Structural Design of a Cross-Laminated Timber (CLT) Single-Family Home

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February 2022
# Table of Contents

1. Introduction .................................................................................................................. 1
   1.1. Introduction to the project ...................................................................................... 1
   1.2. Introduction to the material ................................................................................... 2

2. Applicable Codes and Standards .................................................................................... 6

3. Single-Family CLT Home Design ................................................................................... 7
   3.1 Introduction to Design ............................................................................................. 7
   3.2 Preliminary Design .................................................................................................... 8
   3.3 Wall Panel Design ...................................................................................................... 13
   3.4 Floor and Roof Panel Design ................................................................................... 19
   3.5 Lateral Force-Resistance System (LFRS) Design ....................................................... 22
   3.6 Foundation Design .................................................................................................... 33
   3.7 Connections ............................................................................................................... 35

4. Conclusions .................................................................................................................... 40

5. References .................................................................................................................... 43

Appendix A – Structural Drawings
Appendix B – Structural Design Calculations
Appendix C – Supplementary Design Calculations
Appendix D – Reference Home Plans
Preface

Many in the Architectural/Engineering/Construction (AEC) community have shown interest in using Cross-Laminated Timber (CLT) as a structural building material. CLT is an aesthetically pleasing, warm mass-timber panelized product that offers users a cost-effective, renewable, durable, fire-resistant alternative to traditional building materials, such as masonry, concrete, and light-framing. A significant benefit to developers and community stockholders in the USA is that the raw materials required to produce CLT can be obtained domestically in timber rich rural areas, helping job growth in those areas, shortening supply chains, and reducing reliance on imported materials. Additionally, CLT, being a prefabricated product, gives users access to all the advantages offered by off-site construction methods such as factory quality control, just-in-time delivery, and accelerated construction.

CLT is currently utilized in multi-family residential structures, but it is not widely used for the construction of single-family residences. The cost of the fabricated CLT panels and shipping most often prohibits its use in conventional single-family home design. Another factor discouraging the use of the material in the single-family residential construction sector is that there is limited design aids and prescriptive guidance available for use by engineers.

Additionally, single-family residential projects when compared to larger commercial projects require very little CLT material, so for the manufacturer to justify the economy of such a small order, they may require designers to put forth extra effort and fully design both the panel specification and layout. This increases both the front-end design time and cost, which may be unacceptable for any given single-family project. This is a significant problem for those who would like to consider using CLT for their single-family project and the reason why this report was created. In this report, a CLT structural system alternative design is presented for a single-family residence previously designed using conventional light-framing methods. In this report, the CLT design methodologies, design references, applicable codes, structural analysis, and complete structural design calculations of the CLT panels are presented. The report also points out to potential challenges and shortcomings. Overall, the report offers a unique reference to CLT home design for practicing professionals and researchers.
Acknowledgements

This research was supported by the Pennsylvania Housing Research Center (PHRC). This support is gratefully acknowledged. The opinions expressed in the report are those of the authors only and do not necessarily reflect those of PHRC.

Disclaimer

The material presented in this report is for public information on the subject and the results of the study shall not be relied upon under any circumstances for any specific application or actual projects involving the materials, components, and systems mentioned without consultation by a licensed design professional experienced in the field. Anyone using the material in this report assumes all liability resulting from such use, and the authors, Penn State, or PHRC are not in any way liable for such use.
1. Introduction

1.1. Introduction to the project

In this report, the structural design of a typical single-family residence using CLT panelized construction is presented. CLT is currently more commonly utilized for the construction of multi-family residential and commercial structures; however, some examples of CLT (single-family) homes can be seen (Karacabeyli and Douglas 2013). Two-dimensional flat CLT panel elements make it possible for architects to explore unique, attractive structure forms and floorplans not easily constructed using light-frame methods. Structural engineers can leverage the stiffness and two-way spanning capabilities of the panels to tackle difficult-to-solve design challenges presented by modern-style structures. Although it is largely cost-prohibitive to construct conventional single-family homes using CLT, as production methods mature and availability of design guidance increases, the opportunities may expand. It benefits structural engineers involved in residential construction to be aware of the potential uses for the material, the design resources available and to have a basic understanding of the typical design methodologies and regulatory environment. The goal of this report is to present these items along with a design example to serve as a guide for this type of construction.

The report is organized into the main body and the appendices. In the main body, CLT is introduced as an emerging building construction material. The introduction to the material is kept brief since there is ample information published on this topic already. Next is a short discussion regarding the current regulatory environment, followed by a discussion of the structural design and the presentation of the detailed example. The appendix is broken into three sections. Generalized structural drawings are in Appendix A, full design calculations are in Appendix B, and supplementary design calculations are in Appendix C.

The structural design was performed on a model home provided by a local home builder (S&A), where their design drawings are shown in Appendix D. Previously, a structural design was completed for this same residence using light-frame construction methods. The design was published in 2009 as a chapter in the book titled, “Timber Buildings and Sustainability” (Jellen and Memari 2019). This report is intended as a follow-up to the original design report to present
design of the same residence using an alternative structural system. The intention was to identify benefits and challenges associated with the use of the alternative system.

1.2. Introduction to the material

According the CLT Handbook (Karacabeyli and Douglas 2013 chap. 2), CLT is defined as a prefabricated solid engineered wood product made of at least three orthogonally bonded layers of solid-sawn lumber or structural composite lumber (SCL) that are laminated by gluing of longitudinal and transverse layers with structural adhesives to form a solid rectangular-shaped, straight, and plane timber intended for roof, floor or wall applications (Figure 1).

![Figure 1. Isometric view of a three-layer piece of CLT construction material (Wikimedia Commons contributors 2021).](image)

CLT is manufactured and identified according to ANSI/APA PRG 320 (APA 2020). Engineers utilizing CLT should be familiar with PRG 320. In addition to the testing and manufacturing requirements discussed, this standard also defines the terminology, symbology, grades, and reference design values, which are used throughout the industry. The reference design values can be used for preliminary design if no other information is available; however, manufacturers are required by the standard to publish their own panel specific data. There are currently several
manufacturers that service projects located in the United States. The Engineered Wood Association (APA) lists the major manufacturers along with their product testing reports online ("Manufacturer Directory" 2021.). Most of the manufacturers listed are in the western region of the United States and Canada; However, Nordic and International Beams (IB) have facilities in Quebec and Alabama, respectively. In addition to the manufacturer directory, the APA provides many free downloads for CLT case studies and informational guides.

As a building construction material, CLT is primarily used to prefabricate two-dimensional (2D) load carrying panels that are used as components in floor, roof and wall assemblies (Karacabeyli and Douglas 2013). With proper design, CLT can be used in Type III, IV and V construction as classified by the IBC (Breneman et al. 2019). Examples provided in the 2021 Mass Timber Design manual (Mass Timber Design Manual 2021) show CLT used as an alternative to masonry in multi-story residential or office buildings. Both the U.S. edition (Karacabeyli and Douglas 2013) and the Swedish edition (Borgstrom and Frobel 2019) also show examples of CLT being utilized for the construction of single-family dwellings. In addition, CLT panels can be used to construct elevator and stair shafts as described in a recent white paper published by SmartLam (SMARTLAM 2020). CLT offers the following benefits as a construction material based on the indicated references:

1. According to Borgstrom and Frobel (2019):
   a. High strength-to-weight ratio reduces structure weight, which, in turn, can lower shipping, assembly and foundation costs
   b. Small manufacturing tolerances and good dimensional stability
   c. Retains some load bearing capacity during a fire event
   d. Highly Flexible Large Format Wooden Panel
      i. Factory equipment can produce unique shapes of 2D panels
      ii. panels can be produced with accurate placement of openings
   e. Factory surface finishes can be provided

   a. Safe and Reliable
      i. Proven strength, stiffness, and ductility
      ii. Acceptance by the building code as a construction material
      iii. Can be utilized in fire resistant assemblies
      iv. Good seismic performance
b. Speed of construction can result in cost-savings, schedule savings and early return-on-investment (ROI)

c. Sustainable, healthy material
   i. Low carbon material
   ii. Renewable
   iii. Linked to improved indoor air quality and occupant wellbeing

d. Aesthetically pleasing

e. Lower thermal conductivity compared to masonry, concrete and steel construction

   a. Design Flexibility
   b. Environmental Advantages
      i. Sustainable managed forests
      ii. Renewable
      iii. Outperforms steel and concrete in terms of life-cycle analysis and embodied energy, air pollution and water pollution
      iv. Potentially lower carbon footprint
   c. Less waste
   d. Fire protection
   e. Seismic Performance
      i. Good ductile behavior and energy dissipation
   f. Structural
      i. High axial load-capacity for walls
      ii. Less susceptible to buckling
      iii. High stiffness/strength-to-mass ratio
      iv. High shear strength
      v. Less susceptible to effects of soft-story failure than other platform-type structural systems
      vi. Excellent floor/roof span-to-depth ratios
      vii. Quick, efficient, fast installation

   a. Can be cost competitive with certain concrete, masonry, and steel building types
The following are disadvantages of CLT as a construction material based on the above references:

