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Chapter

Structural Design of a Single-Family Residential Dwelling Using Cross-Laminated Timber (CLT)

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Abstract

Driven by desire to reduce carbon footprint in building construction that in modern times has relied heavily on masonry and concrete whose production is associated with burning excessive amounts of fuel, use of wood offers the ideal alternative. Cross-Laminated Timber (CLT) is an esthetically pleasing, 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. The prefabrication process used to fabricate CLT panels provide users of the construction materials access to all the advantages offered by off-site construction methods such as factory quality control, just-in-time delivery, and accelerated construction. In this chapter, the original light-framing system of a traditional style single-family residential dwelling is converted to a panelized CLT structural support system. The chapter provides the basis of design, typical design process, and explains the challenges associated with using the alternative framing system.

Keywords: residential construction, CLT, panelized construction, alternative construction materials, cross-laminated timber, CLT design, residential structural system, renewable construction material

1. Introduction

In this chapter, the structural design of a typical single-family residence using CLT panelized construction is presented. The information presented in this chapter was largely adapted from a recent CLT research project conducted by the authors for the Pennsylvania Housing Research Center (PHRC) in 2021 [1]. CLT is currently more commonly utilized for the construction of multi-family residential and commercial structures; however, some examples of CLT (single-family) homes are gradually becoming available [2]. 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 chapter is to present these items along with a design example to serve as a guide for this type of construction.

The chapter is organized into three additional sections. 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 structural design was performed on a model home provided by a local (Central Pennsylvania) home builder (S&A). 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" [3]. 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.

2. CLT as a building construction material

The basic desirable sustainable attribute of lumber as a construction material is that it sequesters much carbon dioxide during its growth process while releasing oxygen back into the air, which signifies the favorable carbon impact that wood has on the environment. While up until a couple of decades ago, the use of lumber in construction was limited to light-frame systems due to section size limitation dimension lumber, the use of lamination technology has opened new possibilities.

According to the CLT Handbook [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**).

CLT is manufactured and identified according to ANSI/APA PRG 320 (APA 2020). 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 in this publication 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 [5]. Most of the manufacturers listed are located in the western regions 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.

Figure 1.
Isometric view of a three-layer piece of CLT construction material (Wikimedia commons contributors 2021) [4].

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 [2]. CLT panels are not typically designed to be exposed to moisture and are not intended to be utilized as cladding panels [2], therefore they require protection by the building envelope system to prevent deterioration. With proper design, CLT can be used in Type III, IV and V construction as classified by the IBC [6]. Examples provided in the 2021 Mass Timber Design manual [7] show CLT used as an alternative to masonry walls in multi-story residential or office buildings. Both the U. S. edition [2] and the Swedish edition [8] also show examples of CLT being utilized for the construction of single-family dwellings.

3. Applicable building 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 [9], CLT was first recognized in the 2015 IRC [10] as a construction material for walls and floors; however, no prescriptive guidance was provided in the 2015 residential building code or the 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 [11] 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 [12]. For structural design, the 2015 IBC references the 2015 NDS [13]. Chapter 10 was introduced in the 2015 NDS prescribing reference design values for CLT. Additional design guidance was included throughout the standard where applicable. Further

developed guidance was provided in the 2018 versions of both the IBC and the NDS; however, both the 2015 and 2018 versions lacked any guidance on using CLT for diaphragms and shear walls. The 2021 Special Design Provisions for Wind and Seismic (SDPWS) [14] was the first standard to provide engineering design guidance on these topics.

4. Structural design of the home

In this section the structural design process for the home will be presented. The structural design of the CLT house presented is partitioned into several sub-sections. The following sub-sections are included in this section: Panelizing the Structure, Wall Panel Design, Floor Panel Design, and Lateral Force-Resistance System (LFRS) Design. Connections were mostly designed during the LFRS portion of the design. Allowable stress design (ASD) was the methodology used for design of the CLT panels.

To stay consistent with the original design, the 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 [15]. 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.

4.1 Panelizing the structure

In this section, the structural design of a traditional 2-½-story, single-family home using CLT elements and current design resources is discussed. The residence has 8-foot ceiling heights on both the 1st and 2nd stories, a basement, an attic floor space and a bonus floor space above the attached garage. The structural shell of the dwelling, adapted from the light-framed counterpart, is shown in **Figure 2**. 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 [16].

In this design, the CLT panels are utilized as load-carrying, one-way plate elements, which transfer both conventional gravity loads and lateral wind loads to the concrete foundation. To be consistent with the original light-frame design, the conventional gravity and wind loads were computed based on a project location in State College, PA. As was the case for the original design, seismic loads are assumed not to govern the design of the lateral load resisting system. As described in the CLT Handbook [2], the dwelling utilizes a platform framing system in which the floor and roof panels bear directly on exterior and interior walls. 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 lateral wind loading to designated shear resisting wall panels.

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 (or nested on) a master billet. Upon reviewing the geometry of the building, an 8-foot primary panel module (width) was
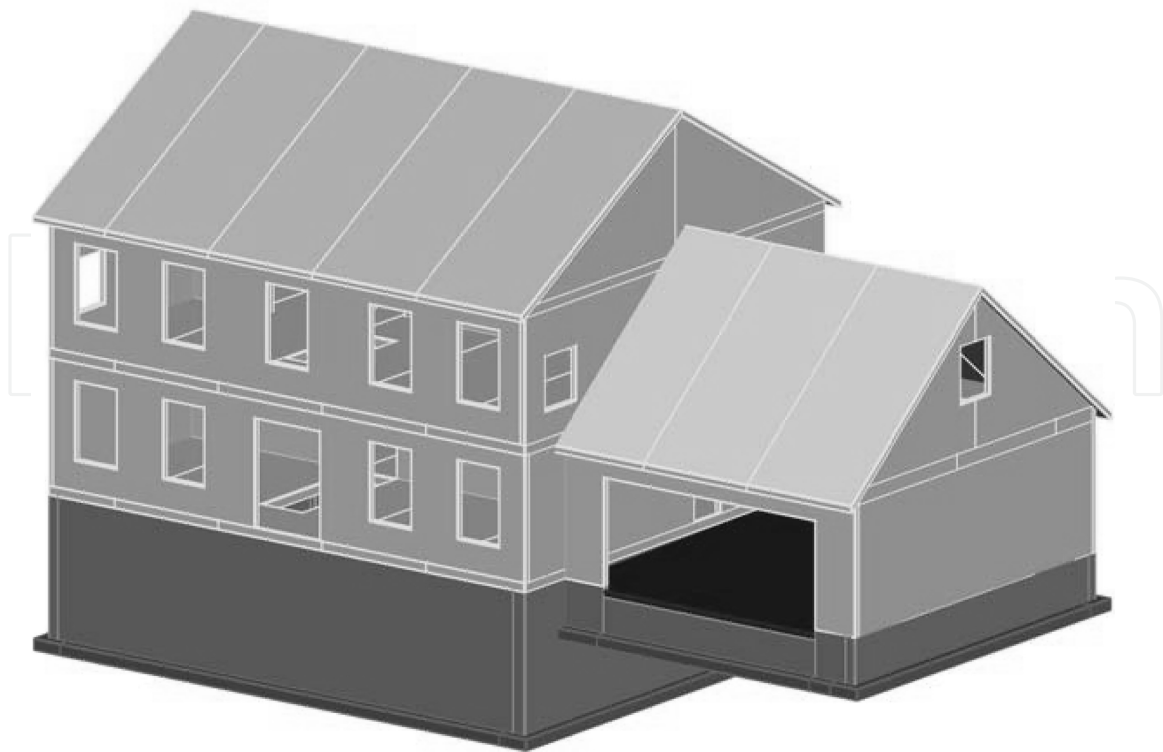
Figure 2.
Rendering of CLT panelized home design.

