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

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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

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Disclaimer

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).

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
2. According to *Mass Timber Design Manual* (2021):
    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

3. According to (*Solid Advantages* (2012):
    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
4. According to Reference Evans (2013):
    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.

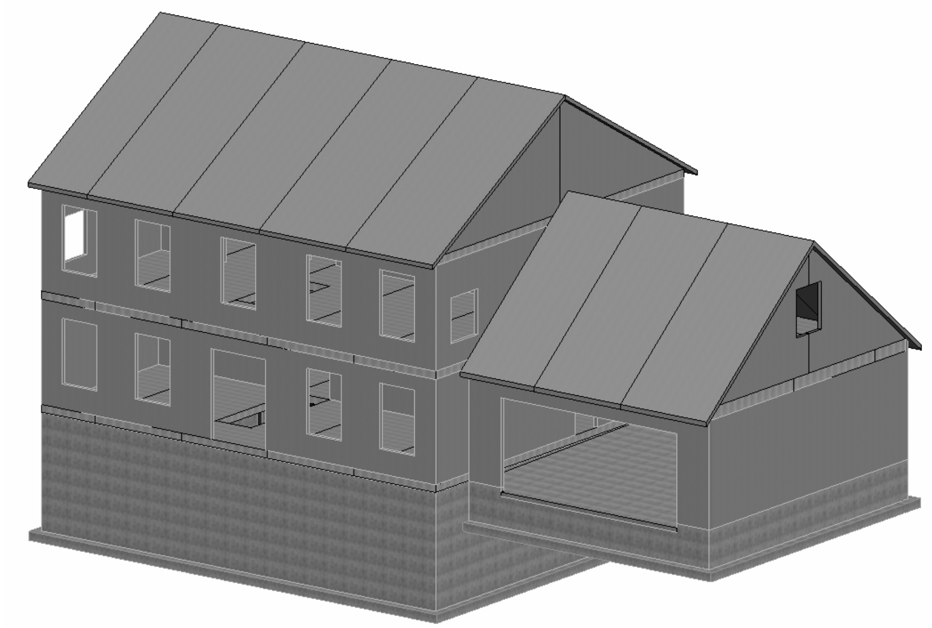


Figure 2. Rendering of CLT Panelized Home Design.

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.

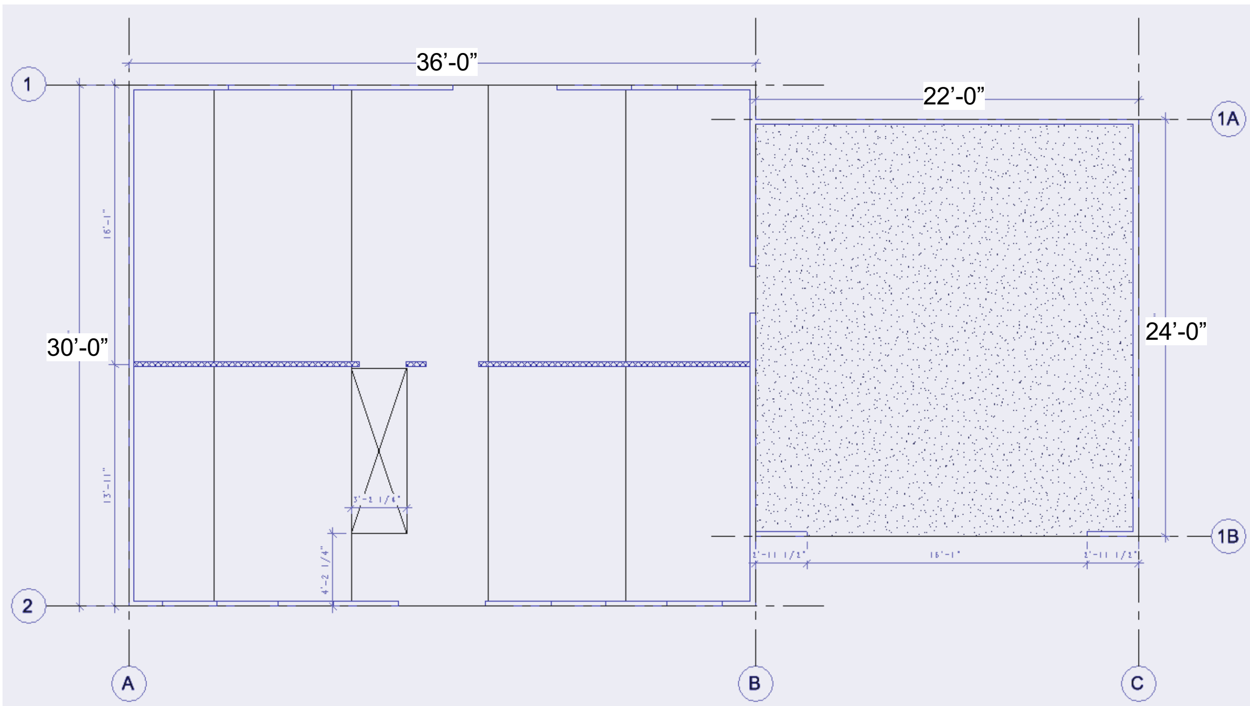


Figure 3. First floor plan.