1. Can have higher costs than competing masonry, concrete, or steel construction
2. Higher relative costs when using low amounts of the material
3. Building code restrictions on timber building heights
4. Mechanical, Electrical, and plumbing costs can be higher due to lack of building cavities
5. Transportation costs can be higher due to the limited number of suppliers
6. Acoustic challenges
7. Vibration challenges
8. Cost can be sensitive to connection detailing
2. Applicable Codes and Standards

The design basis for most conventionally light-framed single-family dwellings is the International Residential Code (IRC). Manufactured and identified in accordance with ANSI/APA PRG 320 (APA 2020), CLT was first recognized in the 2015 IRC (2015 IRC 2015 IRC 2014) as a construction material for walls and floors; however, no prescriptive guidance was provided in the 2015 residential building code or the latest 2021 version. Construction methods utilizing CLT are regulated by Section R301.1.3 as engineered designs and are based on the locally approved version of the International Building Code (IBC).

The 2015 IBC (International Code Council 2014) was the first building code version to incorporate CLT design guidance for the material. Chapters 5 and 6 of the 2015 IBC permitted the use of CLT in wall and floor assemblies within most building construction type categories (American Wood Council 2015). For structural design, the 2015 IBC references the 2015 NDS (American Wood Council, AWC 2015). Chapter 10 was introduced in the 2015 NDS prescribing reference design values for CLT. Additional design guidance was included throughout the standard where needed. Further developed guidance was provided in the 2018 versions of both the IBC and the NDS; however, both the 2015 and 2018 versions lacked guidance on using CLT for diaphragms and shear walls. The 2021 Special Design Provisions for Wind and Seismic (SDPWS) (American Wood Council 2020) was the first standard to provide engineering design guidance on these topics.

The codification of this material was a major milestone for the CLT industry. The standardization of CLT production accomplished by PRG 320, the acceptance of the construction material into the building code and the introduction of design guidance by the NDS all provided the basis needed to safely mass produce and allow for the specification of the material for use in buildings constructed in the United States. Prior to the codification of the material, it was challenging and risky for developers and designers to utilize CLT in building structure projects. Increased acceptance by legislative bodies and increased availability of design guidance has led to increasing interest by the Architecture, Engineering, Construction (AEC) community in CLT as a building construction material.
3. Single-Family CLT Home Design

3.1 Introduction to Design

Currently, CLT is utilized in more modern avant-garde designs, where designers leverage the long-spanning plate-like nature of the wooden slab element. Some modern examples of single-family dwellings constructed using CLT are presented in both the U.S. edition (Karacabeyli and Douglas 2013) and the Swedish edition (Borgstrom and Frobel 2019) of the CLT Handbook.

In this report, the design of a traditional platform style 2-½-story single-family home using CLT elements and current design resources is discussed. The residence has 8-foot ceiling heights for both the 1st and 2nd story, a basement, attic floor space and bonus floor space above the attached garage. The structural shell of the dwelling, adapted from the light-framed counterpart is shown in Figure 2.

![Figure 2. Rendering of CLT Panelized Home Design.](image)

In this design, the CLT panels are utilized as load-carrying plate elements, which transfer both conventional gravity loads, and wind loads to the concrete foundation. To be consistent with the previous light-frame design, the conventional gravity and wind loads were computed based on a
project location of State College, PA. As with the original design, seismic loads are assumed not to govern the design of the lateral load resisting system. As described in The CLT Handbook (Karacabeyli and Douglas 2013), the dwelling utilizes a platform framing system in which the floor and roof panels bear directly on exterior and interior walls. Floor plans are in Appendix A. Floor and roof panels conduct gravity loads such as dead, floor-live and snow loading through wall panels to foundation. The floor panels also serve as diaphragms that transfer wind loading to designated shear resisting wall panels.

3.2 Preliminary Design

For the purposes of this report, it was decided to use CLT panels for the roof, floor, and walls. Platform construction methods were selected due to their similarity to light-framing methods. The original structure utilized a conventional light-framed platform system consisting of dimensional lumber and structural sheathing load bearing elements. The platform framing method was maintained and CLT panels were substituted for the light-framed roof, floor, and wall assemblies. This one-to-one substitution allowed for the CLT alternate design to proceed with only minor floor plan changes. Platform framed CLT methods are likely not the most economical solution for this design; however, by using this method, it becomes possible to demonstrate not only design of the floor elements, but also the wall elements. In an actual design situation, all the building system options should be considered. The CLT Handbook describes platform and balloon framing systems and in their technical guide.

The panelized model shown in Figure 2 was created in Autodesk Revit. According to the Wood Products Council, creation of a 3-D model is necessary to realize the benefits of a prefabricated mass timber system (Woodworks 2019). The model was used initially to determine the panel layout. Adapting a prefabricated CLT panelized approach to an existing floor plan without modifying dimensions or floor plan can be challenging; however, in this case, the impact of the adaptations was minimal.

To minimize panel waste, it is necessary to consider how the panels will be cut from a master billet. The process of efficiently arranging the various required geometric panel shapes on the master billet, for computer numerical control (CNC) cutting is called nesting and generally is
accomplished by computer software (Kremer 2018). The rectangular shape and compatible dimensions of this structural component reduced the difficulty in efficiently panelizing the existing design; however, irregular shape buildings with dimensions not compatible with typical CLT panel dimensions can be difficult to optimize.

Figure 3. First floor plan.

Upon reviewing the geometry of the building, an 8-foot primary panel module (width) was established as the basis for panelization. According to the Engineered Wood Association (APA), typical panel widths for CLT are 2-feet, 4-feet, 8-feet, and 10-feet (APA 2019) with lengths up to 60-feet. It was necessary to consider both the geometry of the main building and the garage when considering a primary panel module. The main exterior dimensions of the building are shown in Figure 3. The factors that influenced the selection of the 8-foot module are as follows:

1. Light-framed construction is typically designed using a 4-foot or 8-foot module, because much of the material used for construction, such as structural sheathing, is manufactured in these widths. An 8-foot module is a logical choice for adapting a light-framed structure for use with CLT panels.
2. Ceiling height for both the 1st and 2nd stories are both 8-feet; therefore, it was logical to select the 8-foot module for the interior and exterior wall panels.

3. The length of the main building is 36-feet; therefore, 4 ½ panels per floor are required. Half-sections could be utilized on other floors.

4. Three 8-foot panels equal to 24-feet can be placed spanning the short 22-foot direction for the garage floor/ceiling structure.

5. The 30-foot width of the building is a convenient and efficient dimension for considering 60 feet long master billets.

![Figure 4. Building section.](image)

CLT walls are used as both interior and exterior load bearing walls. The exterior walls not only transmit axial gravity load, but also transmit in-plane and out-of-plane wind forces. The interior bearing walls transmit gravity load only. As can be seen in detail 7/S-501 located in Appendix A, bypass framing clips, similar to those shown in Figure 5 are utilized to prevent lateral load transfer from the floor diaphragm. To provide usable attic space, the interior wall on the 2nd floor
was utilized for bearing. Unfortunately, as can be seen in Figure 4, this wall does not align with the wall below; therefore, the floor panel below must transfer the interior wall loading through bending action to the supports. This is not an ideal situation; it is better to have the interior bearing walls stacked. Walls bearing within the span of the floor increase the demand of the floor panel on which they bear and could result in increased floor thickness.