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 [17] 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:
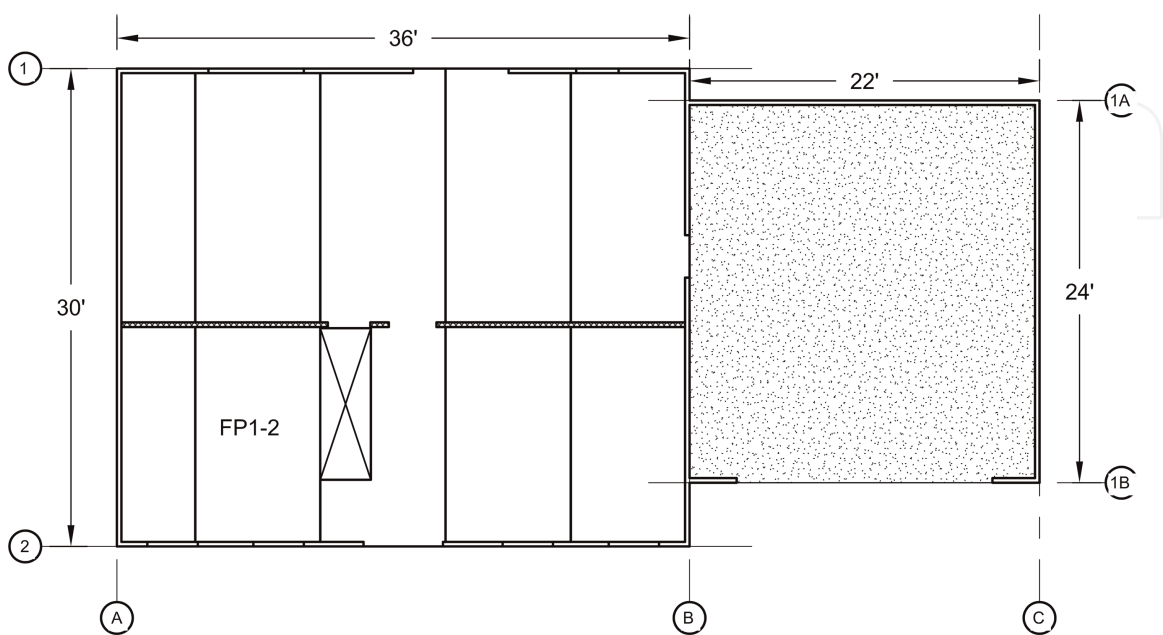

Figure 3.
First floor plan of the house.

1. Typical construction components, such as structural sheathing, have 4 or 8-foot dimensions.

2. Ceiling height for both the first and the second stories is 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.

4.2 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 [13] was utilized as the design basis and the Nordic X-lam Technical Guide [18] was consulted to obtain panel options and design properties.

With minimization of the material use in mind, the X-LAM 89-3S panel was initially selected for consideration. The 89–3 s is a 3-layer, 3 ½-inch thick panel. The panel is certified according PRG 320 as an E1 stress grade panel. Initially, the 2nd story wall panel, WP-5, shown in **Figure 4** was selected for design. It was decided to orient the strong-axis vertically as shown in **Figure 5A**. Typically wall panels are oriented in this fashion to provide greater bending resistance to out-of-plane wind forces.

The structural design limit states influencing wall selection are axial capacity, out-of-plane bending capacity, and the lintel requirement over openings. The axial demand on WP-5, based on controlling ASD load combination Dead (D) + 0.75 Live (L) + 0.75 Snow (S) + (0.75) 0.6 Wind (W), was calculated to be 1213 plf. The 2018 NDS design equations located in Section 3.7 and those in the associated commentary section C3.7 were utilized to calculate the axial capacity on a per foot basis. The column buckling resistance (P_cE) was calculated using the minimum apparent bending stiffness ($EI_{app-min}$) = 0.5184 EI_{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 89–3 s was calculated to be 29,726 plf, which far exceeds the demand of 1214 plf.

