EA_{eff} =	47878.7	kip	EA_{eff} =	71629.6	kip
EI_{eff} =	471490	kip-in^2	EI_{eff} =	248153	kip-in^2	EI_{eff} =	371252	kip-in^2
GA_{eff} =	1075.5	kip	GA_{eff} =	566.1	kip	GA_{eff} =	846.9	kip
EI_{app} =	444475.0	kip	EI_{app} =	235933.9	kip	EI_{app} =	352972.1	kip

STEP DESCRIPTION	COMPUTATION	REFERNCE
	CONNECTOR CALCULATION	
Screw Properties	Diameter d = 8 mm = 0.315 in Total Length l = 240 mm = 9.449 in Penetration Depth l_{ef} = 160 mm = 6.299 in Interlayer Thickness t = 8.89 mm = 0.35 in Angle α = 45 Timber Density ρ_k = 480 kg/m^3 Coefficient of Friction μ = 0 Yield Moment $M_{y,k}$ = 20 Nm Tensile Capacity $f_{tens,k}$ = 17 kN Withdrawal Parameter $f_{ax,k}$ = 11 N/mm^2	Adolf Wurth GmbH & Co. KG ETA-13/0029 Table 2.3 ETA-13/0029 Table 2.4
Characteristic Withdrawal Capacity	$$F_{ax,\alpha,Rk} = \frac{f_{ax,k} d l_{ef}}{1.2\cos(\alpha)^2 + \sin(\alpha)^2}\left(\frac{\rho_k}{350}\right)^{0.8}$$ $$F_{ax,\alpha,Rk} = \frac{(11)(8)(160)}{1.2\cos(45)^2 + \sin(45)^2}\left(\frac{480}{350}\right)^{0.8} = 17179.5 \text{ N}$$	ETA-13/0029 Table 2.3
Characteristic Load-Carrying Capacity	$$F_{Rk} = (\cos(\alpha) + \mu\sin(\alpha))min\begin{cases} F_{ax,\alpha,Rk} \\ f_{tens,k} \end{cases}$$ $$F_{Rk} = (\cos(45) + 0\sin(45))(17kN) = 8.93 \text{ kN}$$	
Slip Modulus	$K_{ser} = 100 l_{ef}$ $K_{ser} = 100(170\ mm) = 16000$ N/mm	ETA-13/0029 Table 2.3
Slip Modulus Ultimate	$K_u = (2/3)K_{ser}$ $K_u = (2/3)(17000\ N/mm) =$ 10667 N/mm Conversions F_{Rk} = 2.0077 kips K_{ser} = 91.3624 k/in K_u = 60.9082 k/in	
	Spacing between connectors in rows at the ends s_e = 152.4 mm = 6 in in rows in the middle s_m = 304.8 mm = 12 in Effective Spacing between s_{eff} = 190.5 mm = 7.5 in Number of rows of connectors n_r = 1 Deformation factor for long term loading for concrete k_{def_c} = 2.5 loading for timber k_{def_t} = 0.9 loading for STS k_{def_sts} = 0.6 Stiffness reduction for ULS ψ_2 = 0.3	ETA-13/0029 Table 2.1
Connector Stiffness Adjustment for Long Term Loading	$$K_{SLS\_LT} = \frac{K_{SER}}{1 + k_{def\_sts}} = \frac{17000\ N/mm}{1 + 0.6} = 10000 \text{ N/mm}$$ $$K_{ULS\_LT} = \frac{K_u}{1 + \psi_2 k_{def\_sts}} = \frac{11333\ N/mm}{1 + (0.3 * 0.6)} = 9039.55 \text{ N/mm}$$	

STEP DESCRIPTION	COMPUTATION	REFERNCE
General Values	EFFECTIVE STIFFNESS Gamma Span Length $L = L_0$ = 24 ft Connector slip Modulus K_{ser} = 16 kN/mm = 91.36 kips/in Connector Spacing s_{eff} = 7.5 in 42.83	Annex B of EN 1995-1-1
Concrete Properties	Slab Depth h_1 = 2.75 in Compressive Strength f'_c = 4000 psi Weight of Concrete w_c = 150 pcf	
CLT Properties	Clt Height h_2 = 6.90 in $EA_{eff,CLT}$ = 90969.6 kip $EI_{eff,CLT}$ = 471490 kip-in^2	CLT Handbook EQ 24 & 25
Modulus of Elasticity	$E_{1\_SLS} = w_c{}^{1.5} 33\sqrt{f'_c}$ $E_{1\_SLS} = (150\ pcf)^{1.5} 33\sqrt{4000\ psi} =$ 3834.25 ksi	ACI 318, 19.2.2.1.a
Area of Concrete	$A_{c\_SLS} = h_c b_c$ $A_{c\_SLS} = (2.75\ in)(12\ in) =$ 33 in^2	EN 1995.1-1 Annex B EQ 3.2
Inertia of Concrete	$I_{1\_SLS} = b_1 h_1{}^2/12$ $I_{1\_SLS} = (12\ in)(2.75\ in)^2/12 =$ 20.8 in^4	EN 1995.1-1 Annex B EQ 3.3
	$E_c A_{c\_SLS} = (3834.3\ ksi)(33\ in^2) =$ 126530 kip $E_c I_{c\_SLS} = (38343\ ksi)(20.8\ in^4) =$ 79740.5 kip-in^2	
Gamma Factor	$\gamma_{1\_SLS} = \left[1 + \frac{\pi^2 E_1 A_1 s_1}{K_1 L_0{}^2}\right]^{-1}$ $\gamma_{1\_SLS} = \left[1 + \frac{\pi^2 (3834.3\ ksi)(33\ in^2)(7.5\ in)}{(91.4\ \ kip/in)(24\ ft)^2}\right]^{-1} =$ 0.44724	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2\_SLS} = \frac{\gamma_1 E_1 A_1 (h_1 + h_2)}{2\gamma_1 E_1 A_1 + 2\gamma_2 EA_{eff}}$ $a_{2\_SLS} = \frac{(0.4472)(3834\ ksi)(33\ in^2)(2.75\ in + 6.90\ in)}{2(0.4472)(3834\ ksi)(33\ in^2) + 2(1.0)(90970\ kip)} =$ 1.85039 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1\_SLS} = \frac{h_1 + h_2}{2} - a_{2\_SLS}$ $a_{1\_SLS} = \frac{6.90\ in + 2.75\ in}{2} - 1.85 =$ 2.97461 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_SLS} = (E_1 I_1 + \gamma_1 E_1 A_1 a_1{}^2) + (EI_{eff} + \gamma_2 EA_{eff} a_2{}^2)$ $(EI)_{eff,comp\_SLS} =$ 580454.8 kip-in^2 + 782966 kip-in^2 = 1363421 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Composite & CLT Effective	$Ratio_{CLT} = \frac{EI_{eff,comp\_SLS}}{EI_{eff,CLT}}$ $Ratio_{CLT} = \frac{1363421\ kip \cdot in^2}{471489.9\ kip \cdot in^2} =$ 2.892	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	APPARENT STIFFNESS	
Bare CLT Stiffness	Length L = 24 ft K Value K_s = 11.5 $EI_{eff,clt}$ = 471490 kip-in^2 $GA_{eff,clt}$ = 1075.5 kips	
	$EI_{app,CLT\_SLS} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{471489.9\ kip \cdot in^2}{1+\frac{(11.5)(471489.9\ kip \cdot in^2)}{(1075.5\ kip)(24\ ft)^2}} =$ 444475 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{444475\ kip \cdot in^2}{471489.9\ kip \cdot in^2} =$ 94.3%	
Composite Section Stiffness	L = 24 ft K_s = 11.5 $EI_{eff,comp}$ = 1363421 kip-in^2 $GA_{eff,clt}$ = 1075.5 kips	
	$EI_{app,comp\_SLS} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{1363421\ kip \cdot in^2}{1+\frac{(11.5)(1363421\ kip \cdot in^2)}{(1075.5\ kip)(24\ ft)^2}} =$ 1159611 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{1159611\ kip \cdot in^2}{1363421\ kip \cdot in^2} =$ 85.1%	
	$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{1159611\ kip \cdot in^2}{444475\ kip \cdot in^2} =$ 2.609	
Apparent Floor Stiffness	L = 24 ft $EI_{app,CLT}$ = 444475 kip-in^2 $EI_{app,comp}$ = 1159611 kip-in^2	
	$K_{app,CLT\_SLS} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$	
	$K_{app,CLT\_SLS} = \frac{48(444475.0\ kip \cdot in^2)}{(24\ ft)^3} =$ 0.89312 kip/in	
	$K_{app,comp\_SLS} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$	
	$K_{app,comp\_SLS} = \frac{48(11159611\ kip \cdot in^2)}{(24\ ft)^3} =$ 2.33011 kip/in	

<table>
<tr><th>STEP DESCRIPTION</th><th>COMPUTATION</th><th>REFERNCE</th></tr>
<tr><td></td><td>BENDING STRESSES & STRAINS</td><td></td></tr>
<tr><td>Ultimate Load Demands</td><td>Length $L = 24$ ft
Shear $V_u = 1.49$ kip
Moment $M_u = 107.57$ kip-in</td><td></td></tr>
<tr><td></td><td>Concrete / CLT
Modulus of Elasticity $E_1 = 3834.25$ ksi, $E_2 = 1800$ ksi
Height $h_1 = 2.75$ in, $h_2 = 6.90$ in
Centroid $a_1 = 2.97461$ in, $a_2 = 1.85039$ in
Gamma Factor $\gamma_1 = 0.44724$, $\gamma_2 = 1$
$EI_{eff,comp\_SLS} = 1363421$ kip-in^2</td><td></td></tr>
<tr><td></td><td>Bending Stress Calculations</td><td></td></tr>
<tr><td>Average Concrete Stress</td><td>$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.447)(3834\,ksi)(2.97\,in)(108\,kip\cdot in)}{1363421\,kip\cdot in^2} =$ 0.402 ksi</td><td>EN 1995.1-1 Annex B EQ 3.7</td></tr>
<tr><td>Extreme Fiber Stress</td><td>$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(3834\,ksi)(2.75\,in)(108\,kip\cdot in)}{1336421\,kip\cdot in^2} =$ 0.416 ksi</td><td>EN 1995.1-1 Annex B EQ 3.8</td></tr>
<tr><td>Stress at Top of Concrete</td><td>$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.402 + 0.416 =$ 0.818 ksi</td><td></td></tr>
<tr><td>Stress at Bottom of Concrete</td><td>$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.402 - 0.416 =$ ##### ksi</td><td></td></tr>
<tr><td>Average CLT Stress</td><td>$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(1800\,ksi)(1.850\,in)(108\,kip\cdot in)}{1363421\,kip\cdot in^2} =$ 0.263 ksi</td><td>EN 1995.1-1 Annex B EQ 3.7</td></tr>
<tr><td>Extreme Fiber Stress</td><td>$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(1800\,ksi)(6.90\,in)(108\,kip\cdot in)}{1363421\,kip\cdot in^2} =$ 0.49 ksi</td><td>EN 1995.1-1 Annex B EQ 3.8</td></tr>
<tr><td>Stress at Top of CLT</td><td>$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} + \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.263 + 0.490 =$ 0.227 ksi</td><td></td></tr>
<tr><td>Stress at Bottom of CLT</td><td>$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.263 - 0.49 =$ -0.75 ksi</td><td></td></tr>
</table>

Serviceability Limit State
Short-Term Loading

STEP DESCRIPTION	COMPUTATION	REFERNCE
	DEFLECTION CALCUATIONS	
	Short-Term / Long-Term	
Loading	Dead Load: w_{DL} = 54.50 plf; Long-Term: w_{DL} = 54.50 plf Superimposed Dead Load: w_{SDL} = 20.00 plf; Long-Term: w_{SDL} = 15.64 plf Live Load: w_{LL} = 50.00 plf; Long-Term: w_{LL} = 10.9 plf Span Length: L = 24 ft; Long-Term: L = 24 ft Stiffness: $EI_{eff,comp\_SLS}$ = 1363421 kip-in^2	
	Short-Term Deflections	
Dead Load	$\Delta_{DL}=\dfrac{5w_{DL}L^4}{384EI_{eff,comp\_SLS}}=\dfrac{5(54.5\,plf)(24\,ft)^4}{384(1363420.9\,kip\cdot in^2)}=$ 0.2984 in	
Superimposed Dead Load	$\Delta_{SDL}=\dfrac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS}}=\dfrac{5(20\,plf)(24\,ft)^4}{384(1363420.9\,kip\cdot in^2)}=$ 0.1095 in	
Live Load	$\Delta_{LL}=\dfrac{5w_{LL}L^4}{384EI_{eff,comp\_SLS}}=\dfrac{5(50\,plf)(24\,ft)^4}{384(1363420.9\,kip-in^2)}=$ 0.2738 in	
	Serviceability Check	
Allowable LL Deflection	$\Delta_{allow,LL}=\dfrac{L}{360}=\dfrac{(24ft)(12\,in)}{360(1\,ft)}=$ 0.80 in	IBC Table 1604.3
Check	$\Delta_{LL}\le\ \Delta_{allow,LL}\rightarrow\Delta_{LL}=0.274\,in\ <\ \Delta_{allow,LL}=0.80\,in$ ∴ ACCEPTABLE	
	Long-Term Deflections	
Dead Load	$\Delta_{DL\_LT}=\dfrac{5w_{DL}L^4}{384EI_{eff,comp\_SLS\_LT}}=\dfrac{5(54.5\,plf)(24\,ft)^4}{384(1363421\,kip\cdot in^2)}=$ 0.2984 in	
Superimposed Dead Load	$\Delta_{SDL\_LT}=\dfrac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS\_LT}}=\dfrac{5(15.6\,plf)(24\,ft)^4}{384(1363421\,kip\cdot in^2)}=$ 0.08563 in	
Live Load	$\Delta_{LL\_LT}=\dfrac{5w_{LL}L^4}{384EI_{eff,comp\_SLS\_LT}}=\dfrac{5(10.9\,plf)(24\,ft)^4}{384(1363421\,kip\cdot in^2)}=$ 0.05968 in	
	Serviceability Check Time Dependent Creep Factor $K_{cr}=$ 2.0 Deflection due to short-term $\Delta_{ST}=$ 0.2738 in Deflection due to long-term $\Delta_{LT}=$ 0.38403 in $\Delta_{LT}=\Delta_{DL\_LT}+\Delta_{SDL\_LT}$	NDS 2018 Section 3.5.1
	$\Delta_{TL}=K_{cr}\Delta_{LT}+\Delta_{ST}$ $\Delta_{TL}=2.0(0.3840\text{ in})+(0.2738\text{ in})=$ 1.042 in	NDS EQ 3.5-1
	$\Delta_{allow,TL}=\dfrac{L}{240}=\dfrac{(24ft)(12\,in)}{240(1\,ft)}=$ 1.20 in	IBC Table 1604.3
	$\Delta_{TL}\le\ \Delta_{allow,TL}\rightarrow\Delta_{TL}=1.042\,in\ <\ \Delta_{allow,TL}=1.20\,in$ ∴ ACCEPTABLE	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	EFFECTIVE STIFFNESS	
	Deformation factor for long term	
	loading for concrete k_{def_c} = 2.5	EN 1995.1-1
	loading for timber k_{def_t} = 0.9	
	loading for STS k_{def_sts} = 0.6	
	Stiffness reduction for ULS ψ_2 = 0.3	ETA-13/0029 Table 2.1
	Effective Spacing between s_{eff} = 190.5 mm = 7.5 in	
	Gamma Span Length L_0 = 24 ft	
	Connector slip Modulus K_{SLS_LT} = 10 kN/mm = 57.10 kips/in	
	Concrete / CLT	
	Modulus of Elasticity E_1 = 3834.25 ksi; E_2 = 1800 ksi	
	Area A_1 = 33 in^2; A_1 = 26.60 in^2	
	Momemnt of Inertia I_1 = 20.7969 in^4; I_1 = 137.863 in^4	
	Height h_1 = 2.75 in; h_2 = 6.90 in	
	EA_{eff,CLT_SLS_LT} = 47878.74 kip	CLT Handbook EQ 24 & 25
	EI_{eff,CLT_SLS_LT} = 248153 kip-in^2	
Modulus of Adjustment for long term loading	$E_{1\_SLS\_LT} = \dfrac{E_1}{1+k_{def\_c}} =$ 1095.5 ksi; $E_{2\_SLS\_LT} = \dfrac{E_2}{1+k_{def\_t}} =$ 947.368 ksi	EN 1995.1-1
Gamma Factor	$\gamma_{1,SLS\_LT} = \left[1+\dfrac{\pi^2 E_{1\_SLS\_LT} A_1 s_{eff}}{K_{ser\_LT} {L_0}^2}\right]^{-1}$ $\gamma_{1,SLS\_LT} = \left[1+\dfrac{\pi^2(1095\ ksi)(33\ in^2)(7.5\ in)}{(57.1\ kip/in)(24\ ft)^2}\right]^{-1} =$ 0.639	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2,SLS\_LT} = \dfrac{\gamma_{1\_SER} E_1 A_1 (h_1+h_2)}{2\gamma_{1\_SER} E_1 A_1 + 2\gamma_2 EA_{eff}}$ $a_{2,SLS\_LT} = \dfrac{(0.639)(1095\ ksi)(33\ in^2)(2.75\ in + 6.90\ in)}{2(0.639)(1095\ ksi)(33\ in^2) + 2(1.0)(47878.7\ kip)} =$ 2.30763 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1,SLS\_LT} = \dfrac{h_1+h_2}{2} - a_{2,SLS\_LT}$ $a_{1,SLS\_LT} = \dfrac{6.90\ in + 2.75\ in}{2} - 2.31 =$ 2.517 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_SLS\_LT} = (E_1 I_1 + \gamma_{1\_SER} E_1 A_1 {a_{1\_SER}}^2) + (EI_{eff} + \gamma_2 EA_{eff} {a_{2\_SER}}^2)$ $(EI)_{eff,comp\_SLS\_LT} =$ 169170 kip-in^2 + 503115 kip-in^2 $(EI)_{eff,comp\_SLT\_LT} =$ 672285.3 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Compsite & CLT Effective	$Ratio_{CLT\_SER} = \dfrac{EI_{eff,comp\_SER}}{EI_{eff,CLT}}$ $Ratio_{CLT\_SER} = \dfrac{672285.3\ kip \cdot in^2}{248152.6\ kip \cdot in^2} =$ 14.04	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	APPARENT STIFFNESS	
Bare CLT Stiffness	Length $L = 24$ ft K Value $K_s = 11.5$ $EI_{eff,clt} = 248153$ kip-in^2 $GA_{eff,clt} = 566.1$ kips	
	$EI_{app,CLT\_SLS\_LT} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{248152.6\ kip \cdot in^2}{1+\frac{(11.5)(248152.6\ kip \cdot in^2)}{(566.1\ kip)(24\ ft)^2}} = 233934$ kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{233934.2\ kip \cdot in^2}{248152.6\ kip \cdot in^2} =$ 94.3%	
Composite Section Stiffness	$L = 24$ ft $K_s = 11.5$ $EI_{eff,comp} = 672285$ kip-in^2 $GA_{eff,clt} = 566.1$ kips	
	$EI_{app,comp\_SLS\_LT} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{672285.3\ kip \cdot in^2}{1+\frac{(11.5)(672285.3\ kip \cdot in^2)}{(566.1\ kip)(24\ ft)^2}} = 577237$ kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{577236.8\ kip \cdot in^2}{672285.3\ kip \cdot in^2} =$ 85.9%	
	$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{577236.8\ kip \cdot in^2}{233934.2\ kip \cdot in^2} = 2.468$	
Apparent Floor Stiffness	$L = 24$ ft $EI_{app,CLT} = 233934$ kip-in^2 $EI_{app,comp} = 577237$ kip-in^2	
	$K_{app,CLT\_SLS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$	
	$K_{app,CLT\_SLS\_LT} = \frac{48(233934.2\ kip \cdot in^2)}{(24\ ft)^3} = 0.47006$ kip/in	
	$K_{app,comp\_SLS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$	
	$K_{app,comp\_SLS\_LT} = \frac{48(577236.8\ kip \cdot in^2)}{(24\ ft)^3} = 1.15989$ kip/in	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length L = 24 ft	
	Shear V_u = 0.97 kip	
	Moment M_u = 70.02 kip-in	
	Concrete / CLT	
	Modulus of Elasticity E_1 = 1095.5 ksi; E_2 = 947.368 ksi	
	Height h_1 = 2.75 in; h_2 = 6.90 in	
	Centroid a_1 = 2.51737 in; a_2 = 2.30763 in	
	Gamma Factor γ_1 = 0.63897; γ_2 = 1	
	$EI_{eff,comp\_SLS\_LT}$ = 672285 kip-in^2	
	Bending Stess Calulations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.718)(1096\ ksi)(2.52\ in)(83.3\ kip \cdot in)}{672285.3\ kip \cdot in^2} =$ 0.184 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(1096\ ksi)(2.75\ in)(83.3\ kip \cdot in)}{720748.4\ kip \cdot in^2} =$ 0.157 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.229 + 0.174 =$ 0.340 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.229 - 0.174 =$ 0.027 ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(1000\ ksi)(2.31\ in)(83.3\ kip \cdot in)}{720748.4\ kip \cdot in^2} =$ 0.228 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(1000\ ksi)(6.90\ in)(83.3\ kip \cdot in)}{720748.4\ kip \cdot in^2} =$ 0.34 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.267 + 0.399 =$ 0.113 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.27 - 0.39 =$ -0.57 ksi	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	DEFLECTION CALCUATIONS	
	Short-Term / Long-Term	
Loading	Dead Load w_{DL} = 54.50 plf; w_{DL} = 54.50 plf	
	Superimposed Dead Load w_{SDL} = 20.00 plf; w_{SDL} = 15.64 plf	
	Live Load w_{LL} = 50.00 plf; w_{LL} = 10.9 plf	
	Span Length L = 24 ft; L = 24 ft	
	Stiffness $EI_{eff,comp\_SLS}$ = 1363421 kip-in^2; $EI_{eff,comp\_SLS\_LT}$ = 672285.3 kip-in^2	
	Short-Term Deflections	
Dead Load	$\Delta_{DL}= \frac{5w_{DL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(54.5\,plf)(25\,ft)^4}{384(1458005.4\,kip\cdot in^2)} =$ 0.2984 in	
Superimposed Dead Load	$\Delta_{SDL}= \frac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(30\,plf)(25\,ft)^4}{384(1458005.4\,kip\cdot in^2)} =$ 0.1095 in	
Live Load	$\Delta_{LL}= \frac{5w_{LL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(50\,plf)(25\,ft)^4}{384(1458005.4\,kip - in^2)} =$ 0.2738 in	
	Servicablility Check	
Allowable LL Deflection	$\Delta_{allow,LL}= \frac{L}{360} = \frac{(25ft)(12\,in)}{360(1\,ft)} =$ 0.80 in	IBC Table 1604.3
Check	$\Delta_{LL} \leq \Delta_{allow,LL} \rightarrow \Delta_{LL}= 0.3014\,in < \Delta_{allow,LL}= 0.83\,in$ $\therefore$ ACCEPTABLE	
	Long-Term Deflections	
Dead Load	$\Delta_{DL\_LT}= \frac{5w_{DL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(54.5\,plf)(25\,ft)^4}{384(720748.4\,kip\cdot in^2)} =$ 0.60516 in	
Superimposed Dead Load	$\Delta_{SDL\_LT}= \frac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(23.5\,plf)(25\,ft)^4}{384(720748.4\,kip\cdot in^2)} =$ 0.17366 in	
Live Load	$\Delta_{LL\_LT}= \frac{5w_{LL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(10.9\,plf)(25\,ft)^4}{384(720748.4\,kip\cdot in^2)} =$ 0.12103 in	
	Servicablility Check	
	Deflection due to short-term Δ_{ST} = 0.2738 in	
	Deflection due to long-term Δ_{LT} = 0.77882 in $\quad \Delta_{LT}= \Delta_{DL\_LT} + \Delta_{SDL\_LT}$	
	Deflection due to Total Load Δ_{TL} = 1.0526 in $\quad \Delta_{TL}= \Delta_{LT} + \Delta_{ST}$	
	$\Delta_{allow,TL}= \frac{L}{240} = \frac{(25ft)(12\,in)}{240(1\,ft)} =$ 1.20 in	IBC Table 1604.3
	$\Delta_{TL} \leq \Delta_{allow,TL} \rightarrow \Delta_{TL}= 1.252\,in < \Delta_{allow,TL}= 1.25\,in$ $\therefore$ ACCEPTABLE	

STEP DESCRIPTION	COMPUTATION	REFERNCE
General Values	EFFECTIVE STIFFNESS Gamma Span Length L = L_0 = 24 ft Connector slip Modulus K_U = 10.6667 kN/mm = 60.91 kips/in Connector Spacing s_{eff} = 7.5 in	Annex B of EN 1995-1-1
Concrete Properties	Slab Depth h_1 = 2.75 in Compressive Strength f_c' = 4000 psi Weight of Concete w_c = 150 pcf	
CLT Properties	Clt Height h_2 = 6.90 in $EA_{eff,CLT}$ = 90969.6 kip $EI_{eff,CLT}$ = 471490 kip-in^2	CLT Handbook EQ 24 & 25
Modulus of Elasticity	$E_{1\_ULS} = w_c{}^{1.5}33\sqrt{f_c'}$ $E_{1\_ULS} = (150\,pcf)^{1.5}33\sqrt{4000\,psi} =$ 3834.25 ksi	ACI 318, 19.2.2.1.a
Area of Concrete	$A_{c\_ULS} = h_c b_c$ $A_{c\_ULS} = (2.75\,in)(12\,in) =$ 33 in^2	EN 1995.1-1 Annex B EQ 3.2
Inhertia of Concrete	$I_{1\_ULS} = b_1 h_1{}^2/12$ $I_{1\_ULS} = (12\,in)(2.75\,in)^2/12 =$ 20.8 in^4	EN 1995.1-1 Annex B EQ 3.3
	$E_c A_{c\_ULS} = (3834.3\,ksi)(33\,in^2) =$ 126530 kip $E_c I_{c\_ULS} = (38343\,ksi)(20.8\,in^4) =$ 79740.5 kip-in^2	
Gamma Factor	$\gamma_{1\_ULS} = \left[1 + \frac{\pi^2 E_1 A_1 s_1}{K_1 L_0{}^2}\right]^{-1}$ $\gamma_{1\_ULS} = \left[1 + \frac{\pi^2(3834.3\,ksi)(33\,in^2)(6\,in)}{(64.7\ kip/in)(25\,ft)^2}\right]^{-1} =$ 0.35039	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2\_ULS} = \frac{\gamma_1 E_1 A_1 (h_1 + h_2)}{2\gamma_1 E_1 A_1 + 2\gamma_2 EA_{eff}}$ $a_{2\_ULS} = \frac{(0.4376)(3834\,ksi)(33\,in^2)(2.75\,in + 6.90\,in)}{2(0.4376)(3834\,ksi)(33\,in^2) + 2(1.0)(90970\,kip)} =$ 1.58101 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1\_ULS} = \frac{h_1 + h_2}{2} - a_{2\_ULS}$ $a_{1\_ULS} = \frac{6.90\,in + 2.75\,in}{2} - 1.82 =$ 3.24399 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_ULS} = (E_1 I_1 + \gamma_1 E_1 A_1 a_1{}^2) + \left(EI_{eff} + \gamma_2 EA_{eff} a_2{}^2\right)$ $(EI)_{eff,comp\_ULS} =$ 546303.3 kip-in^2 + 698876 kip-in^2 = 1245179 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Compsite & CLT Effective	$Ratio_{CLT} = \frac{EI_{eff,comp\_ULS}}{EI_{eff,CLT\_ULS}}$ $Ratio_{CLT} = \frac{1352266\,kip \cdot in^2}{471489.9\,kip \cdot in^2} =$ 2.641	

<table>
<tr><th>STEP DESCRIPTION</th><th>COMPUTATION</th><th>REFERNCE</th></tr>
<tr><td></td><td>APPARENT STIFFNESS</td><td></td></tr>
<tr><td>Bare CLT Stiffness</td><td>Length L = 24 ft
K Value K_s = 11.5
$EI_{eff,clt}$ = 471490 kip-in^2
$GA_{eff,clt}$ = 1075.5 kips
$$EI_{app,CLT\_ULS} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{471489.9\ kip\cdot in^2}{1+\frac{(11.5)(471489.9\ kip\cdot in^2)}{(1075.5\ kip)(25\ ft)^2}} = 444475 \text{ kip-in}^2$$
$$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{446480.6\ kip\cdot in^2}{471489.9\ kip\cdot in^2} = 94.3\%$$</td><td>CLT Handbook EQ 35</td></tr>
<tr><td>Composite Section Stiffness</td><td>L = 24 ft
K_s = 11.5
$EI_{eff,comp}$ = 1245179 kip-in^2
$GA_{eff,clt}$ = 1075.5 kips
$$EI_{app,comp\_ULS} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{1352266\ kip\cdot in^2}{1+\frac{(11.5)(1352266\ kip\cdot in^2)}{(1075.5\ kip)(25\ ft)^2}} = 1072954 \text{ kip-in}^2$$
$$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{1165091\ kip\cdot in^2}{1352266\ kip\cdot in^2} = 86.2\%$$
$$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{1165091\ kip\cdot in^2}{446480.6\ kip\cdot in^2} = 2.414$$</td><td>CLT Handbook EQ 35</td></tr>
<tr><td>Apparent Floor Stiffness</td><td>L = 24 ft
$EI_{app,CLT}$ = 444475 kip-in^2
$EI_{app,comp}$ = 1072954 kip-in^2
$$K_{app,CLT\_ULS} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$$
$$K_{app,CLT\_ULS} = \frac{48(446480.6\ kip\cdot in^2)}{(25\ ft)^3} = 0.89312 \text{ kip/in}$$
$$K_{app,comp\_ULS} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$$
$$K_{app,comp\_ULS} = \frac{48(1165091\ kip\cdot in^2)}{(25\ ft)^3} = 2.15598 \text{ kip/in}$$</td><td></td></tr>
</table>

STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length L = 24 ft	
	Shear V_u = 2.03 kip	
	Moment M_u = 146.36 kip-in	
	Concrete / CLT	
	Modulus of Elasticity E_1 = 3834.25 ksi; E_2 = 1800 ksi	
	Height h_1 = 2.75 in; h_2 = 6.90 in	
	Centroid a_1 = 3.24399 in; a_2 = 1.58101 in	
	Gamma Factor γ_1 = 0.35039; γ_2 = 1	
	$EI_{eff,comp}$ = 1245179 kip-in^2	
	Bending Stress Calculations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.437)(3834\ ksi)(3.00\ in)(170\ kip \cdot in)}{1352266\ kip \cdot in^2} =$ 0.512 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(3834\ ksi)(2.75\ in)(107\ kip \cdot in)}{1352266\ kip \cdot in^2} =$ 0.620 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.633 + 0.663 =$ 1.132 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{+\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.633 - 0.663 =$ ##### ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(1800\ ksi)(1.82\ in)(170\ kip \cdot in)}{1352266\ kip \cdot in^2} =$ 0.335 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(1800\ ksi)(6.90\ in)(170\ kip \cdot in)}{1352266\ kip \cdot in^2} =$ 0.73 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} + \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.413 + 0.781 =$ 0.395 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.43 - 0.71 =$ -1.06 ksi	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	STRENGTH ANALYSIS	
Concrete	Normal Stress in Concrete σ_1 = 512.3 psi Allow Comp Strength of Conc F_c = 2000 psi $F_c = f_c'/2$ $\sigma_1 < F_c \rightarrow \sigma_1 = 632.7\,psi < F_c = 2000\,psi$ ∴ ACCEPTABLE	
Timber	Bending Strength F_b = 2100 psi Format Conversion Factor K_F = 2.54 Resistance Factor Φ = 0.85 Design Bending Strength $F_b' = \Phi F_b K_F = 0.85(2100\,psi)(2.54) =$ 4533.9 psi Tension Strength F_t = 1575 psi Format Conversion Factor K_F = 2.70 Resistance Factor Φ = 0.80 Design Tension Strength $F_t' = \Phi F_t K_F = 0.80(1575\,psi)(2.70) =$ 3402.0 psi Average CLT Stress σ_2 = 334.5 psi Extreme Fiber Stress $\sigma_{m,2}$ = 729.9 psi $\frac{\sigma_2}{F_t'} + \frac{\sigma_{m,2}}{F_b'} \leq 1 \rightarrow \frac{413.1\,psi}{3402\,psi} + \frac{781.0\,psi}{4534\,psi} =$ 0.259 < 1.0 ∴ ACCEPTABLE Shear Strength F_v = 160 psi Format Conversion Factor K_F = 2.88 Resistance Factor Φ = 0.75 Design Shear Strength $F_v' = \phi F_v K_F = 0.75(160\,psi)(2.88) =$ 345.6 psi CLT Modulus of Elasticity E_2 = 1800 ksi NA of timber h = 5.03 in Shear V = 2.03 kip $EI_{eff,comp}$ = 1245179 kip-in^2 $\tau_{2,max} = \frac{E_2 h^2 V_u}{2(EI)_{eff,comp}} = \frac{(1800\,ks)(6.90in)^2(2.27\,kip)}{2(1352266\,kip \cdot in^2)} =$ 37.19 psi $\tau_{2,max} < F_v' \rightarrow \tau_{2,max} = 71.85\,psi < F_v' = 345.6\,psi$ ∴ ACCEPTABLE	EN 1995.1-1 Annex B EQ 3.9

STEP DESCRIPTION	COMPUTATION	REFERNCE
Fastener Load Calculation	STRENGTH ANALYSIS F_{ULT} = 2.00765 kips Gamma Factor γ_1 = 0.35039 ksi Concrete Modulus of Elasticity E_1 = 3834.25 in Concrete Area A_1 = 33.00 in^2 Composite Centroid a_1 = 3.24 in Fastener Spacing s = 7.5 in Ultimate Shear Loading V = 2.0328 kip $EI_{eff,comp\_ULS}$ = 1245179 kip-in^2 $F_q = \frac{\gamma_1 E_1 A_1 a_1 s V}{EI_{eff,comp}}$ $F_q = \frac{(0.437)(3834\ ksi)(33 in^2)(3.00\ in)(6\ in)(2.268\ kip)}{1352266\ kip \cdot in^2} =$ 1.761 kip $F_q < F_{ULT} \rightarrow F_q = 1.67\ kip < F_{ULT} = 2.01\ kip$ ∴ ACCEPTABLE	EN 1995.1-1 Annex B EQ 3.10

<table>
<tr><th>STEP DESCRIPTION</th><th>COMPUTATION</th><th>REFERNCE</th></tr>
<tr><td></td><td>EFFECTIVE STIFFNESS
Deformation factor for long term
loading for concrete k_{def_c} = 2.5
loading for timber k_{def_t} = 0.8
loading for STS k_{def_sts} = 0.6
Stiffness reduction for ULS ψ_2 = 0.3
Effective Spacing between s_{eff} = 190.5 mm = 7.5 in
Gamma Span Length L_0 = 24 ft
Connector slip Modulus K_{U_LT} = 9.03955 kN/mm = 51.62 kips/in</td><td>EN 1995.1-1

ETA-13/0029 Table 2.1</td></tr>
<tr><td></td><td>Concrete / CLT
Modulus of Elasticity E_1 = 3834.25 ksi; E_2 = 1800 ksi
Area A_1 = 33 in^2; A_1 = 49.3448 in^2
Momemnt of Inertia I_1 = 20.7969 in^4; I_1 = 255.751 in^4
Height h_1 = 2.75 in; h_2 = 6.90 in
EA_{eff,CLT_ULT_LT} = 71629.6 in
EI_{eff,CLT_ULT_LT} = 371252 kip-in^2</td><td>CLT Handbook EQ 24 & 25</td></tr>
<tr><td>Modulus of Adjustment for long term loading</td><td>$E_{1,ULT\_LT} = \frac{E_1}{1+\psi_2 k_{def\_c}} =$ 2191 ksi; $E_{2,ULT\_LT} = \frac{E_2}{1+\psi_2 k_{def\_t}} =$ 1451.61 ksi</td><td>EN 1995.1-1</td></tr>
<tr><td>Gamma Factor</td><td>$\gamma_{1_{ULT\_LT}} = \left[1+\frac{\pi^2 E_{1,ULT\_LT} A_1 s_{eff}}{K_{ULT\_LT} {L_0}^2}\right]^{-1}$
$\gamma_{1,ULT\_LT} = \left[1+\frac{\pi^2(2191\ ksi)(33\ in^2)(6\ in)}{(54.84\ kip/in)(25\ ft)^2}\right]^{-1} =$ 0.444</td><td>EN 1995.1-1 Annex B EQ 3.5</td></tr>
<tr><td>Timber to Composite Centroid</td><td>$a_{2,ULT\_LT} = \frac{\gamma_{1\_ULT\_LT} E_{1,ULT\_LT} A_1 (h_1+h_2)}{2\gamma_{1,ULT\_LT} E_{1,ULT\_LT} A_1 + 2\gamma_2 EA_{eff,CLT\_ULT\_LT}}$
$a_{2,ULT\_LT} = \frac{(0.535)(2191.0\ ksi)(33\ in^2)(2.75\ in+6.90\ in)}{2(0.533)(2191.0 ksi)(33\ in^2)+2(1.0)(73718.7\ kip)} =$ 1.494 in</td><td>EN 1995.1-1 Annex B EQ 3.6</td></tr>
<tr><td>Concrete to Composite Centroid</td><td>$a_{1,ULT\_LT} = \frac{h_1+h_2}{2} - a_{2\_ULT}$
$a_{1,ULT\_LT} = \frac{6.90\ in+2.75\ in}{2} - 1.661 =$ 3.331 in</td><td></td></tr>
<tr><td>Effective Comp Stiffness</td><td>$(EI)_{eff,comp\_ULT\_LT} = \left(E_{1,ULT\_LT} I_1 + \gamma_{1,ULT\_LT} E_{1,ULT\_LT} A_1 {a_{1,ULT\_LT}}^2\right) + \left(EI_{eff} + \gamma_2 EA_{eff} {a_{2,ULT}}^2\right)$
$(EI)_{eff,comp\_ULT\_LT} =$ 402060 kip-in^2 + 531177 kip-in^2
$(EI)_{eff,comp\_ULT\_LT} =$ 933237 kip-in^2</td><td>EN 1995.1-1 Annex B EQ 3.1</td></tr>
<tr><td>Ratio of Compsite & CLT Effective</td><td>$Ratio_{CLT\_ULT} = \frac{EI_{eff,comp\_ULT\_LT}}{EI_{eff,CLT\_ULT\_LT}}$
$Ratio_{CLT\_ULT} = \frac{1017512\ kip \cdot in^2}{380963.6\ kip \cdot in^2} =$ 13.03</td><td></td></tr>
</table>