**Figure 5.** Simpson Strong-Tie (SST) SC bypass framing side clip connector (Image from SST C-CF-2020 Catalog)

In the preliminary design stage, it is important to understand those items, such as staggered interior bearing walls, that can increase cost in a CLT project. Increased cost, in comparison to light-framing, is the main reason why CLT is not typically utilized for the construction of traditional-style single-family residences. To gain better insight on the factors that contribute to the recognized CLT cost premium and identify good design practices that can reduce costs, a CLT manufacturer was contacted (Spickler 2020). The following is a summary of the discussion points and recommendations from the interview:

1. *Bring a manufacturer on board as soon as possible. Each manufacturer has its own unique product specifications that can affect the geometry and economy of the project.*

2. *The geometry of a project is important when considering CLT. If economic nesting of the manufacturers standard billet sizes is not possible, a high percentage of waste could result.*

3. *Based on the economy of the structural system, CLT does not typically make sense for single-family homes. There is very little CLT material used in a single-family project in*
comparison to larger multi-story commercial projects. This typically results in disproportionately high shipping costs.

4. If considering CLT for a single-family home project, the design team should anticipate that they will be responsible for the panel layout as well as the engineering design of the panels and connections. The design team should anticipate only a production and minor advisory role of the CLT manufacturer.

5. Most CLT manufacturers use Cadwork as their software platform. Structurlam can accept most 3-D model formats. IFC files are commonly utilized.

6. Some CLT projects utilize model-based project submittals, rather than 2-D shop drawings.

7. For a platform framing system, it is better to align bearing walls if possible. Walls bearing at interior location along a panel span can result in increased panel thicknesses due to increased loading or increased long-term deflection potential.

8. When selecting CLT floor or roof panels, it makes sense to start at the thinnest and lowest grade material option and increase the thickness of the lowest grade material before attempting to increase to a higher-grade material.

The manufacturer’s insight was valuable prior to commencing the design. It is recommended that, if possible, a CLT manufacturer should be consulted prior to considering the use of any prefabricated mass-timber product in a project.

As mentioned previously, conventional external loads were calculated based on the State College, PA area. Local wind and snow loads were obtained from the Applied Technology Council (ATC) Hazards by Location webpage (ATC 2020). A Risk Category II, design wind speed of 115 mph and a ground snow load of 25 psf were obtained from the online service. Tekla Tedds (Tedds) software was then used to determine the Main Wind Force Resisting (MWFR) and Components and Cladding (C&C) wind loading for both the main building and the garage. Tedds was also used to determine balanced, unbalanced, and drifted snow loading for the sloped roofs.

Upon completion of the preliminary design, structural design was conducted to determine actual member and connection specification. Design of the CLT panels was accomplished using a variety of resources and methods each described in their respective sections. Although
preliminary panel design properties can be obtained from PRG 320, it was decided to use manufacturer specific properties since they are readily available. To demonstrate similarities and differences between CLT manufacturers terminology and product offerings, two separate CLT manufacturers were considered; Nordic X-LAM panels were specified for the walls and Katerra panels were specified for the floors and roof. In an actual construction project this would not be the case. Panels would be supplied by one manufacturer.

The structural design was partitioned into sections. The sections include, CLT Wall Panel Design, CLT Floor Panel Design, CLT Lateral Force-Resistance System (LFRS) Design, and lastly the Foundation System review. Connections were designed during the LFRS portion of the design. Allowable stress design (ASD) methodology was primarily used for design of the CLT panels and evaluation of the soil-bearing pressures. Load and Resistance Factor Design (LRFD) methodology was only utilized for the design of the steel beam and concrete foundation elements.

### 3.3 Wall Panel Design

In this section, the initial design and specification of the CLT wall panels is discussed. Final wall verification occurs in the CLT lateral System Design section, when the initial wall selections are analyzed to ensure they can function adequately as shear panels. The wall panels are initially selected based on their capacity to resist the internal axial forces resulting from the application of the prescribed gravity loads and the internal bending forces resulting from the application of out-of-plane wind forces. The primary method of design for the walls was hand calculations. The 2018 NDS (AWC 2017) was utilized as the design basis and the Nordic X-lam Technical Guide (Nordic 2020) was consulted to obtain panel options and design properties.

The goal of this wall panel design was to select the thinnest panel that will resist the design loading. From a structural perspective, CLT wall panels are inefficient (the material is distributed uniformly rather than where needed based on analysis); therefore, it is rational for the designer to want to minimize the use of this expensive material in the walls. Residential wall design loads are relatively small in magnitude compared to those experienced in the walls of taller multi-story structures. Light-framed walls are much more efficient and cost-effective for use in single-family dwellings since they adequately resist the same loading using less material (small wall studs
spaced at intervals and a thin sheathing membrane, rather than a solid piece of thicker wood). From a building enclosure design perspective, the solid wood panels are also problematic when significant building environmental conditioning is required. Thermal bridging is typically an issue and additional cavity framing is often required to provide a location for the insulation.

![Material Design Properties](image)

<table>
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<th>Material Design Properties</th>
<th>CLT stress grade</th>
<th>E1 Layers</th>
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<tr>
<td>Rolling shear modulus, $G_r$ (psi)</td>
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</table>

![Figure 6](image)

**Figure 6.** a.) E1 stress grade reference design values b.) CLT wall panel shown with strong-axis vertical. Images from Nordic (Nordic 2020).

With minimization of the material use in mind, the X-LAM 89-3S panel was initially selected for consideration. The 89-3s is a 3-layer, 3 ½-inch thick panel. The panel is certified according PRG 320 as an E1 stress grade panel. The material design properties for the panel are shown in Figure 6a. Initially, 2nd story wall panel WP-5 (See Appendix A for panel location) was selected for design. It was decided to orient the strong-axis vertical as shown in Figure 6b. Typically wall panels are oriented in this fashion to provide greater bending resistance to out-of-plane wind forces.

Design considerations that influence wall selection are the axial capacity, the out-of-plane bending capacity, and the lintel requirement over openings. WP-5 was selected as a representative panel and the selection of the 2nd floor walls was based on this panel. The panel axial capacity and demand was first determined. An axial demand of 1,213 plf was calculated based on controlling ASD load combination Dead (D) + 0.75 Live (L) + 0.75 Snow (S) + (0.75) 0.6 Wind (W). The 2018 NDS design equations located in Section 3.7 and those in the associated commentary section C3.7 were utilized to calculate the capacity. The 89-3s panel
The column buckling resistance \( (P_{ce}) \) was calculated using the minimum apparent bending stiffness \( (E_{I_{app-min}}) = 0.5184 \ E_{I_{app}} \), as recommended by the CLT handbook section 2.2.2. The apparent bending stiffness, as defined by 2018 NDS Section 10.4.1, was calculated considering a shear deformation factor \( (K_s) \) of 11.8 (pinned support conditions). Other than the material adjustments discussed, design of the CLT panel proceeded as it would for any other wooden compression member. The axial capacity of the 89-3s was calculated to be 29,726 plf, which far exceeds the demand of 1,214 plf.

The unadjusted panel bending capacity was also obtained from the technical guide. Adjusting per the prescribed factors listed in 2018 NDS Table 10.3.1 resulted in a design moment capacity of 5,360 lbf/ft. C&C magnitude wind loading was applied to the panel and a bending demand of 108 lbf-ft was calculated based on ASD load combination 0.6 D + 0.6 W. Once again, the capacity far exceeded the demand. Considering the interaction between axial and bending force, a demand/capacity ratio of 0.023 was calculated using NDS interaction equation C3.9.2-3. The resulting ratio of 0.023 shows that the capacity of the thinnest panel far exceeds the demands. By engineering judgement, no additional strength checks were required.

The final design consideration for the walls was lintel selection for the openings. Proper selection of the lintels proved more challenging than the strength checks. Lintel B-3 associated with panel WP-4 was first selected for analysis. Figure 7 shows the loading for B-3.
Lintel B-3 is loaded uniformly by the 2nd floor and in part by the 2nd story walls, attic floor and roof. The first logical step in selecting a lintel in a CLT wall is to check if the panel can remain continuous and uninterrupted by a discrete header. If the check is satisfactory then the panel can remain continuous without the need for insertion of a stronger beam. Point loads resulting from the B-4 lintel reaction will partly load B-3. Because the wall panel is solid, the assumed distribution of these point loads must be considered. For this project, a 25-degree propagation angle is considered (Gräfe et al. 2018; Wallner-Novak et al. 2017). Some references also suggest distributing the load at 30 degrees with the distribution stopping at a vertical distance of wall-height/4 (Borgstrom and Frobel 2019).