The design moment capacity of the panel, adjusting per the prescribed factors listed in 2018 NDS Table 10.3.1, was calculated to be 5360 lbf/ft. A bending demand based on Component and Cladding (C&C) magnitude wind loading and ASD load combination 0.6 D + 0.6 W, was calculated to be 108 lbf-ft. Once again, the capacity far exceeded the demand. Considering the interaction between axial and bending
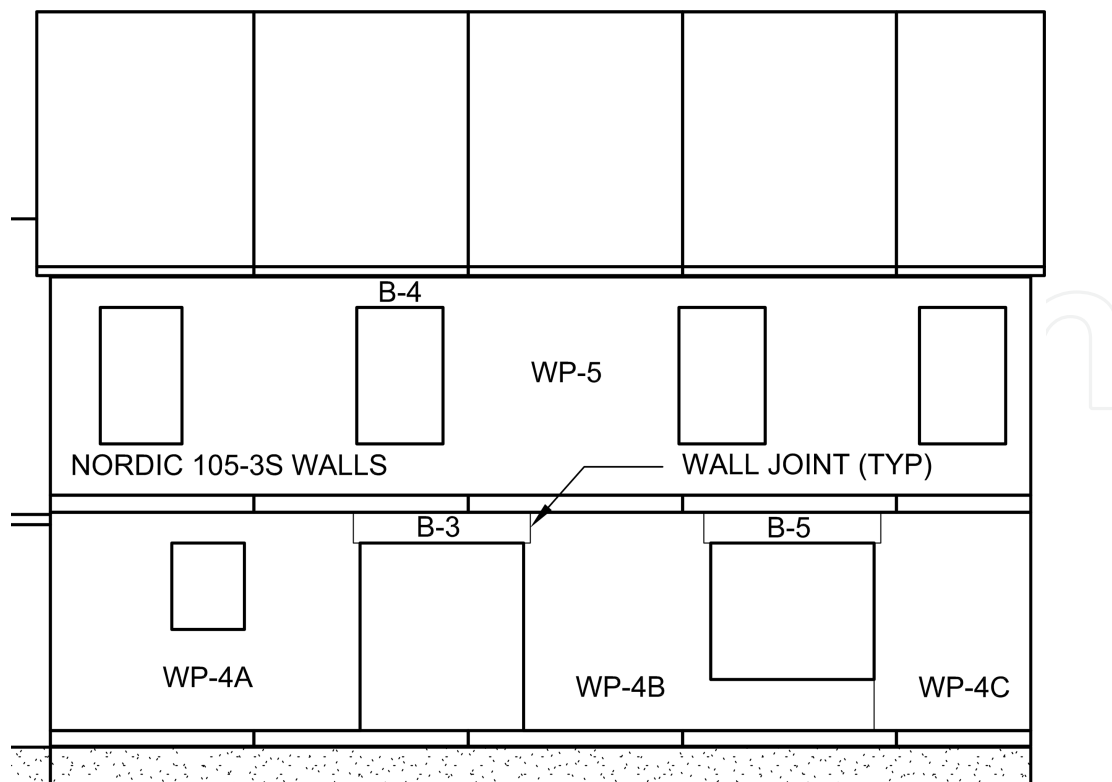
Figure 4.
North elevation of the house.

force, a demand/capacity ratio of 0.023 was calculated using NDS interaction eq.
C3.9.2–3. The resulting ratio of 0.023 shows that the capacity of the thinnest panel far
exceeds the demand. By engineering judgment no additional strength checks were
required.

The lintel requirements for both the 1st and 2nd floors controlled the wall design.
The edgewise mechanical properties for lintels are typically presented by manufac-
turers for the lintel orientations shown in in **Figure 5C** and **D**. Lintels over the
openings in panels WP-4 and WP-5, shown in **Figure 4**, were checked for adequacy.
The lintels over openings in the 2nd floor panel WP-4 were able to remain integrated
in the continuous panel by upsizing the panel to a 105-3S; however, the 1st floor panel
WP-4 was required to be split at the larger openings such as the Lintel B-3 and Lintel
B-4 Locations shown in **Figure 4**.

The assumed external load distribution for the lintels is shown in **Figure 6A**. For
this design, a 25-degree load propagation angle was considered [19, 20]. Some refer-
ences also suggest distributing the load at 30 degrees with the distribution stopping at
a vertical distance of wall-height/4 [8].

The lintel bending capacity was calculated per the provisions of NDS Section 3. In
instances such as the 2nd story, when the lintel is integrated into to the wall panel,
there will be fixed boundary conditions at the bearing creating inflection points near
the bearings. In this situation, the beam stability factor (CL) will not equal 1.0. With
simple span condition such as the 1st story lintel installations, CL can generally be
taken equal to 1.0.

The initial trial 89–3 s lintels installed in the strong-axis vertical orientation
(**Figures 5C** and **6B**) could not meet the slenderness requirement of NDS for bending
members prescribed in Section 3 on either floor. It was necessary to select the wider
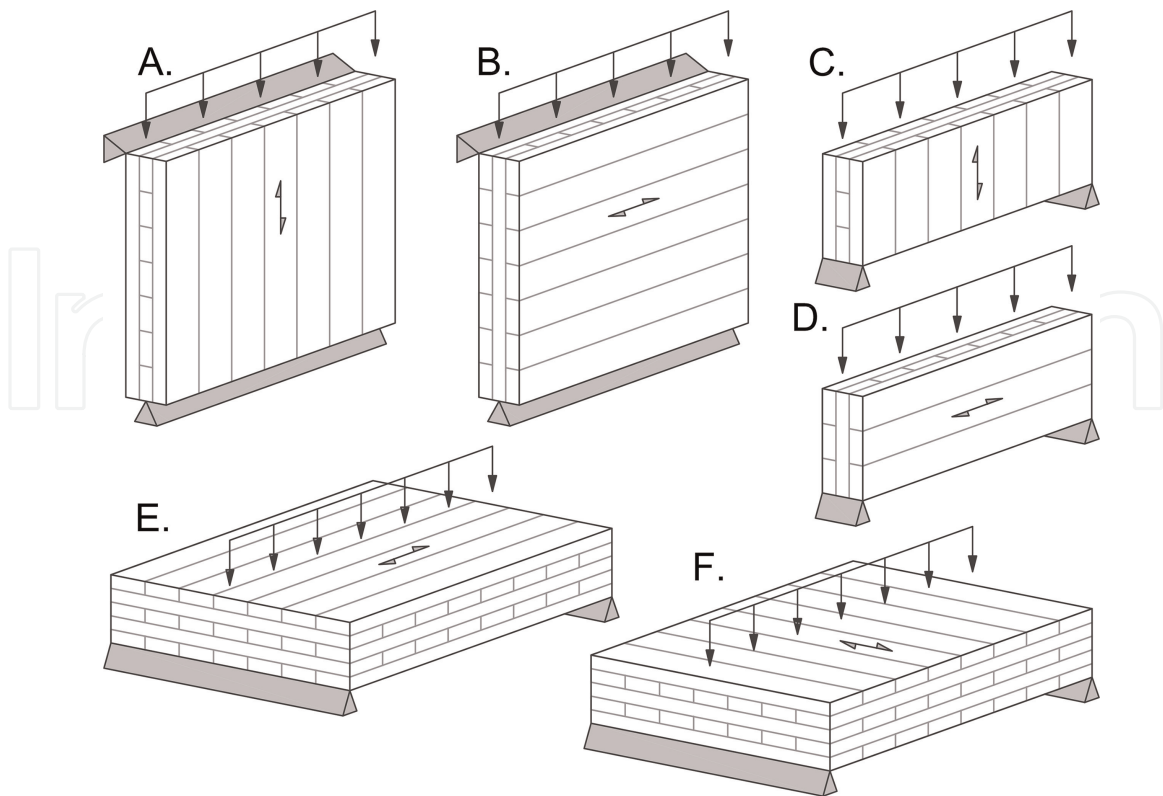105–3 s panel for consideration. For the 2nd floor lintels, a slenderness ratio of 60