STEP DESCRIPTION	COMPUTATION	REFERNCE
	APPARENT STIFFNESS	
Bare CLT Stiffness	Length L = 24 ft K Value K_s = 11.5 EI_{eff,CLT_ULT_LT} = 371252 kip-in^2 GA_{eff,CLT_ULT_LT} = 846.9 kips	
	$EI_{app,CLT\_ULS\_LT} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{380964\ kip\cdot in^2}{1+\frac{(11.5)(380964 kip\cdot in^2)}{(1092.7\ kip)(25\ ft)^2}} =$ 349980 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{364715.4\ kip\cdot in^2}{380963.6\ kip\cdot in^2} =$ 94.3%	
Composite Section Stiffness	L = 24 ft K_s = 11.5 $EI_{eff,comp}$ = 933237 kip-in^2 $GA_{eff,clt}$ = 846.9 kips	
	$EI_{app,comp\_ULS\_LT} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{1017512\ kip\cdot in^2}{1+\frac{(11.5)(1017512\ kip\cdot in^2)}{(1092.7\ kip)(25\ ft)^2}} =$ 809550 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{909313.7\ kip\cdot in^2}{1017512\ \ kip\cdot in^2} =$ 86.7%	
	$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{909313.7\ kip\cdot in^2}{364715.4\ kip\cdot in^2} =$ 2.313	
Apparent Floor Stiffness	L = 24 ft $EI_{app,CLT}$ = 349980 kip-in^2 $EI_{app,comp}$ = 809550 kip-in^2	
	$K_{app,CLT\_ULS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$	
	$K_{app,CLT\_ULS\_LT} = \frac{48(364715.4\ kip\cdot in^2)}{(25\ ft)^3} =$ 0.70325 kip/in	
	$K_{app,comp\_ULS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$	
	$K_{app,comp\_ULS\_LT} = \frac{48(909313.7\ kip\cdot in^2)}{(25\ ft)^3} =$ 1.6267 kip/in	

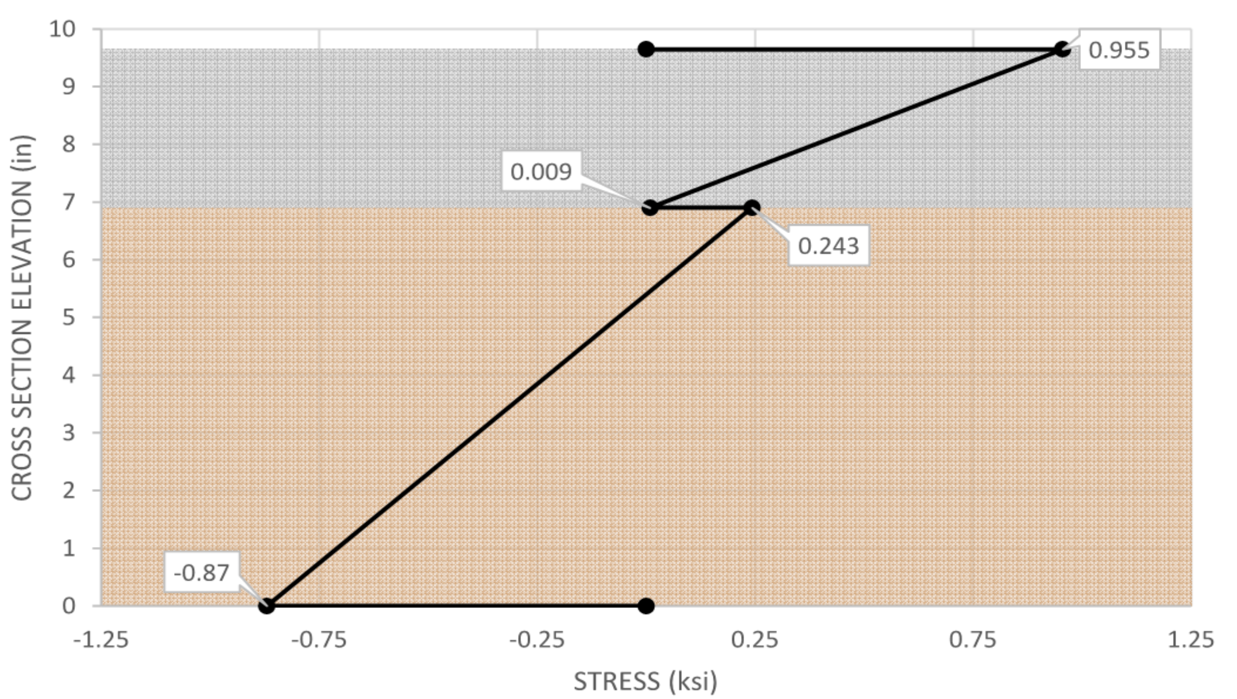
STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length $L = 24$ ft Shear $V_u = 2.03$ kip Moment $M_u = 146.36$ kip-in	
	Concrete / CLT Modulus of Elasticity $E_1 = 2191$ ksi, $E_2 = 1451.61$ ksi Height $h_1 = 2.75$ in, $h_2 = 6.90$ in Centroid $a_1 = 3.33079$ in, $a_2 = 1.49421$ in Gamma Factor $\gamma_1 = 0.44443$, $\gamma_2 = 1$ $EI_{eff,comp} = 933237$ kip-in^2	
	Bending Stess Calulations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.535)(2191\ ksi)(3.16\ in)(170\ kip \cdot in)}{1017512\ kip \cdot in^2} =$ 0.509 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(2191\ ksi)(2.75\ in)(170\ kip \cdot in)}{1017512\ kip \cdot in^2} =$ 0.472 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.620 + 0.504 =$ 0.981 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.620 - 0.504 =$ 0.036 ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(1451.6\ ksi)(1.66\ in)(170\ kip \cdot in)}{1017512\ kip \cdot in^2} =$ 0.34 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(1452\ ksi)(6.90\ in)(170\ kip \cdot in)}{1017512\ kip \cdot in^2} =$ 0.785 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} + \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.403 + 0.837 =$ 0.445 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.43 - 0.837 =$ -1.13 ksi	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	STRENGTH ANALYSIS	
Concrete	Normal Stress in Concrete σ_1 = 508.7 psi	
	Allow Comp Strength of Conc F_c = 2000 psi $F_c = f_c'/2$	
	$\sigma_1 < F_c \rightarrow \sigma_1 = 620.3\, psi < F_c = 2000\, psi$ ∴ ACCEPTABLE	
Timber	Bending Strength F_b = 2100 psi	
	Format Conversion Factor K_F = 2.54	
	Resistance Factor Φ = 0.85	
	Design Bending Stength	
	$F_b' = \Phi F_b K_F = 0.85(2100\, psi)(2.54) =$ 4533.9 psi	
	Tension Strength F_t = 1575 psi	
	Format Conversion Factor K_F = 2.70	
	Resistance Factor Φ = 0.80	
	Design Tension Stength	
	$F_t' = \Phi F_t K_F = 0.80(1575\, psi)(2.70) =$ 3402.0 psi	
	Average CLT Stress σ_2 = 340.2 psi	
	Extreme Fiber Stress $\sigma_{m,2}$ = 785.4 psi	
	$\frac{\sigma_2}{F_t'} + \frac{\sigma_{m,2}}{F_b'} \leq 1 \rightarrow \frac{403.1\, psi}{3402\, psi} + \frac{837.0\, psi}{4534\, psi} =$ 0.273 < 1.0 ∴ ACCEPTABLE	
	Shear Strength F_v = 160 psi	
	Format Conversion Factor K_F = 2.88	
	Resistance Factor Φ = 0.75	
	Design Shear Stength	
	$F_v' = \phi F_v K_F = 0.75(160\, psi)(2.88) =$ 345.6 psi	
	CLT Modulus of Elasticity E_2 = 1451.61 ksi	
	NA of timber h = 4.94 in	
	Shear V = 2.03 kip	
	$EI_{eff,comp}$ = 933237 kip-in^2	
	$\tau_{2,max} = \frac{E_2 {h_2}^2 V_u}{2(EI)_{eff,comp}} = \frac{(1452\, ksi)(6.90 in)^2(2.27\, kip)}{2(1017512\, kip \cdot in^2)} =$ 38.65 psi	EN 1995.1-1 Annex B EQ 3.9
	$\tau_{2,max} < F_v' \rightarrow \tau_{2,max} = 77.01\, psi < F_v' = 345.6\, psi$ ∴ ACCEPTABLE	

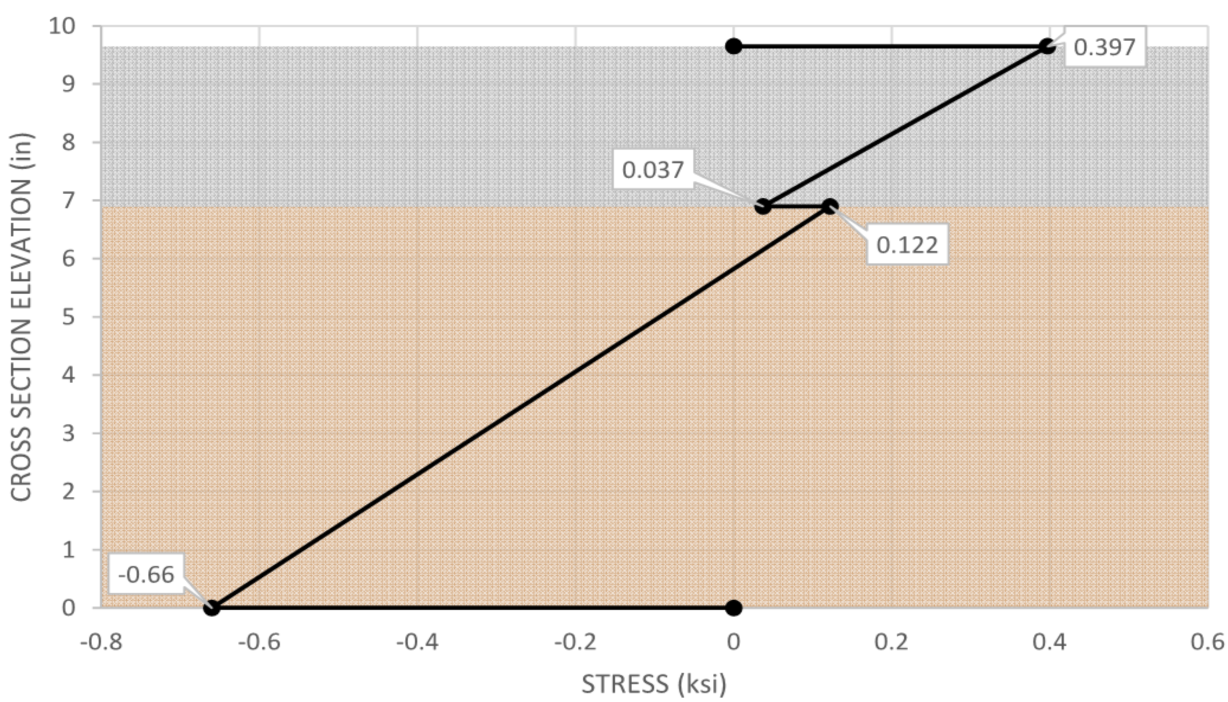
STEP DESCRIPTION	COMPUTATION	REFERNCE
Fastener Load Calculation	STRENGTH ANALYSIS F_{ULT} = 2.00765 kips Gamma Factor γ_1 = 0.44443 ksi Concrete Modulus of Elasticity E_1 = 2191.00 in Concrete Area A_1 = 33.00 in^2 Composite Centroid a_1 = 3.33 in Fastener Spacing s = 7.5 in Ultimate Shear Loading V = 2.0328 kip $EI_{eff,comp}$ = 933237 kip-in^2 $F_q = \frac{\gamma_1 E_1 A_1 a_1 sV}{EI_{eff,comp}}$ $F_q = \frac{(0.535)(2191\ ksi)(33 in^2)(3.16\ in)(6\ in)(2.268\ kip)}{1017512\ kip \cdot in^2} =$ 1.749 kip $F_q < F_{ULT} \rightarrow F_q = 1.638\ kip < F_{ULT} = 2.01\ kip$ ∴ ACCEPTABLE	EN 1995.1-1 Annex B EQ 3.10

STEP DESCRIPTION	COMPUTATION	REFERNCE
	CONNECTOR EFFICIENCY	
Partial Composite Section	Effective Bending Stiffness EI_{eff_SLS} = 1363421 in Deflection Δ_{PC} = 0.27376 in	
Non-Composite Section	Effective Bending Stiffness EI_{eff_SLS} = 551231 in (set K_{ser} close to zero) Deflection Δ_{NC} = 0.67712 in	
Fully Composite	Concrete MOE E_1 = 3834.25 psi Timber MOE E_2 = 1800 psi Width b' = 12 in Width of Transformed Concrete b' = 25.56 in $b' = b(E_1/E_2)$ Height of Timber h_1 = 2.75 in Height of Concrete h_2 = 6.90 in Bottom to Centroid of Transformed Concrete y'_1 = 8.275 in Timber Section y_2 = 3.45 in Area of Transformed Concrete A'_1 = 70.29 in^2 Area of Timber Section A_2 = 82.8 in^2 Moment of Inertia of Transformed Concrete I'_1 = 44.30 in^4 Timber Section I_2 = 328.51 in^4 Centroid of Transformed Section $y_c = \frac{y'_1 A'_1 + y_2 A_2}{A'_1 + A_2}$ = 5.665 in Distance to Concrete Centroid d_1 = 4.29 in Distance to Timber Centroid d_2 = 0.84 in $d = \left\|y_c - \frac{h}{2}\right\|$ Transformed Section Moment of Inertia about NA I_{NA} = 1725.27 in^4 $I_{NA} = \sum I_i + A_i {d_i}^2$ Bending Stiffness $E_2 I_{NA}$ = 3105482 $k\text{-}in^2$ Deflection of Fully Composite Section $\Delta_{FC} = \frac{5wL^4}{384E_2 I_{NA}} = \frac{5(50\,plf)(24ft)^4}{384(3105482\,k \cdot in^2)} =$ 0.12 in	
Shear Connector Efficiency	$Efficiency = \frac{\Delta_{NC} - \Delta_{PC}}{\Delta_{NC} - \Delta_{FC}} = \frac{1.2544" - 0.4582"}{1.2544" - 0.223"} =$ 72.4%	

8 mm dia. ASSY plus VG screw 26 FT SPAN

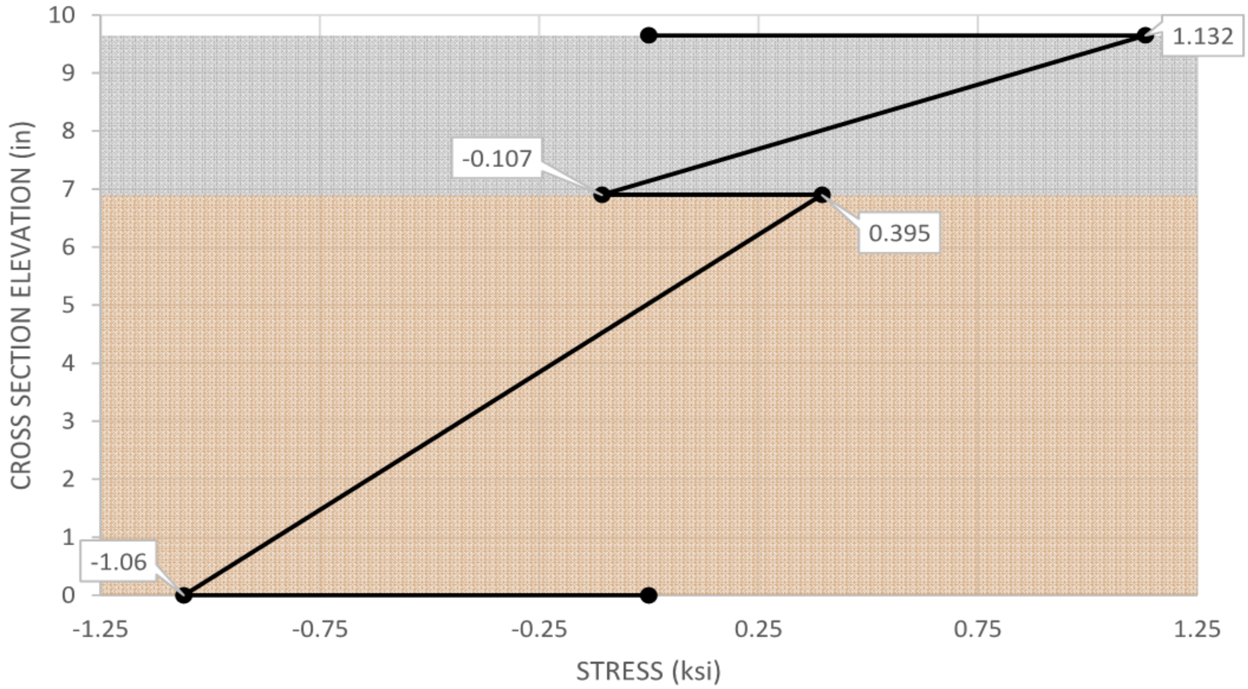


SERVICABILITY LIMIT STATE CROSS-SECTIONAL STRESS DISTRIBUTION

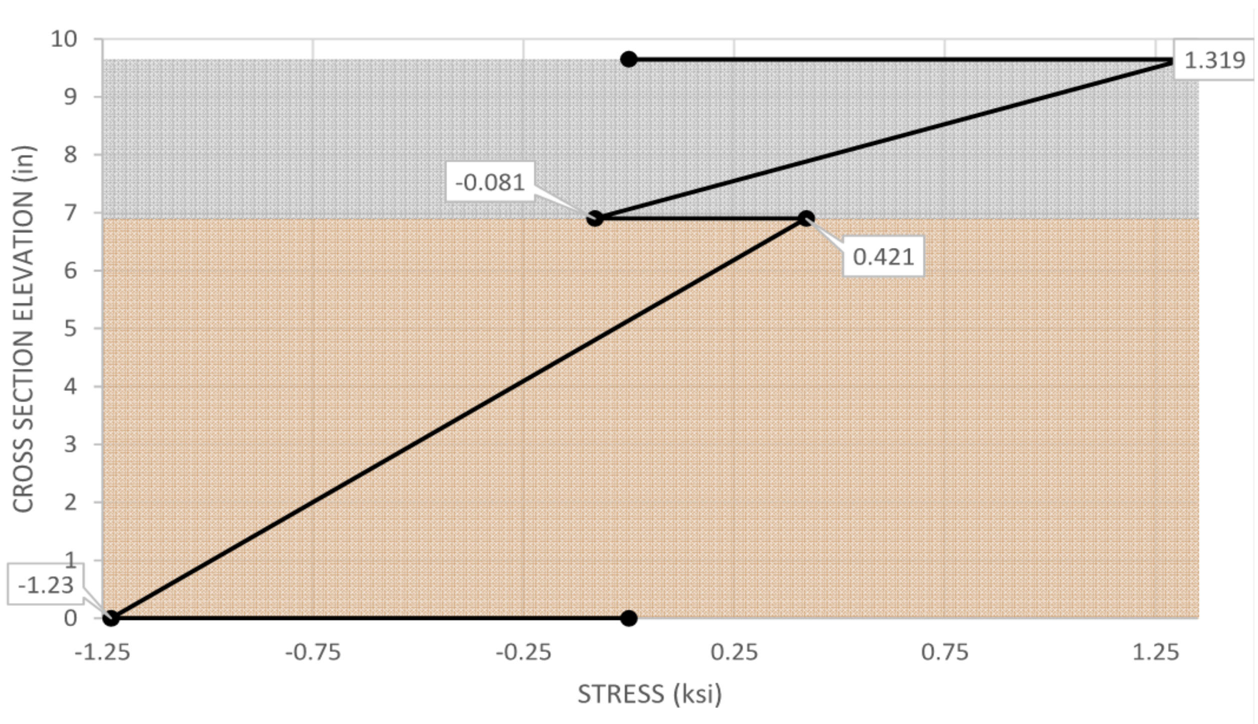


SERVICABILITY LIMIT STATE LONG-TERM CROSS-SECTIONAL STRESS DISTRIBUTION

8 mm dia. ASSY plus VG screw 26 FT SPAN



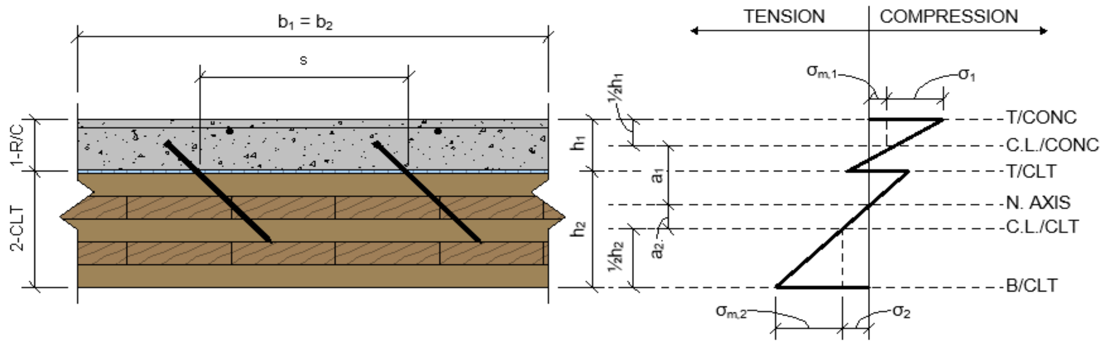
ULTIMATE LIMIT STATE CROSS-SECTIONAL STRESS DISTRIBUTION



ULTIMATE LIMIT STATE LONG-TERM CROSS-SECTIONAL STRESS DISTRIBUTION

STEP DESCRIPTION	COMPUTATION	REFERNCE

8 mm dia. ASSY plus VG screw 26 FT SPAN



Cross-section Stress Diagram

Dead Load Calculation

	Density				Height						
Mass Timber	ρ_t	=	35	pcf	h_t	=	6.90	in	L =	26	ft
Concrete	ρ_c	=	150	pcf	h_c	=	2.75	in			

$$w_{DL} = \rho_t h_t + \rho_c h_c$$
$$w_{DL} = (35\,pcf)(6.90\,in) + (150\,pcf)(2.75\,in) = \text{54.50 psf}$$

Loading Criteria

	Loading				Sustained/Expected			
Dead Load	w_{DL}	=	54.50	psf	m_{DL}	=	100%	
Superimposed Dead Load	w_{SDL}	=	20	psf	m_{SDL}	=	78.2%	
Live Load	w_{LL}	=	50	psf	m_{LL}	=	21.8%	(10.9 psf mean load)

ASCE 7-16

Table C4.3-2

LRFD

Service

Loading Cases

Load Case Factors				Equations	DL	SDL	LL	w_u
Combo	DL	SDL	LL		PSF	PSF	PSF	PLF
G01	1.4	1.4	0.0	$1.4(w_{DL} + w_{SDL})$	76.3	28	0	104.3
G02	1.2	1.2	1.6	$1.2(w_{DL} + w_{SDL}) + 1.6w_{LL}$	65.4	24	80	169.4
S01	1.0	1.0	1.0	$1.0w_{DL} + 1.0w_{SDL} + 1.0w_{LL}$	54.5	20	50	124.5
S02	0.0	1.0	1.0	$1.0w_{SDL} + 1.0w_{LL}$	0	20	50	70
S03	1.0	0.782	0.218	$1.0w_{DL} + 0.782w_{SDL} + 0.218w_{LL}$	54.5	15.64	10.9	81.04

Combo	w_u	V_u	M_u
	PLF	kips	kip-in
G01	104.3	1.356	105.8
G02	169.4	2.202	171.8
S01	124.5	1.619	126.2
S02	70	0.91	70.98
S03	81.04	1.054	82.17

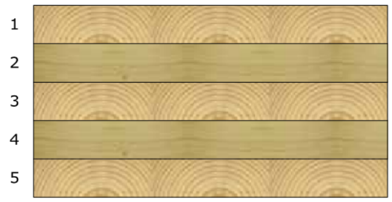
$$V_u = \frac{w_u L}{2}$$
$$M_u = \frac{w_u L^2}{8}$$

STEP DESCRIPTION	COMPUTATION	REFERNCE

CLT Properties

CLT CALCULATIONS

Reference: Structurlam Crosslam



Type: 5-Ply CLT, Grade E1M4, 139 E

h_i = 1.375 in
b = 12 in
h_t = 6.90 in
a = 5.52 in
L = 26.0 ft
K_s = 11.5

Major Strength Axis

$F_{b,0}$ = 2100 psi
E_0 = 1800 ksi
$F_{t,0}$ = 1575 psi
$F_{c,0}$ = 1875 psi
$F_{v,0}$ = 160 psi
$F_{s,0}$ = 50 psi

Minor Strength Axis

$F_{b,90}$ = 875 psi
E_{90} = 1400 ksi
$F_{v,90}$ = 135 psi
$F_{s,90}$ = 45 psi

Reference: CLT Handbook, Chapter 3

CLT Calculations

Layer	E (ksi)	h (in)	z (in)	GA_{eff} G (ksi)	GA_{eff} h/G/b (in^2/kip)	EA_{eff} EA (kip)	EI_{eff} $Ebh^3/12$ (kip-in^2)	EI_{eff} EAz^2 (kip-in^2)	EI_{eff} Sum of Layers (kip-in^2)
1	1800	1.380	2.760	112.5	0.00102	29808	4730.53	227065	231796
2	46.6667	1.380	1.375	8.75	0.01314	772.8	122.643	1461.08	1583.72
3	1800	1.380	0.000	112.5	0.00102	29808	4730.53	0	4730.53
4	46.6667	1.380	1.375	8.75	0.01314	772.8	122.643	1461.08	1583.72
5	1800	1.380	2.760	112.5	0.00102	29808	4730.53	227065	231796
						90969.6			471490

Reference: CLT Handbook Chapter 3 EQ 24

$$EA_{eff} = \sum_{i=1}^{n} E_i b_i h_i$$

$$EI_{eff} = \sum_{i=1}^{n} E_i b_i \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i A_i z_i^2$$

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

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

Cross Sectional Properties (Per foot of width)

	SLS/ULS Short-Term	SLS Long-Term	ULS Long-Term
EA_{eff}	90969.6 kip	47878.7 kip	71629.6 kip
EI_{eff}	471490 kip-in^2	248153 kip-in^2	371252 kip-in^2
GA_{eff}	1075.5 kip	566.1 kip	846.9 kip
EI_{app}	448274.5 kip	235933.9 kip	352972.1 kip

STEP DESCRIPTION	COMPUTATION	REFERNCE
	CONNECTOR CALCULATION	
Screw Properties	Diameter d = 8 mm = 0.315 in	Adolf Wurth GmbH & Co. KG
	Total Length l = 240 mm = 9.449 in	
	Penetration Depth l_{ef} = 160 mm = 6.299 in	
	Interlayer Thickness t = 8.89 mm = 0.35 in	
	Angle α = 45	ETA-13/0029 Table 2.3
	Timber Density ρ_k = 480 kg/m³	
	Coefficient of Friction μ = 0	
	Yield Moment $M_{y,k}$ = 20 Nm	ETA-13/0029 Table 2.4
	Tensile Capacity $f_{tens,k}$ = 17 kN	
	Withdrawal Parameter $f_{ax,k}$ = 11 N/mm²	
Characteristic Withdrawal Capacity	$F_{ax,\alpha,Rk} = \frac{f_{ax,k} d l_{ef}}{1.2\cos(\alpha)^2+\sin(\alpha)^2}\left(\frac{\rho_k}{350}\right)^{0.8}$ $F_{ax,\alpha,Rk} = \frac{(11)(8)(160)}{1.2\cos(45)^2+\sin(45)^2}\left(\frac{480}{350}\right)^{0.8} =$ 17179.5 N	ETA-13/0029 Table 2.3
Characteristic Load-Carrying Capacity	$F_{Rk} = (\cos(\alpha)+\mu\sin(\alpha)) min \begin{cases} F_{ax,\alpha,Rk} \\ f_{tens,k} \end{cases}$ $F_{Rk} = (\cos(45)+0\sin(45))(17kN) =$ 8.93 kN	
Slip Modulus	$K_{ser} = 100 l_{ef}$ $K_{ser} = 100(170\ mm) = 16000$ N/mm	ETA-13/0029 Table 2.3
Slip Modulus Ultimate	$K_u = (2/3)K_{ser}$ $K_u = (2/3)(17000\ N/mm) =$ 10667 N/mm Conversions F_{Rk} = 2.0077 kips K_{ser} = 91.3624 k/in K_u = 60.9082 k/in	
	Spacing between connectors	
	in rows at the ends s_e = 152.4 mm = 6 in	
	in rows in the middle s_m = 304.8 mm = 12 in	
	Effective Spacing between s_{eff} = 190.5 mm = 7.5 in	
	Number of rows of connectors n_r = 1	
	Deformation factor for long term	
	loading for concrete k_{def_c} = 2.5	ETA-13/0029 Table 2.1
	loading for timber k_{def_t} = 0.9	
	loading for STS k_{def_sts} = 0.6	
	Stiffness reduction for ULS ψ_2 = 0.3	
Connector Stiffness Adjustment for Long Term Loading	$K_{SLS\_LT} = \frac{K_{SER}}{1+k_{def\_sts}} = \frac{17000\ N/mm}{1+0.6} =$ 10000 N/mm $K_{ULS\_LT} = \frac{K_u}{1+\psi_2 k_{def\_sts}} = \frac{11333\ N/mm}{1+(0.3*0.6)} =$ 9039.55 N/mm	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	EFFECTIVE STIFFNESS	
General Values	Gamma Span Length $L = L_0 =$ 26 ft Connector slip Modulus $K_{ser} =$ 16 kN/mm = 91.36 kips/in Connector Spacing $s_{eff} =$ 7.5 in	Annex B of EN 1995-1-1
Concrete Properties	Slab Depth $h_1 =$ 2.75 in Compressive Strength $f_c' =$ 4000 psi Weight of Concrete $w_c =$ 150 pcf	
CLT Properties	Clt Height $h_2 =$ 6.90 in $EA_{eff,CLT} =$ 90969.6 kip $EI_{eff,CLT} =$ 471490 kip-in^2	CLT Handbook EQ 24 & 25
Modulus of Elasticity	$E_{1\_SLS} = {w_c}^{1.5} 33\sqrt{f_c'}$ $E_{1\_SLS} = (150\ pcf)^{1.5} 33\sqrt{4000\ psi} =$ 3834.25 ksi	ACI 318, 19.2.2.1.a
Area of Concrete	$A_{c\_SLS} = h_c b_c$ $A_{c\_SLS} = (2.75\ in)(12\ in) =$ 33 in^2	EN 1995.1-1 Annex B EQ 3.2
Inertia of Concrete	$I_{1\_SLS} = b_1 {h_1}^2/12$ $I_{1\_SLS} = (12\ in)(2.75\ in)^2/12 =$ 20.8 in^4	EN 1995.1-1 Annex B EQ 3.3
	$E_c A_{c\_SLS} = (3834.3\ ksi)(33\ in^2) =$ 126530 kip $E_c I_{c\_SLS} = (38343\ ksi)(20.8\ in^4) =$ 79740.5 kip-in^2	
Gamma Factor	$\gamma_{1\_SLS} = \left[1 + \frac{\pi^2 E_1 A_1 s_1}{K_1 {L_0}^2}\right]^{-1}$ $\gamma_{1\_SLS} = \left[1 + \frac{\pi^2 (3834.3\ ksi)(33\ in^2)(7.5\ in)}{(91.4\ \ kip/in)(26\ ft)^2}\right]^{-1} =$ 0.48706	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2\_SLS} = \frac{\gamma_1 E_1 A_1 (h_1 + h_2)}{2\gamma_1 E_1 A_1 + 2\gamma_2 EA_{eff}}$ $a_{2\_SLS} = \frac{(0.4871)(3834\ ksi)(33\ in^2)(2.75\ in + 6.90\ in)}{2(0.4871)(3834\ ksi)(33\ in^2) + 2(1.0)(90970\ kip)} =$ 1.94863 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1\_SLS} = \frac{h_1 + h_2}{2} - a_{2\_SLS}$ $a_{1\_SLS} = \frac{6.90\ in + 2.75\ in}{2} - 1.95 =$ 2.87637 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_SLS} = (E_1 I_1 + \gamma_1 E_1 A_1 {a_1}^2) + (EI_{eff} + \gamma_2 EA_{eff} {a_2}^2)$ $(EI)_{eff,comp\_SLS} =$ 589623.1 kip-in^2 + 816915 kip-in^2 = 1406538 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Composite & CLT Effective	$Ratio_{CLT} = \frac{EI_{eff,comp\_SLS}}{EI_{eff,CLT}}$ $Ratio_{CLT} = \frac{1406538\ kip \cdot in^2}{471489.9\ kip \cdot in^2} =$ 2.983	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	APPARENT STIFFNESS	
Bare CLT Stiffness	Length L = 26 ft K Value K_s = 11.5 $EI_{eff,clt}$ = 471490 kip-in^2 $GA_{eff,clt}$ = 1075.5 kips	
	$EI_{app,CLT\_SLS} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{471489.9\ kip \cdot in^2}{1+\frac{(11.5)(471489.9\ kip \cdot in^2)}{(1075.5\ kip)(26\ ft)^2}} =$ 448274 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{448274.5\ kip \cdot in^2}{471489.9\ kip \cdot in^2} =$ 95.1%	
Composite Section Stiffness	L = 26 ft K_s = 11.5 $EI_{eff,comp}$ = 1406538 kip-in^2 $GA_{eff,clt}$ = 1075.5 kips	
	$EI_{app,comp\_SLS} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{1406538\ kip \cdot in^2}{1+\frac{(11.5)(1406538\ kip \cdot in^2)}{(1075.5\ kip)(26\ ft)^2}} =$ 1218316 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{1218316\ kip \cdot in^2}{1406538\ kip \cdot in^2} =$ 86.6%	
	$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{1218316\ kip \cdot in^2}{448274.5\ kip \cdot in^2} =$ 2.718	
Apparent Floor Stiffness	L = 26 ft $EI_{app,CLT}$ = 448274 kip-in^2 $EI_{app,comp}$ = 1218316 kip-in^2	
	$K_{app,CLT\_SLS} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$	
	$K_{app,CLT\_SLS} = \frac{48(448274.5\ kip \cdot in^2)}{(26\ ft)^3} =$ 0.70847 kip/in	
	$K_{app,comp\_SLS} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$	
	$K_{app,comp\_SLS} = \frac{48(1218316\ kip \cdot in^2)}{(26\ ft)^3} =$ 1.92547 kip/in	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length L = 26 ft	
	Shear V_u = 1.62 kip	
	Moment M_u = 126.24 kip-in	
	Concrete: Modulus of Elasticity E_1 = 3834.25 ksi; CLT: E_2 = 1800 ksi	
	Height: Concrete h_1 = 2.75 in; CLT h_2 = 6.90 in	
	Centroid: Concrete a_1 = 2.87637 in; CLT a_2 = 1.94863 in	
	Gamma Factor: Concrete γ_1 = 0.48706; CLT γ_2 = 1	
	$EI_{eff,comp\_SLS}$ = 1406538 kip-in^2	
	Bending Stress Calculations	
Average Concrete Stress	$\sigma_1 = \dfrac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \dfrac{(0.487)(3834\ ksi)(2.88\ in)(126\ kip \cdot in)}{1406538\ kip \cdot in^2} =$ 0.482 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \dfrac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \dfrac{(0.5)(3834\ ksi)(2.75\ in)(126\ kip \cdot in)}{1406538\ kip \cdot in^2} =$ 0.473 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \dfrac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \dfrac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.482 + 0.473 =$ 0.955 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \dfrac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \dfrac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.482 - 0.473 =$ 0.009 ksi	
Average CLT Stress	$\sigma_2 = \dfrac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \dfrac{(1.0)(1800\ ksi)(1.949\ in)(126\ kip \cdot in)}{1406538\ kip \cdot in^2} =$ 0.315 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \dfrac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \dfrac{(0.5)(1800\ ksi)(6.90\ in)(126\ kip \cdot in)}{1406538\ kip \cdot in^2} =$ 0.557 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \dfrac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} + \dfrac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.315 + 0.557 =$ 0.243 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \dfrac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \dfrac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.315 - 0.56 =$ -0.87 ksi	