**Figure 7.** B-3 lintel loading
The edgewise design properties for the lintel are shown in Figure 8a. The lintel in this check is oriented as shown in Figure 8b. This orientation is beneficial for resisting out-of-plane wind forces; however, notice the effective bending area listed in Figure 8a for bending about the Z-Z axis. Only the center lamination \((b_{\text{eff},90})\), as shown in Figure 9a, can be used to resist bending forces.

**Figure 8.** a.) Lintel design properties b.) Lintel shown with strong-axis vertical. Images from Nordic (Nordic 2020).

The lintel bending capacity was calculated per the provisions of NDS Section 3. Because the lintel is part of the wall, the boundary conditions will be fixed. Due to the fixed boundary condition, a portion of the bottom of the lintel (i.e., negative moment at connection to the wall) will be in compression; therefore, the beam stability factor \((C_L)\) will not equal 1.0. A slenderness ratio of 60 (NDS Section 3.3) was calculated considering an effective length of \(2.06 \times 6\) feet = 12.36 feet (NDS Table 3.3.3 for uniformly distributed loading) and an effective width \((b_{\text{eff},90})\) of 0.75 inch as directed by the manufacturer. Typically, edgewise reference design values are provided by the manufacturer. If they are not provided the effective bending strength and stiffness can be calculated using analytical methods as described in Mahamid (2020) Section 3.5.4. The calculated slenderness ratio of 60 was greater than the limit of 50 prescribed in NDS Section 3.3.3.6; therefore, it is not possible to utilize the 89-3s panel for a lintel in the strong-axis vertical position.
Figure 9. Lintel and effective width for bending shown for, a.) loading parallel to the outermost layers b.) loading perpendicular to outermost layers.

Slenderness continued to be a concern during the initial evaluation of the lintels. Upon discovering that the 89-3s were inadequate, it was decided to check the wider 105-3s. The 105-3s did meet the bending slenderness criteria; however, the bending strength of the single layer was not adequate. Next, the possibility of utilizing the panels oriented with the strong-axis horizontal, as shown in Figure 9b, was investigated. The lintel bending slenderness concerns were resolved; however, in this new orientation the column slenderness limit set forth in NDS Section 3.7.1.4 were not satisfied. To satisfy the column slenderness limit, with the strong-axis in the horizontal position, a 5-layer, 5 ⅝-inch 143-5s panel was required. The addition of the extra two layers was unacceptable, therefore, it was decided to add joints at the larger openings and utilize independent lintels rather than retain a continuous panel on the 1st floor (See 1/S201 and 1/S202).

Prior to finalizing a wall thickness for the 1st floor, the lintels over the smaller openings on the 2nd floor were investigated. As with the 1st floor, the possibility of using continuous panels, rather than jointed panels was investigated first. The largest panel on the 2nd floor would be approximately 8-feet x 36-feet. There are no shipping or erection concerns with these dimensions; therefore, continuous panels can be considered. Previously, it was determined that the 89-3s panel did not satisfy bending slenderness criteria in the strong-axis vertical orientation; therefore, the slightly wider 105-3s panel, which did satisfy the slenderness limit was
investigated for strength. The 105-3s proved to have adequate bending resistance for use on the 2nd story. Due to the significant number of openings on the 2nd floor, it was logical to upsize the panel to 105-3s and keep the panels in one piece rather than considering thinner discontinuous 89-3s panels with joints and independent lintels. The detailing and erection would be simplified with the continuous panels. For consistency, 105-3s were selected for use on the 1st floor and garage with joints as discussed previously. Because it would be likely that there would be left-over material available after the cutting of the wall panels, 105-3s lintels oriented with the strong-axis horizontal were investigated for use as lintels over the larger 1st floor openings. It was determined that the 105-3s lintels were adequate for all the larger 1st floor openings as well as the garage overhead door opening.

### 3.4 Floor and Roof Panel Design

A combination of hand calculations and software-based solutions were utilized for analysis and specification of the floor and roof panels. As with the wall panels, the floor and roof panels were sized on a per-foot basis. When required, RISA 3D software was used to calculate internal forces and estimate deflections considering a 1-foot-wide beam element. Material properties were estimated based on the outer layer wood species properties. An equivalent thickness was calculated based on Equations 1 and 2, where $d_{equiv}$ is the thickness (depth) of the beam and $b$ is the width of the beam (12 inches in this case). Apparent stiffness was considered to include the effect of shear deformation.

$$I_{app} = EI_{app} \div E$$

$$d_{equiv} = 3 \sqrt{\frac{12 {I}_{app}}{b}}$$

In addition to hand calculation and RISA 3D, WoodWorks Sizer (Update 4, AWC 2019) was utilized to perform structural analysis and specification of floor panels.

As mentioned previously, Katerra CLT panels were considered for the floor and roof. Preliminary panel sizes were selected from Katerra CLT Pre-Analysis Span Tables (Katerra 2020b) and are shown in Table 1.
Table 1. Preliminary floor and roof panel selections.

<table>
<thead>
<tr>
<th>Assembly</th>
<th>Table Load ¹ (LL, SDL)</th>
<th>Maximum Actual Span</th>
<th>Allowable Span</th>
<th>Panel Selection</th>
<th>Controlling Limit-State</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st, 2nd Floor</td>
<td>40 PSF, 30 PSF</td>
<td>16.09 FT</td>
<td>16.67 FT</td>
<td>K5-0690</td>
<td>Vibration</td>
</tr>
<tr>
<td>Attic Floor</td>
<td>40 PSF, 30 PSF</td>
<td>18.56 FT</td>
<td>20.50 FT</td>
<td>K7-0970</td>
<td>Vibration</td>
</tr>
<tr>
<td>Garage Floor</td>
<td>40 PSF, 30 PSF</td>
<td>22.00 FT</td>
<td>23.00 FT</td>
<td>K9-1120</td>
<td>Vibration</td>
</tr>
<tr>
<td>Main Roof</td>
<td>20 PSF, 15 PSF</td>
<td>17.97 FT</td>
<td>19.33 FT</td>
<td>K3-0380</td>
<td>Strength</td>
</tr>
<tr>
<td>Garage Roof</td>
<td>20 PSF, 15 PSF</td>
<td>16.26 FT</td>
<td>17.67 FT</td>
<td>K3-0350</td>
<td>Strength</td>
</tr>
</tbody>
</table>

Notes:
1. LL = Live Load, SDL = Sustained Dead Load.

The structural adequacy of floor panels was checked first. Floor panels were assumed to be continuous over intermediate bearing locations and span one-way. 1st floor panel FP1-2 was first checked using Sizer and the results compared to hand calculations. As can be seen in Table 2, analysis results from RISA 3D and Sizer compared closely. To check for discrepancies in methods, the vibration controlled maximum spans, calculated in Sizer, were compared to both the pre-analysis span table values and those computed using Chapter 7 of the CLT Handbook. Results are shown in Table 2. Based on this verification process, the results from the Sizer software package were considered reliable. Analysis of the remaining floor panels was conducted with Sizer alone.

Table 2. Partial results from panel FP1-2 analysis.

<table>
<thead>
<tr>
<th>Method</th>
<th>Moment (K-FT)</th>
<th>Shear (K)</th>
<th>$\Delta_{LL}$ (in)</th>
<th>$\Delta_{TL}$ (in) ¹</th>
<th>Vibration Max. Span (FT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RISA 3D</td>
<td>1.94</td>
<td>0.615</td>
<td>0.125</td>
<td>0.197</td>
<td>-</td>
</tr>
<tr>
<td>Sizer</td>
<td>1.94</td>
<td>0.592</td>
<td>0.130</td>
<td>0.210</td>
<td>16.94</td>
</tr>
<tr>
<td>Chapter 7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>16.81</td>
</tr>
<tr>
<td>Span Tables</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>16.67</td>
</tr>
</tbody>
</table>

Notes:
1. Total deflection is calculated according to NDS Section 3.5.1 with $K_{cr} = 2.0.$
The remaining floor panel checks were straight-forward. All the preliminary floor panel selections listed in Table 1 were verified as adequate. As suggested by the pre-analysis span tables, the controlling limit-state for the floor panels was vibration control.