Figure 5.
A) Axial loading strong-axis vertical; B) axial loading strong-axis horizontal, C) edgewise bending minor axis;
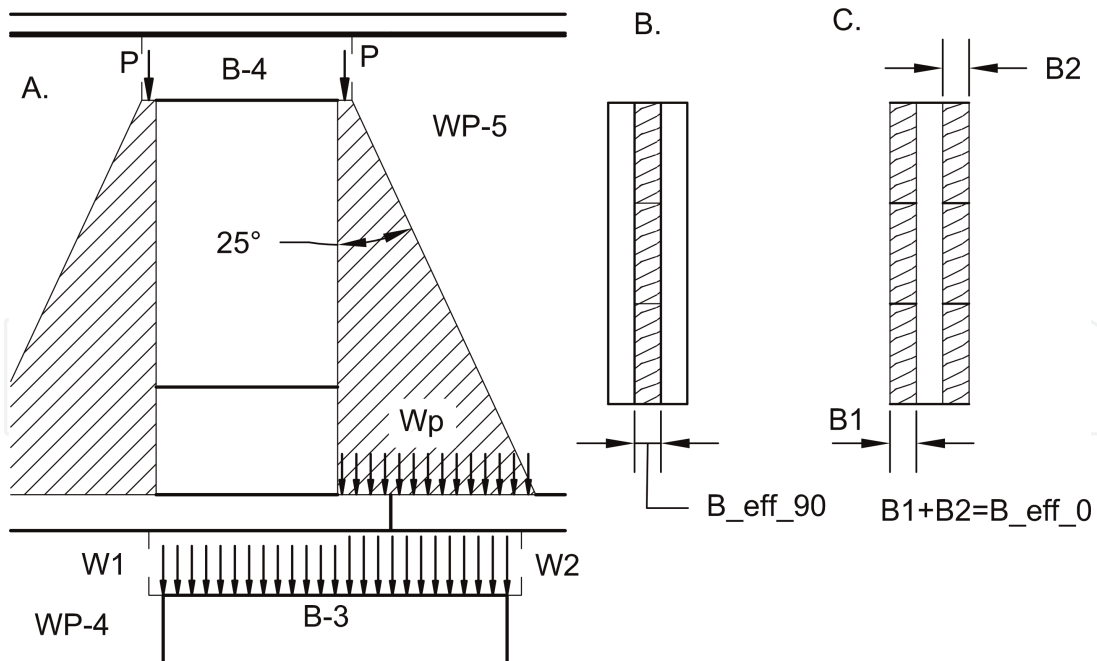*D) edgewise bending major axis; E) flatwise bending major axis; F) flatwise bending minor axis.*

Figure 6.
A.) lintel point load distribution B.) effective width, lintel in strong-axis vertical orientation C.) effective width,
lintel in strong-axis horizontal orientation.

(NDS Section 3.3) was calculated for the initial 89–3 s panels considering an effective length of 2.06 lu = 2.06 x 6 feet = 12.36 feet (NDS Table 3.3.3 for uniformly distributed loading) and an effective width (beff,90) of 0.75 inch. 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 89–3 s panel for a lintel in the strong-axis vertical position. **Figure 6B** depicts the effective width for a lintel installed with the strong-axis vertical. Lintels installed with the strong-axis horizontal, such as the 1st story lintels, have additional effective width to stiffen the beam and increase the overall effective bending width (**Figure 6C**).

The wider 105–3 s panels did meet the slenderness and strength requirements for the openings on the 2nd story and the smaller ones on the 1st; however, they did not meet the strength requirement for the 1st floor larger openings such as B-3 and B-5 shown in **Figure 4**. As mentioned previously, it was necessary to split the panels at these locations and install the lintels with the strong-axis horizontal (**Figures 5D** and **6C**).

4.3 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 Eqs. 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.

$$I_{app} = EI_{app} \div E \tag{1}$$

$$d_{equiv} = \sqrt[3]{\frac{12\,I_{app}}{b}} \tag{2}$$

Preliminary panel sizes were selected from Katerra CLT Pre-Analysis Span Tables [21] and are shown in **Table 1**.

The structural adequacy of floor panels was checked first. Floor panels were designed for one-way major axis bending (**Figure 5E**), spanning continuous over intermediate bearing locations. The 1st floor panel FP1–2 shown in **Figure 3** was first checked using the WoodWorks Sizer software tool [22] and RISA 3D. As can be seen in **Table 2**, the analysis results from RISA 3D and Sizer compared closely.

To check for discrepancies in method, the vibration controlled maximum spans, calculated in Sizer, were compared to both the pre-analysis span table values and those

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

LL = Live Load, SDL = Sustained Dead Load.

Table 1.
Preliminary floor and roof panel selections.

Method	Moment (K-FT)	Shear (K)	Δ_{LL} (in)	Δ_{TL} (in) [1]	Vibration Max. Span (FT)
RISA 3D	1.94	0.615	0.125	0.197	—
Sizer	1.94	0.592	0.130	0.210	16.94
Chapter 7					16.81
Span Tables					16.67

Note: Total deflection is calculated according to NDS Section 3.5.1 with $K_{cr} = 2.0$.

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

computed by hand calculation 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 since it provided the quickest solutions. The remaining floor panel checks were straight-forward using the sizer program. 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 7**, 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 7** shows the free body diagram used as a basis for the garage RISA 3D model.

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 panel was
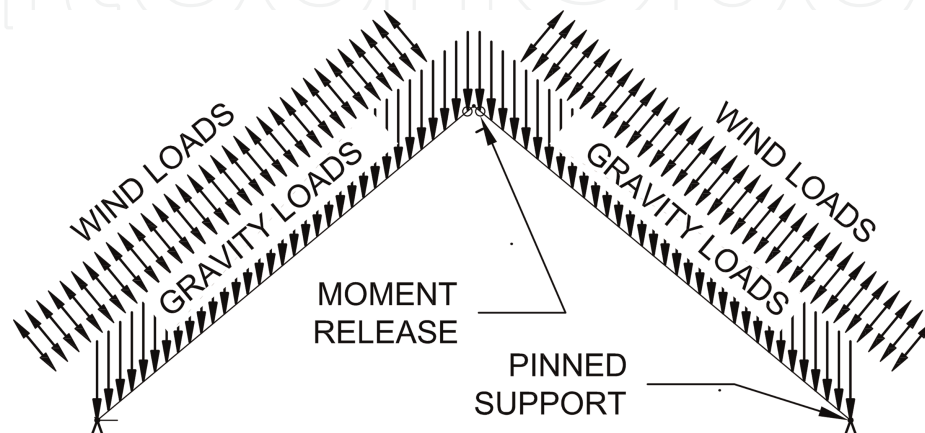

Figure 7.
Garage roof free-body diagram.