<table>
<tr><th>STEP DESCRIPTION</th><th>COMPUTATION</th><th>REFERNCE</th></tr>
<tr><td></td><td>DEFLECTION CALCUATIONS
Short-Term | Long-Term</td><td></td></tr>
<tr><td>Loading</td><td>Dead Load w_{DL} = 54.50 plf | w_{DL} = 54.50 plf
Superimposed Dead Load w_{SDL} = 20.00 plf | w_{SDL} = 15.64 plf
Live Load w_{LL} = 50.00 plf | w_{LL} = 10.9 plf
Span Length L = 26 ft | L = 26 ft
Stiffness $EI_{eff,comp\_SLS}$ = 1406538 kip-in^2</td><td></td></tr>
<tr><td></td><td>Short-Term Deflections</td><td></td></tr>
<tr><td>Dead Load</td><td>$\Delta_{DL}= \frac{5w_{DL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(54.5\,plf)(26\,ft)^4}{384(1406537.8\,kip\cdot in^2)} =$ 0.3984 in</td><td></td></tr>
<tr><td>Superimposed Dead Load</td><td>$\Delta_{SDL}= \frac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(20\,plf)(26\,ft)^4}{384(1406537.8\,kip\cdot in^2)} =$ 0.1462 in</td><td></td></tr>
<tr><td>Live Load</td><td>$\Delta_{LL}= \frac{5w_{LL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(50\,plf)(26\,ft)^4}{384(1406537.8\,kip - in^2)} =$ 0.3655 in</td><td></td></tr>
<tr><td></td><td>Serviceability Check</td><td></td></tr>
<tr><td>Allowable LL Deflection</td><td>$\Delta_{allow,LL}= \frac{L}{360} = \frac{(26ft)(12\,in)}{360(1\,ft)} =$ 0.87 in</td><td>IBC Table 1604.3</td></tr>
<tr><td>Check</td><td>$\Delta_{LL} \leq \Delta_{allow,LL} \rightarrow \Delta_{LL}= 0.366\,in < \Delta_{allow,LL}= 0.87\,in$ ∴ ACCEPTABLE</td><td></td></tr>
<tr><td></td><td>Long-Term Deflections</td><td></td></tr>
<tr><td>Dead Load</td><td>$\Delta_{DL\_LT}= \frac{5w_{DL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(54.5\,plf)(26\,ft)^4}{384(1406538\,kip\cdot in^2)} =$ 0.3984 in</td><td></td></tr>
<tr><td>Superimposed Dead Load</td><td>$\Delta_{SDL\_LT}= \frac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(15.6\,plf)(26\,ft)^4}{384(1406538\,kip\cdot in^2)} =$ 0.11433 in</td><td></td></tr>
<tr><td>Live Load</td><td>$\Delta_{LL\_LT}= \frac{5w_{LL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(10.9\,plf)(26\,ft)^4}{384(1406538\,kip\cdot in^2)} =$ 0.07968 in</td><td></td></tr>
<tr><td></td><td>Serviceability Check
Time Dependent Creep Factor $K_{cr} =$ 2.0
Deflection due to short-term $\Delta_{ST} =$ 0.3655 in
Deflection due to long-term $\Delta_{LT} =$ 0.51273 in $\Delta_{LT}= \Delta_{DL\_LT} + \Delta_{SDL\_LT}$</td><td>NDS 2018 Section 3.5.1</td></tr>
<tr><td></td><td>$\Delta_{TL}= K_{cr}\Delta_{LT} + \Delta_{ST}$
$\Delta_{TL}=$ 2.0(0.5127 in)+(0.3655 in) = 1.391 in</td><td>NDS EQ 3.5-1</td></tr>
<tr><td></td><td>$\Delta_{allow,TL}= \frac{L}{240} = \frac{(26ft)(12\,in)}{240(1\,ft)} =$ 1.30 in</td><td>IBC Table 1604.3</td></tr>
<tr><td></td><td>$\Delta_{TL} \leq \Delta_{allow,TL} \rightarrow \Delta_{TL}= 1.391\,in < \Delta_{allow,TL}= 1.30\,in$ ∴ NOT GOOD</td><td></td></tr>
</table>

Serviceability Limit State
Long-Term Loading

STEP DESCRIPTION	COMPUTATION	REFERNCE
	EFFECTIVE STIFFNESS	
	Deformation factor for long term	
	loading for concrete k_{def_c} = 2.5	EN 1995.1-1
	loading for timber k_{def_t} = 0.9	
	loading for STS k_{def_sts} = 0.6	
	Stiffness reduction for ULS ψ_2 = 0.3	
	Effective Spacing between s_{eff} = 190.5 mm = 7.5 in	ETA-13/0029 Table 2.1
	Gamma Span Length L_0 = 26 ft	
	Connector slip Modulus K_{SLS_LT} = 10 kN/mm = 57.10 kips/in	
	Concrete / CLT	
	Modulus of Elasticity: E_1 = 3834.25 ksi; E_2 = 1800 ksi	
	Area: A_1 = 33 in^2; A_1 = 26.60 in^2	
	Momemnt of Inertia: I_1 = 20.7969 in^4; I_1 = 137.863 in^4	
	Height: h_1 = 2.75 in; h_2 = 6.90 in	
	EA_{eff,CLT_SLS_LT} = 47878.74 kip	CLT Handbook EQ 24 & 25
	EI_{eff,CLT_SLS_LT} = 248153 kip-in^2	
Modulus of Adjustment for long term loading	$E_{1\_SLS\_LT} = \dfrac{E_1}{1+k_{def\_c}} =$ 1095.5 ksi $E_{2\_SLS\_LT} = \dfrac{E_2}{1+k_{def\_t}} =$ 947.368 ksi	EN 1995.1-1
Gamma Factor	$\gamma_{1,SLS\_LT} = \left[1+\dfrac{\pi^2 E_{1\_SLS\_LT} A_1 s_{eff}}{K_{ser\_LT} {L_0}^2}\right]^{-1}$ $\gamma_{1,SLS\_LT} = \left[1+\dfrac{\pi^2(1095\ ksi)(33\ in^2)(7.5\ in)}{(57.1\ kip/in)(26\ ft)^2}\right]^{-1} =$ 0.675	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2,SLS\_LT} = \dfrac{\gamma_{1\_SER} E_1 A_1 (h_1+h_2)}{2\gamma_{1\_SER} E_1 A_1 + 2\gamma_2 EA_{eff}}$ $a_{2,SLS\_LT} = \dfrac{(0.675)(1095\ ksi)(33\ in^2)(2.75\ in+6.90\ in)}{2(0.675)(1095\ ksi)(33\ in^2)+2(1.0)(47878.7\ kip)} =$ 2.37378 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1,SLS\_LT} = \dfrac{h_1+h_2}{2} - a_{2,SLS\_LT}$ $a_{1,SLS\_LT} = \dfrac{6.90\ in + 2.75\ in}{2} - 2.37 =$ 2.451 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_SLS\_LT} = \left(E_1 I_1 + \gamma_{1\_SER} E_1 A_1 {a_{1\_SER}}^2\right) + \left(EI_{eff} + \gamma_2 EA_{eff} {a_{2\_SER}}^2\right)$ $(EI)_{eff,comp\_SLS\_LT} =$ 169409 kip-in^2 + 517941 kip-in^2 $(EI)_{eff,comp\_SLT\_LT} =$ 687349.8 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Compsite & CLT Effective	$Ratio_{CLT\_SER} = \dfrac{EI_{eff,comp\_SER}}{EI_{eff,CLT}}$ $Ratio_{CLT\_SER} = \dfrac{687349.8\ kip\cdot in^2}{248152.6\ kip\cdot in^2} =$ 14.36	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	APPARENT STIFFNESS	
Bare CLT Stiffness	Length $L = 26$ ft K Value $K_s = 11.5$ $EI_{eff,clt} = 248153$ kip-in^2 $GA_{eff,clt} = 566.1$ kips	
	$EI_{app,CLT\_SLS\_LT} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{248152.6\ kip \cdot in^2}{1+\frac{(11.5)(248152.6\ kip \cdot in^2)}{(566.1\ kip)(26\ ft)^2}} = 235934$ kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{235933.9\ kip \cdot in^2}{248152.6\ kip \cdot in^2} = 95.1\%$	
Composite Section Stiffness	$L = 26$ ft $K_s = 11.5$ $EI_{eff,comp} = 687350$ kip-in^2 $GA_{eff,clt} = 566.1$ kips	
	$EI_{app,comp\_SLS\_LT} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{687349.8\ kip \cdot in^2}{1+\frac{(11.5)(687349\ kip \cdot in^2)}{(566.1\ kip)(26\ ft)^2}} = 601121$ kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{601120.9\ kip \cdot in^2}{687349.8\ kip \cdot in^2} = 87.5\%$	
	$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{601120.9\ kip \cdot in^2}{235933.9\ kip \cdot in^2} = 2.548$	
Apparent Floor Stiffness	$L = 26$ ft $EI_{app,CLT} = 235934$ kip-in^2 $EI_{app,comp} = 601121$ kip-in^2	
	$K_{app,CLT\_SLS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$	
	$K_{app,CLT\_SLS\_LT} = \frac{48(235933.9\ kip \cdot in^2)}{(26\ ft)^3} = 0.37288$ kip/in	
	$K_{app,comp\_SLS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$	
	$K_{app,comp\_SLS\_LT} = \frac{48(601120.9\ kip \cdot in^2)}{(26\ ft)^3} = 0.95003$ kip/in	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length $L = 26$ ft Shear $V_u = 1.05$ kip Moment $M_u = 82.17$ kip-in	
	Concrete: Modulus of Elasticity $E_1 = 1095.5$ ksi; Height $h_1 = 2.75$ in; Centroid $a_1 = 2.45122$ in; Gamma Factor $\gamma_1 = 0.67502$ CLT: $E_2 = 947.368$ ksi; $h_2 = 6.90$ in; $a_2 = 2.37378$ in; $\gamma_2 = 1$ $EI_{eff,comp\_SLS\_LT} = 687350$ kip-in^2	
	Bending Stess Calulations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.675)(1096\ ksi)(2.45\ in)(82.2\ kip \cdot in)}{687349.8\ kip \cdot in^2} =$ 0.217 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(1096\ ksi)(2.75\ in)(82.2\ kip \cdot in)}{687349.8\ kip \cdot in^2} =$ 0.180 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.217 + 0.180 =$ 0.397 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.217 - 0.180 =$ 0.037 ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(947.4\ ksi)(2.37\ in)(82.2\ kip \cdot in)}{687349.8\ kip \cdot in^2} =$ 0.269 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(947.5\ ksi)(6.90\ in)(82.2\ kip \cdot in)}{687349.8\ kip \cdot in^2} =$ 0.391 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.269 + 0.391 =$ 0.122 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.269 - 0.39 =$ -0.66 ksi	

<table>
<tr><th>STEP DESCRIPTION</th><th>COMPUTATION</th><th>REFERNCE</th></tr>
<tr><td>Loading</td><td>DEFLECTION CALCUATIONS
Short-Term / Long-Term
Dead Load w_{DL} = 54.50 plf | w_{DL} = 54.50 plf
Superimposed Dead Load w_{SDL} = 20.00 plf | w_{SDL} = 15.64 plf
Live Load w_{LL} = 50.00 plf | w_{LL} = 10.9 plf
Span Length L = 26 ft | L = 26 ft
Stiffness $EI_{eff,comp\_SLS}$ = 1406538 kip-in^2 | $EI_{eff,comp\_SLS\_LT}$ = 687349.8 kip-in^2</td><td></td></tr>
<tr><td></td><td>Short-Term Deflections</td><td></td></tr>
<tr><td>Dead Load</td><td>$\Delta_{DL} = \frac{5w_{DL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(54.5\,plf)(26\,ft)^4}{384(1406537.8\,kip\cdot in^2)} =$ 0.3984 in</td><td></td></tr>
<tr><td>Superimposed Dead Load</td><td>$\Delta_{SDL} = \frac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(20\,plf)(26\,ft)^4}{384(1406537.8\,kip\cdot in^2)} =$ 0.1462 in</td><td></td></tr>
<tr><td>Live Load</td><td>$\Delta_{LL} = \frac{5w_{LL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(50\,plf)(26\,ft)^4}{384(1406537.8\,kip - in^2)} =$ 0.3655 in</td><td></td></tr>
<tr><td></td><td>Servicablility Check</td><td></td></tr>
<tr><td>Allowable LL Deflection</td><td>$\Delta_{allow,LL} = \frac{L}{360} = \frac{(26ft)(12\,in)}{360(1\,ft)} =$ 0.87 in</td><td>IBC Table 1604.3</td></tr>
<tr><td>Check</td><td>$\Delta_{LL} \leq \Delta_{allow,LL} \rightarrow \Delta_{LL} = 0.3655\,in < \Delta_{allow,LL} = 0.87\,in$ ∴ ACCEPTABLE</td><td></td></tr>
<tr><td></td><td>Long-Term Deflections</td><td></td></tr>
<tr><td>Dead Load</td><td>$\Delta_{DL\_LT} = \frac{5w_{DL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(54.5\,plf)(26\,ft)^4}{384(687349.8\,kip\cdot in^2)} =$ 0.81526 in</td><td></td></tr>
<tr><td>Superimposed Dead Load</td><td>$\Delta_{SDL\_LT} = \frac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(15.6\,plf)(26\,ft)^4}{384(687349.8\,kip\cdot in^2)} =$ 0.23396 in</td><td></td></tr>
<tr><td>Live Load</td><td>$\Delta_{LL\_LT} = \frac{5w_{LL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(10.9\,plf)(26\,ft)^4}{384(687349.8\,kip\cdot in^2)} =$ 0.16305 in</td><td></td></tr>
<tr><td></td><td>Servicablility Check
Deflection due to short-term Δ_{ST} = 0.3655 in
Deflection due to long-term Δ_{LT} = 1.04921 in $\Delta_{LT} = \Delta_{DL\_LT} + \Delta_{SDL\_LT}$
Deflection due to Total Load Δ_{TL} = 1.4147 in $\Delta_{TL} = \Delta_{LT} + \Delta_{ST}$</td><td></td></tr>
<tr><td></td><td>$\Delta_{allow,TL} = \frac{L}{240} = \frac{(26ft)(12\,in)}{240(1\,ft)} =$ 1.30 in</td><td>IBC Table 1604.3</td></tr>
<tr><td></td><td>$\Delta_{TL} \leq \Delta_{allow,TL} \rightarrow \Delta_{TL} = 1.415\,in < \Delta_{allow,TL} = 1.30\,in$ ∴ NOT GOOD</td><td></td></tr>
</table>

STEP DESCRIPTION	COMPUTATION	REFERNCE
General Values	EFFECTIVE STIFFNESS Gamma Span Length $L = L_0 =$ 26 ft Connector slip Modulus $K_U =$ 10.6667 kN/mm = 60.91 kips/in Connector Spacing $s_{eff} =$ 7.5 in	Annex B of EN 1995-1-1
Concrete Properties	Slab Depth $h_1 =$ 2.75 in Compressive Strength $f'_c =$ 4000 psi Weight of Concete $w_c =$ 150 pcf	
CLT Properties	Clt Height $h_2 =$ 6.90 in $EA_{eff,CLT} =$ 90969.6 kip $EI_{eff,CLT} =$ 471490 kip-in^2	CLT Handbook EQ 24 & 25
Modulus of Elasticity	$E_{1\_ULS} = w_c^{1.5} 33\sqrt{f'_c}$ $E_{1\_ULS} = (150\ pcf)^{1.5} 33\sqrt{4000\ psi} =$ 3834.25 ksi	ACI 318, 19.2.2.1.a
Area of Concrete	$A_{c\_ULS} = h_c b_c$ $A_{c\_ULS} = (2.75\ in)(12\ in) =$ 33 in^2	EN 1995.1-1 Annex B EQ 3.2
Inhertia of Concrete	$I_{1\_ULS} = b_1 {h_1}^2/12$ $I_{1\_ULS} = (12\ in)(2.75\ in)^2/12 =$ 20.8 in^4	EN 1995.1-1 Annex B EQ 3.3
	$E_c A_{c\_ULS} = (3834.3\ ksi)(33\ in^2) =$ 126530 kip $E_c I_{c\_ULS} = (38343\ ksi)(20.8\ in^4) =$ 79740.5 kip-in^2	
Gamma Factor	$\gamma_{1\_ULS} = \left[1 + \frac{\pi^2 E_1 A_1 s_1}{K_1 {L_0}^2}\right]^{-1}$ $\gamma_{1\_ULS} = \left[1 + \frac{\pi^2 (3834.3\ ksi)(33\ in^2)(7.5\ in)}{(60.9\ kip/in)(26\ ft)^2}\right]^{-1} =$ 0.38764	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2\_ULS} = \frac{\gamma_1 E_1 A_1 (h_1 + h_2)}{2\gamma_1 E_1 A_1 + 2\gamma_2 EA_{eff}}$ $a_{2\_ULS} = \frac{(0.3876)(3834\ ksi)(33\ in^2)(2.75\ in + 6.90\ in)}{2(0.3876)(3834\ ksi)(33\ in^2) + 2(1.0)(90970\ kip)} =$ 1.69021 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1\_ULS} = \frac{h_1 + h_2}{2} - a_{2\_ULS}$ $a_{1\_ULS} = \frac{6.90\ in + 2.75\ in}{2} - 1.69 =$ 3.13479 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_ULS} = (E_1 I_1 + \gamma_1 E_1 A_1 {a_1}^2) + (EI_{eff} + \gamma_2 EA_{eff} {a_2}^2)$ $(EI)_{eff,comp\_ULS} =$ 561738.5 kip-in^2 + 731372 kip-in^2 = 1293111 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Compsite & CLT Effective	$Ratio_{CLT} = \frac{EI_{eff,comp\_ULS}}{EI_{eff,CLT\_ULS}}$ $Ratio_{CLT} = \frac{1293111\ kip \cdot in^2}{471489.9\ kip \cdot in^2} =$ 2.743	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	APPARENT STIFFNESS	
Bare CLT Stiffness	Length L = 26 ft K Value K_s = 11.5 $EI_{eff,clt}$ = 471490 kip-in^2 $GA_{eff,clt}$ = 1075.5 kips	
	$EI_{app,CLT\_ULS} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{471489.9\ kip \cdot in^2}{1+\frac{(11.5)(471489.9\ kip \cdot in^2)}{(1075.5\ kip)(26\ ft)^2}} =$ 448274 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{448274.5\ kip \cdot in^2}{471489.9\ kip \cdot in^2} =$ 95.1%	
Composite Section Stiffness	L = 26 ft K_s = 11.5 $EI_{eff,comp}$ = 1293111 kip-in^2 $GA_{eff,clt}$ = 1075.5 kips	
	$EI_{app,comp\_ULS} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{1293111\ kip \cdot in^2}{1+\frac{(11.5)(1293111\ kip \cdot in^2)}{(1075.5\ kip)(26 ft)^2}} =$ 1132287 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{1132287\ kip \cdot in^2}{1293111\ kip \cdot in^2} =$ 87.6%	
	$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{1132287\ kip \cdot in^2}{448274.5\ kip \cdot in^2} =$ 2.526	
Apparent Floor Stiffness	L = 26 ft $EI_{app,CLT}$ = 448274 kip-in^2 $EI_{app,comp}$ = 1132287 kip-in^2	
	$K_{app,CLT\_ULS} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$	
	$K_{app,CLT\_ULS} = \frac{48(448274.5\ kip \cdot in^2)}{(26\ ft)^3} =$ 0.70847 kip/in	
	$K_{app,comp\_ULS} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$	
	$K_{app,comp\_ULS} = \frac{48(1132287\ kip \cdot in^2)}{(26\ ft)^3} =$ 1.78951 kip/in	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length $L = 26$ ft Shear $V_u = 2.20$ kip Moment $M_u = 171.77$ kip-in	
	Concrete: Modulus of Elasticity $E_1 = 3834.25$ ksi; Height $h_1 = 2.75$ in; Centroid $a_1 = 3.13479$ in; Gamma Factor $\gamma_1 = 0.38764$ CLT: $E_2 = 1800$ ksi; $h_2 = 6.90$ in; $a_2 = 1.69021$ in; $\gamma_2 = 1$ $EI_{eff,comp} = 1293111$ kip-in^2	
	Bending Stress Calculations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.388)(3834\ ksi)(3.13\ in)(172\ kip\cdot in)}{1239311\ kip\cdot in^2} =$ 0.619 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(3834\ ksi)(2.75\ in)(172\ kip\cdot in)}{1293111\ kip\cdot in^2} =$ 0.700 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5E_1 h_1 M_u}{(EI)_{eff}} = 0.619 + 0.700 =$ 1.319 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{+\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5E_1 h_1 M_u}{(EI)_{eff}} = 0.619 - 0.700 =$ ##### ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(1800\ ksi)(1.69\ in)(172\ kip\cdot in)}{1293111\ kip\cdot in^2} =$ 0.404 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(1800\ ksi)(6.90\ in)(172\ kip\cdot in)}{1293111\ kip\cdot in^2} =$ 0.825 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} + \frac{0.5E_2 h_2 M_u}{(EI)_{eff}} = -0.404 + 0.825 =$ 0.421 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5E_2 h_2 M_u}{(EI)_{eff}} = -0.40 - 0.825 =$ -1.23 ksi	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	STRENGTH ANALYSIS	
Concrete	Normal Stress in Concrete σ_1 = 618.9 psi Allow Comp Strength of Conc F_c = 2000 psi $F_c = f_c'/2$ $\sigma_1 < F_c \rightarrow \sigma_1 = 618.9\, psi < F_c = 2000\, psi$ ∴ ACCEPTABLE	
Timber	Bending Strength F_b = 2100 psi Format Conversion Factor K_F = 2.54 Resistance Factor Φ = 0.85 Design Bending Strength $F_b' = \Phi F_b K_F = 0.85(2100\, psi)(2.54) =$ 4533.9 psi	
	Tension Strength F_t = 1575 psi Format Conversion Factor K_F = 2.70 Resistance Factor Φ = 0.80 Design Tension Strength $F_t' = \Phi F_t K_F = 0.80(1575\, psi)(2.70) =$ 3402.0 psi	
	Average CLT Stress σ_2 = 404.1 psi Extreme Fiber Stress $\sigma_{m,2}$ = 824.9 psi $\frac{\sigma_2}{F_t'} + \frac{\sigma_{m,2}}{F_b'} \leq 1 \rightarrow \frac{404.1\, psi}{3402\, psi} + \frac{824.9\, psi}{4534\, psi} =$ 0.301 < 1.0 ∴ ACCEPTABLE	
	Shear Strength F_v = 160 psi Format Conversion Factor K_F = 2.88 Resistance Factor Φ = 0.75 Design Shear Strength $F_v' = \phi F_v K_F = 0.75(160\, psi)(2.88) =$ 345.6 psi	
	CLT Modulus of Elasticity E_2 = 1800 ksi NA of timber h = 5.14 in Shear V = 2.20 kip $EI_{eff,comp}$ = 1293111 kip-in^2	
	$\tau_{2,max} = \frac{E_2 h^2 V_u}{2(EI)_{eff,comp}} = \frac{(1800\, ks)(5.14\, in)^2(2.20\, kip)}{2(1293111\, kip \cdot in^2)} =$ 40.5 psi	EN 1995.1-1 Annex B EQ 3.9
	$\tau_{2,max} < F_v' \rightarrow \tau_{2,max} = 40.5\, psi < F_v' = 345.6\, psi$ ∴ ACCEPTABLE	

STEP DESCRIPTION	COMPUTATION	REFERNCE
Fastener Load Calculation	STRENGTH ANALYSIS F_{ULT} = 2.00765 kips Gamma Factor γ_1 = 0.38764 ksi Concrete Modulus of Elasticity E_1 = 3834.25 in Concrete Area A_1 = 33.00 in^2 Composite Centroid a_1 = 3.13 in Fastener Spacing s = 7.5 in Ultimate Shear Loading V = 2.2022 kip $EI_{eff,comp\_ULS}$ = 1293111 kip-in^2 $F_q = \frac{\gamma_1 E_1 A_1 a_1 s V}{EI_{eff,comp}}$ $F_q = \frac{(0.388)(3834\ ksi)(33 in^2)(3.13\ in)(7.5\ in)(2.20\ kip)}{1293111\ kip \cdot in^2} =$ 1.964 kip $F_q < F_{ULT} \rightarrow F_q = 1.964\ kip < F_{ULT} = 2.01\ kip$ ∴ ACCEPTABLE	EN 1995.1-1 Annex B EQ 3.10

Ultimate Limit State
Long-Term Loading

STEP DESCRIPTION	COMPUTATION	REFERNCE
	EFFECTIVE STIFFNESS	
	Deformation factor for long term	
	loading for concrete k_{def_c} = 2.5	EN 1995.1-1
	loading for timber k_{def_t} = 0.9	
	loading for STS k_{def_sts} = 0.6	
	Stiffness reduction for ULS ψ_2 = 0.3	
	Effective Spacing between s_{eff} = 190.5 mm = 7.5 in	ETA-13/0029 Table 2.1
	Gamma Span Length L_0 = 26 ft	
	Connector slip Modulus K_{U_LT} = 9.03955 kN/mm = 51.62 kips/in	
	Concrete / CLT	
	Modulus of Elasticity: E_1 = 3834.25 ksi; E_2 = 1800 ksi	
	Area: A_1 = 33 in^2; A_1 = 50.5387 in^2	
	Momemnt of Inertia: I_1 = 20.7969 in^4; I_1 = 261.939 in^4	
	Height: h_1 = 2.75 in; h_2 = 6.90 in	
	EA_{eff,CLT_ULT_LT} = 71629.6 in EI_{eff,CLT_ULT_LT} = 371252 kip-in^2	CLT Handbook EQ 24 & 25
Modulus of Adjustment for long term loading	$E_{1,ULT\_LT}=\frac{E_1}{1+\psi_2 k_{def\_c}}=$ 2191 ksi $E_{2,ULT\_LT}=\frac{E_2}{1+\psi_2 k_{def\_t}}=$ 1417.32 ksi	EN 1995.1-1
Gamma Factor	$\gamma_{1_{ULT\_LT}}=\left[1+\frac{\pi^2 E_{1,ULT\_LT}A_1 s_{eff}}{K_{ULT\_LT}{L_0}^2}\right]^{-1}$ $\gamma_{1,ULT\_LT}=\left[1+\frac{\pi^2(2191\ ksi)(33\ in^2)(7.5\ in)}{(51.6\ kip/in)(26\ ft)^2}\right]^{-1}=$ 0.484	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2,ULT\_LT}=\frac{\gamma_{1\_ULT\_LT}E_{1,ULT\_LT}A_1(h_1+h_2)}{2\gamma_{1,ULT\_LT}E_{1,ULT\_LT}A_1+2\gamma_2 EA_{eff,CLT\_ULT\_LT}}$ $a_{2,ULT\_LT}=\frac{(0.484)(2191.0\ ksi)(33\ in^2)(2.75\ in+6.90\ in)}{2(0.48)(2191.0ksi)(33\ in^2)+2(1.0)(71629.6\ kip)}=$ 1.584 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1,ULT\_LT}=\frac{h_1+h_2}{2}-a_{2\_ULT}$ $a_{1,ULT\_LT}=\frac{6.90\ in+2.75\ in}{2}-1.58=$ 3.241 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_ULT\_LT}=\left(E_{1,ULT\_LT}I_1+\gamma_{1,ULT\_LT}E_{1,ULT\_LT}A_1{a_{1,ULT\_LT}}^2\right)+\left(EI_{eff}+\gamma_2 EA_{eff}{a_{2,ULT}}^2\right)$ $(EI)_{eff,comp\_ULT\_LT}=$ 413304 kip-in^2 + 550994 kip-in^2 $(EI)_{eff,comp\_ULT\_LT}=$ 964298 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Compsite & CLT Effective	$Ratio_{CLT\_ULT}=\frac{EI_{eff,comp\_ULT\_LT}}{EI_{eff,CLT\_ULT\_LT}}$ $Ratio_{CLT\_ULT}=\frac{964297.9\ kip\cdot in^2}{371251.9\ kip\cdot in^2}=$ 13.46	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	APPARENT STIFFNESS	
Bare CLT Stiffness	Length $L = 26$ ft K Value $K_s = 11.5$ $EI_{eff,CLT\_ULT\_LT} = 371252$ kip-in^2 $GA_{eff,CLT\_ULT\_LT} = 846.9$ kips	
	$EI_{app,CLT\_ULS\_LT} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{371251.9\ kip\cdot in^2}{1+\frac{(11.5)(371252 kip\cdot in^2)}{(846.9\ kip)(26\ ft)^2}} =$ 352972 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{352972.1\ kip\cdot in^2}{371251.9\ kip\cdot in^2} =$ 95.1%	
Composite Section Stiffness	$L = 26$ ft $K_s = 11.5$ $EI_{eff,comp} = 964298$ kip-in^2 $GA_{eff,clt} = 846.9$ kips	
	$EI_{app,comp\_ULS\_LT} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{964297.9\ kip\cdot in^2}{1+\frac{(11.5)(964298\ kip\cdot in^2)}{(846.9\ kip)(26\ ft)^2}} =$ 849964 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{849964\ kip\cdot in^2}{964297.9\ kip\cdot in^2} =$ 88.1%	
	$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{849964\ kip\cdot in^2}{352972.1\ kip\cdot in^2} =$ 2.408	
Apparent Floor Stiffness	$L = 26$ ft $EI_{app,CLT} = 352972$ kip-in^2 $EI_{app,comp} = 849964$ kip-in^2	
	$K_{app,CLT\_ULS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$	
	$K_{app,CLT\_ULS\_LT} = \frac{48(352972.1\ kip\cdot in^2)}{(26\ ft)^3} =$ 0.55785 kip/in	
	$K_{app,comp\_ULS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$	
	$K_{app,comp\_ULS\_LT} = \frac{48(849964.0\ kip\cdot in^2)}{(26\ ft)^3} =$ 1.34332 kip/in	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length $L = 26$ ft Shear $V_u = 2.20$ kip Moment $M_u = 171.77$ kip-in	
	Concrete: Modulus of Elasticity $E_1 = 2191$ ksi; Height $h_1 = 2.75$ in; Centroid $a_1 = 3.24092$ in; Gamma Factor $\gamma_1 = 0.48422$ CLT: $E_2 = 1417.32$ ksi; $h_2 = 6.90$ in; $a_2 = 1.58408$ in; $\gamma_2 = 1$ $EI_{eff,comp} = 964298$ kip-in^2	
	Bending Stess Calulations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.484)(2191\ ksi)(3.24\ in)(172\ kip \cdot in)}{964297.9\ kip \cdot in^2} =$ 0.612 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(2191\ ksi)(2.75\ in)(172\ kip \cdot in)}{964297.9\ kip \cdot in^2} =$ 0.537 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.612 + 0.537 =$ 1.149 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.612 - 0.537 =$ 0.076 ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(1417.3\ ksi)(1.58\ in)(172\ kip \cdot in)}{964297.9\ kip \cdot in^2} =$ 0.40 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(1417\ ksi)(6.90\ in)(172\ kip \cdot in)}{964297.9\ kip \cdot in^2} =$ 0.871 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} + \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.40 + 0.871 =$ 0.471 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.40 - 0.871 =$ -1.3 ksi	

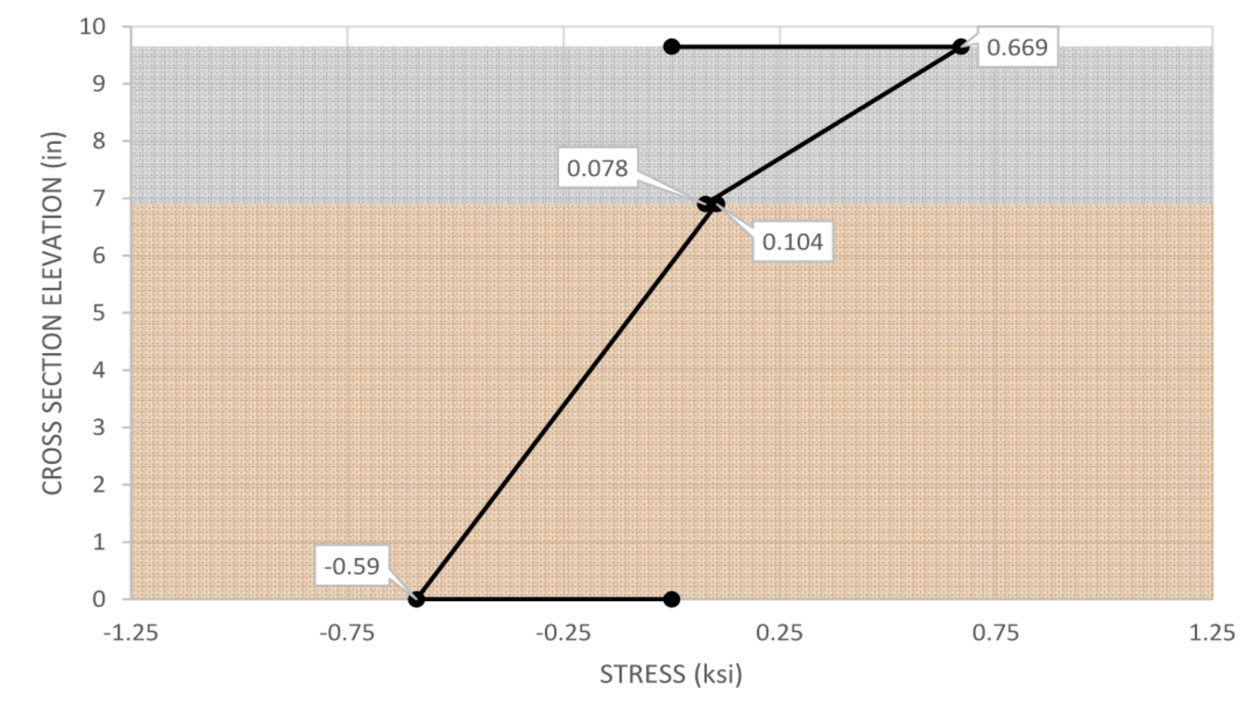
STEP DESCRIPTION	COMPUTATION	REFERNCE
	STRENGTH ANALYSIS	
Concrete	Normal Stress in Concrete σ_1 = 612.5 psi Allow Comp Strength of Conc F_c = 2000 psi $F_c = f_c'/2$ $\sigma_1 < F_c \rightarrow \sigma_1 = 612.5\,psi < F_c = 2000\,psi$ ∴ ACCEPTABLE	
Timber	Bending Strength F_b = 2100 psi Format Conversion Factor K_F = 2.54 Resistance Factor Φ = 0.85 Design Bending Stength $F_b' = \Phi F_b K_F = 0.85(2100\,psi)(2.54) =$ 4533.9 psi	
	Tension Strength F_t = 1575 psi Format Conversion Factor K_F = 2.70 Resistance Factor Φ = 0.80 Design Tension Stength $F_t' = \Phi F_t K_F = 0.80(1575\,psi)(2.70) =$ 3402.0 psi	
	Average CLT Stress σ_2 = 399.9 psi Extreme Fiber Stress $\sigma_{m,2}$ = 871.0 psi $\frac{\sigma_2}{F_t'} + \frac{\sigma_{m,2}}{F_b'} \leq 1 \rightarrow \frac{399.9\,psi}{3402\,psi} + \frac{871.0\,psi}{4534\,psi} =$ 0.310 < 1.0 ∴ ACCEPTABLE	
	Shear Strength F_v = 160 psi Format Conversion Factor K_F = 2.88 Resistance Factor Φ = 0.75 Design Shear Stength $F_v' = \phi F_v K_F = 0.75(160\,psi)(2.88) =$ 345.6 psi	
	CLT Modulus of Elasticity E_2 = 1417.32 ksi NA of timber h = 5.03 in Shear V = 2.20 kip $EI_{eff,comp}$ = 964298 kip-in^2	
	$\tau_{2,max} = \frac{E_2 {h_2}^2 V_u}{2(EI)_{eff,comp}} = \frac{(1417\,ksi)(5.03\,in)^2(2.20\,kip)}{2(964297.9\,kip \cdot in^2)} =$ 41.01 psi	EN 1995.1-1 Annex B EQ 3.9
	$\tau_{2,max} < F_v' \rightarrow \tau_{2,max} = 41.01\,psi < F_v' = 345.6\,psi$ ∴ ACCEPTABLE	

STEP DESCRIPTION	COMPUTATION	REFERNCE
Fastener Load Calculation	STRENGTH ANALYSIS F_{ULT} = 2.00765 kips Gamma Factor γ_1 = 0.48422 ksi Concrete Modulus of Elasticity E_1 = 2191.00 in Concrete Area A_1 = 33.00 in^2 Composite Centroid a_1 = 3.24 in Fastener Spacing s = 7.5 in Ultimate Shear Loading V = 2.2022 kip $EI_{eff,comp}$ = 964298 kip-in^2 $F_q = \frac{\gamma_1 E_1 A_1 a_1 sV}{EI_{eff,comp}}$ $F_q = \frac{(0.484)(2191\ ksi)(33 in^2)(3.24\ in)(7.5\ in)(2.20\ kip)}{964297.9\ kip \cdot in^2} =$ 1.943 kip $F_q < F_{ULT} \rightarrow F_q = 1.943\ kip < F_{ULT} = 2.01\ kip$ ∴ ACCEPTABLE	EN 1995.1-1 Annex B EQ 3.10