Upon completion of the floor panel design, the preliminary roof panel sizes were verified. As can be seen in Figure 4, the roof is designed to function without the need for interior bearing. The decision to detail the roof in this manner was made largely to eliminate obstruction in the most usable central portion of the attic and to avoid loading the interior span of the attic floor below. To analyze the roof panels, independent RISA 3D models were created for both the main roof and the garage roof. The analytical models not only provided the internal forces and deflections required to determine adequate panel sizes, but also provided joint forces, which were used to determine connection requirements at the peak and base of the panels. Figure 10 shows the free body diagram used as a basis for the garage RISA 3D model.

![Garage roof free-body diagram](image)

**Figure 10.** Garage roof free-body diagram.

The Garage panels were checked first, and based on the pre-analysis tables, a K3-0350 panel was selected for analysis. Upon review of the design loads, it was clear that due to the adjacent higher main portion of the building, the drifted snow load would control the design. When analyzed, the deflection of the K3-0350 panels exceeded the typical L/240 live load and L/180
total load deflection limits. The K3-0380 was subsequently analyzed and failed to meet the
deflection criteria. The thicker K3-0410 panel was analyzed and satisfied both deflection and
strength criteria.

The same process was followed for the selection of the main roof panels. Like the Garage
panel, the initial pre-analysis table panel selection (K3-0380) did not satisfy the deflection
criteria. There was no snow drift possible on the main roof, but due to the roof slope, an
unbalanced snow loading was required to be investigated. To satisfy deflection criteria, the
thicker K3-0410 was also required.

3.5 Lateral Force-Resistance System (LFRS) Design

The lateral system design was the most challenging aspect of this home design. The CLT
panels' in-plane stiffness and strength were large and there was little concern regarding their
adequacy to function properly in the system; however, the regulations governing design of
interconnecting components within the lateral force resisting system were difficult to navigate for
panel-to-panel connections. Three areas of lateral system design that lacked substantive
guidance were:

1. Diaphragm deformation and rigidity.
2. Connection design for diaphragms and shear walls
3. Shear wall design in general, especially in wind driven designs.

To perform the LFRS design, many references were required to be reviewed and used to
produce a confident design. While such efforts are expected to be part of a study as presented
here, the outcome should help reduce some of the challenges for designers of CLT homes.
Pertinent references along with design challenges faced will be discussed throughout the
section. Connection design will also be discussed in this section since many of the connections
are subjected to forces resulting from lateral forces.
Figure 11 identifies many of the LFRS components. Additional details are provided on the drawings located in Appendix A. The CLT floor and roof panels act as rigid diaphragms (in this case) transferring wind loads to designated shear segments located within the wall panels. The shear wall boundaries, outlined in Figure 11, are fictitious and defined by the anchorage to the floor panels. A segmental approach, based on the mandatory requirements set forth in Appendix B of the 2021 SDPWS was utilized to apportion the shear wall segments. Appendix B does not permit shear walls to be designed using Force-Transfer Around Opening (FTAO) or Perforated Shear Wall methods.

Hardware was required to ensure the continuity of the LFRS. Straps are used to transfer tensile overturning forces to the foundation. Straps are also utilized as splices to resist diaphragm chord forces. In addition to functioning as lintel, the CLT material above the wall openings on the 2nd floor is also utilized as both a chord and collector to transfer attic floor diaphragm loading. Establishing the load transfer path on the first floor, however, proved not to be as
straightforward due to the joints at headers; therefore, it was decided to utilize the 2nd floor CLT edge laminations, oriented parallel to the shear resisting segments, to function as chords. This approach follows that used by Spickler in a CLT horizontal diaphragm design example (Spickler et al. 2015). The chord delineation can be seen in detail 5/S501.

Initially, the design of the horizontal diaphragms was considered. To determine whether the panels possessed adequate internal shear strength, the panel edgewise shear stress ($F_v$) was required. The allowable design value for edgewise shear stress was obtained from Katerra guidance (Katerra 2020a). Katerra capacities were presented in terms of allowable shear capacity, which indicates that the 2.0 ASD reduction factor, required in Section 4.1.4 of the 2021 SDPWS, is included in the published value. According to PRG-320 Section 8.5.6.2 published values for $F_v$ are required to be reduced by a factor of 2.1 from that of the tested value. According to 2021 SDPWS Section 4.5.4.3, in addition to the required reduction factor, an overstrength factor of 1.5 is required to be applied to the wind demand for diaphragm design.

The reduction and overstrength factors are applied to ensure that if diaphragm failure were to occur, it would proceed in a ductile manner at the connections, rather than an abrupt shear failure of the main load carrying elements. According to Breneman (Breneman and Line 2020), one of the engineering goals of the diaphragm design is to ensure that the CLT panels and chord members can achieve their target shear capacity in this ductile manner. The requirements set forth in 2021 SDPWS Section 4.5.4 were included to encourage this goal of a safe ductile horizontal diaphragm.

The roof level diaphragm was the first to be verified. The roof panels, in this design, are only intended to function as a diaphragm in the east-west direction. The upper-half of the attic gable walls transfer a small amount of out-of-plane wind loading through the roof to the shear panels on the 2nd floor. In the north-south direction, out-of-plane wind forces are transferred directly to the attic floor diaphragm which in turn transfers them to the shear walls along wall lines A and B. To ensure integrity of the diaphragm in the east-west direction, tensile chords made continuous using Simpson Strong-Tie LSTA-9 straps were established at the peak (See 1/S104). A shear capacity of approximately 10,000 plf was calculated considering a load duration factor of 1.6 ($C_d$) and the gross thickness of the panel as directed in both the Katerra guide and PRG-320. The calculated shear demand of 12 plf was insignificant.
The roof panels are somewhat efficient in resisting the gravity loads; however, they are excessively oversized for the shear demand. From a material efficiency perspective, a light-framed construction system that utilizes a thin shear resisting sheathing membrane and deeper modular members spaced at intervals would be a more material-efficient system of construction for this case. There may be other aspects of design to consider, however, such as installation, cost, speed-of-construction, envelope performance, construction schedule, etc., so often the choice of structural systems is not directly related to the material efficiency.

<table>
<thead>
<tr>
<th>Table 4.2.2 Maximum Diaphragm Aspect Ratios</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Flat or Sloped Diaphragms)</td>
</tr>
<tr>
<td>Sheathed Wood-Frame Diaphragm Assemblies</td>
</tr>
<tr>
<td>Wood structural panel, unblocked</td>
</tr>
<tr>
<td>Wood structural panel, blocked</td>
</tr>
<tr>
<td>Single-layer horizontally-sheathed lumber</td>
</tr>
<tr>
<td>Single-layer diagonally-sheathed lumber</td>
</tr>
<tr>
<td>Double-layer diagonally-sheathed lumber</td>
</tr>
</tbody>
</table>

Figure 12. 2021 SDPWS Table 4.2.2 reproduced.

The analysis and design of the attic floor diaphragm was conducted next. A detailed analysis was performed on the attic floor to develop an improved understanding of the performance and capabilities of CLT panels functioning in the role of diaphragm. Initially, the geometry of the diaphragm was considered. For the design of a sheathed, light-framed diaphragm, a designer would reference SDPWS Table 4.2.2 (reproduced in Figure 12) for guidance. Satisfying the length-to-width (L/W) ratio listed in the table, for the assembly under consideration, would provide confidence that the diaphragm or sub diaphragm could achieve full strength without buckling. No codified length-to-width ratio was identified for CLT diaphragms; therefore, the buckling criteria for the assembly was unclear. As a guide, the 4:1 ratio listed for blocked diaphragms was adopted. In this case, the L/W ratio for the attic diaphragm was computed at 1.2, which is well under the adopted limit; therefore, it was assumed that the CLT panels could achieve their full shear capacities.

A determination of diaphragm flexibility was the next design decision. Based on Section 1604.4 of the 2018 IBC and Section 4.1.7.2 of the 2021 SDPWS, a diaphragm can be considered rigid if the deflection of the diaphragm is less than or equal to twice that of the average deflection of
the adjoining shear walls. The rigidity of the attic diaphragm was checked in the east-west direction. Perforations along Grid Lines 1 and 2 (in shear walls) create significant difference in stiffness between these lines; therefore, it was necessary to calculate the stiffness of the diaphragm to properly distribute lateral forces to individual wall segments. Wall lengths along Grid Lines A and B are largely non-perforated and similar in length; therefore, the difference in distribution of lateral forces between a rigid and flexible diaphragm analysis would be negligible.