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.

4.4 Lateral force-resistance system design (LFRS)

The in-plane stiffness of the floor and wall panels is utilized to provide stability to the structure and transmit lateral wind or seismic load to the foundation. For the State College locale, wind governs the design of the lateral force resistance system. It is conventional to design the LFRS system to respond in a linear elastic manner when subjected to lateral wind loading. [2]. Allowing energy dissipation through permanent deformation of the structure is not necessary for wind design. A number of references that will be discussed in this section were used for the LFRS system design.

Figure 8 identifies many of the LFRS components. The CLT floor and roof panels in this case act as rigid diaphragms transferring wind loads to designated shear segments located within the CLT wall panels. The shear wall (SW1–8) boundaries, outlined in **Figure 8**, 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
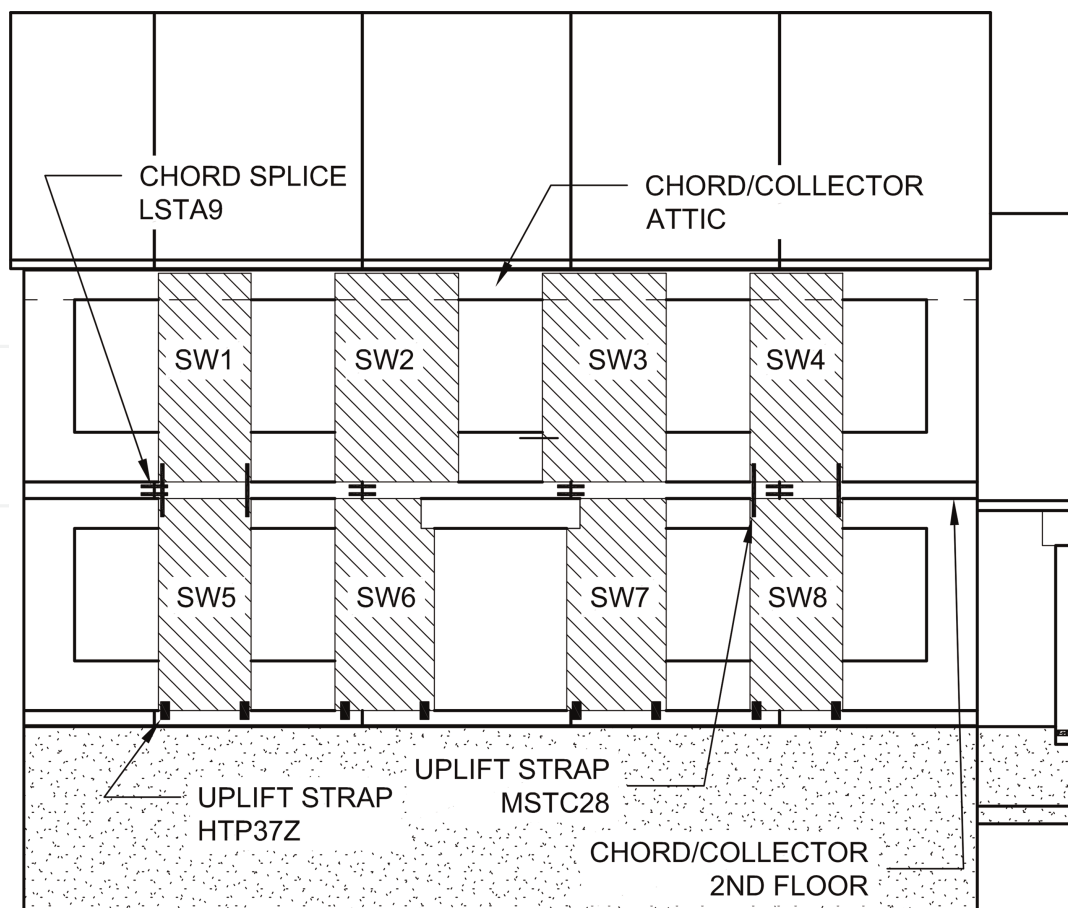

Figure 8.
LFRS components, southern building elevation.

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. As an alternative to utilizing Appendix B as a basis for shear wall and diaphragm design, 2021 SDPWS Section 4.1.2.2 permits CLT shear walls and diaphragms to be designed using alternative procedures that are in accordance with principles of engineering mechanics.

In addition to defining the main load carry components (shear walls, diaphragms), the boundary elements and connections that linked these components also had to be established. In order to ensure continuity of load path at the boundaries, hardware was required at some locations. Straps are used to transfer tensile overturning shear wall chord forces from floor-to-floor and also to the foundation. Straps are also utilized as splices to maintain the continuity of the 2nd floor diaphragm chords. In addition to functioning as a lintel, the CLT material above the wall openings on the 2nd floor also functions as both a chord and collector to transfer attic floor diaphragm loading to shear walls below. This was not possible on the 1st floor due to the split walls; 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 [23]. The chord delineation can be seen in **Figure 9A**.