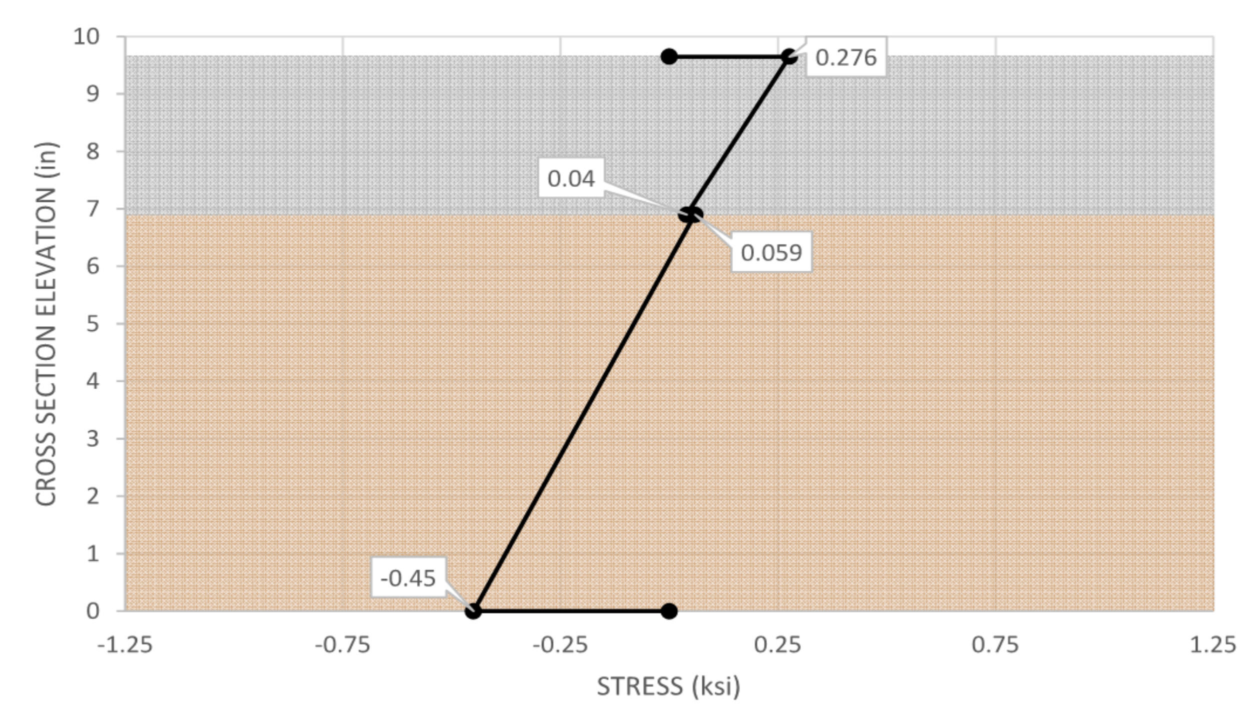
STEP DESCRIPTION	COMPUTATION					REFERNCE
	CONNECTOR EFFICIENCY					
Partial Composite Section	Effective Bending Stiffness	EI_{eff_SLS} =	1406538	in		
	Deflection	Δ_{PC} =	0.36551	in		
Non-Composite Section	Effective Bending Stiffness	EI_{eff_SLS} =	551231	in	(set K_{ser} close to zero)	
	Deflection	Δ_{NC} =	0.93264	in		
Fully Composite	Concrete MOE	E_1 =	3834.25	psi		
	Timber MOE	E_2 =	1800	psi		
	Width	b' =	12	in		
	Width of Transformed Concrete	b' =	25.56	in	$b' = b(E_1/E_2)$	
	Height of Timber	h_1 =	2.75	in		
	Height of Concrete	h_2 =	6.90	in		
	Bottom to Centroid of					
	Transformed Concrete	y'_1 =	8.275	in		
	Timber Section	y_2 =	3.45	in		
	Area of Transformed Concrete	A'_1 =	70.29	in^2		
	Area of Timber Section	A_2 =	82.8	in^2		
	Moment of Inertia of					
	Transformed Concrete	I'_1 =	44.30	in^4		
	Timber Section	I_2 =	328.51	in^4		
	Centroid of Transformed Section					
		$y_c = \frac{y_1' A_1' + y_2 A_2}{A_1' + A_2}$ =	5.665	in		
	Distance to Concrete Centroid	d_1 =	4.29	in	$d = \left\lvert y_c - \frac{h}{2} \right\rvert$	
	Distance to Timber Centroid	d_2 =	0.84	in		
	Transformed Section					
	Moment of Inertia about NA	I_{NA} =	1725.27	in^4	$I_{NA} = \sum I_i + A_i {d_i}^2$	
	Bending Stiffness	$E_2 I_{NA}$ =	3105482	$k\text{-}in^2$		
	Deflection of Fully Composite Section					
	$\Delta_{FC} = \frac{5wL^4}{384E_2 I_{NA}} = \frac{5(50\,plf)(26ft)^4}{384(3105482\,k \cdot in^2)} =$ 0.166 in					
Shear Connector Efficiency	$Efficiency = \frac{\Delta_{NC} - \Delta_{PC}}{\Delta_{NC} - \Delta_{FC}} = \frac{0.9326'' - 0.3655''}{0.9326'' - 0.166''} =$ 73.9%					

Appendix B - HBV Calculation

90 mm HBV 22 feet SPAN

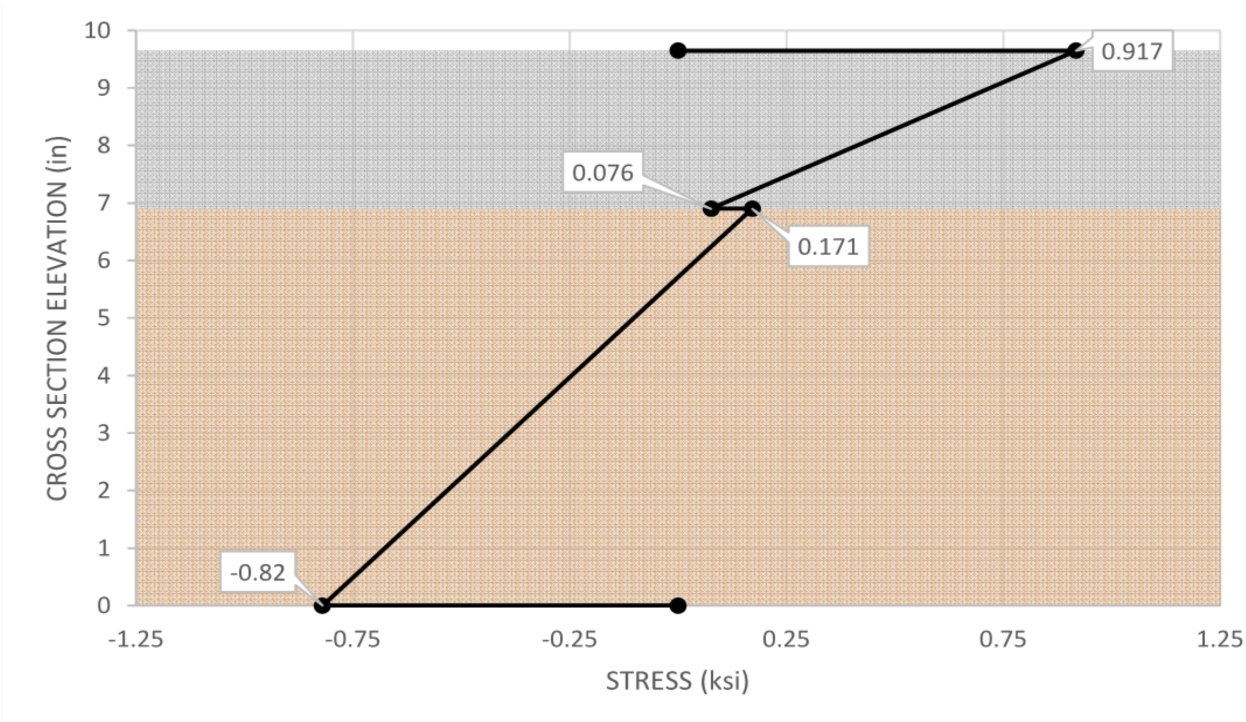


SERVICABILITY LIMIT STATE CROSS-SECTIONAL STRESS DISTRIBUTION

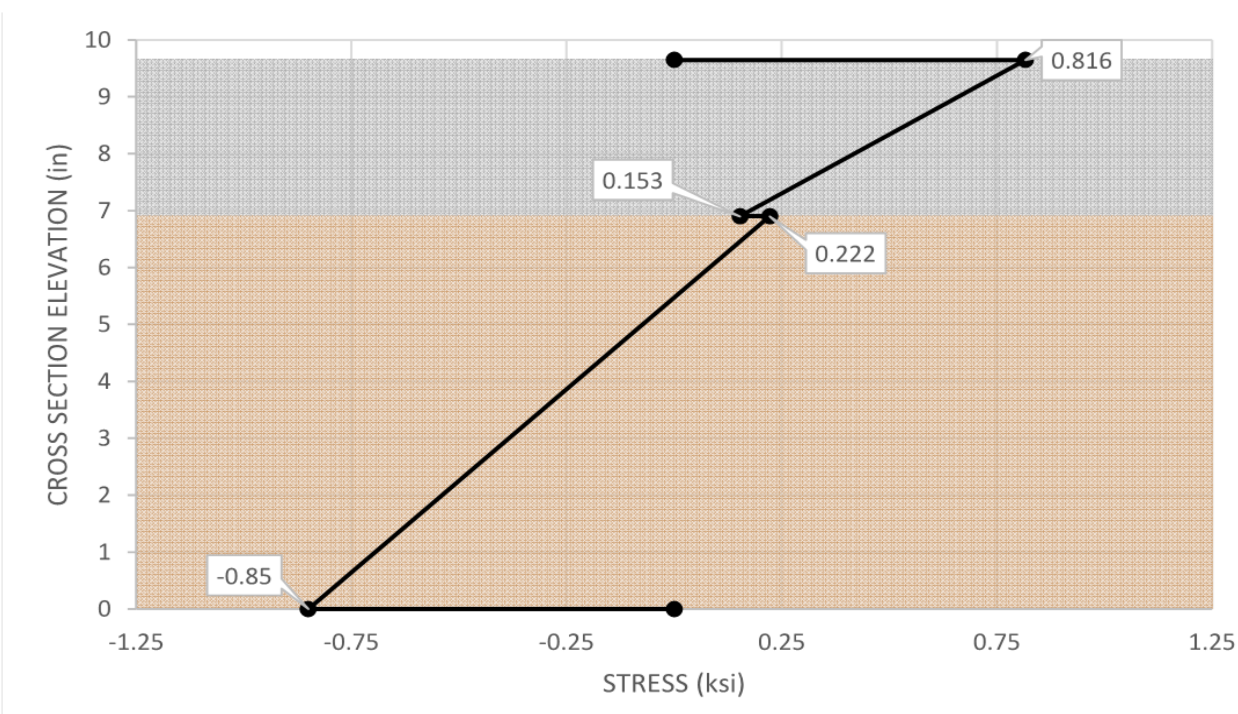


SERVICABILITY LIMIT STATE LONG-TERM CROSS-SECTIONAL STRESS DISTRIBUTION

90 mm HBV 22 feet SPAN



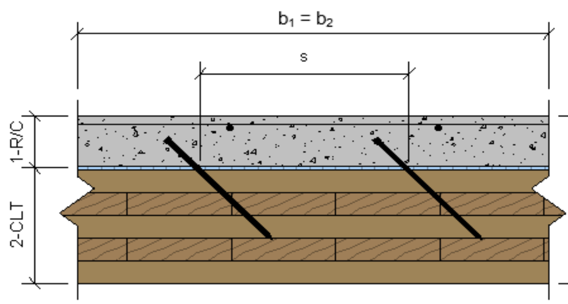
ULTIMATE LIMIT STATE CROSS-SECTIONAL STRESS DISTRIBUTION



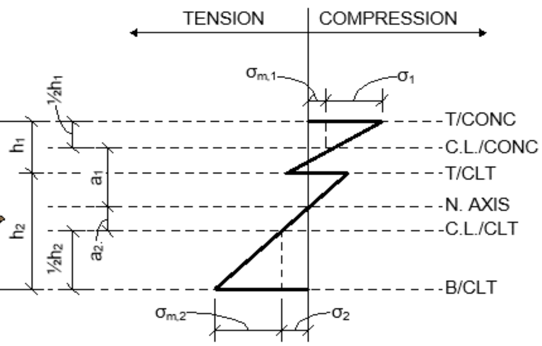
ULTIMATE LIMIT STATE LONG-TERM CROSS-SECTIONAL STRESS DISTRIBUTION

STEP DESCRIPTION	COMPUTATION	REFERNCE

90 mm HBV 22 feet SPAN



Cross-section



Stress Diagram

Dead Load Calculation

		Density			Height				
Mass Timber	ρ_t =	35	pcf	h_t =	6.90	in	L =	22	ft
Concrete	ρ_c =	150	pcf	h_c =	2.75	in			

$$w_{DL} = \rho_t h_t + \rho_c h_c$$
$$w_{DL} = (35\, pcf)(6.90\, in) + (150\, pcf)(2.75\, in) = \text{54.50 psf}$$

Loading Criteria

		Loading			Sustained/Expected		
Dead Load	w_{DL} =	54.50	psf	m_{DL} =	100%		ASCE 7-16
Superimposed Dead Load	w_{SDL} =	20	psf	m_{SDL} =	78.2%		
Live Load	w_{LL} =	50	psf	m_{LL} =	21.8%	(10.9 psf mean load)	Table C4.3-2

LRFD / **Service**

Loading Cases								
Load Case Factors				Equations	DL	SDL	LL	w_u
Combo	DL	SDL	LL		PSF	PSF	PSF	PLF
G01	1.4	1.4	0.0	$1.4(w_{DL} + w_{SDL})$	76.3	28	0	104.3
G02	1.2	1.2	1.6	$1.2(w_{DL} + w_{SDL}) + 1.6w_{LL}$	65.4	24	80	169.4
S01	1.0	1.0	1.0	$1.0w_{DL} + 1.0w_{SDL} + 1.0w_{LL}$	54.5	20	50	124.5
S02	0.0	1.0	1.0	$1.0w_{SDL} + 1.0w_{LL}$	0	20	50	70
S03	1.0	0.782	0.218	$1.0w_{DL} + 0.782w_{SDL} + 0.218w_{LL}$	54.5	15.64	10.9	81.04

Combo	w_u	V_u	M_u
	PLF	kips	kip-in
G01	104.3	1.147	75.72
G02	169.4	1.863	123
S01	124.5	1.37	90.39
S02	70	0.77	50.82
S03	81.04	0.891	58.84

$$V_u = \frac{w_u L}{2}$$
$$M_u = \frac{w_u L^2}{8}$$

STEP DESCRIPTION	COMPUTATION	REFERNCE

CLT Properties

CLT CALCULATIONS

Type: 5-Ply CLT, Grade E1M4, 139 E

h_i = 1.375 in

b = 12 in

h_t = 6.90 in

a = 5.52 in

L = 22.0 ft

K_s = 11.5

Reference: Structurlam Crosslam

Major Strength Axis

$F_{b,0}$ = 2100 psi

E_0 = 1800 ksi

$F_{t,0}$ = 1575 psi

$F_{c,0}$ = 1875 psi

$F_{v,0}$ = 160 psi

$F_{s,0}$ = 50 psi

Minor Strength Axis

$F_{b,90}$ = 875 psi

E_{90} = 1400 ksi

$F_{v,90}$ = 135 psi

$F_{s,90}$ = 45 psi

CLT Calculations

				GA_{eff}		EA_{eff}	EI_{eff}		
Layer	E (ksi)	h (in)	z (in)	G (ksi)	h/G/b (in^2/kip)	EA (kip)	$Ebh^3/12$ (kip-in^2)	EAz^2 (kip-in^2)	Sum of Layers (kip-in^2)
1	1800	1.380	2.760	112.5	0.00102	29808	4730.53	227065	231796
2	46.6667	1.380	1.375	8.75	0.01314	772.8	122.643	1461.08	1583.72
3	1800	1.380	0.000	112.5	0.00102	29808	4730.53	0	4730.53
4	46.6667	1.380	1.375	8.75	0.01314	772.8	122.643	1461.08	1583.72
5	1800	1.380	2.760	112.5	0.00102	29808	4730.53	227065	231796
						90969.6			471490

Reference: CLT Handbook, Chapter 3

$$EA_{eff} = \sum_{i=1}^{n} E_i b_i h_i$$

$$EI_{eff} = \sum_{i=1}^{n} E_i b_i \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i A_i z_i^2$$

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

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

Reference: CLT Handbook Chapter 3 EQ 24

Cross Sectional Properties (Per foot of width)

SLS/ULS Short-Term			SLS Long-Term			ULS Long-Term		
EA_{eff} =	90969.6	kip	EA_{eff} =	47878.7	kip	EA_{eff} =	71629.6	kip
EI_{eff} =	471490	kip-in^2	EI_{eff} =	248153	kip-in^2	EI_{eff} =	371252	kip-in^2
GA_{eff} =	1075.5	kip	GA_{eff} =	566.1	kip	GA_{eff} =	846.9	kip
EI_{app} =	439686.3	kip	EI_{app} =	235933.9	kip	EI_{app} =	352972.1	kip

STEP DESCRIPTION	COMPUTATION	REFERNCE
	CONNECTOR CALCUATION	
	Length of Connector Strip L = 1000 mm = 39.37 in	
	Thickness of the interlayer d_{zs} = 8.89 mm = 0.35 in	DIBt Z-9.1-557
	1 mm base unit d_0 = 1 mm	
Characteristic Load-Carrying Capacity	$T_k = 160 - 8.0(d_{zs}/d_0)^{0.5}$ $T_k = 160 - 8.0(8.89/1)^{0.5} =$ 136.1 kN	
Acceptable Shear Load	$zul.T = 90 - 4.5(d_{zs}/d_0)^{0.5}$ $zul.T = 90 - 4.5(8.89/1)^{0.5} =$ 76.58 kN	
Rated value of Load Bearing Capacity	$T_d = T_k/1.25 \approx 1.42\, zul.T$ $T_d = (136.1\ kN/mm)/1.25 =$ 108.9 kN	
Slip Modulus	$K_{ser} = 825 - 250(d_{zs}/d_0)^{0.2}$ $K_{ser} = 825 - 250(8.89/1)^{0.2} =$ 438 kN/mm	
Slip Modulus Ultimate	$K_u = (2/3)K_{ser}$ $K_u = (2/3)(438.0\ kN/mm) =$ 292 kN/mm Converstions $T_k =$ 30.6071 kips $K_{ser} =$ 2501 k/in $K_u =$ 1667.33 k/in	
	Spacing between connectors in rows at the ends s_e = 1372 mm = 54 in in rows in the middle s_m = 1372 mm = 54 in Effective Spacing between s_{eff} = 1372 mm = 54 in Number of rows of connectors n_r = 1 Deformation factor for long term loading for concrete k_{def_c} = 2.5 loading for timber k_{def_t} = 0.9 loading for STS $k_{defHBV\_}$ = 0.6 Stiffness reduction for ULS ψ_2 = 0.3	DIBt Z-9.1-557 Table A1.1 EN 1990
Connector Stiffness Adjustment for Long Term Loading	$K_{SLS\_LT} = \dfrac{K_{SER}}{1 + k_{def\_HBV}} = \dfrac{438.0\ kN/mm}{1 + 0.6} =$ 273.745 kN/mm $K_{ULS\_LT} = \dfrac{K_u}{1 + \psi_2 k_{def\_sts}} = \dfrac{292\, kN/mm}{1 + (0.3 * 0.6)} =$ 247.453 kN/mm	

Serviceability Limit State
Short-Term Loading

STEP DESCRIPTION	COMPUTATION	REFERNCE
General Values	EFFECTIVE STIFFNESS Gamma Span Length $L = L_0 =$ 22 ft Connector slip Modulus $K_{ser} =$ 437.992 kN/mm = 2501.00 kips/in Connector Spacing $s_{eff} =$ 54 in	Annex B of EN 1995-1-1
Concrete Properties	Slab Depth $h_1 =$ 2.75 in Compressive Strength $f_c' =$ 4000 psi Weight of Concrete $w_c =$ 150 pcf	
CLT Properties	Clt Height $h_2 =$ 6.90 in $EA_{eff,CLT} =$ 90969.6 kip $EI_{eff,CLT} =$ 471490 kip-in^2	CLT Handbook EQ 24 & 25
Modulus of Elasticity	$E_{1\_SLS} = w_c{}^{1.5} 33\sqrt{f_c'}$ $E_{1\_SLS} = (150\ pcf)^{1.5} 33\sqrt{4000\ psi} =$ 3834.25 ksi	ACI 318, 19.2.2.1.a
Area of Concrete	$A_{c\_SLS} = h_c b_c$ $A_{c\_SLS} = (2.75\ in)(12\ in) =$ 33 in^2	EN 1995.1-1 Annex B EQ 3.2
Inertia of Concrete	$I_{1\_SLS} = b_1 h_1{}^2/12$ $I_{1\_SLS} = (12\ in)(2.75\ in)^2/12 =$ 20.8 in^4 $E_c A_{c\_SLS} = (3834.3\ ksi)(33\ in^2) =$ 126530 kip $E_c I_{c\_SLS} = (38343\ ksi)(20.8\ in^4) =$ 79740.5 kip-in^2	EN 1995.1-1 Annex B EQ 3.3
Gamma Factor	$\gamma_{1\_SLS} = \left[1 + \frac{\pi^2 E_1 A_1 s_1}{K_1 L_0{}^2}\right]^{-1}$ $\gamma_{1\_SLS} = \left[1 + \frac{\pi^2 (3834.3\ ksi)(33\ in^2)(54\ in)}{(2501\ \ kip/in)(22\ ft)^2}\right]^{-1} =$ 0.72105	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2\_SLS} = \frac{\gamma_1 E_1 A_1 (h_1 + h_2)}{2\gamma_1 E_1 A_1 + 2\gamma_2 EA_{eff}}$ $a_{2\_SLS} = \frac{(0.7211)(3834\ ksi)(33\ in^2)(2.75\ in + 6.90\ in)}{2(0.7211)(3834\ ksi)(33\ in^2) + 2(1.0)(90970\ kip)} =$ 2.41601 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1\_SLS} = \frac{h_1 + h_2}{2} - a_{2\_SLS}$ $a_{1\_SLS} = \frac{6.90\ in + 2.75\ in}{2} - 2.42 =$ 2.40899 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_SLS} = (E_1 I_1 + \gamma_1 E_1 A_1 a_1{}^2) + (EI_{eff} + \gamma_2 EA_{eff} a_2{}^2)$ $(EI)_{eff,comp\_SLS} =$ 609196.7 kip-in^2 + 1002487 kip-in^2 = 1611684 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Composite & CLT Effective	$Ratio_{CLT} = \frac{EI_{eff,comp\_SLS}}{EI_{eff,CLT}}$ $Ratio_{CLT} = \frac{1611684\ kip \cdot in^2}{471489.9\ kip \cdot in^2} =$ 3.418	

Serviceability Limit State
Short-Term Loading

STEP DESCRIPTION	COMPUTATION	REFERNCE
	APPARENT STIFFNESS	
Bare CLT Stiffness	Length $L = 22$ ft K Value $K_s = 11.5$ $EI_{eff,clt} = 471490$ kip-in^2 $GA_{eff,clt} = 1075.5$ kips	
	$EI_{app,CLT\_SLS} = \dfrac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \dfrac{471489.9\ kip\cdot in^2}{1+\frac{(11.5)(471489.9\ kip\cdot in^2)}{(1075.5\ kip)(22\ ft)^2}} =$ 439686 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \dfrac{EI_{app,CLT}}{EI_{eff,CLT}} = \dfrac{439686.3\ kip\cdot in^2}{471489.9\ kip\cdot in^2} =$ 93.3%	
Composite Section Stiffness	$L = 22$ ft $K_s = 11.5$ $EI_{eff,comp} = 1611684$ kip-in^2 $GA_{eff,clt} = 1075.5$ kips	
	$EI_{app,comp\_SLS} = \dfrac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \dfrac{1611684\ kip\cdot in^2}{1+\frac{(11.5)(1611684\ kip\cdot in^2)}{(1075.5\ kip)(23\ ft)^2}} =$ 1292187 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \dfrac{EI_{app,,comp}}{EI_{eff,comp}} = \dfrac{1292187\ kip\cdot in^2}{1611684\ kip\cdot in^2} =$ 80.2%	
	$Ratio_{CLT} = \dfrac{EI_{app,comp}}{EI_{app,cLT}} = \dfrac{1292187\ kip\cdot in^2}{439686.3\ kip\cdot in^2} =$ 2.939	
Apparent Floor Stiffness	$L = 22$ ft $EI_{app,CLT} = 439686$ kip-in^2 $EI_{app,comp} = 1292187$ kip-in^2	
	$K_{app,CLT\_SLS} = \dfrac{P}{\Delta} = \dfrac{48EI_{app,CLT}}{L^3}$	
	$K_{app,CLT\_SLS} = \dfrac{48(439686.3\ kip\cdot in^2)}{(22\ ft)^3} =$ 1.14702 kip/in	
	$K_{app,comp\_SLS} = \dfrac{P}{\Delta} = \dfrac{48EI_{app,comp}}{L^3}$	
	$K_{app,comp\_SLS} = \dfrac{48(1292187\ kip\cdot in^2)}{(22\ ft)^3} =$ 3.37097 kip/in	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length L = 22 ft Shear V_u = 1.37 kip Moment M_u = 90.39 kip-in	
	Concrete / CLT Modulus of Elasticity E_1 = 3834.25 ksi; E_2 = 1800 ksi Height h_1 = 2.75 in; h_2 = 6.90 in Centroid a_1 = 2.40899 in; a_2 = 2.41601 in Gamma Factor γ_1 = 0.72105; γ_2 = 1 $EI_{eff,comp\_SLS}$ = 1611684 kip-in^2	
	Bending Stress Calculations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.721)(3834\ ksi)(2.41\ in)(90.4\ kip \cdot in)}{1611684\ kip \cdot in^2} =$ 0.374 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(3834\ ksi)(2.75\ in)(90.4\ kip \cdot in)}{1611684\ kip \cdot in^2} =$ 0.296 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.374 + 0.296 =$ 0.669 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.374 - 0.296 =$ 0.078 ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(1800\ ksi)(2.416\ in)(90.4\ kip \cdot in)}{1611684\ kip \cdot in^2} =$ 0.244 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(1800\ ksi)(6.90\ in)(90.4\ kip \cdot in)}{1611684\ kip \cdot in^2} =$ 0.348 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} + \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.244 + 0.348 =$ 0.104 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.24 - 0.348 =$ -0.59 ksi	

<table>
<tr><th>STEP DESCRIPTION</th><th>COMPUTATION</th><th>REFERNCE</th></tr>
<tr><td></td><td>DEFLECTION CALCUATIONS</td><td></td></tr>
<tr><td>Loading</td><td>Short-Term
Dead Load w_{DL} = 54.50 plf
Superimposed Dead Load w_{SDL} = 20.00 plf
Live Load w_{LL} = 50.00 plf
Span Length L = 22 ft
Stiffness $EI_{eff,comp\_SLS}$ = 1611684 kip-in^2

Long-Term
w_{DL} = 54.50 plf
w_{SDL} = 15.64 plf
w_{LL} = 10.9 plf
L = 22 ft</td><td></td></tr>
<tr><td></td><td>Short-Term Deflections</td><td></td></tr>
<tr><td>Dead Load</td><td>$\Delta_{DL}=\dfrac{5w_{DL}L^4}{384EI_{eff,comp\_SLS}}=\dfrac{5(54.5\,plf)(22\,ft)^4}{384(1611683.6\,kip\cdot in^2)}=$ 0.1782 in</td><td></td></tr>
<tr><td>Superimposed Dead Load</td><td>$\Delta_{SDL}=\dfrac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS}}=\dfrac{5(20\,plf)(22\,ft)^4}{384(1611683.6\,kip\cdot in^2)}=$ 0.0654 in</td><td></td></tr>
<tr><td>Live Load</td><td>$\Delta_{LL}=\dfrac{5w_{LL}L^4}{384EI_{eff,comp\_SLS}}=\dfrac{5(50\,plf)(22\,ft)^4}{384(1611683.6\,kip-in^2)}=$ 0.1635 in</td><td></td></tr>
<tr><td></td><td>Serviceability Check</td><td></td></tr>
<tr><td>Allowable LL Deflection</td><td>$\Delta_{allow,LL}=\dfrac{L}{360}=\dfrac{(22ft)(12\,in)}{360(1\,ft)}=$ 0.73 in</td><td>IBC Table 1604.3</td></tr>
<tr><td>Check</td><td>$\Delta_{LL}\leq\ \Delta_{allow,LL}\rightarrow\Delta_{LL}=0.164\,in\ <\ \Delta_{allow,LL}=0.73\,in$ ∴ ACCEPTABLE</td><td></td></tr>
<tr><td></td><td>Long-Term Deflections</td><td></td></tr>
<tr><td>Dead Load</td><td>$\Delta_{DL\_LT}=\dfrac{5w_{DL}L^4}{384EI_{eff,comp\_SLS\_LT}}=\dfrac{5(54.5\,plf)(22\,ft)^4}{384(1611684\,kip\cdot in^2)}=$ 0.17823 in</td><td></td></tr>
<tr><td>Superimposed Dead Load</td><td>$\Delta_{SDL\_LT}=\dfrac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS\_LT}}=\dfrac{5(15.6\,plf)(22\,ft)^4}{384(1611684\,kip\cdot in^2)}=$ 0.05115 in</td><td></td></tr>
<tr><td>Live Load</td><td>$\Delta_{LL\_LT}=\dfrac{5w_{LL}L^4}{384EI_{eff,comp\_SLS\_LT}}=\dfrac{5(10.9\,plf)(22\,ft)^4}{384(1611684\,kip\cdot in^2)}=$ 0.03565 in</td><td></td></tr>
<tr><td></td><td>Serviceability Check
Time Dependent Creep Factor $K_{cr}=$ 2.0
Deflection due to short-term $\Delta_{ST}=$ 0.1635 in
Deflection due to long-term $\Delta_{LT}=$ 0.22938 in $\Delta_{LT}=\Delta_{DL\_LT}+\Delta_{SDL\_LT}$</td><td>NDS 2018 Section 3.5.1</td></tr>
<tr><td></td><td>$\Delta_{TL}=K_{cr}\Delta_{LT}+\Delta_{ST}$
$\Delta_{TL}=2.0(0.2294\text{ in})+(0.1635\text{ in})=$ 0.622 in</td><td>NDS EQ 3.5-1</td></tr>
<tr><td></td><td>$\Delta_{allow,TL}=\dfrac{L}{240}=\dfrac{(22ft)(12\,in)}{240(1\,ft)}=$ 1.10 in</td><td>IBC Table 1604.3</td></tr>
<tr><td></td><td>$\Delta_{TL}\leq\ \Delta_{allow,TL}\rightarrow\Delta_{TL}=0.622\,in\ <\ \Delta_{allow,TL}=1.10\,in$ ∴ ACCEPTABLE</td><td></td></tr>
</table>

Serviceability Limit State
Long-Term Loading

STEP DESCRIPTION	COMPUTATION	REFERNCE
	EFFECTIVE STIFFNESS	
	Deformation factor for long term	
	loading for concrete k_{def_c} = 2.5	EN 1995.1-1
	loading for timber k_{def_t} = 0.9	
	loading for HBV k_{def_HBV} = 0.6	
	Stiffness reduction for ULS ψ_2 = 0.3	
	Effective Spacing between s_{eff} = 1372 mm = 54 in	ETA-13/0029 Table 2.1
	Gamma Span Length L_0 = 22 ft	
	Connector slip Modulus K_{SLS_LT} = 273.745 kN/mm = 1563.12 kips/in	
	Concrete / CLT	
	Modulus of Elasticity: E_1 = 3834.25 ksi; E_2 = 1800 ksi	
	Area: A_1 = 33 in^2; A_1 = 26.60 in^2	
	Momemnt of Inertia: I_1 = 20.7969 in^4; I_1 = 137.863 in^4	
	Height: h_1 = 2.75 in; h_2 = 6.90 in	
	EA_{eff,CLT_SLS_LT} = 47878.74 kip EI_{eff,CLT_SLS_LT} = 248153 kip-in^2	CLT Handbook EQ 24 & 25
Modulus of Adjustment for long term loading	$E_{1\_SLS\_LT} = \dfrac{E_1}{1+k_{def\_c}} =$ 1095.5 ksi $E_{2\_SLS\_LT} = \dfrac{E_2}{1+k_{def\_t}} =$ 947.368 ksi	EN 1995.1-1
Gamma Factor	$\gamma_{1,SLS\_LT} = \left[1+\dfrac{\pi^2 E_{1\_SLS\_LT} A_1 s_{eff}}{K_{ser\_LT} {L_0}^2}\right]^{-1}$ $\gamma_{1,SLS\_LT} = \left[1+\dfrac{\pi^2 (1095\,ksi)(33\,in^2)(54\,in)}{(1563\,kip/in)(22\,ft)^2}\right]^{-1} =$ 0.850	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2,SLS\_LT} = \dfrac{\gamma_{1\_SER} E_1 A_1 (h_1+h_2)}{2\gamma_{1\_SER} E_1 A_1 + 2\gamma_2 EA_{eff}}$ $a_{2,SLS\_LT} = \dfrac{(0.850)(1095\,ksi)(33\,in^2)(2.75\,in+6.90\,in)}{2(0.850)(1095\,ksi)(33\,in^2)+2(1.0)(47878.7\,kip)} =$ 2.65063 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1,SLS\_LT} = \dfrac{h_1+h_2}{2} - a_{2,SLS\_LT}$ $a_{1,SLS\_LT} = \dfrac{6.90\,in + 2.75\,in}{2} - 2.650 =$ 2.174 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_SLS\_LT} = \left(E_1 I_1 + \gamma_{1\_SER} E_1 A_1 {a_{1\_SER}}^2\right) + \left(EI_{eff} + \gamma_2 EA_{eff} {a_{2\_SER}}^2\right)$ $(EI)_{eff,comp\_SLS\_LT} =$ 168018 kip-in^2 + 584540 kip-in^2 $(EI)_{eff,comp\_SLT\_LT} =$ 752558.1 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Compsite & CLT Effective	$Ratio_{CLT\_SER} = \dfrac{EI_{eff,comp\_SER}}{EI_{eff,CLT}}$ $Ratio_{CLT\_SER} = \dfrac{752558.1\,kip\cdot in^2}{248152.6\,kip\cdot in^2} =$ 15.72	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	APPARENT STIFFNESS	
Bare CLT Stiffness	Length L = 22 ft K Value K_s = 11.5 $EI_{eff,clt}$ = 248153 kip-in^2 $GA_{eff,clt}$ = 566.1 kips	
	$EI_{app,CLT\_SLS\_LT} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{248152.6\ kip\cdot in^2}{1+\frac{(11.5)(248152.6\ kip\cdot in^2)}{(566.1\ kip)(22\ ft)^2}} =$ 231414 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{231413.8\ kip\cdot in^2}{248152.6\ kip\cdot in^2} =$ 93.3%	
Composite Section Stiffness	L = 22 ft K_s = 11.5 $EI_{eff,comp}$ = 752558 kip-in^2 $GA_{eff,clt}$ = 566.1 kips	
	$EI_{app,comp\_SLS\_LT} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{752558.1\ kip\cdot in^2}{1+\frac{(11.5)(752558\ kip\cdot in^2)}{(566.1\ kip)(22\ ft)^2}} =$ 617175 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{617175.4\ kip\cdot in^2}{752558.1\ kip\cdot in^2} =$ 82.0%	
	$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{617175.4\ kip\cdot in^2}{231413.8\ kip\cdot in^2} =$ 2.667	
Apparent Floor Stiffness	L = 22 ft $EI_{app,CLT}$ = 231414 kip-in^2 $EI_{app,comp}$ = 617175 kip-in^2	
	$K_{app,CLT\_SLS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$	
	$K_{app,CLT\_SLS\_LT} = \frac{48(231413.8\ kip\cdot in^2)}{(22\ ft)^3} =$ 0.6037 kip/in	
	$K_{app,comp\_SLS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$	
	$K_{app,comp\_SLS\_LT} = \frac{48(617175.4\ kip\cdot in^2)}{(22\ ft)^3} =$ 1.61005 kip/in	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length $L = 22$ ft Shear $V_u = 0.89$ kip Moment $M_u = 58.84$ kip-in	
	Concrete: Modulus of Elasticity $E_1 = 1095.5$ ksi; Height $h_1 = 2.75$ in; Centroid $a_1 = 2.17437$ in; Gamma Factor $\gamma_1 = 0.84972$ CLT: $E_2 = 947.368$ ksi; $h_2 = 6.90$ in; $a_2 = 2.65063$ in; $\gamma_2 = 1$ $EI_{eff,comp\_SLS\_LT} = 752558$ kip-in^2	
	Bending Stess Calulations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.850)(1096\ ksi)(2.17\ in)(58.8\ kip \cdot in)}{752558.1\ kip \cdot in^2} =$ 0.158 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(1096\ ksi)(2.75\ in)(58.8\ kip \cdot in)}{752558.1\ kip \cdot in^2} =$ 0.118 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.158 + 0.118 =$ 0.276 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.158 - 0.118 =$ 0.040 ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(947.4\ ksi)(2.65\ in)(58.8\ kip \cdot in)}{752558.1\ kip \cdot in^2} =$ 0.196 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(947.4\ ksi)(6.90\ in)(58.8\ kip \cdot in)}{752558.1\ kip \cdot in^2} =$ 0.256 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.196 + 0.256 =$ 0.059 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.20 - 0.256 =$ -0.45 ksi	

Serviceability Limit State
Long-Term Loading

STEP DESCRIPTION	COMPUTATION	REFERNCE
	DEFLECTION CALCUATIONS	
Loading	Short-Term: Dead Load w_{DL} = 54.50 plf; Superimposed Dead Load w_{SDL} = 20.00 plf; Live Load w_{LL} = 50.00 plf; Span Length L = 22 ft; Stiffness $EI_{eff,comp\_SLS}$ = 1611684 kip-in^2 Long-Term: w_{DL} = 54.50 plf; w_{SDL} = 15.64 plf; w_{LL} = 10.9 plf; L = 22 ft; $EI_{eff,comp\_SLS\_LT}$ = 752558.1 kip-in^2	
	Short-Term Deflections	
Dead Load	$\Delta_{DL} = \frac{5w_{DL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(54.5\,plf)(24\,ft)^4}{384(1621574.8\,kip \cdot in^2)} =$ 0.1782 in	
Superimposed Dead Load	$\Delta_{SDL} = \frac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(20\,plf)(24\,ft)^4}{384(1621574.8\,kip \cdot in^2)} =$ 0.0654 in	
Live Load	$\Delta_{LL} = \frac{5w_{LL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(50\,plf)(24\,ft)^4}{384(1621574.8\,kip - in^2)} =$ 0.1635 in	
	Servicablility Check	
Allowable LL Deflection	$\Delta_{allow,LL} = \frac{L}{360} = \frac{(22ft)(12\,in)}{360(1\,ft)} =$ 0.73 in	IBC Table 1604.3
Check	$\Delta_{LL} \leq \Delta_{allow,LL} \rightarrow \Delta_{LL} = 0.164\,in < \Delta_{allow,LL} = 0.73\,in$ ∴ ACCEPTABLE	
	Long-Term Deflections	
Dead Load	$\Delta_{DL\_LT} = \frac{5w_{DL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(54.5\,plf)(22\,ft)^4}{384(752558.1\,kip \cdot in^2)} =$ 0.38171 in	
Superimposed Dead Load	$\Delta_{SDL\_LT} = \frac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(15.6\,plf)(22\,ft)^4}{384(752558.1\,kip \cdot in^2)} =$ 0.10954 in	
Live Load	$\Delta_{LL\_LT} = \frac{5w_{LL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(10.9\,plf)(22\,ft)^4}{384(752558.1\,kip \cdot in^2)} =$ 0.07634 in	
	Servicablility Check Deflection due to short-term Δ_{ST} = 0.1635 in Deflection due to long-term Δ_{LT} = 0.49125 in $\Delta_{LT} = \Delta_{DL\_LT} + \Delta_{SDL\_LT}$ Deflection due to Total Load Δ_{TL} = 0.6548 in $\Delta_{TL} = \Delta_{LT} + \Delta_{ST}$	
	$\Delta_{allow,TL} = \frac{L}{240} = \frac{(22ft)(12\,in)}{240(1\,ft)} =$ 1.10 in	IBC Table 1604.3
	$\Delta_{TL} \leq \Delta_{allow,TL} \rightarrow \Delta_{TL} = 0.655\,in < \Delta_{allow,TL} = 1.10\,in$ ∴ ACCEPTABLE	