An analysis was conducted to estimate both the attic diaphragm deflection and the adjoining 2nd floor shear wall average deflection. An average shear wall deflection of 0.284 inches was estimated based on provisions in the 2021 SDPWS Section B.4 and suggestions put forth in the Swedish CLT handbook (Borgstrom and Frobel 2019). The deflection of the diaphragm was estimated at 0.092 inches, based on calculation methods like those used by Spickler (Spickler et al. 2015). The diaphragm deflection of 0.092 inches is significantly less than the average shear wall deflection of 2 x 0.284 inches = 0.568 inches; therefore, the diaphragm can be considered rigid. DeStafano suggests that it is reasonable to assume that untopped CLT diaphragms with L/W ratios less than 2:1 is rigid (DeStafano and Way 2020). Based on the analysis and DeStafano’s suggestions, all floor diaphragms will be considered rigid in both directions.
Based on the conclusions of the flexibility analysis, a rigid diaphragm analysis was conducted to determine the proper distribution of the wind forces in the east-west direction. As required in 2021 SDPWS Section B.2.5, shear forces were distributed according to relative segment stiffness, which in this case is determined by panel length since the material and thickness of the panels is consistent throughout the story. Only segments with height-to-length (h/l) aspect ratios less than 4, as suggested in 2021 SDPWS Section B.3.1 are considered. The lower limit of 2, required in the section, was not adhered to. It was unclear whether this lower limit is applicable for structures subject to wind only. Based on review of Chapter 4 in the CLT handbook and NEHRP Recommended Seismic Provisions for New Buildings and Other Structures section C14.5.2 (FEMA 2020), it was interpreted that the requirements specified in
the 2021 SDPWS Appendix B are based on capacity design principle, and are focused on the response of CLT panels subjected to seismic loading and non-linear behavior.

Figure 13 shows the parameters used in the rigid diaphragm analysis. Methods utilized by Breyer (Breyer et al. 2003) and the U.S. Department of Housing and Urban Development (HUD) (NAHB Research Center 2001) in their publications were utilized to conduct the analysis. Table 3 shows the distribution of the lateral wind force from the attic diaphragm to the 2nd floor exterior shear wall segments. For comparison, the distribution is also shown for flexible diaphragm. As can be seen in Table 3, there are slight differences in the shear magnitude due to torsional loading.

### Table 3. Comparison of rigid and flexible attic diaphragm shear load distribution

<table>
<thead>
<tr>
<th></th>
<th>Wall Line 1 (plf)</th>
<th>Wall Line 2 (plf)</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rigid Diaphragm</td>
<td>133</td>
<td>145</td>
<td>7.5%</td>
</tr>
<tr>
<td>Flexible Diaphragm</td>
<td>123</td>
<td>157</td>
<td>-8.3%</td>
</tr>
</tbody>
</table>

After determining the distribution of the diaphragm shear load, the forces resulting from overturning action were calculated for each wall segment. Based on the large, calculated roof panel shear capacity, it was assumed that the remaining diaphragm and shear wall panels were adequate to resist in-plane shear loading; therefore, no further strength checks were performed. Both the compressive pressure ($f_c$) and the tensile force ($T$), resulting from the propensity of the panel to overturn when subjected to shear loading, were calculated. Figure 14a depicts the panel forces.

Conservatively, considering the self-weight of the CLT panels only and ASD load combination 0.6 D + 0.6 W, the tensile forces were calculated for each shear wall segment. Along Wall Line 2, only SW1 required tensile anchorage. No anchorage was required for those segments along Wall Line 1. To resist the tensile forces, Simpson Strong-Tie MSTC28 straps were specified. The ST6224 straps, depicted in Figure 14b, have adequate capacity to resist the calculated tensile force; however, for continuity of load path, the force had to be directly transferred to the panel below. The 2nd floor panel created a separation between the two panels preventing
installation of the required number of nails for the shorter ST6224 strap. The longer MSTC28 strap was required to span this distance. Because the MSTC28 had excess capacity, calculations were performed to reduce the number of nails required from 18 to 10 per side. Even with this reduction and consideration of the overstrength factor prescribed in 2021 SDPWS Section B.3.4.3, the MSTC28 capacity of 1966 lbf was more than adequate to resist the demand of 279 lbf.

The bearing capacity of the CLT floor panel below the compressive leg of each overturning shear panel was also checked. It was assumed that during an overturning event, a perpendicular to the grain bearing failure would occur in the floor panels resulting from compressive pressure applied from the stiffer, vertically oriented laminations of the shear wall panel. For the bearing check, the overturning analysis was repeated considering ASD load combination D + 0.75(0.6 W) + 0.75 S and adding the collateral roof and floor dead load to the self-weight. Based on equation 6.11 in the Swedish CLT Handbook (Borgstrom and Frobel 2019), bearing area was estimated considering the combined width of the two vertically-oriented wall laminations and 25% of the segment length. The maximum calculated bearing pressure of 82 psi was significantly less than the allowable floor capacity of 425 psi.

![Figure 14](image)

**Figure 14.** a.) Shear panel overturning free-body diagram b.) Shear panel tension strap.
Following the overturning analysis, the floor panel-to-shear wall segment shear transfer connection requirements were determined. The design shear load was 145 plf. A frictional resistance of between 73-145 plf was estimated, but not utilized for design. By engineering judgement, it was conservatively considered unreliable. Effective shear wall shear transfer was provided throughout the building by dedicated Simpson Strong-Tie ABR9020 brackets shown in Figure 15. The brackets were selected from the Simpson Strong-Tie mass timber construction catalog (Strong-Tie 2020). Two brackets were specified for the top and bottom of each contributing shear panel with a maximum spacing restricted to 6-foot. Additionally, brackets are to be installed within the first 12-inches of each segment end as instructed in Section B.3.1.4 of the 2021 SDPWS.

![Figure 15. Simpson Strong-Tie shear transfer bracket (Strong-Tie 2020).](image)

Upon completion of the 2nd story shear wall analysis and hardware specification, the 2nd floor horizontal diaphragm analysis was conducted. A rigid diaphragm analysis was conducted to determine distribution of forces; however, due to the proximity of the stair opening to its adjacent panel, an additional analysis was conducted to determine detailing requirements for the stair opening (see 1,2,3/S-102). Initially, the following criteria (Malone and Rice 2011) was used to evaluate the significance of the opening:

1. Depth of the opening ($D_o$) no greater than 15% of the diaphragm depth.
   a. $D_o = 3.19 \text{ ft} < 0.15 \times (36 \text{ ft}) = 5.40 \text{ ft}$; therefore, not significant.
2. Length of the opening ($L_o$) no greater than 15% of the diaphragm length.
a. \( L_o = 7.15 \text{ ft} > 0.15 \times (30 \text{ ft}) = 4.5 \text{ ft}; \) therefore, significant.

3. Distance from diaphragm edge to opening edge (\( D_o \)) less than three (3) times the larger opening.
   a. \( D_o = 6.55 \text{ ft} < 3 \times (7.15 \text{ ft}) = 21.45 \text{ ft}; \) therefore, significant.

4. Diaphragm portion on all sides of the opening satisfies the maximum aspect ratio requirements.
   a. Yes, all four (4) component diaphragms satisfy the assumed maximum aspect ratio of 4:1.

Malone and Rice (2011) suggest that if all four of these points are true then the opening is not likely significant. In this case, however, two of the four criteria are false; therefore, further analysis is warranted.

A simplified method, often utilized to determine detailing requirements around openings in steel girders (Blodgett 1966) and concrete diaphragms (Taylor et al. 2015), was utilized to examine the chord forces above and below the stair opening. Figure 16 shows the assumed internal forces generated by bending around an opening in a steel beam. Global shear is assessed at the midpoint of the opening and distributed to both the top and bottom segment based on the cross-sectional area of the respective segments. Imbalanced shear generates a localized bending moment at the edges of the hole, which must be resisted by the existing material or additional reinforcement. It is common to assume that the point of contraflexure occurs at the midpoint of the opening; therefore, the beam can be sectioned, and forces summed about this point of zero moment. This provides for a convenient means of determining the secondary moment at the edges of the hole.
Only the localized bending effects were considered for the analysis of the stair opening. In the case of a steel girder, the effects of the bending moment due to external loading on the top and bottom segments should be examined to determine if the tensile and compressive force created by global effects is significant. In this case, it is assumed that the main diaphragm chords completely resist the compressive and tensile couple resulting from bending due to external forces.