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.

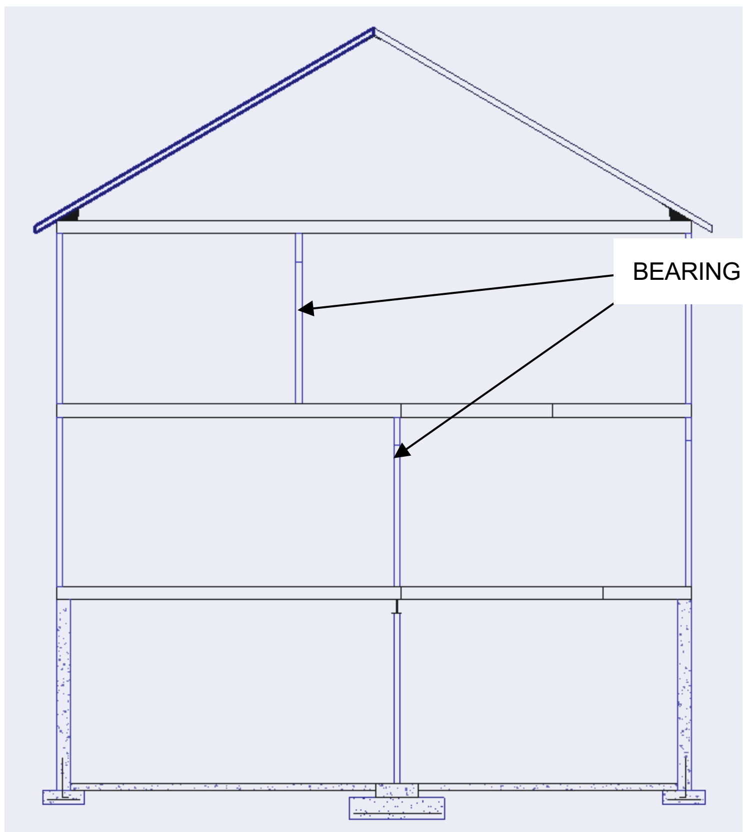


Figure 4. Building section.

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.

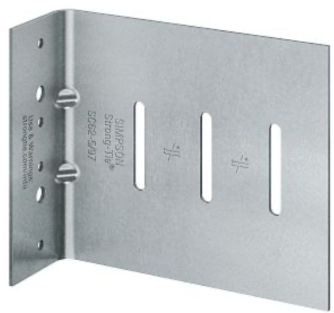


Figure 5. Simpson Strong-Tie (SST) SC bypass framing side clip connector (Image from SST C-CF-2020 Catalog)

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

CLT stress grade	E1	
	Layers	
Orientation	Longitudinal	Transversal
Species combination	S-P-F	S-P-F
Stress class	1950f-1.7E MSR	No. 3
Bending at extreme fiber, F_b (psi)	1,950	500
Shear parallel to grain, F_v (psi)	135	135
Rolling shear, F_s (psi)	45	45
Compression parallel to grain, F_c (psi)	1,800	650
Compression perpendicular to grain, F_{cp} (psi)	425	425
Tension parallel to grain, F_t (psi)	1,375	250
Modulus of elasticity, E (psi)	1,700,000	1,200,000
Shear modulus, G (psi)	106,250	75,000
Rolling shear modulus, G_s (psi)	10,625	7,500

a.)

b.)

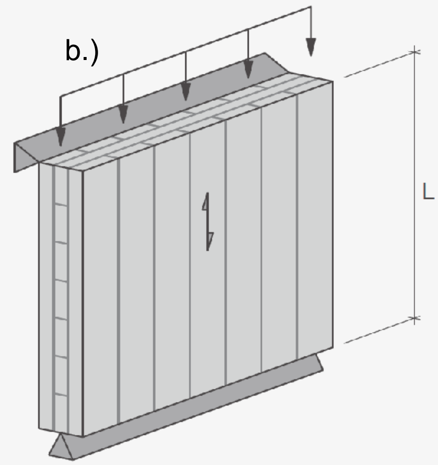


Figure 6. a.) E1 stress grade reference design values b.) CLT wall panel shown with strong-axis vertical. Images from Nordic (Nordic 2020).

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 reference design properties were obtained from the Nordic technical guide. Design capacity was calculated based 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 89-3s 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.