Ultimate Limit State
Short-Term Loading

STEP DESCRIPTION	COMPUTATION	REFERNCE
General Values	EFFECTIVE STIFFNESS Gamma Span Length $L = L_0 =$ 22 ft Connector slip Modulus $K_U =$ 291.994 kN/mm = 1667.33 kips/in Connector Spacing $s_{eff} =$ 54 in	Annex B of EN 1995-1-1
Concrete Properties	Slab Depth $h_1 =$ 2.75 in Compressive Strength $f_c' =$ 4000 psi Weight of Concete $w_c =$ 150 pcf	
CLT Properties	Clt Height $h_2 =$ 6.90 in $EA_{eff,CLT} =$ 90969.6 kip $EI_{eff,CLT} =$ 471490 kip-in^2	CLT Handbook EQ 24 & 25
Modulus of Elasticity	$E_{1\_ULS} = {w_c}^{1.5} 33\sqrt{f_c'}$ $E_{1\_ULS} = (150\ pcf)^{1.5} 33\sqrt{4000\ psi} =$ 3834.25 ksi	ACI 318, 19.2.2.1.a
Area of Concrete	$A_{c\_ULS} = h_c b_c$ $A_{c\_ULS} = (2.75\ in)(12\ in) =$ 33 in^2	EN 1995.1-1 Annex B EQ 3.2
Inhertia of Concrete	$I_{1\_ULS} = b_1 {h_1}^2/12$ $I_{1\_ULS} = (12\ in)(2.75\ in)^2/12 =$ 20.8 in^4	EN 1995.1-1 Annex B EQ 3.3
	$E_c A_{c\_ULS} = (3834.3\ ksi)(33\ in^2) =$ 126530 kip $E_c I_{c\_ULS} = (38343\ ksi)(20.8\ in^4) =$ 79740.5 kip-in^2	
Gamma Factor	$\gamma_{1\_ULS} = \left[1 + \frac{\pi^2 E_1 A_1 s_1}{K_1 {L_0}^2}\right]^{-1}$ $\gamma_{1\_ULS} = \left[1 + \frac{\pi^2 (3834.3\ ksi)(33\ in^2)(54\ in)}{(1667\ kip/in)(22\ ft)^2}\right]^{-1} =$ 0.63279	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2\_ULS} = \frac{\gamma_1 E_1 A_1 (h_1 + h_2)}{2\gamma_1 E_1 A_1 + 2\gamma_2 EA_{eff}}$ $a_{2\_ULS} = \frac{(0.6328)(3834\ ksi)(33\ in^2)(2.75\ in + 6.90\ in)}{2(0.6328)(3834\ ksi)(33\ in^2) + 2(1.0)(90970\ kip)} =$ 2.25872 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1\_ULS} = \frac{h_1 + h_2}{2} - a_{2\_ULS}$ $a_{1\_ULS} = \frac{6.90\ in + 2.75\ in}{2} - 2.26 =$ 2.56628 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_ULS} = (E_1 I_1 + \gamma_1 E_1 A_1 {a_1}^2) + \left(EI_{eff} + \gamma_2 EA_{eff} {a_2}^2\right)$ $(EI)_{eff,comp\_ULS} =$ 607046.4 kip-in^2 + 935598 kip-in^2 = 1542645 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Compsite & CLT Effective	$Ratio_{CLT} = \frac{EI_{eff,comp\_ULS}}{EI_{eff,CLT\_ULS}}$ $Ratio_{CLT} = \frac{1542645\ kip \cdot in^2}{471489.9\ kip \cdot in^2} =$ 3.272	

<table>
<tr><th>STEP DESCRIPTION</th><th>COMPUTATION</th><th>REFERNCE</th></tr>
<tr><td></td><td>APPARENT STIFFNESS</td><td></td></tr>
<tr><td>Bare CLT Stiffness</td><td>Length L = 22 ft
K Value K_s = 11.5
$EI_{eff,clt}$ = 471490 kip-in^2
$GA_{eff,clt}$ = 1075.5 kips

$EI_{app,CLT\_ULS} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{471489.9\ kip \cdot in^2}{1+\frac{(11.5)(471489.9\ kip \cdot in^2)}{(1075.5\ kip)(22\ ft)^2}} =$ 439686 kip-in^2

$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{439686.3\ kip \cdot in^2}{471489.9\ kip \cdot in^2} =$ 93.3%</td><td>CLT Handbook EQ 35</td></tr>
<tr><td>Composite Section Stiffness</td><td>L = 22 ft
K_s = 11.5
$EI_{eff,comp}$ = 1542645 kip-in^2
$GA_{eff,clt}$ = 1075.5 kips

$EI_{app,comp\_ULS} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{1542645\ kip \cdot in^2}{1+\frac{(11.5)(1542645\ kip \cdot in^2)}{(1075.5\ kip)(22\ ft)^2}} =$ 1247427 kip-in^2

$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{1247427\ kip \cdot in^2}{1542654\ kip \cdot in^2} =$ 80.9%

$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{1247427\ kip \cdot in^2}{439686.3\ kip \cdot in^2} =$ 2.837</td><td>CLT Handbook EQ 35</td></tr>
<tr><td>Apparent Floor Stiffness</td><td>L = 22 ft
$EI_{app,CLT}$ = 439686 kip-in^2
$EI_{app,comp}$ = 1247427 kip-in^2

$K_{app,CLT\_ULS} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$

$K_{app,CLT\_ULS} = \frac{48(439686.3\ kip \cdot in^2)}{(22\ ft)^3} =$ 1.14702 kip/in

$K_{app,comp\_ULS} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$

$K_{app,comp\_ULS} = \frac{48(1247427\ kip \cdot in^2)}{(22\ ft)^3} =$ 3.2542 kip/in</td><td></td></tr>
</table>

STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length $L = 22$ ft Shear $V_u = 1.86$ kip Moment $M_u = 122.98$ kip-in	
	Concrete: Modulus of Elasticity $E_1 = 3834.25$ ksi; Height $h_1 = 2.75$ in; Centroid $a_1 = 2.56628$ in; Gamma Factor $\gamma_1 = 0.63279$ CLT: $E_2 = 1800$ ksi; $h_2 = 6.90$ in; $a_2 = 2.25872$ in; $\gamma_2 = 1$ $EI_{eff,comp} = 1542645$ kip-in^2	
	Bending Stress Calculations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.633)(3834\ ksi)(2.57\ in)(123\ kip \cdot in)}{1542645\ kip \cdot in^2} =$ 0.496 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(3834\ ksi)(2.75\ in)(123\ kip \cdot in)}{1542645\ kip \cdot in^2} =$ 0.420 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.496 + 0.420 =$ 0.917 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{+\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.496 - 0.420 =$ 0.076 ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(1800\ ksi)(2.26\ in)(123\ kip \cdot in)}{1542645\ kip \cdot in^2} =$ 0.324 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(1800\ ksi)(6.90\ in)(123\ kip \cdot in)}{1542645\ kip \cdot in^2} =$ 0.495 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} + \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.324 + 0.495 =$ 0.171 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.32 - 0.495 =$ -0.82 ksi	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	STRENGTH ANALYSIS	
Concrete	Normal Stress in Concrete σ_1 = 496.4 psi	
	Allow Comp Strength of Conc F_c = 2000 psi $F_c = f_c'/2$	
	$\sigma_1 < F_c \rightarrow \sigma_1 = 496.4\,psi < F_c = 2000\,psi$ ∴ ACCEPTABLE	
Timber	Bending Strength F_b = 2100 psi	
	Format Conversion Factor K_F = 2.54	
	Resistance Factor Φ = 0.85	
	Design Bending Strength	
	$F_b' = \Phi F_b K_F = 0.85(2100\,psi)(2.54) =$ 4533.9 psi	
	Tension Strength F_t = 1575 psi	
	Format Conversion Factor K_F = 2.70	
	Resistance Factor Φ = 0.80	
	Design Tension Strength	
	$F_t' = \Phi F_t K_F = 0.80(1575\,psi)(2.70) =$ 3402.0 psi	
	Average CLT Stress σ_2 = 324.1 psi	
	Extreme Fiber Stress $\sigma_{m,2}$ = 495.1 psi	
	$\frac{\sigma_2}{F_t'} + \frac{\sigma_{m,2}}{F_b'} \leq 1 \rightarrow \frac{324.1\,psi}{3402\,psi} + \frac{495.1\,psi}{4534\,psi} =$ 0.204 < 1.0 ∴ ACCEPTABLE	
	Shear Strength F_v = 160 psi	
	Format Conversion Factor K_F = 2.88	
	Resistance Factor Φ = 0.75	
	Design Shear Strength	
	$F_v' = \phi F_v K_F = 0.75(160\,psi)(2.88) =$ 345.6 psi	
	CLT Modulus of Elasticity E_2 = 1800 ksi	
	NA of timber h = 5.71 in	
	Shear V = 1.86 kip	
	$EI_{eff,comp}$ = 1542645 kip-in^2	
	$\tau_{2,max} = \frac{E_2 h^2 V_u}{2(EI)_{eff,comp}} = \frac{(1800\,ksi)(5.71\,in)^2(1.86\,kip)}{2(1542645\,kip \cdot in^2)} =$ 35.43 psi	EN 1995.1-1 Annex B EQ 3.9
	$\tau_{2,max} < F_v' \rightarrow \tau_{2,max} = 35.43\,psi < F_v' = 345.6\,psi$ ∴ ACCEPTABLE	

STEP DESCRIPTION	COMPUTATION	REFERNCE
Fastener Load Calculation	STRENGTH ANALYSIS F_{ULT} = 76.5828 kips Gamma Factor γ_1 = 0.63279 ksi Concrete Modulus of Elasticity E_1 = 3834.25 in Concrete Area A_1 = 33.00 in^2 Composite Centroid a_1 = 2.57 in Fastener Spacing s = 54 in Ultimate Shear Loading V = 1.8634 kip $EI_{eff,comp\_ULS}$ = 1542645 kip-in^2 $F_q = \frac{\gamma_1 E_1 A_1 a_1 sV}{EI_{eff,comp}}$ $F_q = \frac{(0.633)(3834\ ksi)(33 in^2)(2.57\ in)(54\ in)(1.86\ kip)}{1542645\ kip \cdot in^2} =$ 13.4 kip $F_q < F_{ULT} \rightarrow F_q = 13.4\ kip < F_{ULT} = 76.6\ kip$ ∴ ACCEPTABLE	EN 1995.1-1 Annex B EQ 3.10

STEP DESCRIPTION	COMPUTATION	REFERNCE
	EFFECTIVE STIFFNESS	
	Deformation factor for long term	
	loading for concrete k_{def_c} = 2.5	EN 1995.1-1
	loading for timber k_{def_t} = 0.9	
	loading for HBV k_{def_HBV} = 0.6	
	Stiffness reduction for ULS ψ_2 = 0.3	
	Effective Spacing between s_{eff} = 1372 mm = 54 in	ETA-13/0029 Table 2.1
	Gamma Span Length L_0 = 22 ft	
	Connector slip Modulus K_{U_LT} = 247.453 kN/mm = 1412.99 kips/in	
	Concrete / CLT	
	Modulus of Elasticity E_1 = 3834.25 ksi; E_2 = 1800 ksi	
	Area A_1 = 33 in^2; A_1 = 50.5387 in^2	
	Momemnt of Inertia I_1 = 20.7969 in^4; I_1 = 261.939 in^4	
	Height h_1 = 2.75 in; h_2 = 6.90 in	
	EA_{eff,CLT_ULT_LT} = 71629.6 in	CLT Handbook EQ 24 & 25
	EI_{eff,CLT_ULT_LT} = 371252 kip-in^2	
Modulus of Adjustment for long term loading	$E_{1,ULT\_LT} = \dfrac{E_1}{1+\psi_2 k_{def\_c}} =$ 2191 ksi; $E_{2,ULT\_LT} = \dfrac{E_2}{1+\psi_2 k_{def\_t}} =$ 1417.32 ksi	EN 1995.1-1
Gamma Factor	$\gamma_{1_{ULT\_LT}} = \left[1+\dfrac{\pi^2 E_{1,ULT\_LT} A_1 s_{eff}}{K_{ULT\_LT} {L_0}^2}\right]^{-1}$ $\gamma_{1,ULT\_LT} = \left[1+\dfrac{\pi^2 (2191\ ksi)(33\ in^2)(54\ in)}{(1413\ \ kip/in)(22\ ft)^2}\right]^{-1} =$ 0.719	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2,ULT\_LT} = \dfrac{\gamma_{1\_ULT\_LT} E_{1,ULT\_LT} A_1 (h_1+h_2)}{2\gamma_{1,ULT\_LT} E_{1,ULT\_LT} A_1 + 2\gamma_2 EA_{eff,CLT\_ULT\_LT}}$ $a_{2,ULT\_LT} = \dfrac{(0.719)(2191.0\ ksi)(33\ in^2)(2.75\ in+6.90\ in)}{2(0.719)(2191.0 ksi)(33\ in^2)+2(1.0)(71629.6\ kip)} =$ 2.029 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1,ULT\_LT} = \dfrac{h_1+h_2}{2} - a_{2\_ULT}$ $a_{1,ULT\_LT} = \dfrac{6.90\ in + 2.75\ in}{2} - 2.03 =$ 2.796 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_ULT\_LT} = \left(E_{1,ULT\_LT} I_1 + \gamma_{1,ULT\_LT} E_{1,ULT\_LT} A_1 {a_{1,ULT\_LT}}^2\right) + \left(EI_{eff} + \gamma_2 EA_{eff} {a_{2,ULT}}^2\right)$ $(EI)_{eff,comp\_ULT\_LT} =$ 451912 kip-in^2 + 666061 kip-in^2 $(EI)_{eff,comp\_ULT\_LT} =$ 1117973 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Compsite & CLT Effective	$Ratio_{CLT\_ULT} = \dfrac{EI_{eff,comp\_ULT\_LT}}{EI_{eff,CLT\_ULT\_LT}}$ $Ratio_{CLT\_ULT} = \dfrac{1117973\ kip \cdot in^2}{371251.9\ kip \cdot in^2} =$ 15.61	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	APPARENT STIFFNESS	
Bare CLT Stiffness	Length L = 22 ft K Value K_s = 11.5 EI_{eff,CLT_ULT_LT} = 371252 kip-in^2 GA_{eff,CLT_ULT_LT} = 846.9 kips	
	$EI_{app,CLT\_ULS\_LT} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{371252\ kip\cdot in^2}{1+\frac{(11.5)(371252\ kip\cdot in^2)}{(846.9\ kip)(22\ ft)^2}} =$ 346210 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{346209.7\ kip\cdot in^2}{371251.9\ kip\cdot in^2} =$ 93.3%	
Composite Section Stiffness	L = 22 ft K_s = 11.5 $EI_{eff,comp}$ = 1117973 kip-in^2 $GA_{eff,clt}$ = 846.9 kips	
	$EI_{app,comp\_ULS\_LT} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{1117973\ kip\cdot in^2}{1+\frac{(11.5)(1117973 kip\cdot in^2)}{(846.9\ kip)(22\ ft)^2}} =$ 918013 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{918012.6\ kip\cdot in^2}{1117973\ kip\cdot in^2} =$ 82.1%	
	$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{918012.6\ kip\cdot in^2}{346209.7\ kip\cdot in^2} =$ 2.652	
Apparent Floor Stiffness	L = 22 ft $EI_{app,CLT}$ = 346210 kip-in^2 $EI_{app,comp}$ = 918013 kip-in^2	
	$K_{app,CLT\_ULS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$	
	$K_{app,CLT\_ULS\_LT} = \frac{48(346209.7\ kip\cdot in^2)}{(22\ ft)^3} =$ 0.90317 kip/in	
	$K_{app,comp\_ULS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$	
	$K_{app,comp\_ULS\_LT} = \frac{48(918012.6\ kip\cdot in^2)}{(22\ ft)^3} =$ 2.39485 kip/in	

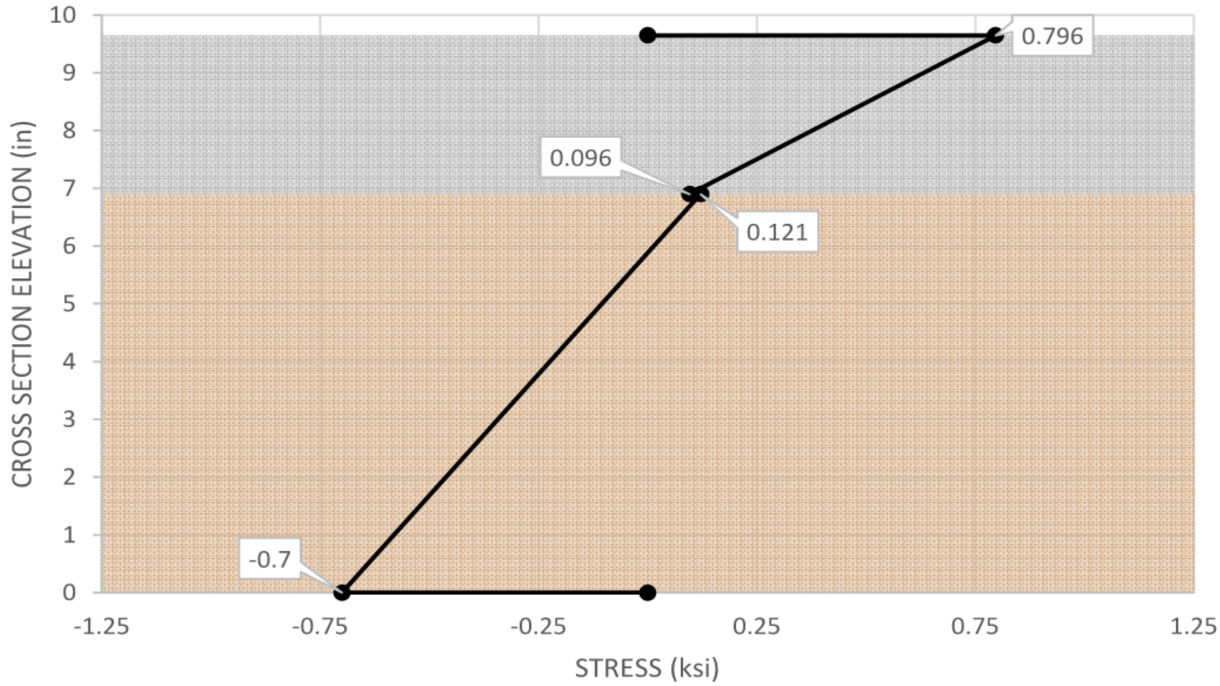
STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length $L =$ 22 ft Shear $V_u =$ 1.86 kip Moment $M_u =$ 122.98 kip-in	
	Concrete: Modulus of Elasticity $E_1 =$ 2191 ksi; Height $h_1 =$ 2.75 in; Centroid $a_1 =$ 2.79627 in; Gamma Factor $\gamma_1 =$ 0.71876 CLT: $E_2 =$ 1417.32 ksi; $h_2 =$ 6.90 in; $a_2 =$ 2.02873 in; $\gamma_2 =$ 1 $EI_{eff,comp} =$ 1117973 kip-in^2	
	Bending Stess Calulations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.719)(2191\ ksi)(2.80\ in)(123\ kip \cdot in)}{1117973\ kip \cdot in^2} =$ 0.484 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(2191\ ksi)(2.75\ in)(123\ kip \cdot in)}{1117973\ kip \cdot in^2} =$ 0.331 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.484 + 0.331 =$ 0.816 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.484 - 0.331 =$ 0.153 ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(1417.3\ ksi)(2.03 in)(123\ kip \cdot in)}{1117973\ kip \cdot in^2} =$ 0.316 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(1417\ ksi)(6.90\ in)(123\ kip \cdot in)}{1117973\ kip \cdot in^2} =$ 0.538 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} + \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.316 + 0.538 =$ 0.222 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.32 - 0.538 =$ -0.85 ksi	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	STRENGTH ANALYSIS	
Concrete	Normal Stress in Concrete σ_1 = 484.4 psi Allow Comp Strength of Conc F_c = 2000 psi $F_c = f_c'/2$ $\sigma_1 < F_c \rightarrow \sigma_1 = 484.4\, psi < F_c = 2000\, psi$ ∴ ACCEPTABLE	
Timber	Bending Strength F_b = 2100 psi Format Conversion Factor K_F = 2.54 Resistance Factor Φ = 0.85 Design Bending Stength $F_b' = \Phi F_b K_F = 0.85(2100\, psi)(2.54) =$ 4533.9 psi	
	Tension Strength F_t = 1575 psi Format Conversion Factor K_F = 2.70 Resistance Factor Φ = 0.80 Design Tension Stength $F_t' = \Phi F_t K_F = 0.80(1575\, psi)(2.70) =$ 3402.0 psi	
	Average CLT Stress σ_2 = 316.3 psi Extreme Fiber Stress $\sigma_{m,2}$ = 537.9 psi $\frac{\sigma_2}{F_t'} + \frac{\sigma_{m,2}}{F_b'} \leq 1 \rightarrow \frac{316.3\, psi}{3402\, psi} + \frac{537.9\, psi}{4534\, psi} =$ 0.212 < 1.0 ∴ ACCEPTABLE	
	Shear Strength F_v = 160 psi Format Conversion Factor K_F = 2.88 Resistance Factor Φ = 0.75 Design Shear Stength $F_v' = \phi F_v K_F = 0.75(160\, psi)(2.88) =$ 345.6 psi	
	CLT Modulus of Elasticity E_2 = 1417.32 ksi NA of timber h = 5.48 in Shear V = 1.86 kip $EI_{eff,comp}$ = 1117973 kip-in^2	
	$\tau_{2,max} = \frac{E_2 h_2{}^2 V_u}{2(EI)_{eff,comp}} = \frac{(1417\, ksi)(5.48 in)^2(1.86\, kip)}{2(1117973\, kip \cdot in^2)} =$ 35.45 psi	EN 1995.1-1 Annex B EQ 3.9
	$\tau_{2,max} < F_v' \rightarrow \tau_{2,max} = 35.45\, psi < F_v' = 345.6\, psi$ ∴ ACCEPTABLE	

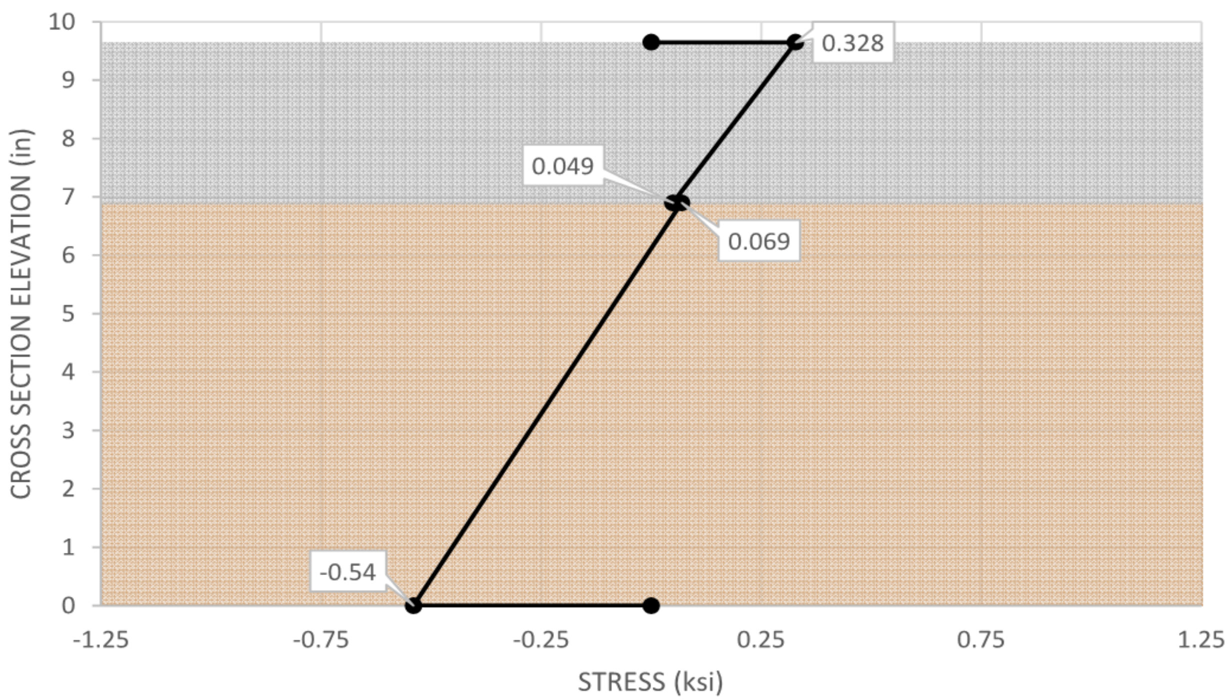
STEP DESCRIPTION	COMPUTATION	REFERNCE
Fastener Load Calculation	STRENGTH ANALYSIS F_{ULT} = 76.5828 kips Gamma Factor γ_1 = 0.71876 ksi Concrete Modulus of Elasticity E_1 = 2191.00 in Concrete Area A_1 = 33.00 in^2 Composite Centroid a_1 = 2.80 in Fastener Spacing s = 54 in Ultimate Shear Loading V = 1.8634 kip $EI_{eff,comp}$ = 1117973 kip-in^2 $F_q = \frac{\gamma_1 E_1 A_1 a_1 sV}{EI_{eff,comp}}$ $F_q = \frac{(0.719)(2191\ ksi)(33in^2)(2.80\ in)(54\ in)(1.86\ kip)}{1117973 kip \cdot in^2} =$ 13.08 kip $F_q < F_{ULT} \rightarrow F_q = 13.08\ kip < F_{ULT} = 76.6\ kip$ ∴ ACCEPTABLE	EN 1995.1-1 Annex B EQ 3.10

STEP DESCRIPTION	COMPUTATION	REFERNCE
	CONNECTOR EFFICIENCY	
Partial Composite Section	Effective Bending Stiffness EI_{eff_SLS} = 1611684 in	
	Deflection Δ_{PC} = 0.16352 in	
Non-Composite Section	Effective Bending Stiffness EI_{eff_SLS} = 551231 in (set K_{ser} close to zero)	
	Deflection Δ_{NC} = 0.47809 in	
Fully Composite	Concrete MOE E_1 = 3834.25 psi	
	Timber MOE E_2 = 1800 psi	
	Width b' = 12 in	
	Width of Transformed Concrete b' = 25.56 in $b' = b(E_1/E_2)$	
	Height of Timber h_1 = 2.75 in	
	Height of Concrete h_2 = 6.90 in	
	Bottom to Centroid of	
	Transformed Concrete y'_1 = 8.275 in	
	Timber Section y_2 = 3.45 in	
	Area of Transformed Concrete A'_1 = 70.29 in^2	
	Area of Timber Section A_2 = 82.8 in^2	
	Moment of Inertia of	
	Transformed Concrete I'_1 = 44.30 in^4	
	Timber Section I_2 = 328.51 in^4	
	Centroid of Transformed Section	
	$y_c = \frac{y_1' A_1' + y_2 A_2}{A_1' + A_2}$ = 5.665 in	
	Distance to Concrete Centroid d_1 = 4.29 in $d = \left\lvert y_c - \frac{h}{2} \right\rvert$	
	Distance to Timber Centroid d_2 = 0.84 in	
	Transformed Section	
	Moment of Inertia about NA I_{NA} = 1725.27 in^4 $I_{NA} = \sum I_i + A_i {d_i}^2$	
	Bending Stiffness $E_2 I_{NA}$ = 3105482 k-in^2	
	Deflection of Fully Composite Section	
	$\Delta_{FC} = \frac{5wL^4}{384 E_2 I_{NA}} = \frac{5(50\,plf)(22ft)^4}{384(3105482\,k \cdot in^2)} =$ 0.085 in	
Shear Connector Efficiency	$Efficiency = \frac{\Delta_{NC} - \Delta_{PC}}{\Delta_{NC} - \Delta_{FC}} = \frac{0.4781'' - 0.1635''}{0.4781'' - 0.085''} =$ 80.0%	

90 mm HBV 24 feet SPAN

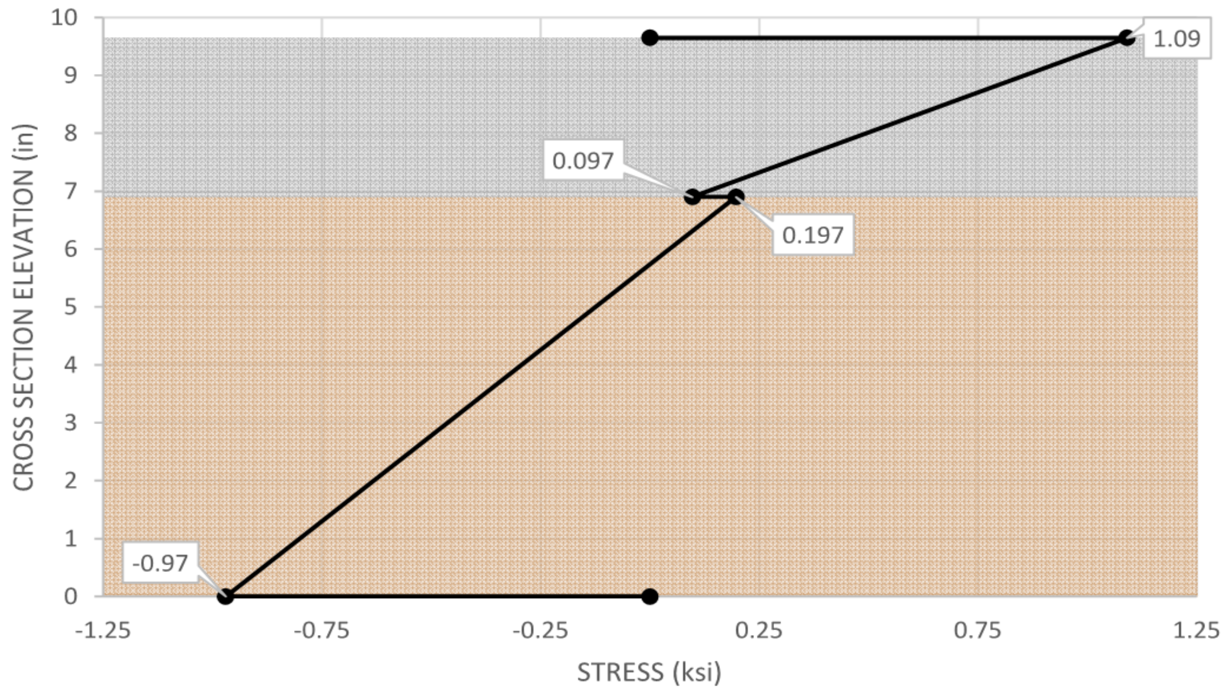


SERVICABILITY LIMIT STATE CROSS-SECTIONAL STRESS DISTRIBUTION

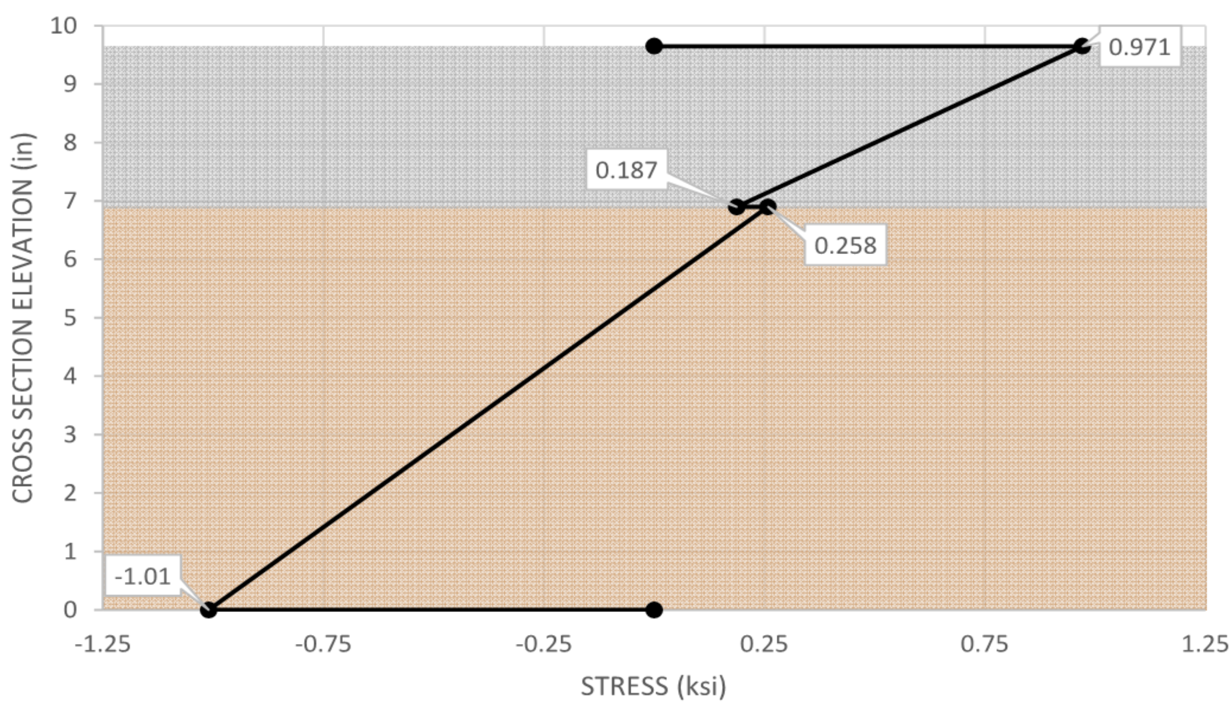


SERVICABILITY LIMIT STATE LONG-TERM CROSS-SECTIONAL STRESS DISTRIBUTION

90 mm HBV 24 feet SPAN

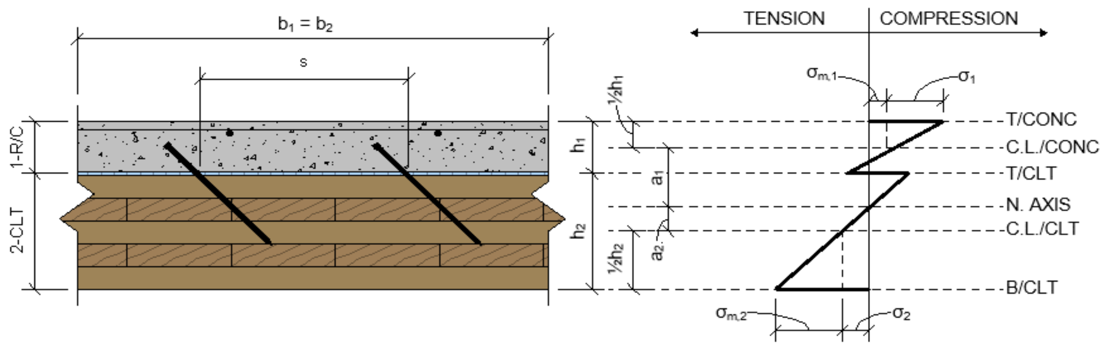


ULTIMATE LIMIT STATE CROSS-SECTIONAL STRESS DISTRIBUTION



ULTIMATE LIMIT STATE LONG-TERM CROSS-SECTIONAL STRESS DISTRIBUTION

STEP DESCRIPTION	COMPUTATION	REFERNCE



Cross-section Stress Diagram

Dead Load Calculation

		Density		Height		
Mass Timber	ρ_t =	35 pcf	h_t =	6.90 in	L =	24 ft
Concrete	ρ_c =	150 pcf	h_c =	2.75 in		

$$w_{DL} = \rho_t h_t + \rho_c h_c$$
$$w_{DL} = (35\ pcf)(6.90\ in) + (150\ pcf)(2.75\ in) = \ 54.50\ \text{psf}$$

Loading Criteria

	Loading		Sustained/Expected		Reference
Dead Load	w_{DL} =	54.50 psf	m_{DL} =	100%	ASCE 7-16
Superimposed Dead Load	w_{SDL} =	20 psf	m_{SDL} =	78.2%	
Live Load	w_{LL} =	50 psf	m_{LL} =	21.8% (10.9 psf mean load)	Table C4.3-2

LRFD / Service

Loading Cases

Load Case Factors				Equations	DL	SDL	LL	w_u
Combo	DL	SDL	LL		PSF	PSF	PSF	PLF
G01	1.4	1.4	0.0	$1.4(w_{DL} + w_{SDL})$	76.3	28	0	104.3
G02	1.2	1.2	1.6	$1.2(w_{DL} + w_{SDL}) + 1.6w_{LL}$	65.4	24	80	169.4
S01	1.0	1.0	1.0	$1.0w_{DL} + 1.0w_{SDL} + 1.0w_{LL}$	54.5	20	50	124.5
S02	0.0	1.0	1.0	$1.0w_{SDL} + 1.0w_{LL}$	0	20	50	70
S03	1.0	0.782	0.218	$1.0w_{DL} + 0.782w_{SDL} + 0.218w_{LL}$	54.5	15.64	10.9	81.04

Combo	w_u	V_u	M_u
	PLF	kips	kip-in
G01	104.3	1.252	90.12
G02	169.4	2.033	146.4
S01	124.5	1.494	107.6
S02	70	0.84	60.48
S03	81.04	0.972	70.02

$$V_u = \frac{w_u L}{2}$$
$$M_u = \frac{w_u L^2}{8}$$

STEP DESCRIPTION	COMPUTATION	REFERNCE
CLT Properties	CLT CALCULATIONS	Structurlam Crosslam

CLT CALCULATIONS

Type: 5-Ply CLT, Grade E1M4, 139 E

h_i = 1.375 in
b = 12 in
h_t = 6.90 in
a = 5.52 in
L = 24.0 ft
K_s = 11.5

Reference: Structurlam Crosslam

Major Strength Axis

$F_{b,0}$ = 2100 psi
E_0 = 1800 ksi
$F_{t,0}$ = 1575 psi
$F_{c,0}$ = 1875 psi
$F_{v,0}$ = 160 psi
$F_{s,0}$ = 50 psi