After defining the main load carrying components of the LFRS and boundary element, it was necessary to design the roof and floor diaphragms. To determine whether or not the panels possessed adequate internal shear strength, the panel edgewise shear stress (Fv) was obtained from Katerra guidance [24]. 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 Fv are required to be reduced by a factor of 2.1 from that of the tested value. According to 2021 SDPWS
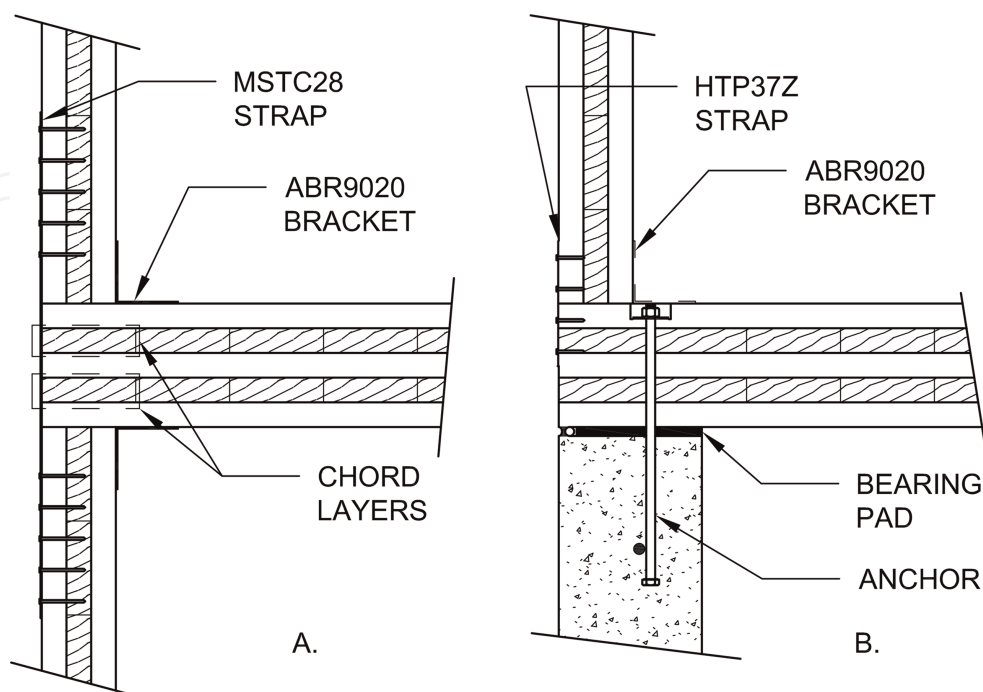

Figure 9.
A.) 2nd story floor intersection detail B.) foundation-floor intersection detail.

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 [25], 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 diaphragm was reviewed 1st. The roof diaphragm is only required to resist a small amount of lateral wind load in the plan east-west direction. Load in the north-south direction is primarily transferred through the attic floor. The demand on the roof diaphragm relative to the capacity of the panels is low. A shear capacity of approximately 10,000 plf was calculated for the K3–0410 panels sized previously for gravity loading. The calculated shear demand of 12 plf was insignificant.

The geometry of the remaining floor diaphragms was reviewed next. According to ASCE 7 Section 26, the building would qualify as a simple diaphragm building. It was assumed that the building would also qualify as a torsionally regular building exempt from the torsional wind load cases in Figure 27.4–8 (ASCE Appendix D1.3). No codified length-to-width ratio limitations were identified, therefore Table 4.2.2 of the 2021 SDPWS was used to gauge the likely effectiveness/efficiency of the diaphragm. A maximum length-to-width ratio of 36-feet/30-feet = 1.2 was calculated. This was lower than the 4:1 ratio established in the table for a block diaphragm.

Next, an investigation of diaphragm flexibility was conducted. 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 [8]. The deflection of the dia-phragm was estimated at 0.092 inches based on calculation methods like those used by Spickler [23]. 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 dia-phragm 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 [26]. 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 are 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 concluded that this lower limit is not applicable for structures subject to wind only. Based on the review of Chapter 4 in the CLT handbook and NEHRP Recommended Seismic Provisions for New Buildings and Other Structures section C14.5.2 [27], 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. The 2:1 aspect ratio appears to be a lower bound geometric marker that comes from seismic testing and represents a transitional point between panel rocking behavior and panel sliding behavior [28]. The more desirable rocking behavior permits greater deformation and energy dissipation during a seismic event. This lower bound limit would be irrelevant in a linear elastic analysis.

The rigid diaphragm analysis was conducted on the attic diaphragm. **Figure 10** shows the loading. Methods utilized by Breyer [29] and the U.S. Department of Housing and Urban Development (HUD) [30] 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.

The shear loading was distributed to the panels based on the results of the rigid diaphragm analysis. By engineering judgment, it was assumed that the panels had adequate shear strength based on the large, calculated roof panel shear capacity.

After determining the distribution of the diaphragm shear load, the chord forces resulting from overturning action were calculated for each wall segment.
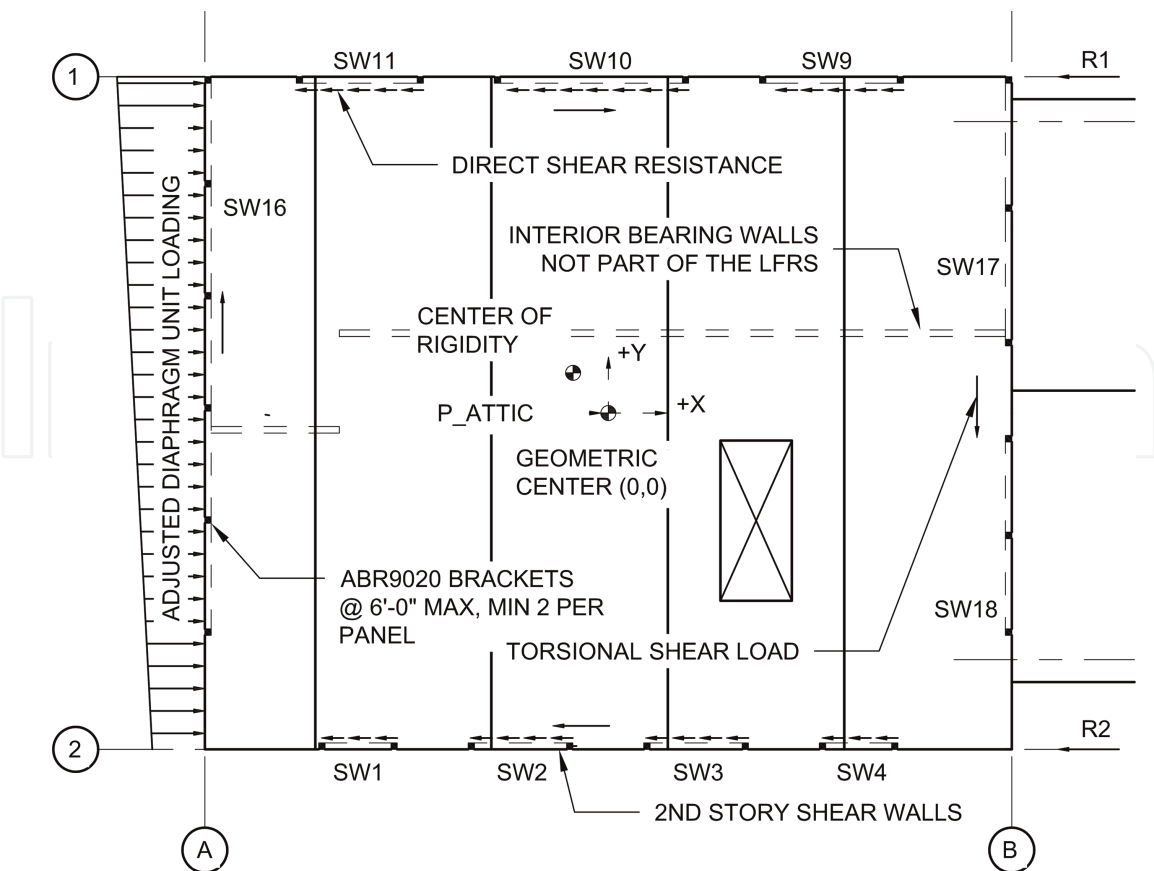

Figure 10.
Attic diaphragm rigid diaphragm analysis.