The stair opening on the 2nd floor is in a region of low shear due to its proximity to midspan. Regions Sub 1 and Sub 2, as shown in 3/S102, were defined for the analysis and the sub-chord forces associated with the distributed shear were computed. A maximum localized sub-chord force of 39 lbf was computed, which is insignificant and therefore neglected. It is assumed for all intents and purposes, the splice detail shown in (4/S-102) is sufficient to transfer this minimal force at locations 1 and 2 (3/S102). No further analysis was conducted regarding openings.
The same analysis that was conducted for the attic diaphragm and 2\textsuperscript{nd} story shear walls was also conducted for the 2\textsuperscript{nd} floor diaphragm and 1\textsuperscript{st} floor shear walls. Analysis concluded that both the wall and floor sizes as determined in previous steps were adequate. The ABR9020 shear connector specification determined for the 2\textsuperscript{nd} story was also determined to be acceptable for the 1\textsuperscript{st} story connections as well. Differing from the 2\textsuperscript{nd} story specification, however, was the tension hold-downs required to stabilize the 1\textsuperscript{st} story shear wall segments. Whereas tension resistance was only required for a few panels on the 2nd story, nearly all of the wall segments on the 1st floor required hold downs. For simplicity it was decided to install Simpson Strong-Tie HTP37Z straps on all segment ends.

To conclude the LFRS design and determine foundation anchorage requirements, a global overturning analysis was conducted. The results of the analysis indicated that the heavy CLT structure had more than enough weight to resist both overturning and sliding due to lateral wind loading. Based on this analysis it was determined that only minimum foundation anchorage would be required. Detail 6/S-501 shows the anchorage requirements. Minimal anchorage was provided to ensure positive attachment to the foundation. An elastomeric bearing pad was provided to bridge inconsistencies in the top of wall finish and to help seal the joint.

### 3.6 Foundation Design

Foundation design was relatively simple and resulting foundation elements were similar in size to those required for the light-framed wood structure previously designed (Jellen and Memari 2019). The foundation specification is shown on 2/S-101. The W8x18 girder utilized for the light-framed structure was adequate for midspan support of the CLT floor system as well.

Concentrated load checks were conducted according to the Steel Construction Manual (SCM) (American Institute of Steel Construction (AISC) 2011) Specification Section J10 at the column bearings. All checks passed; however, a maximum LRFD factored reaction of 48.2 Kips did approach the limit of 51.1 kips calculated for the web compression buckling check. Additionally, to pass the web sidesway buckling check, rotational restraint was required at all interior bearing points. In some cases, the column connection could be relied upon for restraint; however, dedicated restraint was preferred in this instance due to the larger magnitude column reactions resulting from the heavier CLT structure. Figure 17 shows the specified restraint detail. The columns were also sized at the same time the girder was checked. Due to the heavier column
loads, a thicker-wall 3.5-inch diameter (0.216 inch thick) adjustable column was required in lieu of the thinner 11-gauge column utilized for the light framed design.

**Figure 17.** Steel girder rotational restraint detail.

The foundation wall specification was like that of its light-framed counterpart; however, the footing sizes were slightly different. The increased weight of the CLT structure required a 24-inch-wide plain concrete wall footing in lieu of the 18-inch-wide footing utilized for the light-framed structure. Interior column pad-footings increased in size from the 4 ft - 0 in x 4 ft - 0 in x 10 in thick pads utilized for the light framed structure to two 4 ft - 6 in x 4 ft - 6 in x 12 in pads and a 4 ft - 0 in x 4 ft - 0 in x 12 in pad. In general, there was a need for larger foundation elements due to the increased weight of the structure; however, the slight increases in required material were minimal and not likely to affect the foundation costs significantly.
3.7 Connections

The most significant connections designed for this structure are identified on 2/S-203. They include the Roof Peak Connection, Roof-Floor Connection, Floor Intersection Detail, Foundation-Floor Intersection Detail, the Interior Top-of-Wall Detail, and the Girder Bearing Detail. The details for these connections are shown on S-501. In addition to the details identified in 2/S-203, the panel-to-panel splice was also designed (See 2/S-501). The connection design was largely conducted according to recommendations put forth in Chapters 3-5 of the CLT Handbook, the 2018 NDS, and the 2021 SDPWS. Discrete, dowel-type fasteners were used for all connections. Lag screws, structural-screw fasteners, bolts, and nails are all utilized to complete critical connections. The individual connection types will be discussed in the subsequent sections.

The roof connections will be discussed first. As mentioned previously, no ridge beam is provided; therefore, it was necessary to design the base and peak connections to both facilitate erection and resist outward thrust generated by the geometry of the roof members. The intent is to utilize bent plates at the peak and base to act as erection aids as well as permanent connections. To act as base stops, wooden blocks cut from CLT scraps are fastened to the attic floor with structural screws at intervals.

The anticipated construction sequence is that the bent steel plates will be attached to both the base and peak locations on the first panel to be erected. This first panel is then craned into position with the base bent plate resting against the base stop. The contractor will be required to position properly and temporarily brace the first panel. The base bent plate is then attached to the second panel. The second panel is lifted into position, the base bent plate rests against the stop, the panel peak is rotated into position resting on the other leg of the peak plate and the connections are made.

Initially, the roof peak connection was designed. As shown in 3/S-501, three \( \frac{1}{4} \)-inch thick bent steel connectors per panel were specified. The legs of the connector are to be fastened to each CLT roof panel using four \( \frac{3}{4} \)-inch x 3-inch lag screws. The connection for the peak was designed considering the gravity loads only. The erection load case was assumed to control the design and was evaluated per ASD load combination \( D + 0.75 \cdot L_R \) (Roof Live) + 0.75 (0.6 W).
Due to the geometry, the lag screw connection was subject to both withdrawal and lateral loading. The forces shown in Figure 18 FBD-3 were resolved into components parallel (y-axis) and perpendicular (x-axis) to the fastener axis as shown in FBD-4. Withdrawal and Lateral design values were calculated per 2018 NDS, Chapter 12 using adjustment factors defined in Chapter 10, with consideration of the calculation adjustments recommended in the CLT Handbook. Withdrawal perpendicular to the plane of the CLT panels is discussed in Chapter 5 of the CLT Handbook. Section 6.3 recommends adherence to NDS Chapter 12.2 for design; therefore, the procedure is no different, in respect to withdrawal, than that used for dimensional lumber. Lateral design for fasteners greater than ¼-inch and installed perpendicular to the plane of the panel, however, requires modification to compensate for the alternating CLT laminations. 2018 NDS Section 12.3 was referenced for design; however, the dowel bearing lengths were reduced by a factor of $F_{e,\text{parallel}} / F_{e,\text{perpendicular}}$ to compensate for the different dowel bearing strengths associated with each penetrated cross lamination. The dowel bearing strength for the lamination at the shear plane, which was shear parallel to the grain in this instance, was considered for use in the yield-limit equations.

The roof base connection was next designed. This connection, as can be seen in 4/S-501, is complicated and the design was multi-faceted. As mentioned previously, bracket B1 is to be bolted to the roof panel prior to erection. Just like the peak connection, three brackets per panel are installed. Through-bolts were specified at the base connection to improve joint durability, which is important because the bracket will be utilized as an erection aid and will likely be subject to minor impacts with the block. Bracket B1 is nailed to the wood block. The bracket

![Figure 18. Roof peak connection free-body diagrams.](image)
transfers the thrust load to the block by bearing and the nails are intended to transfer shear created by uplift and lateral forces to the block.

Structural screw fasteners are specified to transfer shear and the eccentric axial force, shown in Figure 19, from the block to the 2nd floor panel. MyTiCon structural screws were evaluated and selected from their catalog (MyTiCon 2019). Initially, the ASSY Ecofast screw was considered, but discarded. The Ecofast partially threaded screw, as depicted in Figure 19, was not adequate to resist the pull-through force generated by the eccentric uplift force. ASSY VG CSK all-thread screws were next considered. The pull-through limit-state does not apply to fully threaded screws; therefore, the tensile capacity is controlled by withdrawal. It was determined that a screw spacing of 10-inches-on-center was adequate to resist the combined loading.