Minor Strength Axis

$F_{b,90}$ = 875 psi
E_{90} = 1400 ksi
$F_{v,90}$ = 135 psi
$F_{s,90}$ = 45 psi

CLT Calculations

Layer	E (ksi)	h (in)	z (in)	GA_{eff}: G (ksi)	GA_{eff}: h/G/b (in^2/kip)	EA_{eff}: EA (kip)	EI_{eff}: $Ebh^3/12$ (kip-in^2)	EI_{eff}: EAz^2 (kip-in^2)	EI_{eff}: Sum of Layers (kip-in^2)
1	1800	1.380	2.760	112.5	0.00102	29808	4730.53	227065	231796
2	46.6667	1.380	1.375	8.75	0.01314	772.8	122.643	1461.08	1583.72
3	1800	1.380	0.000	112.5	0.00102	29808	4730.53	0	4730.53
4	46.6667	1.380	1.375	8.75	0.01314	772.8	122.643	1461.08	1583.72
5	1800	1.380	2.760	112.5	0.00102	29808	4730.53	227065	231796
						90969.6			471490

Reference: CLT Handbook, Chapter 3

$$EA_{eff} = \sum_{i=1}^{n} E_i b_i h_i$$

Reference: CLT Handbook Chapter 3 EQ 24

$$EI_{eff} = \sum_{i=1}^{n} E_i b_i \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i A_i z_i^2$$

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

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

Cross Sectional Properties (Per foot of width)

SLS/ULS Short-Term

EA_{eff} = 90969.6 kip
EI_{eff} = 471490 kip-in^2
GA_{eff} = 1075.5 kip
EI_{app} = 444475.0 kip

SLS Long-Term

EA_{eff} = 47878.7 kip
EI_{eff} = 248153 kip-in^2
GA_{eff} = 566.1 kip
EI_{app} = 235933.9 kip

ULS Long-Term

EA_{eff} = 71629.6 kip
EI_{eff} = 371252 kip-in^2
GA_{eff} = 846.9 kip
EI_{app} = 352972.1 kip

STEP DESCRIPTION	COMPUTATION	REFERNCE
	CONNECTOR CALCUATION	
	Length of Connector Strip L = 1000 mm = 39.37 in	
	Thickness of the interlayer d_{zs} = 8.89 mm = 0.35 in	DIBt Z-9.1-557
	1 mm base unit d_0 = 1 mm	
Characteristic Load-Carrying Capacity	$T_k = 160 - 8.0(d_{zs}/d_0)^{0.5}$ $T_k = 160 - 8.0(8.89/1)^{0.5} =$ 136.1 kN	
Acceptable Shear Load	$zul.T = 90 - 4.5(d_{zs}/d_0)^{0.5}$ $zul.T = 90 - 4.5(8.89/1)^{0.5} =$ 76.58 kN	
Rated value of Load Bearing Capacity	$T_d = T_k/1.25 \approx 1.42\, zul.T$ $T_d = (136.1\ kN/mm)/1.25 =$ 108.9 kN	
Slip Modulus	$K_{ser} = 825 - 250(d_{zs}/d_0)^{0.2}$ $K_{ser} = 825 - 250(8.89/1)^{0.2} =$ 438 kN/mm	
Slip Modulus Ultimate	$K_u = (2/3)K_{ser}$ $K_u = (2/3)(438.0\ kN/mm) =$ 292 kN/mm Converstions $T_k =$ 30.6071 kips $K_{ser} =$ 2501 k/in $K_u =$ 1667.33 k/in	
	Spacing between connectors in rows at the ends s_e = 1524 mm = 60 in in rows in the middle s_m = 1524 mm = 60 in Effective Spacing between s_{eff} = 1524 mm = 60 in Number of rows of connectors n_r = 1 Deformation factor for long term loading for concrete k_{def_c} = 2.5 loading for timber k_{def_t} = 0.9 loading for STS $k_{defHBV\_}$ = 0.6 Stiffness reduction for ULS ψ_2 = 0.3	DIBt Z-9.1-557 Table A1.1 EN 1990
Connector Stiffness Adjustment for Long Term Loading	$K_{SLS\_LT} = \frac{K_{SER}}{1 + k_{def\_HBV}} = \frac{438.0\ kN/mm}{1 + 0.6} =$ 273.745 kN/mm $K_{ULS\_LT} = \frac{K_u}{1 + \psi_2 k_{def\_sts}} = \frac{292\, kN/mm}{1 + (0.3 * 0.6)} =$ 247.453 kN/mm	

Serviceability Limit State
Short-Term Loading

STEP DESCRIPTION	COMPUTATION	REFERNCE
General Values	EFFECTIVE STIFFNESS Gamma Span Length $L = L_0 =$ 24 ft Connector slip Modulus $K_{ser} =$ 437.992 kN/mm = 2501.00 kips/in Connector Spacing $s_{eff} =$ 60 in	Annex B of EN 1995-1-1
Concrete Properties	Slab Depth $h_1 =$ 2.75 in Compressive Strength $f_c' =$ 4000 psi Weight of Concrete $w_c =$ 150 pcf	
CLT Properties	Clt Height $h_2 =$ 6.90 in $EA_{eff,CLT} =$ 90969.6 kip $EI_{eff,CLT} =$ 471490 kip-in^2	CLT Handbook EQ 24 & 25
Modulus of Elasticity	$E_{1\_SLS} = {w_c}^{1.5} 33\sqrt{f_c'}$ $E_{1\_SLS} = (150\ pcf)^{1.5} 33\sqrt{4000\ psi} =$ 3834.25 ksi	ACI 318, 19.2.2.1.a
Area of Concrete	$A_{c\_SLS} = h_c b_c$ $A_{c\_SLS} = (2.75\ in)(12\ in) =$ 33 in^2	EN 1995.1-1 Annex B EQ 3.2
Inertia of Concrete	$I_{1\_SLS} = b_1 {h_1}^2/12$ $I_{1\_SLS} = (12\ in)(2.75\ in)^2/12 =$ 20.8 in^4	EN 1995.1-1 Annex B EQ 3.3
	$E_c A_{c\_SLS} = (3834.3\ ksi)(33\ in^2) =$ 126530 kip $E_c I_{c\_SLS} = (38343\ ksi)(20.8\ in^4) =$ 79740.5 kip-in^2	
Gamma Factor	$\gamma_{1\_SLS} = \left[1 + \frac{\pi^2 E_1 A_1 s_1}{K_1 {L_0}^2}\right]^{-1}$ $\gamma_{1\_SLS} = \left[1 + \frac{\pi^2 (3834.3\ ksi)(33\ in^2)(60\ in)}{(2501\ \ kip/in)(24\ ft)^2}\right]^{-1} =$ 0.73465	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2\_SLS} = \frac{\gamma_1 E_1 A_1 (h_1 + h_2)}{2\gamma_1 E_1 A_1 + 2\gamma_2 EA_{eff}}$ $a_{2\_SLS} = \frac{(0.7346)(3834\ ksi)(33\ in^2)(2.75\ in + 6.90\ in)}{2(0.7346)(3834\ ksi)(33\ in^2) + 2(1.0)(90970\ kip)} =$ 2.43854 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1\_SLS} = \frac{h_1 + h_2}{2} - a_{2\_SLS}$ $a_{1\_SLS} = \frac{6.90\ in + 2.75\ in}{2} - 2.44 =$ 2.38646 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_SLS} = (E_1 I_1 + \gamma_1 E_1 A_1 {a_1}^2) + \left(EI_{eff} + \gamma_2 EA_{eff} {a_2}^2\right)$ $(EI)_{eff,comp\_SLS} =$ 609136.1 kip-in^2 + 1012440 kip-in^2 = 1621576 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Composite & CLT Effective	$Ratio_{CLT} = \frac{EI_{eff,comp\_SLS}}{EI_{eff,CLT}}$ $Ratio_{CLT} = \frac{1621576\ kip \cdot in^2}{471489.9\ kip \cdot in^2} =$ 3.439	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	APPARENT STIFFNESS	
Bare CLT Stiffness	Length L = 24 ft K Value K_s = 11.5 $EI_{eff,clt}$ = 471490 kip-in^2 $GA_{eff,clt}$ = 1075.5 kips	
	$EI_{app,CLT\_SLS} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{471489.9\ kip \cdot in^2}{1+\frac{(11.5)(471489.9\ kip \cdot in^2)}{(1075.5\ kip)(24\ ft)^2}} =$ 444475 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{444475\ kip \cdot in^2}{471489.9\ kip \cdot in^2} =$ 94.3%	
Composite Section Stiffness	L = 24 ft K_s = 11.5 $EI_{eff,comp}$ = 1621576 kip-in^2 $GA_{eff,clt}$ = 1075.5 kips	
	$EI_{app,comp\_SLS} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{1621576\ kip \cdot in^2}{1+\frac{(11.5)(1621576\ kip \cdot in^2)}{(1075.5\ kip)(24\ ft)^2}} =$ 1341214 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{1341214\ kip \cdot in^2}{1621576\ kip \cdot in^2} =$ 82.7%	
	$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{1341214\ kip \cdot in^2}{444475\ kip \cdot in^2} =$ 3.018	
Apparent Floor Stiffness	L = 24 ft $EI_{app,CLT}$ = 444475 kip-in^2 $EI_{app,comp}$ = 1341214 kip-in^2	
	$K_{app,CLT\_SLS} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$	
	$K_{app,CLT\_SLS} = \frac{48(444475.0\ kip \cdot in^2)}{(24\ ft)^3} =$ 0.89312 kip/in	
	$K_{app,comp\_SLS} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$	
	$K_{app,comp\_SLS} = \frac{48(1341214\ kip \cdot in^2)}{(24\ ft)^3} =$ 2.69502 kip/in	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length $L = 24$ ft Shear $V_u = 1.49$ kip Moment $M_u = 107.57$ kip-in	
	Concrete / CLT Modulus of Elasticity $E_1 = 3834.25$ ksi; $E_2 = 1800$ ksi Height $h_1 = 2.75$ in; $h_2 = 6.90$ in Centroid $a_1 = 2.38646$ in; $a_2 = 2.43854$ in Gamma Factor $\gamma_1 = 0.73465$; $\gamma_2 = 1$ $EI_{eff,comp\_SLS} = 1621576$ kip-in^2	
	Bending Stress Calculations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.735)(3834\ ksi)(2.39\ in)(108\ kip \cdot in)}{1621576\ kip \cdot in^2} =$ 0.446 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(3834\ ksi)(2.75\ in)(108\ kip \cdot in)}{1621576\ kip \cdot in^2} =$ 0.350 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.446 + 0.350 =$ 0.796 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.446 - 0.350 =$ 0.096 ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(1800\ ksi)(2.439\ in)(108\ kip \cdot in)}{1621576\ kip \cdot in^2} =$ 0.291 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(1800\ ksi)(6.90\ in)(108\ kip \cdot in)}{1621576\ kip \cdot in^2} =$ 0.412 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} + \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.291 + 0.412 =$ 0.121 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.29 - 0.412 =$ -0.70 ksi	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	DEFLECTION CALCUATIONS	
	Short-Term / Long-Term	
Loading	Dead Load w_{DL} = 54.50 plf; Long-Term: w_{DL} = 54.50 plf	
	Superimposed Dead Load w_{SDL} = 20.00 plf; Long-Term: w_{SDL} = 15.64 plf	
	Live Load w_{LL} = 50.00 plf; Long-Term: w_{LL} = 10.9 plf	
	Span Length L = 24 ft; Long-Term: L = 24 ft	
	Stiffness $EI_{eff,comp\_SLS}$ = 1621576 kip-in^2	
	Short-Term Deflections	
Dead Load	$\Delta_{DL} = \frac{5w_{DL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(54.5\,plf)(24\,ft)^4}{384(1621575.6\,kip \cdot in^2)} =$ 0.2509 in	
Superimposed Dead Load	$\Delta_{SDL} = \frac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(20\,plf)(24\,ft)^4}{384(1621575.6\,kip \cdot in^2)} =$ 0.0921 in	
Live Load	$\Delta_{LL} = \frac{5w_{LL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(50\,plf)(24\,ft)^4}{384(1621575.6\,kip - in^2)} =$ 0.2302 in	
	Serviceability Check	
Allowable LL Deflection	$\Delta_{allow,LL} = \frac{L}{360} = \frac{(24ft)(12\,in)}{360(1\,ft)} =$ 0.80 in	IBC Table 1604.3
Check	$\Delta_{LL} \leq \Delta_{allow,LL} \rightarrow \Delta_{LL} = 0.23\,in < \Delta_{allow,LL} = 0.80\,in$ ∴ ACCEPTABLE	
	Long-Term Deflections	
Dead Load	$\Delta_{DL\_LT} = \frac{5w_{DL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(54.5\,plf)(24\,ft)^4}{384(1621576\,kip \cdot in^2)} =$ 0.25089 in	
Superimposed Dead Load	$\Delta_{SDL\_LT} = \frac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(15.6\,plf)(24\,ft)^4}{384(1621576\,kip \cdot in^2)} =$ 0.07200 in	
Live Load	$\Delta_{LL\_LT} = \frac{5w_{LL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(10.9\,plf)(24\,ft)^4}{384(1621576\,kip \cdot in^2)} =$ 0.05018 in	
	Serviceability Check	
	Time Dependent Creep Factor K_{cr} = 2.0	NDS 2018 Section 3.5.1
	Deflection due to short-term Δ_{ST} = 0.2302 in	
	Deflection due to long-term Δ_{LT} = 0.32289 in $\Delta_{LT} = \Delta_{DL\_LT} + \Delta_{SDL\_LT}$	
	$\Delta_{TL} = K_{cr}\Delta_{LT} + \Delta_{ST}$	NDS EQ 3.5-1
	$\Delta_{TL} =$ 2.0(0.3223 in)+(0.2302 in) = 0.876 in	
	$\Delta_{allow,TL} = \frac{L}{240} = \frac{(24ft)(12\,in)}{240(1\,ft)} =$ 1.20 in	IBC Table 1604.3
	$\Delta_{TL} \leq \Delta_{allow,TL} \rightarrow \Delta_{TL} = 0.876\,in < \Delta_{allow,TL} = 1.20\,in$ ∴ ACCEPTABLE	

<table>
<tr><th>STEP DESCRIPTION</th><th>COMPUTATION</th><th>REFERNCE</th></tr>
<tr><td></td><td>EFFECTIVE STIFFNESS
Deformation factor for long term
loading for concrete k_{def_c} = 2.5
loading for timber k_{def_t} = 0.9
loading for HBV k_{def_HBV} = 0.6
Stiffness reduction for ULS ψ_2 = 0.3
Effective Spacing between s_{eff} = 1524 mm = 60 in
Gamma Span Length L_0 = 24 ft
Connector slip Modulus K_{SLS_LT} = 273.745 kN/mm = 1563.12 kips/in</td><td>EN 1995.1-1

ETA-13/0029
Table 2.1</td></tr>
<tr><td></td><td>Concrete: Modulus of Elasticity E_1 = 3834.25 ksi; Area A_1 = 33 in^2; Momemnt of Inertia I_1 = 20.7969 in^4; Height h_1 = 2.75 in
CLT: E_2 = 1800 ksi; A_1 = 26.60 in^2; I_1 = 137.863 in^4; h_2 = 6.90 in
EA_{eff,CLT_SLS_LT} = 47878.74 kip
EI_{eff,CLT_SLS_LT} = 248153 kip-in^2</td><td>CLT Handbook
EQ 24 & 25</td></tr>
<tr><td>Modulus of Adjustment for long term loading</td><td>$E_{1\_SLS\_LT} = \frac{E_1}{1+k_{def\_c}} =$ 1095.5 ksi $E_{2\_SLS\_LT} = \frac{E_2}{1+k_{def\_t}} =$ 947.368 ksi</td><td>EN 1995.1-1</td></tr>
<tr><td>Gamma Factor</td><td>$\gamma_{1,SLS\_LT} = \left[1+\frac{\pi^2 E_{1\_SLS\_LT} A_1 s_{eff}}{K_{ser\_LT} {L_0}^2}\right]^{-1}$
$\gamma_{1,SLS\_LT} = \left[1+\frac{\pi^2 (1095\,ksi)(33\,in^2)(60\,in)}{(1563\,kip/in)(24\,ft)^2}\right]^{-1} =$ 0.858</td><td>EN 1995.1-1
Annex B EQ 3.5</td></tr>
<tr><td>Timber to Composite Centroid</td><td>$a_{2,SLS\_LT} = \frac{\gamma_{1\_SER} E_1 A_1 (h_1+h_2)}{2\gamma_{1\_SER} E_1 A_1 + 2\gamma_2 EA_{eff}}$
$a_{2,SLS\_LT} = \frac{(0.858)(1095\,ksi)(33\,in^2)(2.75\,in+6.90\,in)}{2(0.858)(1095\,ksi)(33\,in^2)+2(1.0)(47878.7\,kip)} =$ 2.66259 in</td><td>EN 1995.1-1
Annex B EQ 3.6</td></tr>
<tr><td>Concrete to Composite Centroid</td><td>$a_{1,SLS\_LT} = \frac{h_1+h_2}{2} - a_{2,SLS\_LT}$
$a_{1,SLS\_LT} = \frac{6.90\,in+2.75\,in}{2} - 2.663 =$ 2.162 in</td><td></td></tr>
<tr><td>Effective Comp Stiffness</td><td>$(EI)_{eff,comp\_SLS\_LT} = \left(E_1 I_1 + \gamma_{1\_SER} E_1 A_1 {a_{1\_SER}}^2\right) + \left(EI_{eff} + \gamma_2 EA_{eff} {a_{2\_SER}}^2\right)$
$(EI)_{eff,comp\_SLS\_LT} =$ 167871 kip-in^2 + 587584 kip-in^2
$(EI)_{eff,comp\_SLT\_LT} =$ 755454.8 kip-in^2</td><td>EN 1995.1-1
Annex B EQ 3.1</td></tr>
<tr><td>Ratio of Compsite & CLT Effective</td><td>$Ratio_{CLT\_SER} = \frac{EI_{eff,comp\_SER}}{EI_{eff,CLT}}$
$Ratio_{CLT\_SER} = \frac{755454.8\,kip \cdot in^2}{248152.6\,kip \cdot in^2} =$ 15.78</td><td></td></tr>
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<tr><th>STEP DESCRIPTION</th><th>COMPUTATION</th><th>REFERNCE</th></tr>
<tr><td></td><td>APPARENT STIFFNESS</td><td></td></tr>
<tr><td>Bare CLT Stiffness</td><td>Length L = 24 ft
K Value K_s = 11.5
$EI_{eff,clt}$ = 248153 kip-in^2
$GA_{eff,clt}$ = 566.1 kips

$EI_{app,CLT\_SLS\_LT} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{248152.6\ kip\cdot in^2}{1+\frac{(11.5)(248152.6\ kip\cdot in^2)}{(566.1\ kip)(24\ ft)^2}} =$ 233934 kip-in^2

$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{233934.2\ kip\cdot in^2}{248152.6\ kip\cdot in^2} =$ 94.3%</td><td>CLT Handbook EQ 35</td></tr>
<tr><td>Composite Section Stiffness</td><td>L = 24 ft
K_s = 11.5
$EI_{eff,comp}$ = 755455 kip-in^2
$GA_{eff,clt}$ = 566.1 kips

$EI_{app,comp\_SLS\_LT} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{755454.8\ kip\cdot in^2}{1+\frac{(11.5)(755455\ kip\cdot in^2)}{(566.1\ kip)(24\ ft)^2}} =$ 637498 kip-in^2

$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{637497.6\ kip\cdot in^2}{755454.8\ kip\cdot in^2} =$ 84.4%

$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{637497.6\ kip\cdot in^2}{233934.2\ kip\cdot in^2} =$ 2.725</td><td>CLT Handbook EQ 35</td></tr>
<tr><td>Apparent Floor Stiffness</td><td>L = 24 ft
$EI_{app,CLT}$ = 233934 kip-in^2
$EI_{app,comp}$ = 637498 kip-in^2

$K_{app,CLT\_SLS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$

$K_{app,CLT\_SLS\_LT} = \frac{48(233934.2\ kip\cdot in^2)}{(24\ ft)^3} =$ 0.47006 kip/in

$K_{app,comp\_SLS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$

$K_{app,comp\_SLS\_LT} = \frac{48(637497.6\ kip\cdot in^2)}{(24\ ft)^3} =$ 1.28098 kip/in</td><td></td></tr>
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STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length L = 24 ft Shear V_u = 0.97 kip Moment M_u = 70.02 kip-in	
	Concrete: Modulus of Elasticity E_1 = 1095.5 ksi; Height h_1 = 2.75 in; Centroid a_1 = 2.16241 in; Gamma Factor γ_1 = 0.85828 CLT: E_2 = 947.368 ksi; h_2 = 6.90 in; a_2 = 2.66259 in; γ_2 = 1 $EI_{eff,comp\_SLS\_LT}$ = 755455 kip-in^2	
	Bending Stess Calulations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.858)(1096\ ksi)(2.16\ in)(70.0\ kip \cdot in)}{755454.8\ kip \cdot in^2} =$ 0.188 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(1096\ ksi)(2.75\ in)(70.0\ kip \cdot in)}{755454.8\ kip \cdot in^2} =$ 0.140 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.188 + 0.140 =$ 0.328 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.188 - 0.140 =$ 0.049 ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(947.4\ ksi)(2.66\ in)(70.0\ kip \cdot in)}{755454.8\ kip \cdot in^2} =$ 0.234 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(947.4\ ksi)(6.90\ in)(70.0\ kip \cdot in)}{755454.8\ kip \cdot in^2} =$ 0.303 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.234 + 0.303 =$ 0.069 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.23 - 0.303 =$ -0.54 ksi	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	DEFLECTION CALCUATIONS	
Loading	Short-Term: Dead Load w_{DL} = 54.50 plf; Superimposed Dead Load w_{SDL} = 20.00 plf; Live Load w_{LL} = 50.00 plf; Span Length L = 24 ft; Stiffness $EI_{eff,comp\_SLS}$ = 1621576 kip-in^2 Long-Term: w_{DL} = 54.50 plf; w_{SDL} = 15.64 plf; w_{LL} = 10.9 plf; L = 24 ft; $EI_{eff,comp\_SLS\_LT}$ = 755454.8 kip-in^2	
	Short-Term Deflections	
Dead Load	$\Delta_{DL} = \frac{5w_{DL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(54.5\,plf)(24\,ft)^4}{384(1621574.8\,kip\cdot in^2)} =$ 0.2509 in	
Superimposed Dead Load	$\Delta_{SDL} = \frac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(20\,plf)(24\,ft)^4}{384(1621574.8\,kip\cdot in^2)} =$ 0.0921 in	
Live Load	$\Delta_{LL} = \frac{5w_{LL}L^4}{384EI_{eff,comp\_SLS}} = \frac{5(50\,plf)(24\,ft)^4}{384(1621574.8\,kip - in^2)} =$ 0.2302 in	
	Servicablility Check	
Allowable LL Deflection	$\Delta_{allow,LL} = \frac{L}{360} = \frac{(24ft)(12\,in)}{360(1\,ft)} =$ 0.80 in	IBC Table 1604.3
Check	$\Delta_{LL} \leq \Delta_{allow,LL} \rightarrow \Delta_{LL} = 0.230\,in < \Delta_{allow,LL} = 0.80\,in$ ∴ ACCEPTABLE	
	Long-Term Deflections	
Dead Load	$\Delta_{DL\_LT} = \frac{5w_{DL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(54.5\,plf)(24\,ft)^4}{384(755454.8\,kip\cdot in^2)} =$ 0.53854 in	
Superimposed Dead Load	$\Delta_{SDL\_LT} = \frac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(15.6\,plf)(24\,ft)^4}{384(755454.8\,kip\cdot in^2)} =$ 0.15455 in	
Live Load	$\Delta_{LL\_LT} = \frac{5w_{LL}L^4}{384EI_{eff,comp\_SLS\_LT}} = \frac{5(10.9\,plf)(24\,ft)^4}{384(755454.8\,kip\cdot in^2)} =$ 0.10771 in	
	Servicablility Check Deflection due to short-term Δ_{ST} = 0.2302 in Deflection due to long-term Δ_{LT} = 0.69308 in $\quad \Delta_{LT} = \Delta_{DL\_LT} + \Delta_{SDL\_LT}$ Deflection due to Total Load Δ_{TL} = 0.9233 in $\quad \Delta_{TL} = \Delta_{LT} + \Delta_{ST}$	
	$\Delta_{allow,TL} = \frac{L}{240} = \frac{(24ft)(12\,in)}{240(1\,ft)} =$ 1.20 in	IBC Table 1604.3
	$\Delta_{TL} \leq \Delta_{allow,TL} \rightarrow \Delta_{TL} = 0.923\,in < \Delta_{allow,TL} = 1.20\,in$ ∴ ACCEPTABLE	

STEP DESCRIPTION	COMPUTATION	REFERNCE
General Values	EFFECTIVE STIFFNESS Gamma Span Length $L = L_0 =$ 24 ft Connector slip Modulus $K_U =$ 291.994 kN/mm = 1667.33 kips/in Connector Spacing $s_{eff} =$ 60 in	Annex B of EN 1995-1-1
Concrete Properties	Slab Depth $h_1 =$ 2.75 in Compressive Strength $f_c' =$ 4000 psi Weight of Concete $w_c =$ 150 pcf	
CLT Properties	Clt Height $h_2 =$ 6.90 in $EA_{eff,CLT} =$ 90969.6 kip $EI_{eff,CLT} =$ 471490 kip-in^2	CLT Handbook EQ 24 & 25
Modulus of Elasticity	$E_{1\_ULS} = w_c{}^{1.5} 33\sqrt{f_c'}$ $E_{1\_ULS} = (150\ pcf)^{1.5} 33\sqrt{4000\ psi} =$ 3834.25 ksi	ACI 318, 19.2.2.1.a
Area of Concrete	$A_{c\_ULS} = h_c b_c$ $A_{c\_ULS} = (2.75\ in)(12\ in) =$ 33 in^2	EN 1995.1-1 Annex B EQ 3.2
Inhertia of Concrete	$I_{1\_ULS} = b_1 h_1{}^2/12$ $I_{1\_ULS} = (12\ in)(2.75\ in)^2/12 =$ 20.8 in^4 $E_c A_{c\_ULS} = (3834.3\ ksi)(33\ in^2) =$ 126530 kip $E_c I_{c\_ULS} = (38343\ ksi)(20.8\ in^4) =$ 79740.5 kip-in^2	EN 1995.1-1 Annex B EQ 3.3
Gamma Factor	$\gamma_{1\_ULS} = \left[1 + \frac{\pi^2 E_1 A_1 s_1}{K_1 L_0{}^2}\right]^{-1}$ $\gamma_{1\_ULS} = \left[1 + \frac{\pi^2 (3834.3\ ksi)(33\ in^2)(60\ in)}{(1667\ \ kip/in)(24\ ft)^2}\right]^{-1} =$ 0.64859	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2\_ULS} = \frac{\gamma_1 E_1 A_1 (h_1 + h_2)}{2\gamma_1 E_1 A_1 + 2\gamma_2 EA_{eff}}$ $a_{2\_ULS} = \frac{(0.6486)(3834\ ksi)(33\ in^2)(2.75\ in + 6.90\ in)}{2(0.6486)(3834\ ksi)(33\ in^2) + 2(1.0)(90970\ kip)} =$ 2.28837 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1\_ULS} = \frac{h_1 + h_2}{2} - a_{2\_ULS}$ $a_{1\_ULS} = \frac{6.90\ in + 2.75\ in}{2} - 2.29 =$ 2.53663 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_ULS} = (E_1 I_1 + \gamma_1 E_1 A_1 a_1{}^2) + \left(EI_{eff} + \gamma_2 EA_{eff} a_2{}^2\right)$ $(EI)_{eff,comp\_ULS} =$ 607796.2 kip-in^2 + 947866 kip-in^2 = 1555662 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Compsite & CLT Effective	$Ratio_{CLT} = \frac{EI_{eff,comp\_ULS}}{EI_{eff,CLT\_ULS}}$ $Ratio_{CLT} = \frac{1555662\ kip \cdot in^2}{471489.9\ kip \cdot in^2} =$ 3.299	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	APPARENT STIFFNESS	
Bare CLT Stiffness	Length L = 24 ft K Value K_s = 11.5 $EI_{eff,clt}$ = 471490 kip-in^2 $GA_{eff,clt}$ = 1075.5 kips	
	$EI_{app,CLT\_ULS} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{471489.9\ kip\cdot in^2}{1+\frac{(11.5)(471489.9\ kip\cdot in^2)}{(1075.5\ kip)(24\ ft)^2}} =$ 444475 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{444475\ kip\cdot in^2}{471489.9\ kip\cdot in^2} =$ 94.3%	
Composite Section Stiffness	L = 24 ft K_s = 11.5 $EI_{eff,comp}$ = 1555662 kip-in^2 $GA_{eff,clt}$ = 1075.5 kips	
	$EI_{app,comp\_ULS} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{1555662\ kip\cdot in^2}{1+\frac{(11.5)(1555662\ kip\cdot in^2)}{(1075.5\ kip)(24\ ft)^2}} =$ 1295803 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{1295803\ kip\cdot in^2}{1555662\ kip\cdot in^2} =$ 83.3%	
	$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{1295803\ kip\cdot in^2}{444475\ kip\cdot in^2} =$ 2.915	
Apparent Floor Stiffness	L = 24 ft $EI_{app,CLT}$ = 444475 kip-in^2 $EI_{app,comp}$ = 1295803 kip-in^2	
	$K_{app,CLT\_ULS} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$	
	$K_{app,CLT\_ULS} = \frac{48(444475.0\ kip\cdot in^2)}{(24\ ft)^3} =$ 0.89312 kip/in	
	$K_{app,comp\_ULS} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$	
	$K_{app,comp\_ULS} = \frac{48(1295803\ kip\cdot in^2)}{(24\ ft)^3} =$ 2.60377 kip/in	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length $L = 24$ ft Shear $V_u = 2.03$ kip Moment $M_u = 146.36$ kip-in	
	Concrete: Modulus of Elasticity $E_1 = 3834.25$ ksi; Height $h_1 = 2.75$ in; Centroid $a_1 = 2.53663$ in; Gamma Factor $\gamma_1 = 0.64859$ CLT: $E_2 = 1800$ ksi; $h_2 = 6.90$ in; $a_2 = 2.28837$ in; $\gamma_2 = 1$ $EI_{eff,comp} = 1555662$ kip-in^2	
	Bending Stress Calculations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.649)(3834\ ksi)(2.54\ in)(146\ kip \cdot in)}{1555662\ kip \cdot in^2} =$ 0.594 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(3834\ ksi)(2.75\ in)(146\ kip \cdot in)}{1555662\ kip \cdot in^2} =$ 0.496 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.594 + 0.496 =$ 1.090 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{+\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.594 - 0.496 =$ 0.097 ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(1800\ ksi)(2.29\ in)(146\ kip \cdot in)}{1555662\ kip \cdot in^2} =$ 0.388 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(1800\ ksi)(6.90\ in)(146\ kip \cdot in)}{1555662\ kip \cdot in^2} =$ 0.584 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} + \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.388 + 0.584 =$ 0.197 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.39 - 0.584 =$ -0.97 ksi	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	STRENGTH ANALYSIS	
Concrete	Normal Stress in Concrete σ_1 = 593.5 psi Allow Comp Strength of Conc F_c = 2000 psi $F_c = f_c'/2$ $\sigma_1 < F_c \rightarrow \sigma_1 = 593.5\ psi < F_c = 2000\ psi$ ∴ ACCEPTABLE	
Timber	Bending Strength F_b = 2100 psi Format Conversion Factor K_F = 2.54 Resistance Factor Φ = 0.85 Design Bending Strength $F_b' = \Phi F_b K_F = 0.85(2100\ psi)(2.54) =$ 4533.9 psi Tension Strength F_t = 1575 psi Format Conversion Factor K_F = 2.70 Resistance Factor Φ = 0.80 Design Tension Strength $F_t' = \Phi F_t K_F = 0.80(1575\ psi)(2.70) =$ 3402.0 psi Average CLT Stress σ_2 = 387.5 psi Extreme Fiber Stress $\sigma_{m,2}$ = 584.3 psi $\frac{\sigma_2}{F_t'} + \frac{\sigma_{m,2}}{F_b'} \leq 1 \rightarrow \frac{387.5\ psi}{3402\ psi} + \frac{584.3\ psi}{4534\ psi} =$ 0.243 < 1.0 ∴ ACCEPTABLE Shear Strength F_v = 160 psi Format Conversion Factor K_F = 2.88 Resistance Factor Φ = 0.75 Design Shear Strength $F_v' = \phi F_v K_F = 0.75(160\ psi)(2.88) =$ 345.6 psi CLT Modulus of Elasticity E_2 = 1800 ksi NA of timber h = 5.74 in Shear V = 2.03 kip $EI_{eff,comp}$ = 1555662 kip-in^2 $\tau_{2,max} = \frac{E_2 h^2 V_u}{2(EI)_{eff,comp}} = \frac{(1800\ ksi)(5.74\ in)^2(2.03\ kip)}{2(1555662\ kip \cdot in^2)} =$ 38.73 psi $\tau_{2,max} < F_v' \rightarrow \tau_{2,max} = 38.73\ psi < F_v' = 345.6\ psi$ ∴ ACCEPTABLE	EN 1995.1-1 Annex B EQ 3.9

STEP DESCRIPTION	COMPUTATION	REFERNCE
Fastener Load Calculation	STRENGTH ANALYSIS F_{ULT} = 76.5828 kips Gamma Factor γ_1 = 0.64859 ksi Concrete Modulus of Elasticity E_1 = 3834.25 in Concrete Area A_1 = 33.00 in^2 Composite Centroid a_1 = 2.54 in Fastener Spacing s = 60 in Ultimate Shear Loading V = 2.0328 kip $EI_{eff,comp\_ULS}$ = 1555662 kip-in^2 $F_q = \frac{\gamma_1 E_1 A_1 a_1 s V}{EI_{eff,comp}}$ $F_q = \frac{(0.649)(3834\ ksi)(33 in^2)(2.54\ in)(60\ in)(2.033\ kip)}{1555662\ kip \cdot in^2} =$ 16.32 kip $F_q < F_{ULT} \rightarrow F_q = 16.32\ kip < F_{ULT} = 76.6\ kip$ ∴ ACCEPTABLE	EN 1995.1-1 Annex B EQ 3.10

Ultimate Limit State
Long-Term Loading

STEP DESCRIPTION	COMPUTATION	REFERNCE
	EFFECTIVE STIFFNESS	
	Deformation factor for long term	
	loading for concrete k_{def_c} = 2.5	EN 1995.1-1
	loading for timber k_{def_t} = 0.9	
	loading for HBV k_{def_HBV} = 0.6	
	Stiffness reduction for ULS ψ_2 = 0.3	
	Effective Spacing between s_{eff} = 1524 mm = 60 in	ETA-13/0029 Table 2.1
	Gamma Span Length L_0 = 24 ft	
	Connector slip Modulus K_{U_LT} = 247.453 kN/mm = 1412.99 kips/in	
	Concrete / CLT	
	Modulus of Elasticity E_1 = 3834.25 ksi; E_2 = 1800 ksi	
	Area A_1 = 33 in^2; A_1 = 50.5387 in^2	
	Momemnt of Inertia I_1 = 20.7969 in^4; I_1 = 261.939 in^4	
	Height h_1 = 2.75 in; h_2 = 6.90 in	
	EA_{eff,CLT_ULT_LT} = 71629.6 in EI_{eff,CLT_ULT_LT} = 371252 kip-in^2	CLT Handbook EQ 24 & 25
Modulus of Adjustment for long term loading	$E_{1,ULT\_LT} = \frac{E_1}{1+\psi_2 k_{def\_c}} =$ 2191 ksi $E_{2,ULT\_LT} = \frac{E_2}{1+\psi_2 k_{def\_t}} =$ 1417.32 ksi	EN 1995.1-1
Gamma Factor	$\gamma_{1_{ULT\_LT}} = \left[1+\frac{\pi^2 E_{1,ULT\_LT} A_1 s_{eff}}{K_{ULT\_LT} {L_0}^2}\right]^{-1}$ $\gamma_{1,ULT\_LT} = \left[1+\frac{\pi^2(2191\ ksi)(33\ in^2)(60\ in)}{(1413\ \ kip/in)(24\ ft)^2}\right]^{-1} =$ 0.732	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2,ULT\_LT} = \frac{\gamma_{1\_ULT\_LT} E_{1,ULT\_LT} A_1 (h_1+h_2)}{2\gamma_{1,ULT\_LT} E_{1,ULT\_LT} A_1 + 2\gamma_2 EA_{eff,CLT\_ULT\_LT}}$ $a_{2,ULT\_LT} = \frac{(0.732)(2191.0\ ksi)(33\ in^2)(2.75\ in+6.90\ in)}{2(0.732)(2191.0 ksi)(33\ in^2)+2(1.0)(71629.6\ kip)} =$ 2.051 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1,ULT\_LT} = \frac{h_1+h_2}{2} - a_{2\_ULT}$ $a_{1,ULT\_LT} = \frac{6.90\ in+2.75\ in}{2} - 2.05 =$ 2.774 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_ULT\_LT} = \left(E_{1,ULT\_LT} I_1 + \gamma_{1,ULT\_LT} E_{1,ULT\_LT} A_1 {a_{1,ULT\_LT}}^2\right) + \left(EI_{eff} + \gamma_2 EA_{eff} {a_{2,ULT}}^2\right)$ $(EI)_{eff,comp\_ULT\_LT} =$ 453096 kip-in^2 + 672544 kip-in^2 $(EI)_{eff,comp\_ULT\_LT} =$ 1125640 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Compsite & CLT Effective	$Ratio_{CLT\_ULT} = \frac{EI_{eff,comp\_ULT\_LT}}{EI_{eff,CLT\_ULT\_LT}}$ $Ratio_{CLT\_ULT} = \frac{1125640\ kip\cdot in^2}{371251.9\ kip\cdot in^2} =$ 15.71	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	APPARENT STIFFNESS	
Bare CLT Stiffness	Length $L = 24$ ft K Value $K_s = 11.5$ $EI_{eff,CLT\_ULT\_LT} = 371252$ kip-in^2 $GA_{eff,CLT\_ULT\_LT} = 846.9$ kips	
	$EI_{app,CLT\_ULS\_LT} = \dfrac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \dfrac{371252\ kip\cdot in^2}{1+\frac{(11.5)(371252\ kip\cdot in^2)}{(846.9\ kip)(24\ ft)^2}} =$ 349980 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \dfrac{EI_{app,CLT}}{EI_{eff,CLT}} = \dfrac{349980.3\ kip\cdot in^2}{371251.9\ kip\cdot in^2} =$ 94.3%	
Composite Section Stiffness	$L = 24$ ft $K_s = 11.5$ $EI_{eff,comp} = 1125640$ kip-in^2 $GA_{eff,clt} = 846.9$ kips	
	$EI_{app,comp\_ULS\_LT} = \dfrac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \dfrac{1125640\ kip\cdot in^2}{1+\frac{(11.5)(1125640 kip\cdot in^2)}{(846.9\ kip)(24\ ft)^2}} =$ 950482 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \dfrac{EI_{app,,comp}}{EI_{eff,comp}} = \dfrac{950481.6\ kip\cdot in^2}{1125640\ kip\cdot in^2} =$ 84.4%	
	$Ratio_{CLT} = \dfrac{EI_{app,comp}}{EI_{app,cLT}} = \dfrac{950481.6\ kip\cdot in^2}{349980.3\ kip\cdot in^2} =$ 2.716	
Apparent Floor Stiffness	$L = 24$ ft $EI_{app,CLT} = 349980$ kip-in^2 $EI_{app,comp} = 950482$ kip-in^2	
	$K_{app,CLT\_ULS\_LT} = \dfrac{P}{\Delta} = \dfrac{48EI_{app,CLT}}{L^3}$	
	$K_{app,CLT\_ULS\_LT} = \dfrac{48(349980.3\ kip\cdot in^2)}{(24\ ft)^3} =$ 0.70325 kip/in	
	$K_{app,comp\_ULS\_LT} = \dfrac{P}{\Delta} = \dfrac{48EI_{app,comp}}{L^3}$	
	$K_{app,comp\_ULS\_LT} = \dfrac{48(950482\ kip\cdot in^2)}{(24\ ft)^3} =$ 1.90989 kip/in	