	Wall Line 1 (plf)	Wall Line 2 (plf)	Percent Difference
Rigid Diaphragm	133	145	7.5%
Flexible Diaphragm	123	157	- 8.3%

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

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 11A** 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 11B**, 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 strap 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
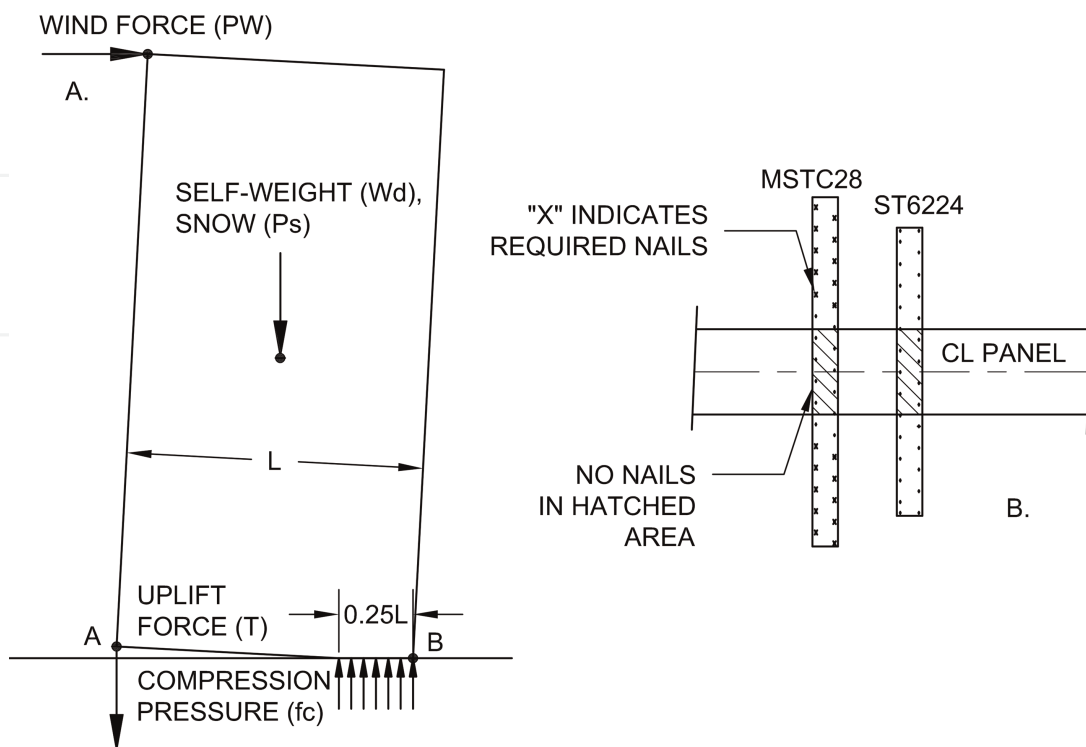

Figure 11.
A.) shear panel overturning chord forces B.) tension strap to transfer chord forces.

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 eq. 6.11 in the Swedish CLT Handbook [8], 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.

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 and 145 plf was estimated, but not utilized for design. By engineering judgment, 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 9A** and **B**. The brackets were selected from the Simpson Strong-Tie mass timber construction catalog [31]. 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.

The same analysis that was conducted for the attic diaphragm and the 2nd story shear walls was also conducted for the 2nd floor diaphragm and 1st 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 2nd story was also determined to be acceptable for the 1st story connections as well. Differing from the 2nd story specification, however, was the tension hold-downs required to stabilize the 1st 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 9B shows the floor-to-wall foundation connection. Minimal anchorage was provided to ensure positive attachment to the foundation. An elastomeric bearing pad was specified to bridge inconsistencies in the top of the wall finish and to help seal the joint.

4.5 Connections

The most significant connections designed for this structure include the roof peak connection, roof-floor connection, floor intersection detail and the foundation-floor intersection detail. 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 **Figure 12B**, three ¼-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 ⅜ -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 LR (Roof Live) + 0.75 (0.6 W).

Due to the geometry, the lag screw connection was subject to both withdrawal and lateral loading. 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 $F_{e\_parallel}/F_{e\_perpendicular}$ 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.

Next, the roof base connection was designed. This connection, as can be seen in **Figure 13**, 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
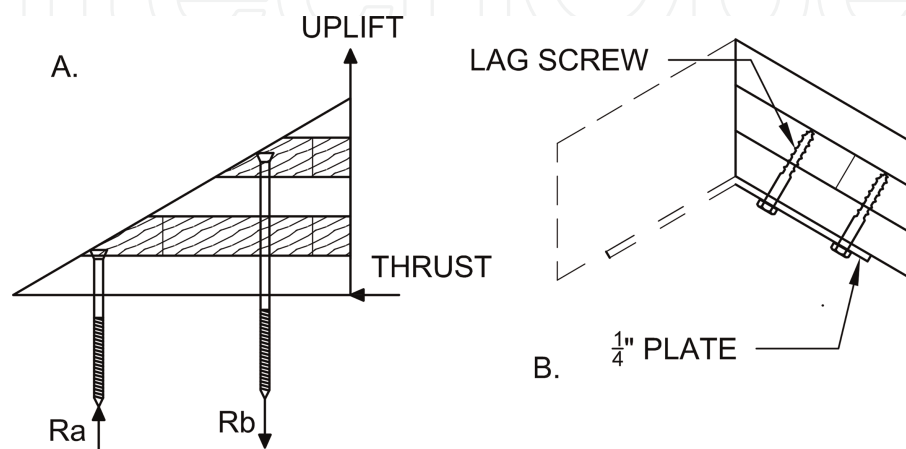