![Figure 19. Wooden block eccentric force resolution.](image)

Next the angled screw connection, shown in 4/S-501, between the wall and attic floor was designed. The purpose of this connection is to provide a dedicated uplift connection between the wall and the floor system and to transfer chord forces between the attic diaphragm and the top-of-wall chord. 2021 SDPWS Section 4.5.4.2 requires a separate shear and uplift connection.
Additionally, due to the connections’ relationship with the wall chord, the connection must also meet the ductility criteria required in Section 4.5.1. As discussed earlier, ABR9020 brackets are utilized on the interior to transfer the diaphragm shear to the wall. Uplift could technically be resisted by the weight of the structure, but a dedicated fastener improves reliability of the connection and alleviates concerns regarding differential movement between the walls and floors.

The angled screw connection was designed for direct tension from roof uplift and longitudinal shear from the diaphragm. The joint was assumed to be a pinned connection and transfer no moment. MyTiCon Table S.1.2 (MyTiCon 2019) was used to evaluate the geometry factor ($C_a$). Lateral capacity was calculated per NDS Section 12.3 and SDPWS Sections 4.1.4 and 4.5.4. The withdrawal capacity was calculated and reduced by the angle-to-grain reduction factor listed by MTC Solutions in Table RDV.1.2 (MTC Solutions 2020); however, once again the pull-through limit controlled the design.

The next connection to be mentioned is the panel-to-panel splice detail. The single spline panel splice detail, shown in 2/S-501, was utilized for all the roof and floor panels. The panel-to-panel connection is a diaphragm shear transfer detail and therefore is subject to 2021 SDPWS Sections 4.1.4 and 4.5.4. Spline splice design is well documented. MyTiCon provides standard spline specifications in their design catalog for structural-screw fasteners (MyTiCon 2019). Spickler details a splice in his horizontal diaphragm design example (Spickler et al. 2015), and Brenneman also discusses typical splice design in his presentation (Breneman and Line 2020). This connection is used to transfer diaphragm bending generated shear between panels. The panels are routed, and a plywood spline is fitted. The routed section is typically larger than the spline to provide for fit tolerance. It is most typical to use structural screws in this connection; however, non-structural screws are sometimes used along the edges as a construction aid. The 2nd floor diaphragm shear controlled the design of this connection. The magnitude of the shear was relatively low due to light residential loading. 5/16-inch Ecofast screws spaced at 48-inch were adequate to resist the demand.

The remaining connections, such as the Floor-Intersection Detail, Foundation-Floor Intersection Detail, the Interior Top-of-Wall Detail, and the Girder Bearing Detail were all straightforward designs and relied on the same principles previously discussed for the other connections. The design elements for the Floor Intersection Detail, the Foundation-Floor Intersection detail, and
the Girder Bearing Detail were previously discussed in the LFRS and foundation design sections. The only noteworthy item to mention regarding the Interior Top-of-Wall detail (7/S-501) is the top connection. The interior walls are not designed as shear walls and to ensure that lateral load does not inadvertently transfer to the interior walls from diaphragms, bypass-framing clips were provided at the top to allow relative slip between the floor and the wall. This should be considered when detailing the interior finish. Additionally, in seismically controlled regions it is important to note that the detailing of members not part of the LFRS, such as the interior wall, is subject to connection requirements set forth in the 2021 SDPWS.
4. Conclusions

In general, the design of a single-family residence CLT structural system posed many challenges that had to be addressed/overcome in this project, mainly because of lack of prior work done for this type of building. Currently, CLT systems are typically not economical for single-family residences and if they are to be considered, then the complete design, including panelization, should be accomplished ahead of time by the designer. This increases the front-end design time required by the professional as well as the design fee. It is difficult to justify the increase in design effort when considering the typically available budget for design allocated in the traditional light-framed construction workflow. According to NAHB (Ford 2020), the total allotted architectural and engineering (not only structural) budget for a typical single-family home was approximately $4,335.

If CLT is to be considered for use in single-family projects, then the efficiency of the workflow should be maximized. During this design, valuable lessons were learned regarding efficient workflow, which will help residential building designers working on CLT single-family projects. The following is a list of the lessons:

1. Adapting an existing building plan for use with CLT panels can be difficult if the geometry of the structure does not match typical CLT panel dimensions.
2. Interior Bearing walls, not stacked with the wall or beam below, could result in increased floor panel thickness.
3. Floor and roof panel structural evaluation for out-of-plane bending and deflection is a relatively easy process when simple-span conditions exist, and the bending and shear diagrams can be easily created. Software should be utilized for continuous spans with complex loading.
4. Time saving, prescriptive aids do not exist for CLT design such as those utilized for light-framed construction.
5. The lateral design posed additional challenges for a CLT residential project. CLT diaphragm design is well documented in the literature; however, the requirements for CLT shear wall design is not; especially for wind-driven designs.
6. In general, the needed calculations (utilizing current design resources) tend to be much longer than that needed for conventional wood-frame home design, and the resulting
design will turn out to be overly conservative, especially where the design of shear walls is concerned.

7. If any efficiency is to be brought into a CLT design, the reliance on hand calculations should be minimized. Using 2021 SDPWS Section 4.1.2.2 as a basis, rather than the prescriptive provisions, would likely result in a cleaner design. This section provides the option for, “approved alternate procedures that are in accordance with the principles of engineering mechanics.” An FEA software program like Dlubal’s RFEM could be utilized to analyze the structure, thus providing the opportunity for a much more efficient design completed in less time.

8. If working with CLT structures on a consistent basis, a drafting program like Cadwork that can export model data directly to CAD/CAM Systems should be considered for documentation and panelization design.

9. Using CLT panels for residential walls can be inefficient, therefore. One could alternatively consider hybridizing the structure by using light-framed walls.

10. If using CLT walls, however, it is recommended designing the wall lintels first. In this project report, the wall lintels controlled the design. The CLT wall panels had more than adequate axial and bending capacity considering the applied wind loading.

11. For wall and lintel design, always check the slenderness prior to performing further structural checks.

12. Due to the geometry of the floor plan, various floor thicknesses were required for this design. This could be a problem for the manufacturer.

13. The proper application of the overstrength and reduction factors required for diaphragm and shear wall design by the 2021 SDPWS need further clarification. Some clarification and examples of their application would be helpful for designers.

14. Plate buckling criteria for both shear walls and diaphragms need to be further developed specifically for CLT. There are no aspect ratios provided for horizontal diaphragms, and it is not clear why there is a lower limit of 2 for the shear wall aspect ratio in 2021 SDPWS in regard to wind design.

15. For wind-controlled designs, the capacity-based design requirements presented in the 2021 SDPWS are overly conservative. Further discussions and clarifications are needed to relate the SDPWS requirements to a linear-elastic design approach.

16. Diaphragm openings should be evaluated to determine detailing requirements around the perimeter of the opening.
17. Detailing around large openings in the diaphragm should be assessed; however, the sub-chord forces associated with shear are likely to be low due to the low magnitudes of external loading residential structures experience. Centralized openings will be subject to lower sub-chord forces than ones closer to the edges of the diaphragm.

18. CLT connection design needs much more effort and innovation compared to conventional wood-frame. Connections within the LFRS require careful review of the SDPWS design requirements and clear identification of load path. The design information for connections is not concisely located in one document currently. For this design several resources were required to perform a reliable design.

Light-framed construction is still the most economical construction system for traditional-style, single-family homes. The system is familiar to most contractors and the material is readily available. The units of construction are modular and construction using this method can be accomplished by the homeowner if required. There are many benefits to using this system; however, there are also many well-known inefficiencies in the construction system. Most revolve around inefficient workflow.

Currently the inefficiencies inherent in light-framed construction methods do not outweigh the economic savings and familiarity of the system within the industry. Light-framed construction methods will likely remain the most popular single-family residential construction system until a time when economic drivers such as material availability/cost, building code requirements, or homeowner demand for modern structures boost the economy of using alternative construction materials such as CLT to a point where they are cost-competitive with light-framed construction.
5. References

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Appendix A – Structural Drawings

To be published in late 2022.
Appendix B – Structural Design Calculations

To be published in late 2022.
Appendix C – Supplementary Design Calculations

To be published in late 2022.
Appendix D – Reference Home Plans

To be published in late 2022.