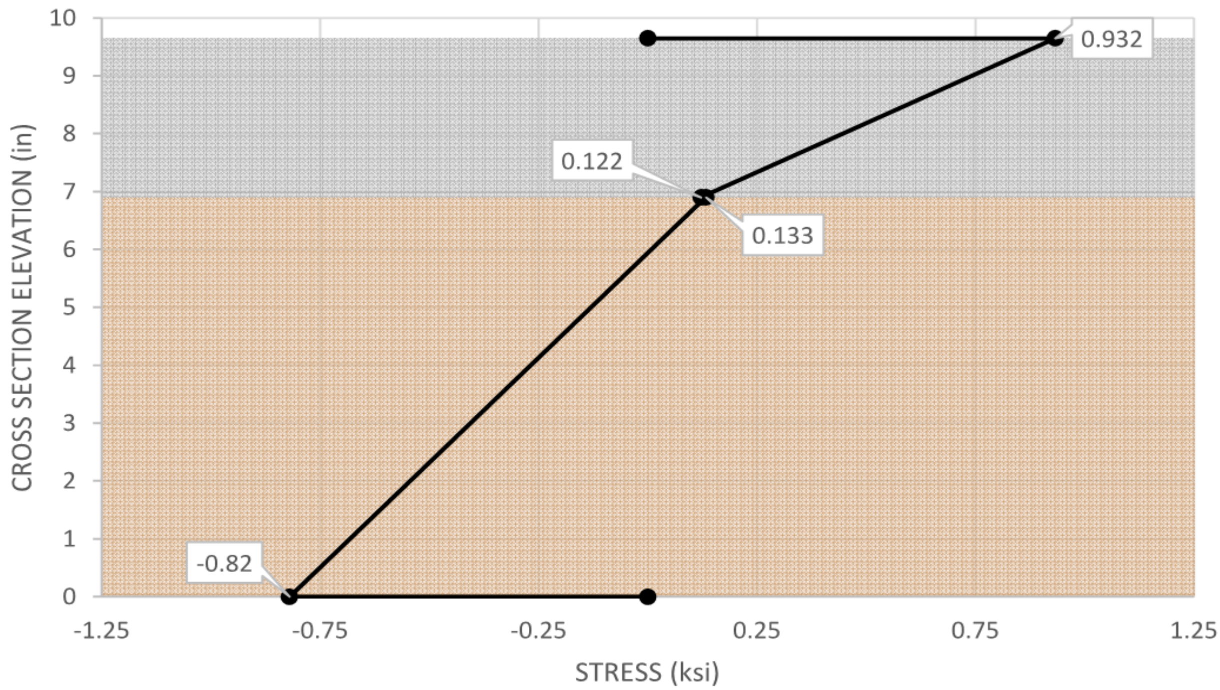
STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length $L = 24$ ft Shear $V_u = 2.03$ kip Moment $M_u = 146.36$ kip-in	
	Concrete: Modulus of Elasticity $E_1 = 2191$ ksi; Height $h_1 = 2.75$ in; Centroid $a_1 = 2.77409$ in; Gamma Factor $\gamma_1 = 0.73242$ CLT: $E_2 = 1417.32$ ksi; $h_2 = 6.90$ in; $a_2 = 2.05091$ in; $\gamma_2 = 1$ $EI_{eff,comp} = 1125640$ kip-in^2	
	Bending Stess Calulations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.732)(2191\ ksi)(2.77\ in)(146\ kip \cdot in)}{1125640\ kip \cdot in^2} =$ 0.579 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(2191\ ksi)(2.75\ in)(146\ kip \cdot in)}{1125640\ kip \cdot in^2} =$ 0.392 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.579 + 0.392 =$ 0.971 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.579 - 0.392 =$ 0.187 ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(1417.3\ ksi)(2.05 in)(146\ kip \cdot in)}{1125640\ kip \cdot in^2} =$ 0.378 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(1417\ ksi)(6.90\ in)(146\ kip \cdot in)}{1125640\ kip \cdot in^2} =$ 0.636 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} + \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.379 + 0.636 =$ 0.258 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.38 - 0.636 =$ -1.01 ksi	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	STRENGTH ANALYSIS	
	Normal Stress in Concrete σ_1 = 578.8 psi	
Concrete	Allow Comp Strength of Conc F_c = 2000 psi $F_c = f_c'/2$	
	$\sigma_1 < F_c \rightarrow \sigma_1 = 578.8\,psi < F_c = 2000\,psi$ ∴ ACCEPTABLE	
Timber	Bending Strength F_b = 2100 psi	
	Format Conversion Factor K_F = 2.54	
	Resistance Factor Φ = 0.85	
	Design Bending Stength	
	$F_b' = \Phi F_b K_F = 0.85(2100\,psi)(2.54) =$ 4533.9 psi	
	Tension Strength F_t = 1575 psi	
	Format Conversion Factor K_F = 2.70	
	Resistance Factor Φ = 0.80	
	Design Tension Stength	
	$F_t' = \Phi F_t K_F = 0.80(1575\,psi)(2.70) =$ 3402.0 psi	
	Average CLT Stress σ_2 = 378.0 psi	
	Extreme Fiber Stress $\sigma_{m,2}$ = 635.8 psi	
	$\frac{\sigma_2}{F_t'} + \frac{\sigma_{m,2}}{F_b'} \leq 1 \rightarrow \frac{378.0\,psi}{3402\,psi} + \frac{635.8\,psi}{4534\,psi} =$ 0.251 < 1.0 ∴ ACCEPTABLE	
	Shear Strength F_v = 160 psi	
	Format Conversion Factor K_F = 2.88	
	Resistance Factor Φ = 0.75	
	Design Shear Stength	
	$F_v' = \phi F_v K_F = 0.75(160\,psi)(2.88) =$ 345.6 psi	
	CLT Modulus of Elasticity E_2 = 1417.32 ksi	
	NA of timber h = 5.50 in	
	Shear V = 2.03 kip	
	$EI_{eff,comp}$ = 1125640 kip-in^2	
	$\tau_{2,max} = \frac{E_2 {h_2}^2 V_u}{2(EI)_{eff,comp}} = \frac{(1417\,ksi)(5.50in)^2(2.03\,kip)}{2(1125640\,kip \cdot in^2)} =$ 38.73 psi	EN 1995.1-1 Annex B EQ 3.9
	$\tau_{2,max} < F_v' \rightarrow \tau_{2,max} = 38.73\,psi < F_v' = 345.6\,psi$ ∴ ACCEPTABLE	

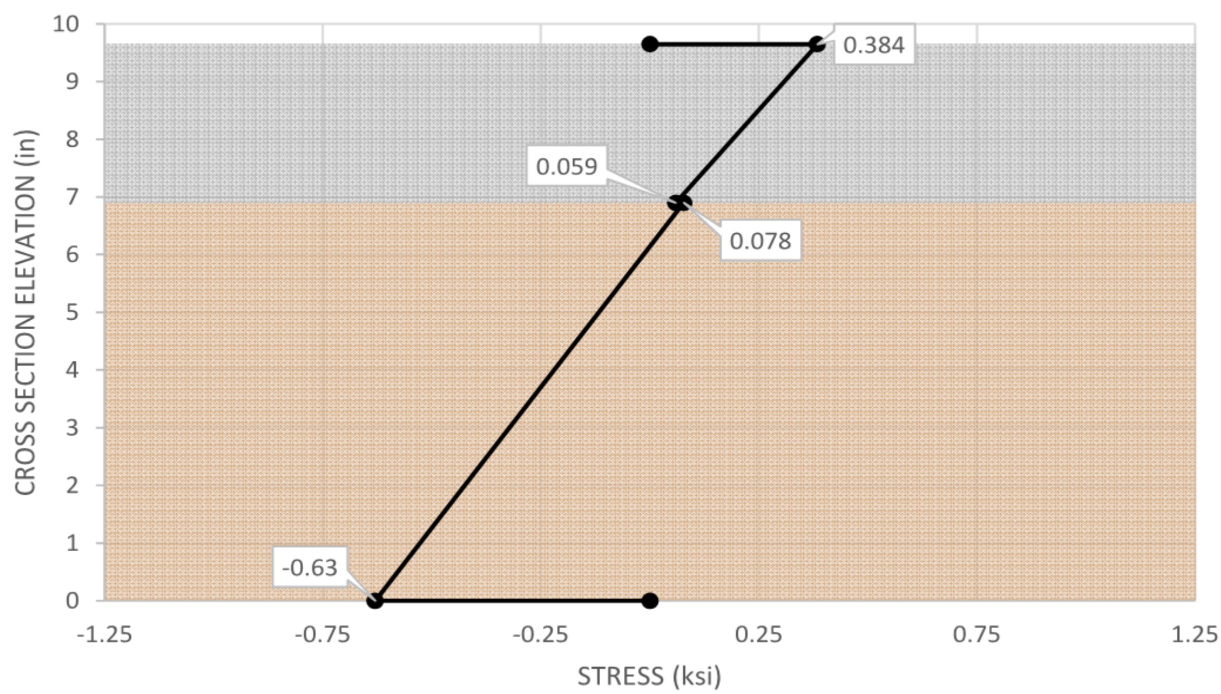
STEP DESCRIPTION	COMPUTATION	REFERNCE
Fastener Load Calculation	STRENGTH ANALYSIS F_{ULT} = 76.5828 kips Gamma Factor γ_1 = 0.73242 ksi Concrete Modulus of Elasticity E_1 = 2191.00 in Concrete Area A_1 = 33.00 in^2 Composite Centroid a_1 = 2.77 in Fastener Spacing s = 60 in Ultimate Shear Loading V = 2.0328 kip $EI_{eff,comp}$ = 1125640 kip-in^2 $F_q = \frac{\gamma_1 E_1 A_1 a_1 sV}{EI_{eff,comp}}$ $F_q = \frac{(0.732)(2191\ ksi)(33in^2)(2.77\ in)(60\ in)(2.03\ kip)}{1125640\ kip \cdot in^2} =$ 15.92 kip $F_q < F_{ULT} \rightarrow F_q = 15.92\ kip < F_{ULT} = 76.6\ kip$ ∴ ACCEPTABLE	EN 1995.1-1 Annex B EQ 3.10

STEP DESCRIPTION	COMPUTATION						REFERNCE
	CONNECTOR EFFICIENCY						
Partial Composite Section	Effective Bending Stiffness	EI_{eff_SLS}	=	1621576	in		
	Deflection	Δ_{PC}	=	0.23018	in		
Non-Composite Section	Effective Bending Stiffness	EI_{eff_SLS}	=	551231	in	(set K_{ser} close to zero)	
	Deflection	Δ_{NC}	=	0.47809	in		
Fully Composite	Concrete MOE	E_1	=	3834.25	psi		
	Timber MOE	E_2	=	1800	psi		
	Width	b'	=	12	in		
	Width of Transformed Concrete	b'	=	25.56	in	$b' = b(E_1/E_2)$	
	Height of Timber	h_1	=	2.75	in		
	Height of Concrete	h_2	=	6.90	in		
	Bottom to Centroid of						
	Transformed Concrete	y'_1	=	8.275	in		
	Timber Section	y_2	=	3.45	in		
	Area of Transformed Concrete	A'_1	=	70.29	in^2		
	Area of Timber Section	A_2	=	82.8	in^2		
	Moment of Inertia of						
	Transformed Concrete	I'_1	=	44.30	in^4		
	Timber Section	I_2	=	328.51	in^4		
	Centroid of Transformed Section						
		$y_c = \frac{y_1' A_1' + y_2 A_2}{A_1' + A_2}$	=	5.665	in		
	Distance to Concrete Centroid	d_1	=	4.29	in	$d = \left\lvert y_c - \frac{h}{2} \right\rvert$	
	Distance to Timber Centroid	d_2	=	0.84	in		
	Transformed Section						
	Moment of Inertia about NA	I_{NA}	=	1725.27	in^4	$I_{NA} = \sum I_i + A_i {d_i}^2$	
	Bending Stiffness	$E_2 I_{NA}$	=	3105482	$k\text{-}in^2$		
	Deflection of Fully Composite Section						
	$\Delta_{FC} = \frac{5wL^4}{384 E_2 I_{NA}} = \frac{5(50\,plf)(24ft)^4}{384(3105482\; k \cdot in^2)} =$ 0.120 in						
Shear Connector Efficiency	$Efficiency = \frac{\Delta_{NC} - \Delta_{PC}}{\Delta_{NC} - \Delta_{FC}} = \frac{0.6771" - 0.2302"}{0.6771" - 0.120"} =$ 69.3%						

90 mm HBV 26 feet SPAN

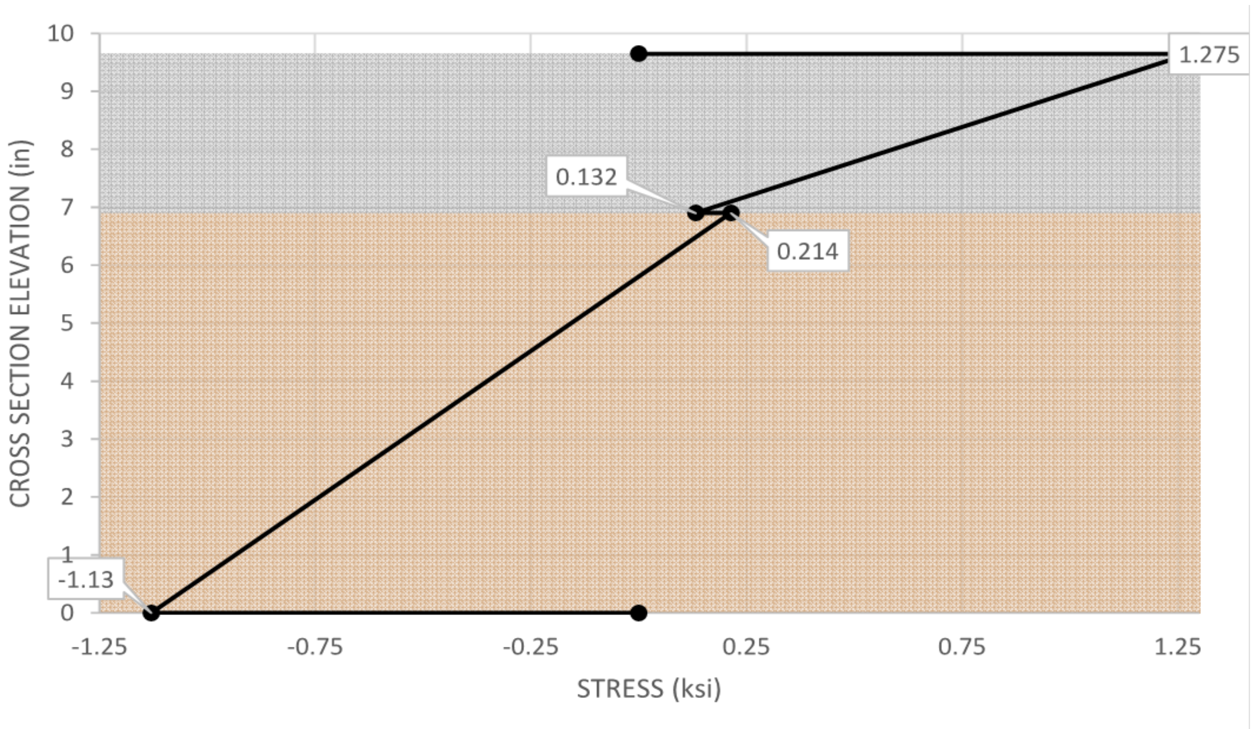


SERVICABILITY LIMIT STATE CROSS-SECTIONAL STRESS DISTRIBUTION

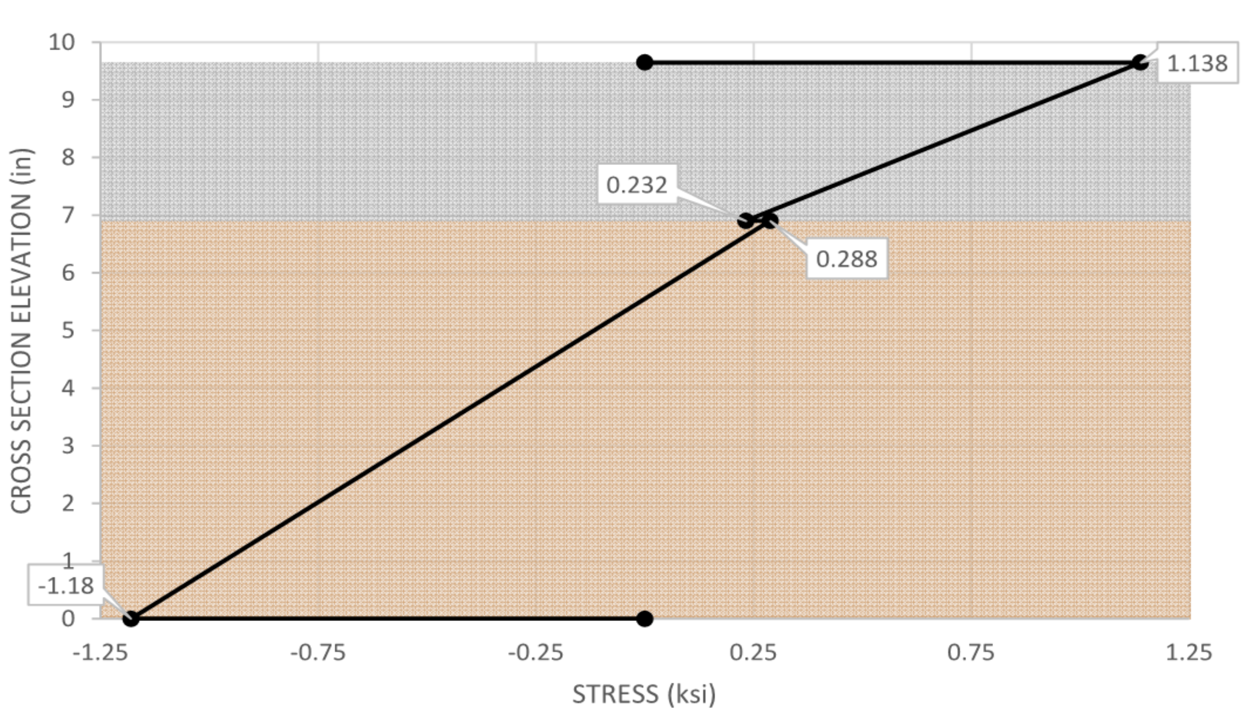


SERVICABILITY LIMIT STATE LONG-TERM CROSS-SECTIONAL STRESS DISTRIBUTION

90 mm HBV 26 feet SPAN



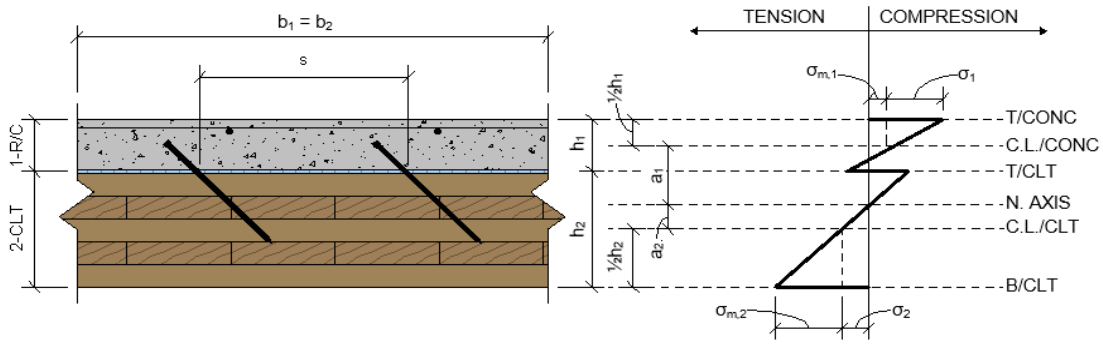
ULTIMATE LIMIT STATE CROSS-SECTIONAL STRESS DISTRIBUTION



ULTIMATE LIMIT STATE LONG-TERM CROSS-SECTIONAL STRESS DISTRIBUTION

STEP DESCRIPTION	COMPUTATION	REFERNCE

90 mm HBV 26 feet SPAN



Cross-section Stress Diagram

Dead Load Calculation

		Density			Height		
Mass Timber	ρ_t =	35	pcf	h_t =	6.90	in	L = 26 ft
Concrete	ρ_c =	150	pcf	h_c =	2.75	in	

$$w_{DL} = \rho_t h_t + \rho_c h_c$$
$$w_{DL} = (35\,pcf)(6.90\,in) + (150\,pcf)(2.75\,in) = \text{ 54.50 psf}$$

Loading Criteria

		Loading			Sustained/Expected		
Dead Load	w_{DL} =	54.50	psf	m_{DL} =	100%		ASCE 7-16
Superimposed Dead Load	w_{SDL} =	20	psf	m_{SDL} =	78.2%		
Live Load	w_{LL} =	50	psf	m_{LL} =	21.8%	(10.9 psf mean load)	Table C4.3-2

Loading Cases

	Load Case Factors			Equations	DL	SDL	LL	w_u
	Combo	DL	SDL	LL		PSF	PSF	PSF	PLF
LRFD	G01	1.4	1.4	0.0	$1.4(w_{DL} + w_{SDL})$	76.3	28	0	104.3
	G02	1.2	1.2	1.6	$1.2(w_{DL} + w_{SDL}) + 1.6w_{LL}$	65.4	24	80	169.4
Service	S01	1.0	1.0	1.0	$1.0w_{DL} + 1.0w_{SDL} + 1.0w_{LL}$	54.5	20	50	124.5
	S02	0.0	1.0	1.0	$1.0w_{SDL} + 1.0w_{LL}$	0	20	50	70
	S03	1.0	0.782	0.218	$1.0w_{DL} + 0.782w_{SDL} + 0.218w_{LL}$	54.5	15.64	10.9	81.04

Combo	w_u PLF	V_u kips	M_u kip-in
G01	104.3	1.356	105.8
G02	169.4	2.202	171.8
S01	124.5	1.619	126.2
S02	70	0.91	70.98
S03	81.04	1.054	82.17

$$V_u = \frac{w_u L}{2}$$
$$M_u = \frac{w_u L^2}{8}$$

STEP DESCRIPTION	COMPUTATION	REFERNCE

CLT Properties

CLT CALCULATIONS

Type: 5-Ply CLT, Grade E1M4, 139 E

h_i = 1.375 in
b = 12 in
h_t = 6.90 in
a = 5.52 in
L = 26.0 ft
K_s = 11.5

Structurlam Crosslam

Major Strength Axis

$F_{b,0}$ = 2100 psi
E_0 = 1800 ksi
$F_{t,0}$ = 1575 psi
$F_{c,0}$ = 1875 psi
$F_{v,0}$ = 160 psi
$F_{s,0}$ = 50 psi

Minor Strength Axis

$F_{b,90}$ = 875 psi
E_{90} = 1400 ksi
$F_{v,90}$ = 135 psi
$F_{s,90}$ = 45 psi

CLT Handbook, Chapter 3

CLT Calculations									
				GA_{eff}		EA_{eff}	EI_{eff}		
Layer	E (ksi)	h (in)	z (in)	G (ksi)	h/G/b (in^2/kip)	EA (kip)	$Ebh^3/12$ (kip-in^2)	EAz^2 (kip-in^2)	Sum of Layers (kip-in^2)
1	1800	1.380	2.760	112.5	0.00102	29808	4730.53	227065	231796
2	46.6667	1.380	1.375	8.75	0.01314	772.8	122.643	1461.08	1583.72
3	1800	1.380	0.000	112.5	0.00102	29808	4730.53	0	4730.53
4	46.6667	1.380	1.375	8.75	0.01314	772.8	122.643	1461.08	1583.72
5	1800	1.380	2.760	112.5	0.00102	29808	4730.53	227065	231796
						90969.6			471490

CLT Handbook Chapter 3 EQ 24

$$EA_{eff} = \sum_{i=1}^{n} E_i b_i h_i$$

$$EI_{eff} = \sum_{i=1}^{n} E_i b_i \frac{h_i^3}{12} + \sum_{i=1}^{n} E_i A_i z_i^2$$

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

$$EI_{app} = \frac{EI_{eff}}{1 + \frac{K_s EI_{eff}}{GA_{eff} L^2}}$$

Cross Sectional Properties (Per foot of width)

SLS/ULS Short-Term

EA_{eff} = 90969.6 kip
EI_{eff} = 471490 kip-in^2
GA_{eff} = 1075.5 kip
EI_{app} = 448274.5 kip

SLS Long-Term

EA_{eff} = 47878.7 kip
EI_{eff} = 248153 kip-in^2
GA_{eff} = 566.1 kip
EI_{app} = 235933.9 kip

ULS Long-Term

EA_{eff} = 71629.6 kip
EI_{eff} = 371252 kip-in^2
GA_{eff} = 846.9 kip
EI_{app} = 352972.1 kip

STEP DESCRIPTION	COMPUTATION	REFERNCE
	CONNECTOR CALCUATION	
	Length of Connector Strip L = 1000 mm = 39.37 in	
	Thickness of the interlayer d_{zs} = 8.89 mm = 0.35 in	DIBt Z-9.1-557
	1 mm base unit d_0 = 1 mm	
Characteristic Load-Carrying Capacity	$T_k = 160 - 8.0(d_{zs}/d_0)^{0.5}$ $T_k = 160 - 8.0(8.89/1)^{0.5} =$ 136.1 kN	
Acceptable Shear Load	$zul.T = 90 - 4.5(d_{zs}/d_0)^{0.5}$ $zul.T = 90 - 4.5(8.89/1)^{0.5} =$ 76.58 kN	
Rated value of Load Bearing Capacity	$T_d = T_k/1.25 \approx 1.42\, zul.T$ $T_d = (136.1\ kN/mm)/1.25 =$ 108.9 kN	
Slip Modulus	$K_{ser} = 825 - 250(d_{zs}/d_0)^{0.2}$ $K_{ser} = 825 - 250(8.89/1)^{0.2} =$ 438 kN/mm	
Slip Modulus Ultimate	$K_u = (2/3)K_{ser}$ $K_u = (2/3)(438.0\ kN/mm) =$ 292 kN/mm Converstions $T_k =$ 30.6071 kips $K_{ser} =$ 2501 k/in $K_u =$ 1667.33 k/in	
	Spacing between connectors in rows at the ends s_e = 1524 mm = 60 in in rows in the middle s_m = 1829 mm = 72 in Effective Spacing between s_{eff} = 1600 mm = 63 in Number of rows of connectors n_r = 1 Deformation factor for long term loading for concrete k_{def_c} = 2.5 loading for timber k_{def_t} = 0.9 loading for STS $k_{defHBV\_}$ = 0.6 Stiffness reduction for ULS ψ_2 = 0.3	DIBt Z-9.1-557 Table A1.1 EN 1990
Connector Stiffness Adjustment for Long Term Loading	$K_{SLS\_LT} = \frac{K_{SER}}{1 + k_{def\_HBV}} = \frac{438.0\ kN/mm}{1 + 0.6} =$ 273.745 kN/mm $K_{ULS\_LT} = \frac{K_u}{1 + \psi_2 k_{def\_sts}} = \frac{292\, kN/mm}{1 + (0.3 * 0.6)} =$ 247.453 kN/mm	

STEP DESCRIPTION	COMPUTATION	REFERNCE
General Values	EFFECTIVE STIFFNESS Gamma Span Length $L = L_0 =$ 26 ft Connector slip Modulus $K_{ser} =$ 437.992 kN/mm = 2501.00 kips/in Connector Spacing $s_{eff} =$ 63 in	Annex B of EN 1995-1-1
Concrete Properties	Slab Depth $h_1 =$ 2.75 in Compressive Strength $f_c' =$ 4000 psi Weight of Concrete $w_c =$ 150 pcf	
CLT Properties	Clt Height $h_2 =$ 6.90 in $EA_{eff,CLT} =$ 90969.6 kip $EI_{eff,CLT} =$ 471490 kip-in^2	CLT Handbook EQ 24 & 25
Modulus of Elasticity	$E_{1\_SLS} = w_c{}^{1.5}33\sqrt{f_c'}$ $E_{1\_SLS} = (150\ pcf)^{1.5}33\sqrt{4000\ psi} =$ 3834.25 ksi	ACI 318, 19.2.2.1.a
Area of Concrete	$A_{c\_SLS} = h_c b_c$ $A_{c\_SLS} = (2.75\ in)(12\ in) =$ 33 in^2	EN 1995.1-1 Annex B EQ 3.2
Inertia of Concrete	$I_{1\_SLS} = b_1 h_1{}^2/12$ $I_{1\_SLS} = (12\ in)(2.75\ in)^2/12 =$ 20.8 in^4 $E_c A_{c\_SLS} = (3834.3\ ksi)(33\ in^2) =$ 126530 kip $E_c I_{c\_SLS} = (38343\ ksi)(20.8\ in^4) =$ 79740.5 kip-in^2	EN 1995.1-1 Annex B EQ 3.3
Gamma Factor	$\gamma_{1\_SLS} = \left[1 + \frac{\pi^2 E_1 A_1 s_1}{K_1 L_0{}^2}\right]^{-1}$ $\gamma_{1\_SLS} = \left[1 + \frac{\pi^2 (3834.3\ ksi)(33\ in^2)(63\ in)}{(2501\ \ kip/in)(26\ ft)^2}\right]^{-1} =$ 0.75577	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2\_SLS} = \frac{\gamma_1 E_1 A_1 (h_1 + h_2)}{2\gamma_1 E_1 A_1 + 2\gamma_2 EA_{eff}}$ $a_{2\_SLS} = \frac{(0.7558)(3834\ ksi)(33\ in^2)(2.75\ in + 6.90\ in)}{2(0.7558)(3834\ ksi)(33\ in^2) + 2(1.0)(90970\ kip)} =$ 2.47272 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1\_SLS} = \frac{h_1 + h_2}{2} - a_{2\_SLS}$ $a_{1\_SLS} = \frac{6.90\ in + 2.75\ in}{2} - 2.47 =$ 2.35228 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_SLS} = (E_1 I_1 + \gamma_1 E_1 A_1 a_1{}^2) + (EI_{eff} + \gamma_2 EA_{eff} a_2{}^2)$ $(EI)_{eff,comp\_SLS} =$ 608867.8 kip-in^2 + 1027711 kip-in^2 = 1636579 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Composite & CLT Effective	$Ratio_{CLT} = \frac{EI_{eff,comp\_SLS}}{EI_{eff,CLT}}$ $Ratio_{CLT} = \frac{1636579\ kip \cdot in^2}{471489.9\ kip \cdot in^2} =$ 3.471	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	APPARENT STIFFNESS	
Bare CLT Stiffness	Length L = 26 ft K Value K_s = 11.5 $EI_{eff,clt}$ = 471490 kip-in^2 $GA_{eff,clt}$ = 1075.5 kips	
	$EI_{app,CLT\_SLS} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{471489.9\ kip \cdot in^2}{1+\frac{(11.5)(471489.9\ kip \cdot in^2)}{(1075.5\ kip)(26\ ft)^2}} =$ 448274 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{448274.5\ kip \cdot in^2}{471489.9\ kip \cdot in^2} =$ 95.1%	
Composite Section Stiffness	L = 26 ft K_s = 11.5 $EI_{eff,comp}$ = 1636579 kip-in^2 $GA_{eff,clt}$ = 1075.5 kips	
	$EI_{app,comp\_SLS} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{1636579\ kip \cdot in^2}{1+\frac{(11.5)(1636759\ kip \cdot in^2)}{(1075.5\ kip)(26\ ft)^2}} =$ 1387212 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{1387212 kip \cdot in^2}{1636579\ kip \cdot in^2} =$ 84.8%	
	$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{1387212\ kip \cdot in^2}{448274.5\ kip \cdot in^2} =$ 3.095	
Apparent Floor Stiffness	L = 26 ft $EI_{app,CLT}$ = 448274 kip-in^2 $EI_{app,comp}$ = 1387212 kip-in^2	
	$K_{app,CLT\_SLS} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$	
	$K_{app,CLT\_SLS} = \frac{48(448274.5\ kip \cdot in^2)}{(26\ ft)^3} =$ 0.70847 kip/in	
	$K_{app,comp\_SLS} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$	
	$K_{app,comp\_SLS} = \frac{48(1387212\ kip \cdot in^2)}{(26\ ft)^3} =$ 2.1924 kip/in	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length $L = 26$ ft Shear $V_u = 1.62$ kip Moment $M_u = 126.24$ kip-in	
	Concrete: Modulus of Elasticity $E_1 = 3834.25$ ksi; Height $h_1 = 2.75$ in; Centroid $a_1 = 2.35228$ in; Gamma Factor $\gamma_1 = 0.75577$ CLT: $E_2 = 1800$ ksi; $h_2 = 6.90$ in; $a_2 = 2.47272$ in; $\gamma_2 = 1$ $EI_{eff,comp\_SLS} = 1636579$ kip-in^2	
	Bending Stress Calculations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.756)(3834\,ksi)(2.35\,in)(126\,kip\cdot in)}{1636579\,kip\cdot in^2} =$ 0.526 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(3834\,ksi)(2.75\,in)(126\,kip\cdot in)}{1636579\,kip\cdot in^2} =$ 0.407 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.526 + 0.407 =$ 0.932 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.526 - 0.407 =$ 0.119 ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(1800\,ksi)(2.473\,in)(126\,kip\cdot in)}{1636579\,kip\cdot in^2} =$ 0.343 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(1800\,ksi)(6.90\,in)(126\,kip\cdot in)}{1636579\,kip\cdot in^2} =$ 0.479 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} + \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.343 + 0.479 =$ 0.136 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.34 - 0.479 =$ -0.82 ksi	

<table>
<tr><th>STEP DESCRIPTION</th><th>COMPUTATION</th><th>REFERNCE</th></tr>
<tr><td></td><td>DEFLECTION CALCUATIONS</td><td></td></tr>
<tr><td>Loading</td><td>Short-Term
Dead Load w_{DL} = 54.50 plf
Superimposed Dead Load w_{SDL} = 20.00 plf
Live Load w_{LL} = 50.00 plf
Span Length L = 26 ft
Stiffness $EI_{eff,comp\_SLS}$ = 1636579 kip-in^2

Long-Term
w_{DL} = 54.50 plf
w_{SDL} = 15.64 plf
w_{LL} = 10.9 plf
L = 26 ft</td><td></td></tr>
<tr><td></td><td>Short-Term Deflections</td><td></td></tr>
<tr><td>Dead Load</td><td>$\Delta_{DL}=\dfrac{5w_{DL}L^4}{384EI_{eff,comp\_SLS}}=\dfrac{5(54.5\,plf)(26\,ft)^4}{384(1636578.9\,kip\cdot in^2)}=$ 0.3424 in</td><td></td></tr>
<tr><td>Superimposed Dead Load</td><td>$\Delta_{SDL}=\dfrac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS}}=\dfrac{5(20\,plf)(26\,ft)^4}{384(1636578.9\,kip\cdot in^2)}=$ 0.1257 in</td><td></td></tr>
<tr><td>Live Load</td><td>$\Delta_{LL}=\dfrac{5w_{LL}L^4}{384EI_{eff,comp\_SLS}}=\dfrac{5(50\,plf)(26\,ft)^4}{384(1636578.9\,kip-in^2)}=$ 0.3141 in</td><td></td></tr>
<tr><td></td><td>Serviceability Check</td><td></td></tr>
<tr><td>Allowable LL Deflection</td><td>$\Delta_{allow,LL}=\dfrac{L}{360}=\dfrac{(26ft)(12\,in)}{360(1\,ft)}=$ 0.87 in</td><td>IBC Table 1604.3</td></tr>
<tr><td>Check</td><td>$\Delta_{LL}\leq\ \Delta_{allow,LL}\rightarrow\Delta_{LL}=0.314\,in\ <\ \Delta_{allow,LL}=0.87\,in$ ∴ ACCEPTABLE</td><td></td></tr>
<tr><td></td><td>Long-Term Deflections</td><td></td></tr>
<tr><td>Dead Load</td><td>$\Delta_{DL\_LT}=\dfrac{5w_{DL}L^4}{384EI_{eff,comp\_SLS\_LT}}=\dfrac{5(54.5\,plf)(26\,ft)^4}{384(1636579\,kip\cdot in^2)}=$ 0.34240 in</td><td></td></tr>
<tr><td>Superimposed Dead Load</td><td>$\Delta_{SDL\_LT}=\dfrac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS\_LT}}=\dfrac{5(15.6\,plf)(26\,ft)^4}{384(1636579\,kip\cdot in^2)}=$ 0.09826 in</td><td></td></tr>
<tr><td>Live Load</td><td>$\Delta_{LL\_LT}=\dfrac{5w_{LL}L^4}{384EI_{eff,comp\_SLS\_LT}}=\dfrac{5(10.9\,plf)(26\,ft)^4}{384(1663579\,kip\cdot in^2)}=$ 0.06848 in</td><td></td></tr>
<tr><td></td><td>Serviceability Check
Time Dependent Creep Factor $K_{cr}=$ 2.0
Deflection due to short-term $\Delta_{ST}=$ 0.3141 in
Deflection due to long-term $\Delta_{LT}=$ 0.44066 in $\Delta_{LT}=\Delta_{DL\_LT}+\Delta_{SDL\_LT}$</td><td>NDS 2018 Section 3.5.1</td></tr>
<tr><td></td><td>$\Delta_{TL}=K_{cr}\Delta_{LT}+\Delta_{ST}$
$\Delta_{TL}=2.0(0.4407\text{ in})+(0.3141\text{ in})=$ 1.195 in</td><td>NDS EQ 3.5-1</td></tr>
<tr><td></td><td>$\Delta_{allow,TL}=\dfrac{L}{240}=\dfrac{(26ft)(12\,in)}{240(1\,ft)}=$ 1.30 in</td><td>IBC Table 1604.3</td></tr>
<tr><td></td><td>$\Delta_{TL}\leq\ \Delta_{allow,TL}\rightarrow\Delta_{TL}=1.195\,in\ <\ \Delta_{allow,TL}=1.30\,in$ ∴ ACCEPTABLE</td><td></td></tr>
</table>