Figure 12.
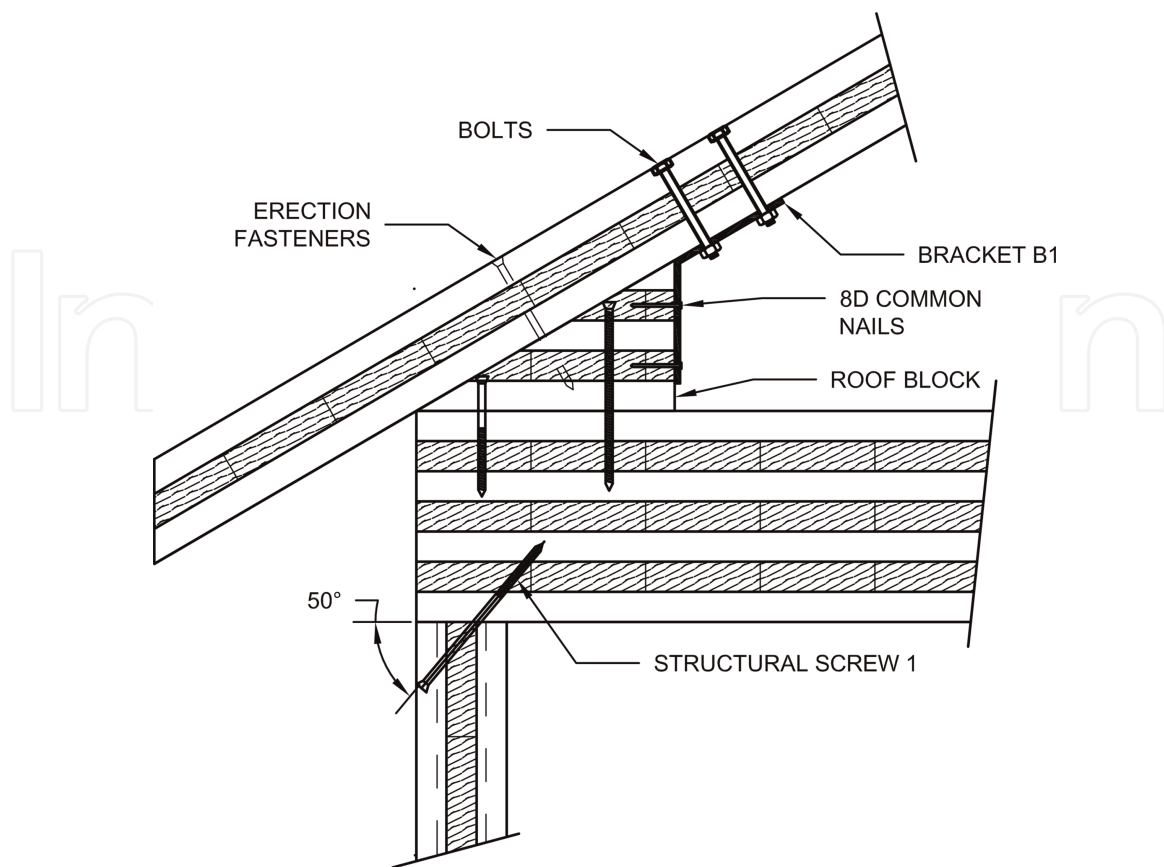*A.) roof panel base block forces B.) roof peak connection.*

Figure 13.
Roof-attic floor connection detail.

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 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 12B**, from the block to the 2nd floor panel. MyTiCon structural screws were evaluated and selected from their catalog [32]. Initially, the ASSY Ecofast screw was considered, but discarded. The Ecofast partially threaded screw, as depicted in **Figure 12B**, 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.

Next, the angled screw connection, shown in **Figure 13**, 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 connection's 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.
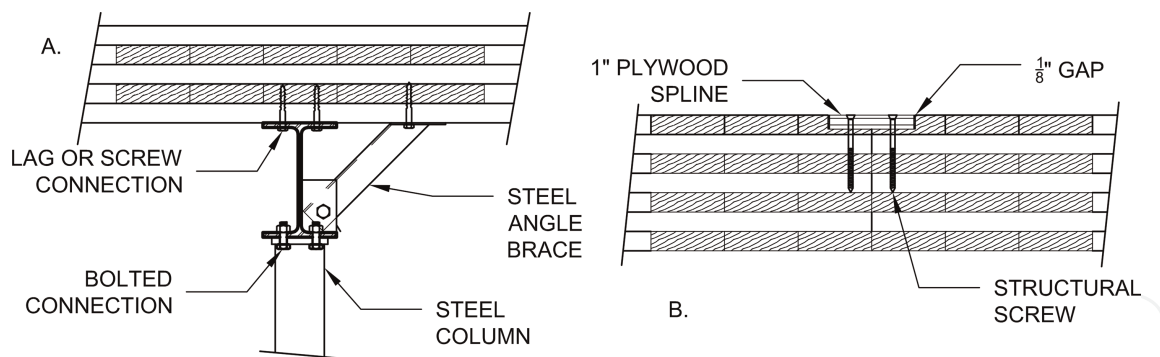
Figure 14.
A.) steel girder-to-floor detail B.) panel-to-panel splice detail.

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 [32] was used to evaluate the geometry factor (C_Δ). 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 [33]; 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 **Figure 14B**, 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 [32]. Spickler details a splice in his horizontal diaphragm design example [23], and Brenneman also discusses typical splice design in his presentation [25]. 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 (**Figure 9A**), Foundation-Floor Intersection Detail (**Figure 9B**), the Interior Top-of-Wall Detail, and the Girder Bearing Detail (**Figure 14A**) were all straightforward designs and relied on the same principles previously discussed for the other connections.

It is also noteworthy to mention interior Top-of-Wall detail. As shown in **Figure 10**, 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.

5. Conclusions

CLT is a versatile building material that can be used to construct the structural system for single-family homes. The addition of CLT to the international building

code and the NDS allows for increased use and confidence in the material by potential users. CLT is typically a more costly structural system for conventional single-family dwellings when compared to light-wood framing; however, it can have some advantages when large spans or unique geometric shapes are required. If CLT is to be considered for use in single-family projects, then the efficiency of the workflow should be maximized. The following is a list of the recommendations to help improve workflow in a residential CLT project:

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. Select dimensions that are compatible with the chosen manufacturers standard panel sizes.

3. For a single-family dwelling design, the design team should anticipate that it will be responsible for the full panel layout and the complete engineering design of the panels.

4. Very little prescriptive design aids are currently available for designers and hand calculations can be time consuming. Consider utilizing the CLT design software packages that are commercially available.

5. Avoid expensive connections and interior bearing wall whenever possible.

6. Create a digital 3D building information model (BIM) of the panelized structure. Some manufacturers can use the model directly for fabrication.

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