STEP DESCRIPTION	COMPUTATION	REFERNCE
	EFFECTIVE STIFFNESS	
	Deformation factor for long term	
	loading for concrete k_{def_c} = 2.5	EN 1995.1-1
	loading for timber k_{def_t} = 0.9	
	loading for HBV k_{def_HBV} = 0.6	
	Stiffness reduction for ULS ψ_2 = 0.3	
	Effective Spacing between s_{eff} = 1600 mm = 63 in	ETA-13/0029 Table 2.1
	Gamma Span Length L_0 = 26 ft	
	Connector slip Modulus K_{SLS_LT} = 273.745 kN/mm = 1563.12 kips/in	
	Concrete / CLT	
	Modulus of Elasticity E_1 = 3834.25 ksi; E_2 = 1800 ksi	
	Area A_1 = 33 in^2; A_1 = 26.60 in^2	
	Momemnt of Inertia I_1 = 20.7969 in^4; I_1 = 137.863 in^4	
	Height h_1 = 2.75 in; h_2 = 6.90 in	
	EA_{eff,CLT_SLS_LT} = 47878.74 kip	CLT Handbook EQ 24 & 25
	EI_{eff,CLT_SLS_LT} = 248153 kip-in^2	
Modulus of Adjustment for long term loading	$E_{1\_SLS\_LT} = \dfrac{E_1}{1+k_{def\_c}} =$ 1095.5 ksi; $E_{2\_SLS\_LT} = \dfrac{E_2}{1+k_{def\_t}} =$ 947.368 ksi	EN 1995.1-1
Gamma Factor	$\gamma_{1,SLS\_LT} = \left[1+\dfrac{\pi^2 E_{1\_SLS\_LT} A_1 s_{eff}}{K_{ser\_LT} {L_0}^2}\right]^{-1}$ $\gamma_{1,SLS\_LT} = \left[1+\dfrac{\pi^2(1095\,ksi)(33\,in^2)(63\,in)}{(1563\,kip/in)(26\,ft)^2}\right]^{-1} =$ 0.871	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2,SLS\_LT} = \dfrac{\gamma_{1\_SER} E_1 A_1 (h_1+h_2)}{2\gamma_{1\_SER} E_1 A_1 + 2\gamma_2 EA_{eff}}$ $a_{2,SLS\_LT} = \dfrac{(0.871)(1095\,ksi)(33\,in^2)(2.75\,in+6.90\,in)}{2(0.871)(1095\,ksi)(33\,in^2)+2(1.0)(47878.7\,kip)} =$ 2.68052 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1,SLS\_LT} = \dfrac{h_1+h_2}{2} - a_{2,SLS\_LT}$ $a_{1,SLS\_LT} = \dfrac{6.90\,in+2.75\,in}{2} - 2.68 =$ 2.144 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_SLS\_LT} = \left(E_1 I_1 + \gamma_{1\_SER} E_1 A_1 {a_{1\_SER}}^2\right) + \left(EI_{eff} + \gamma_2 EA_{eff} {a_{2\_SER}}^2\right)$ $(EI)_{eff,comp\_SLS\_LT} =$ 167637 kip-in^2 + 592171 kip-in^2 $(EI)_{eff,comp\_SLT\_LT} =$ 759808.0 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Compsite & CLT Effective	$Ratio_{CLT\_SER} = \dfrac{EI_{eff,comp\_SER}}{EI_{eff,CLT}}$ $Ratio_{CLT\_SER} = \dfrac{759808.0\,kip\cdot in^2}{248152.6\,kip\cdot in^2} =$ 15.87	

<table>
<tr><th>STEP DESCRIPTION</th><th>COMPUTATION</th><th>REFERNCE</th></tr>
<tr><td></td><td>APPARENT STIFFNESS</td><td></td></tr>
<tr><td>Bare CLT Stiffness</td><td>Length L = 26 ft
K Value K_s = 11.5
$EI_{eff,clt}$ = 248153 kip-in^2
$GA_{eff,clt}$ = 566.1 kips
$$EI_{app,CLT\_SLS\_LT} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{248152.6\ kip\cdot in^2}{1+\frac{(11.5)(248152.6\ kip\cdot in^2)}{(566.1\ kip)(26\ ft)^2}} = \quad 235934 \text{ kip-in}^2$$
$$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{235933.9\ kip\cdot in^2}{248152.6\ kip\cdot in^2} = \quad 95.1\%$$</td><td>CLT Handbook EQ 35</td></tr>
<tr><td>Composite Section Stiffness</td><td>L = 26 ft
K_s = 11.5
$EI_{eff,comp}$ = 759808 kip-in^2
$GA_{eff,clt}$ = 566.1 kips
$$EI_{app,comp\_SLS\_LT} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{759808\ kip\cdot in^2}{1+\frac{(11.5)(759808\ kip\cdot in^2)}{(566.1\ kip)(26\ ft)^2}} = \quad 655816 \text{ kip-in}^2$$
$$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{655816.24\ kip\cdot in^2}{759808\ kip\cdot in^2} = \quad 86.3\%$$
$$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{617175.4\ kip\cdot in^2}{231413.8\ kip\cdot in^2} = \quad 2.78$$</td><td>CLT Handbook EQ 35</td></tr>
<tr><td>Apparent Floor Stiffness</td><td>L = 26 ft
$EI_{app,CLT}$ = 235934 kip-in^2
$EI_{app,comp}$ = 655816 kip-in^2
$$K_{app,CLT\_SLS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$$
$$K_{app,CLT\_SLS\_LT} = \frac{48(235933.8\ kip\cdot in^2)}{(26\ ft)^3} = \quad 0.37288 \text{ kip/in}$$
$$K_{app,comp\_SLS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$$
$$K_{app,comp\_SLS\_LT} = \frac{48(655816.2\ kip\cdot in^2)}{(26\ ft)^3} = \quad 1.03648 \text{ kip/in}$$</td><td></td></tr>
</table>

STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length $L = 26$ ft Shear $V_u = 1.05$ kip Moment $M_u = 82.17$ kip-in	
	Concrete: Modulus of Elasticity $E_1 = 1095.5$ ksi; Height $h_1 = 2.75$ in; Centroid $a_1 = 2.14448$ in; Gamma Factor $\gamma_1 = 0.87129$ CLT: $E_2 = 947.368$ ksi; $h_2 = 6.90$ in; $a_2 = 2.68052$ in; $\gamma_2 = 1$ $EI_{eff,comp\_SLS\_LT} = 759808$ kip-in^2	
	Bending Stess Calulations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.871)(1096\ ksi)(2.14\ in)(82.2\ kip \cdot in)}{759808\ kip \cdot in^2} =$ 0.221 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(1096\ ksi)(2.75\ in)(82.2\ kip \cdot in)}{7598081\ kip \cdot in^2} =$ 0.163 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.221 + 0.163 =$ 0.384 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.221 - 0.163 =$ 0.058 ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(947.4\ ksi)(2.68\ in)(82.2\ kip \cdot in)}{759808 kip \cdot in^2} =$ 0.275 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(947.4\ ksi)(6.90\ in)(82.2\ kip \cdot in)}{79808\ kip \cdot in^2} =$ 0.353 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.275 + 0.353 =$ 0.079 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.28 - 0.353 =$ -0.63 ksi	

Serviceability Limit State
Long-Term Loading

STEP DESCRIPTION	COMPUTATION	REFERNCE
	DEFLECTION CALCUATIONS	
	Short-Term / Long-Term	
Loading	Dead Load w_{DL} = 54.50 plf; w_{DL} = 54.50 plf Superimposed Dead Load w_{SDL} = 20.00 plf; w_{SDL} = 15.64 plf Live Load w_{LL} = 50.00 plf; w_{LL} = 10.9 plf Span Length L = 26 ft; L = 26 ft Stiffness $EI_{eff,comp\_SLS}$ = 1636579 kip-in^2; $EI_{eff,comp\_SLS\_LT}$ = 759808.0 kip-in^2	
	Short-Term Deflections	
Dead Load	$\Delta_{DL}=\dfrac{5w_{DL}L^4}{384EI_{eff,comp\_SLS}}=\dfrac{5(54.5\,plf)(26\,ft)^4}{384(1636578.9\,kip\cdot in^2)}=$ 0.3424 in	
Superimposed Dead Load	$\Delta_{SDL}=\dfrac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS}}=\dfrac{5(20\,plf)(26\,ft)^4}{384(1636578.9\,kip\cdot in^2)}=$ 0.1257 in	
Live Load	$\Delta_{LL}=\dfrac{5w_{LL}L^4}{384EI_{eff,comp\_SLS}}=\dfrac{5(50\,plf)(26\,ft)^4}{384(1636578.9\,kip-in^2)}=$ 0.3141 in	
	Servicablility Check	
Allowable LL Deflection	$\Delta_{allow,LL}=\dfrac{L}{360}=\dfrac{(26ft)(12\,in)}{360(1\,ft)}=$ 0.87 in	IBC Table 1604.3
Check	$\Delta_{LL}\le\ \Delta_{allow,LL}\rightarrow\Delta_{LL}=0.314\,in\ <\ \Delta_{allow,LL}=0.87\,in$ ∴ ACCEPTABLE	
	Long-Term Deflections	
Dead Load	$\Delta_{DL\_LT}=\dfrac{5w_{DL}L^4}{384EI_{eff,comp\_SLS\_LT}}=\dfrac{5(54.5\,plf)(26\,ft)^4}{384(759808\,kip\cdot in^2)}=$ 0.73751 in	
Superimposed Dead Load	$\Delta_{SDL\_LT}=\dfrac{5w_{SDL}L^4}{384EI_{eff,comp\_SLS\_LT}}=\dfrac{5(15.6\,plf)(26\,ft)^4}{384(759808\,kip\cdot in^2)}=$ 0.21165 in	
Live Load	$\Delta_{LL\_LT}=\dfrac{5w_{LL}L^4}{384EI_{eff,comp\_SLS\_LT}}=\dfrac{5(10.9\,plf)(26\,ft)^4}{384(759808kip\cdot in^2)}=$ 0.1475 in	
	Servicablility Check Deflection due to short-term $\Delta_{ST}=$ 0.3141 in Deflection due to long-term $\Delta_{LT}=$ 0.94916 in $\Delta_{LT}=\Delta_{DL\_LT}+\Delta_{SDL\_LT}$ Deflection due to Total Load $\Delta_{TL}=$ 1.2633 in $\Delta_{TL}=\Delta_{LT}+\Delta_{ST}$	
	$\Delta_{allow,TL}=\dfrac{L}{240}=\dfrac{(26ft)(12\,in)}{240(1\,ft)}=$ 1.30 in	IBC Table 1604.3
	$\Delta_{TL}\le\ \Delta_{allow,TL}\rightarrow\Delta_{TL}=1.263\,in\ <\ \Delta_{allow,TL}=1.30\,in$ ∴ ACCEPTABLE	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	EFFECTIVE STIFFNESS	
General Values	Gamma Span Length $L = L_0 =$ 26 ft Connector slip Modulus $K_U =$ 291.994 kN/mm = 1667.33 kips/in Connector Spacing $s_{eff} =$ 63 in	Annex B of EN 1995-1-1
Concrete Properties	Slab Depth $h_1 =$ 2.75 in Compressive Strength $f'_c =$ 4000 psi Weight of Concete $w_c =$ 150 pcf	
CLT Properties	Clt Height $h_2 =$ 6.90 in $EA_{eff,CLT} =$ 90969.6 kip $EI_{eff,CLT} =$ 471490 kip-in^2	CLT Handbook EQ 24 & 25
Modulus of Elasticity	$E_{1\_ULS} = w_c{}^{1.5} 33\sqrt{f'_c}$ $E_{1\_ULS} = (150\ pcf)^{1.5} 33\sqrt{4000\ psi} =$ 3834.25 ksi	ACI 318, 19.2.2.1.a
Area of Concrete	$A_{c\_ULS} = h_c b_c$ $A_{c\_ULS} = (2.75\ in)(12\ in) =$ 33 in^2	EN 1995.1-1 Annex B EQ 3.2
Inhertia of Concrete	$I_{1\_ULS} = b_1 h_1{}^2/12$ $I_{1\_ULS} = (12\ in)(2.75\ in)^2/12 =$ 20.8 in^4	EN 1995.1-1 Annex B EQ 3.3
	$E_c A_{c\_ULS} = (3834.3\ ksi)(33\ in^2) =$ 126530 kip $E_c I_{c\_ULS} = (38343\ ksi)(20.8\ in^4) =$ 79740.5 kip-in^2	
Gamma Factor	$\gamma_{1\_ULS} = \left[1 + \frac{\pi^2 E_1 A_1 s_1}{K_1 L_0{}^2}\right]^{-1}$ $\gamma_{1\_ULS} = \left[1 + \frac{\pi^2 (3834.3\ ksi)(33\ in^2)(63\ in)}{(1667\ \ kip/in)(26\ ft)^2}\right]^{-1} =$ 0.67352	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2\_ULS} = \frac{\gamma_1 E_1 A_1 (h_1 + h_2)}{2\gamma_1 E_1 A_1 + 2\gamma_2 EA_{eff}}$ $a_{2\_ULS} = \frac{(0.6735)(3834\ ksi)(33\ in^2)(2.75\ in + 6.90\ in)}{2(0.6735)(3834\ ksi)(33\ in^2) + 2(1.0)(90970\ kip)} =$ 2.33379 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1\_ULS} = \frac{h_1 + h_2}{2} - a_{2\_ULS}$ $a_{1\_ULS} = \frac{6.90\ in + 2.75\ in}{2} - 2.33 =$ 2.49121 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_ULS} = (E_1 I_1 + \gamma_1 E_1 A_1 a_1{}^2) + (EI_{eff} + \gamma_2 EA_{eff} a_2{}^2)$ $(EI)_{eff,comp\_ULS} =$ 608634.1 kip-in^2 + 966961 kip-in^2 = 1575595 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Compsite & CLT Effective	$Ratio_{CLT} = \frac{EI_{eff,comp\_ULS}}{EI_{eff,CLT\_ULS}}$ $Ratio_{CLT} = \frac{1575595\ kip \cdot in^2}{471489.9\ kip \cdot in^2} =$ 3.342	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	APPARENT STIFFNESS	
Bare CLT Stiffness	Length $L = 26$ ft K Value $K_s = 11.5$ $EI_{eff,clt} = 471490$ kip-in^2 $GA_{eff,clt} = 1075.5$ kips	
	$EI_{app,CLT\_ULS} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{471489.9\ kip \cdot in^2}{1+\frac{(11.5)(471489.9\ kip \cdot in^2)}{(1075.5\ kip)(26\ ft)^2}} =$ 448274 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{448274.5\ kip \cdot in^2}{471489.9\ kip \cdot in^2} =$ 95.1%	
Composite Section Stiffness	$L = 26$ ft $K_s = 11.5$ $EI_{eff,comp} = 1575595$ kip-in^2 $GA_{eff,clt} = 1075.5$ kips	
	$EI_{app,comp\_ULS} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{1575595\ kip \cdot in^2}{1+\frac{(11.5)(1575595\ kip \cdot in^2)}{(1075.5\ kip)(26\ ft)^2}} =$ 1343146 kip-in^2	CLT Handbook EQ 35
	$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{1343146\ kip \cdot in^2}{1575595\ kip \cdot in^2} =$ 85.2%	
	$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{1343146\ kip \cdot in^2}{448274.5\ kip \cdot in^2} =$ 2.996	
Apparent Floor Stiffness	$L = 26$ ft $EI_{app,CLT} = 448274$ kip-in^2 $EI_{app,comp} = 1343146$ kip-in^2	
	$K_{app,CLT\_ULS} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$	
	$K_{app,CLT\_ULS} = \frac{48(448274.5 kip \cdot in^2)}{(26\ ft)^3} =$ 0.70847 kip/in	
	$K_{app,comp\_ULS} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$	
	$K_{app,comp\_ULS} = \frac{48(1343146\ kip \cdot in^2)}{(26\ ft)^3} =$ 2.12276 kip/in	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	BENDING STRESSES & STRAINS	
Ultimate Load Demands	Length L = 26 ft	
	Shear V_u = 2.20 kip	
	Moment M_u = 171.77 kip-in	
	Concrete / CLT	
	Modulus of Elasticity E_1 = 3834.25 ksi; E_2 = 1800 ksi	
	Height h_1 = 2.75 in; h_2 = 6.90 in	
	Centroid a_1 = 2.49121 in; a_2 = 2.33379 in	
	Gamma Factor γ_1 = 0.67352; γ_2 = 1	
	$EI_{eff,comp}$ = 1575595 kip-in^2	
	Bending Stress Calculations	
Average Concrete Stress	$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.674)(3834\,ksi)(2.49\,in)(171\,kip \cdot in)}{1575595\,kip \cdot in^2} =$ 0.701 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(3834\,ksi)(2.75\,in)(171\,kip \cdot in)}{1575595\,kip \cdot in^2} =$ 0.575 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of Concrete	$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.701 + 0.575 =$ 1.276 ksi	
Stress at Bottom of Concrete	$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{+\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.701 - 0.575 =$ 0.127 ksi	
Average CLT Stress	$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(1800\,ksi)(2.33\,in)(171\,kip \cdot in)}{1575595\,kip \cdot in^2} =$ 0.458 ksi	EN 1995.1-1 Annex B EQ 3.7
Extreme Fiber Stress	$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(1800\,ksi)(6.90\,in)(171\,kip \cdot in)}{1575595\,kip \cdot in^2} =$ 0.677 ksi	EN 1995.1-1 Annex B EQ 3.8
Stress at Top of CLT	$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} + \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.459 + 0.677 =$ 0.219 ksi	
Stress at Bottom of CLT	$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.46 - 0.677 =$ -1.13 ksi	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	STRENGTH ANALYSIS	
	Normal Stress in Concrete σ_1 = 701.4 psi	
Concrete	Allow Comp Strength of Conc F_c = 2000 psi $F_c = f_c'/2$	
	$\sigma_1 < F_c \rightarrow \sigma_1 = 701.4 psi < F_c = 2000\, psi$ ∴ ACCEPTABLE	
Timber	Bending Strength F_b = 2100 psi	
	Format Conversion Factor K_F = 2.54	
	Resistance Factor Φ = 0.85	
	Design Bending Strength	
	$F_b' = \Phi F_b K_F = 0.85(2100\, psi)(2.54) =$ 4533.9 psi	
	Tension Strength F_t = 1575 psi	
	Format Conversion Factor K_F = 2.70	
	Resistance Factor Φ = 0.80	
	Design Tension Strength	
	$F_t' = \Phi F_t K_F = 0.80(1575\, psi)(2.70) =$ 3402.0 psi	
	Average CLT Stress σ_2 = 458.0 psi	
	Extreme Fiber Stress $\sigma_{m,2}$ = 677.0 psi	
	$\frac{\sigma_2}{F_t'} + \frac{\sigma_{m,2}}{F_b'} \leq 1 \rightarrow \frac{458\, psi}{3402\, psi} + \frac{677\, psi}{4534\, psi} =$ 0.284 < 1.0 ∴ ACCEPTABLE	
	Shear Strength F_v = 160 psi	
	Format Conversion Factor K_F = 2.88	
	Resistance Factor Φ = 0.75	
	Design Shear Strength	
	$F_v' = \phi F_v K_F = 0.75(160\, psi)(2.88) =$ 345.6 psi	
	CLT Modulus of Elasticity E_2 = 1800 ksi	
	NA of timber h = 5.78 in	
	Shear V = 2.20 kip	
	$EI_{eff,comp}$ = 1575595 kip-in^2	
	$\tau_{2,max} = \frac{E_2 h^2 V_u}{2(EI)_{eff,comp}} = \frac{(1800\, ksi)(5.78\, in)^2(2.20\, kip)}{2(1575595\, kip \cdot in^2)} =$ 42.08 psi	EN 1995.1-1 Annex B EQ 3.9
	$\tau_{2,max} < F_v' \rightarrow \tau_{2,max} = 42.08\, psi < F_v' = 345.6\, psi$ ∴ ACCEPTABLE	

STEP DESCRIPTION	COMPUTATION	REFERNCE
Fastener Load Calculation	STRENGTH ANALYSIS F_{ULT} = 76.5828 kips Gamma Factor γ_1 = 0.67352 ksi Concrete Modulus of Elasticity E_1 = 3834.25 in Concrete Area A_1 = 33.00 in^2 Composite Centroid a_1 = 2.49 in Fastener Spacing s = 63 in Ultimate Shear Loading V = 2.2022 kip $EI_{eff,comp\_ULS}$ = 1575595 kip-in^2 $F_q = \frac{\gamma_1 E_1 A_1 a_1 s V}{EI_{eff,comp}}$ $F_q = \frac{(0.676)(3834\ ksi)(33 in^2)(2.49\ in)(63\ in)(2.20\ kip)}{1575595\ kip \cdot in^2} =$ 18.69 kip $F_q < F_{ULT} \rightarrow F_q = 18.69\ kip < F_{ULT} = 76.6\ kip$ ∴ ACCEPTABLE	EN 1995.1-1 Annex B EQ 3.10

STEP DESCRIPTION	COMPUTATION	REFERNCE
	EFFECTIVE STIFFNESS	
	Deformation factor for long term	
	loading for concrete k_{def_c} = 2.5	EN 1995.1-1
	loading for timber k_{def_t} = 0.9	
	loading for HBV k_{def_HBV} = 0.6	
	Stiffness reduction for ULS ψ_2 = 0.3	
	Effective Spacing between s_{eff} = 1600 mm = 63 in	ETA-13/0029 Table 2.1
	Gamma Span Length L_0 = 26 ft	
	Connector slip Modulus K_{U_LT} = 247.453 kN/mm = 1412.99 kips/in	
	Concrete / CLT	
	Modulus of Elasticity E_1 = 3834.25 ksi; E_2 = 1800 ksi	
	Area A_1 = 33 in^2; A_1 = 50.5387 in^2	
	Momemnt of Inertia I_1 = 20.7969 in^4; I_1 = 261.939 in^4	
	Height h_1 = 2.75 in; h_2 = 6.90 in	
	EA_{eff,CLT_ULT_LT} = 71629.6 in	CLT Handbook EQ 24 & 25
	EI_{eff,CLT_ULT_LT} = 371252 kip-in^2	
Modulus of Adjustment for long term loading	$E_{1,ULT\_LT} = \frac{E_1}{1+\psi_2 k_{def\_c}} =$ 2191 ksi; $E_{2,ULT\_LT} = \frac{E_2}{1+\psi_2 k_{def\_t}} =$ 1417.32 ksi	EN 1995.1-1
Gamma Factor	$\gamma_{1_{ULT\_LT}} = \left[1+\frac{\pi^2 E_{1,ULT\_LT} A_1 s_{eff}}{K_{ULT\_LT} {L_0}^2}\right]^{-1}$ $\gamma_{1,ULT\_LT} = \left[1+\frac{\pi^2(2191\ ksi)(33\ in^2)(63\ in)}{(1413\ \ kip/in)(26\ ft)^2}\right]^{-1} =$ 0.754	EN 1995.1-1 Annex B EQ 3.5
Timber to Composite Centroid	$a_{2,ULT\_LT} = \frac{\gamma_{1\_ULT\_LT} E_{1,ULT\_LT} A_1 (h_1+h_2)}{2\gamma_{1,ULT\_LT} E_{1,ULT\_LT} A_1 + 2\gamma_2 EA_{eff,CLT\_ULT\_LT}}$ $a_{2,ULT\_LT} = \frac{(0.754)(2191.0\ ksi)(33\ in^2)(2.75\ in+6.90\ in)}{2(0.754)(2191.0 ksi)(33\ in^2)+2(1.0)(71629.6\ kip)} =$ 2.085 in	EN 1995.1-1 Annex B EQ 3.6
Concrete to Composite Centroid	$a_{1,ULT\_LT} = \frac{h_1+h_2}{2} - a_{2\_ULT}$ $a_{1,ULT\_LT} = \frac{6.90\ in + 2.75\ in}{2} - 2.09 =$ 2.74 in	
Effective Comp Stiffness	$(EI)_{eff,comp\_ULT\_LT} = \left(E_{1,ULT\_LT} I_1 + \gamma_{1,ULT\_LT} E_{1,ULT\_LT} A_1 {a_{1,ULT\_LT}}^2\right) + \left(EI_{eff} + \gamma_2 EA_{eff} {a_{2,ULT}}^2\right)$ $(EI)_{eff,comp\_ULT\_LT} =$ 454764 kip-in^2 + 682550 kip-in^2 $(EI)_{eff,comp\_ULT\_LT} =$ 1137314 kip-in^2	EN 1995.1-1 Annex B EQ 3.1
Ratio of Compsite & CLT Effective	$Ratio_{CLT\_ULT} = \frac{EI_{eff,comp\_ULT\_LT}}{EI_{eff,CLT\_ULT\_LT}}$ $Ratio_{CLT\_ULT} = \frac{1137314\ kip \cdot in^2}{371251.9\ kip \cdot in^2} =$ 15.88	

STEP DESCRIPTION	COMPUTATION	REFERNCE
	APPARENT STIFFNESS	
Bare CLT Stiffness	Length L = 26 ft K Value K_s = 11.5 EI_{eff,CLT_ULT_LT} = 371252 kip-in^2 GA_{eff,CLT_ULT_LT} = 846.9 kips	
	$$EI_{app,CLT\_ULS\_LT} = \frac{EI_{eff,CLT}}{1+\frac{K_s EI_{eff,CLT}}{GA_{eff}L^2}} = \frac{371252\ kip\cdot in^2}{1+\frac{(11.5)(371252\ kip\cdot in^2)}{(846.9\ kip)(26\ ft)^2}} = \quad 352972 \text{ kip-in}^2$$	CLT Handbook EQ 35
	$$Eff\ \% = \frac{EI_{app,CLT}}{EI_{eff,CLT}} = \frac{3452972.1\ kip\cdot in^2}{371251.9\ kip\cdot in^2} = \quad 95.1\%$$	
Composite Section Stiffness	L = 26 ft K_s = 11.5 $EI_{eff,comp}$ = 1137314 kip-in^2 $GA_{eff,clt}$ = 846.9 kips	
	$$EI_{app,comp\_ULS\_LT} = \frac{EI_{eff,comp}}{1+\frac{K_s EI_{eff,comp}}{GA_{eff}L^2}} = \frac{11137314\ kip\cdot in^2}{1+\frac{(11.5)(11137314 kip\cdot in^2)}{(846.9\ kip)(26\ ft)^2}} = \quad 981585 \text{ kip-in}^2$$	CLT Handbook EQ 35
	$$Eff\ \% = \frac{EI_{app,,comp}}{EI_{eff,comp}} = \frac{981584.5\ kip\cdot in^2}{1137314\ kip\cdot in^2} = \quad 86.3\%$$	
	$$Ratio_{CLT} = \frac{EI_{app,comp}}{EI_{app,cLT}} = \frac{98154.5\ kip\cdot in^2}{352972.1\ kip\cdot in^2} = \quad 2.781$$	
Apparent Floor Stiffness	L = 26 ft $EI_{app,CLT}$ = 352972 kip-in^2 $EI_{app,comp}$ = 981585 kip-in^2	
	$$K_{app,CLT\_ULS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,CLT}}{L^3}$$	
	$$K_{app,CLT\_ULS\_LT} = \frac{48(352972.1\ kip\cdot in^2)}{(26\ ft)^3} = \quad 0.55785 \text{ kip/in}$$	
	$$K_{app,comp\_ULS\_LT} = \frac{P}{\Delta} = \frac{48EI_{app,comp}}{L^3}$$	
	$$K_{app,comp\_ULS\_LT} = \frac{48(981584.5\ kip\cdot in^2)}{(26\ ft)^3} = \quad 1.55133 \text{ kip/in}$$	

<table>
<tr><th>STEP DESCRIPTION</th><th>COMPUTATION</th><th>REFERNCE</th></tr>
<tr><td></td><td>BENDING STRESSES & STRAINS</td><td></td></tr>
<tr><td>Ultimate Load Demands</td><td>Length $L = 26$ ft
Shear $V_u = 2.20$ kip
Moment $M_u = 171.77$ kip-in

Concrete / CLT
Modulus of Elasticity $E_1 = 2191$ ksi, $E_2 = 1417.32$ ksi
Height $h_1 = 2.75$ in, $h_2 = 6.90$ in
Centroid $a_1 = 2.74031$ in, $a_2 = 2.08469$ in
Gamma Factor $\gamma_1 = 0.75366$, $\gamma_2 = 1$
$EI_{eff,comp} = 1137314$ kip-in^2

Bending Stess Calulations</td><td></td></tr>
<tr><td>Average Concrete Stress</td><td>$$\sigma_1 = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} = \frac{(0.754)(2191\ ksi)(2.74\ in)(172\ kip \cdot in)}{1137314\ \ kip \cdot in^2} = \quad 0.683\ \text{ksi}$$</td><td>EN 1995.1-1 Annex B EQ 3.7</td></tr>
<tr><td>Extreme Fiber Stress</td><td>$$\sigma_{m,1} = \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = \frac{(0.5)(2191\ ksi)(2.75\ in)(172\ kip \cdot in)}{1137314\ kip \cdot in^2} = \quad 0.455\ \text{ksi}$$</td><td>EN 1995.1-1 Annex B EQ 3.8</td></tr>
<tr><td>Stress at Top of Concrete</td><td>$$\sigma_{t,1} = \sigma_1 + \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} + \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.683 + 0.455 = \quad 1.138\ \text{ksi}$$</td><td></td></tr>
<tr><td>Stress at Bottom of Concrete</td><td>$$\sigma_{b,1} = \sigma_1 - \sigma_{m,1} = \frac{\gamma_1 E_1 a_1 M_u}{(EI)_{eff}} - \frac{0.5 E_1 h_1 M_u}{(EI)_{eff}} = 0.683 - 0.455 = \quad 0.228\ \text{ksi}$$</td><td></td></tr>
<tr><td>Average CLT Stress</td><td>$$\sigma_2 = \frac{\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} = \frac{(1.0)(1417.3\ ksi)(2.09\ in)(172\ kip \cdot in)}{1137314\ kip \cdot in^2} = \quad 0.446\ \text{ksi}$$</td><td>EN 1995.1-1 Annex B EQ 3.7</td></tr>
<tr><td>Extreme Fiber Stress</td><td>$$\sigma_{m,2} = \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = \frac{(0.5)(1417\ ksi)(6.90\ in)(172\ kip \cdot in)}{1137314 kip \cdot in^2} = \quad 0.739\ \text{ksi}$$</td><td>EN 1995.1-1 Annex B EQ 3.8</td></tr>
<tr><td>Stress at Top of CLT</td><td>$$\sigma_{t,2} = -\sigma_2 + \sigma_{m,2} = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} + \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.446 + 0.739 = \quad 0.292\ \text{ksi}$$</td><td></td></tr>
<tr><td>Stress at Bottom of CLT</td><td>$$\sigma_{b,2} = -(\sigma_2 + \sigma_{m,2}) = \frac{-\gamma_2 E_2 a_2 M_u}{(EI)_{eff}} - \frac{0.5 E_2 h_2 M_u}{(EI)_{eff}} = -0.45 - 0.739 = \quad -1.18\ \text{ksi}$$</td><td></td></tr>
</table>

STEP DESCRIPTION	COMPUTATION	REFERNCE
	STRENGTH ANALYSIS	
	Normal Stress in Concrete σ_1 = 683.4 psi	
Concrete	Allow Comp Strength of Conc F_c = 2000 psi $F_c = f_c'/2$	
	$\sigma_1 < F_c \rightarrow \sigma_1 = 683.4\,psi < F_c = 2000\,psi$ ∴ ACCEPTABLE	
Timber	Bending Strength F_b = 2100 psi	
	Format Conversion Factor K_F = 2.54	
	Resistance Factor Φ = 0.85	
	Design Bending Stength	
	$F_b' = \Phi F_b K_F = 0.85(2100\,psi)(2.54) =$ 4533.9 psi	
	Tension Strength F_t = 1575 psi	
	Format Conversion Factor K_F = 2.70	
	Resistance Factor Φ = 0.80	
	Design Tension Stength	
	$F_t' = \Phi F_t K_F = 0.80(1575\,psi)(2.70) =$ 3402.0 psi	
	Average CLT Stress σ_2 = 446.3 psi	
	Extreme Fiber Stress $\sigma_{m,2}$ = 738.5 psi	
	$\frac{\sigma_2}{F_t'} + \frac{\sigma_{m,2}}{F_b'} \leq 1 \rightarrow \frac{446.3\,psi}{3402\,psi} + \frac{738.5\,psi}{4534\,psi} =$ 0.294 < 1.0 ∴ ACCEPTABLE	
	Shear Strength F_v = 160 psi	
	Format Conversion Factor K_F = 2.88	
	Resistance Factor Φ = 0.75	
	Design Shear Stength	
	$F_v' = \phi F_v K_F = 0.75(160\,psi)(2.88) =$ 345.6 psi	
	CLT Modulus of Elasticity E_2 = 1417.32 ksi	
	NA of timber h = 5.53 in	
	Shear V = 2.20 kip	
	$EI_{eff,comp}$ = 1137314 kip-in^2	
	$\tau_{2,max} = \frac{E_2 {h_2}^2 V_u}{2(EI)_{eff,comp}} = \frac{(1417\,ksi)(5.53\,in)^2(2.20\,kip)}{2(1137314\,kip \cdot in^2)} =$ 42.03 psi	EN 1995.1-1 Annex B EQ 3.9
	$\tau_{2,max} < F_v' \rightarrow \tau_{2,max} = 42.03\,psi < F_v' = 345.6\,psi$ ∴ ACCEPTABLE	

STEP DESCRIPTION	COMPUTATION	REFERNCE
Fastener Load Calculation	STRENGTH ANALYSIS F_{ULT} = 76.5828 kips Gamma Factor γ_1 = 0.75366 ksi Concrete Modulus of Elasticity E_1 = 2191.00 in Concrete Area A_1 = 33.00 in^2 Composite Centroid a_1 = 2.74 in Fastener Spacing s = 63 in Ultimate Shear Loading V = 2.2022 kip $EI_{eff,comp}$ = 1137314 kip-in^2 $F_q = \frac{\gamma_1 E_1 A_1 a_1 sV}{EI_{eff,comp}}$ $F_q = \frac{(0.754)(2191\ ksi)(33in^2)(2.74\ in)(53\ in)(2.20\ kip)}{1137314\ kip \cdot in^2} =$ 18.22 kip $F_q < F_{ULT} \rightarrow F_q = 18.22\ kip < F_{ULT} = 76.6\ kip$ ∴ ACCEPTABLE	EN 1995.1-1 Annex B EQ 3.10

STEP DESCRIPTION	COMPUTATION					REFERNCE
	CONNECTOR EFFICIENCY					
Partial Composite Section	Effective Bending Stiffness	EI_{eff_SLS} =	1636579	in		
	Deflection	Δ_{PC} =	0.31413	in		
Non-Composite Section	Effective Bending Stiffness	EI_{eff_SLS} =	551231	in	(set K_{ser} close to zero)	
	Deflection	Δ_{NC} =	0.93264	in		
Fully Composite	Concrete MOE	E_1 =	3834.25	psi		
	Timber MOE	E_2 =	1800	psi		
	Width	b' =	12	in		
	Width of Transformed Concrete	b' =	25.56	in	$b' = b(E_1/E_2)$	
	Height of Timber	h_1 =	2.75	in		
	Height of Concrete	h_2 =	6.90	in		
	Bottom to Centroid of					
	Transformed Concrete	y'_1 =	8.275	in		
	Timber Section	y_2 =	3.45	in		
	Area of Transformed Concrete	A'_1 =	70.29	in^2		
	Area of Timber Section	A_2 =	82.8	in^2		
	Moment of Inertia of					
	Transformed Concrete	I'_1 =	44.30	in^4		
	Timber Section	I_2 =	328.51	in^4		
	Centroid of Transformed Section					
	$y_c = \frac{y'_1 A'_1 + y_2 A_2}{A'_1 + A_2}$	=	5.665	in		
	Distance to Concrete Centroid	d_1 =	4.29	in	$d = \left\| y_c - \frac{h}{2} \right\|$	
	Distance to Timber Centroid	d_2 =	0.84	in		
	Transformed Section					
	Moment of Inertia about NA	I_{NA} =	1725.27	in^4	$I_{NA} = \sum I_i + A_i {d_i}^2$	
	Bending Stiffness	$E_2 I_{NA}$ =	3105482	k-in^2		
	Deflection of Fully Composite Section					
	$\Delta_{FC} = \frac{5wL^4}{384E_2 I_{NA}} = \frac{5(50\,plf)(26ft)^4}{384(3105482\,k \cdot in^2)} =$		0.166	in		
Shear Connector Efficiency	$Efficiency = \frac{\Delta_{NC} - \Delta_{PC}}{\Delta_{NC} - \Delta_{FC}} = \frac{0.9326" - 0.314"}{0.9326" - 0.166"} =$		80.6%			