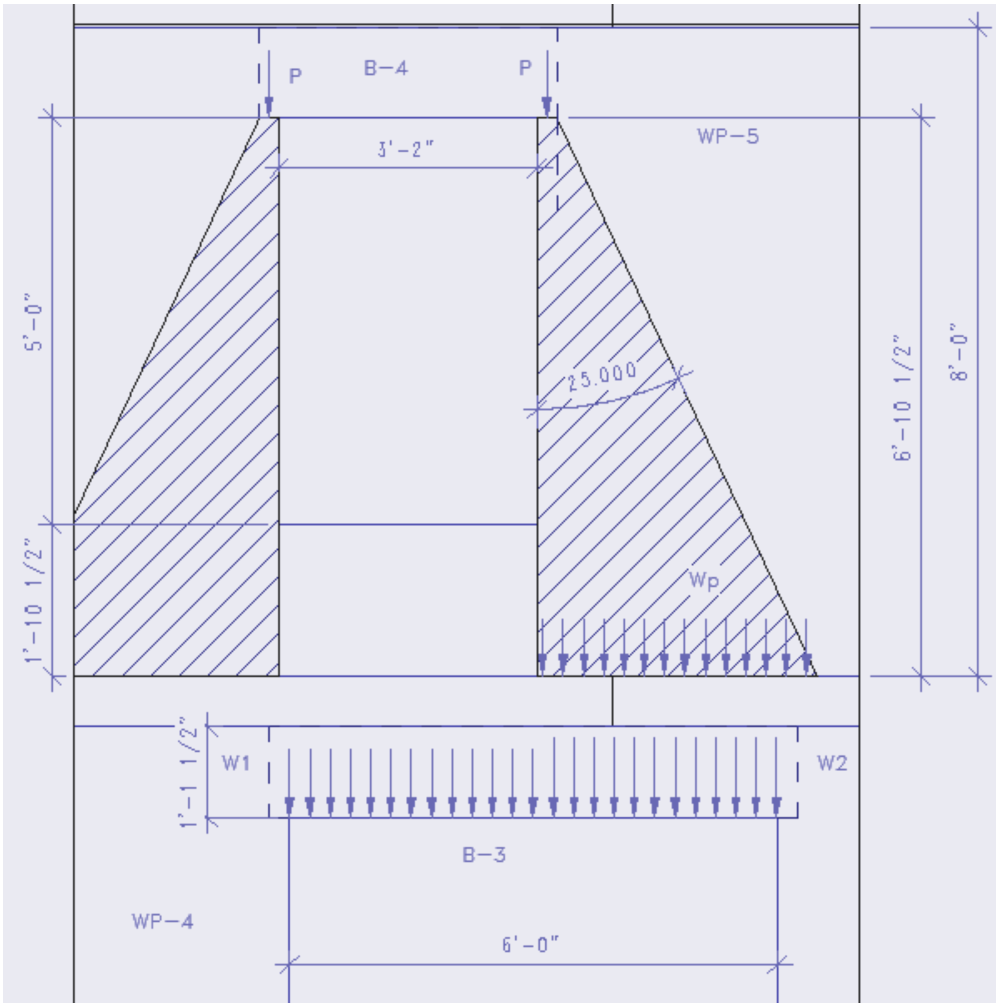


Figure 7. B-3 lintel loading

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).

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 ($b_{eff,90}$), as shown in Figure 9a, can be used to resist bending forces.

Shear Walls, Lintels, and Diaphragms – Design Properties

CLT stress grade		
Layup combination	89-3s	105-3s
Loading parallel to outermost layers		
Edgewise bending (z-z)		
Effective width for bending, $b_{eff,90}$ (in.) [(a)]	0.75	1.38
Shear capacity, $F_{v,90}$ (psi) [(b)]	190	190
Shear rigidity, $G_v t_{v,90}$ (10^6 lbf/ft) [(c)]	1.52	1.79
Loading perpendicular to outermost layers		
Edgewise bending (z-z)		
Effective width for bending, $b_{eff,0}$ (in.) [(a)]	2.75	2.75
Shear capacity, $F_{v,0}$ (psi) [(b)]	155	155
Shear rigidity, $G_v t_{v,0}$ (10^6 lbf/ft) [(c)]	1.52	1.79

a.)

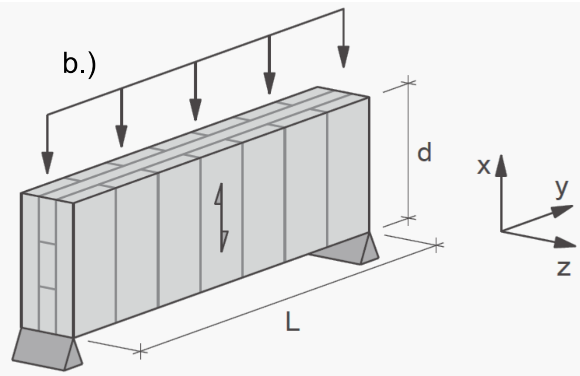


Figure 8. a.) Lintel design properties b.) Lintel shown with strong-axis vertical. Images from Nordic (Nordic 2020).

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 l_u = 2.06 x 6 feet = 12.36 feet (NDS Table 3.3.3 for uniformly distributed loading) and an effective width ($b_{eff,90}$) 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.

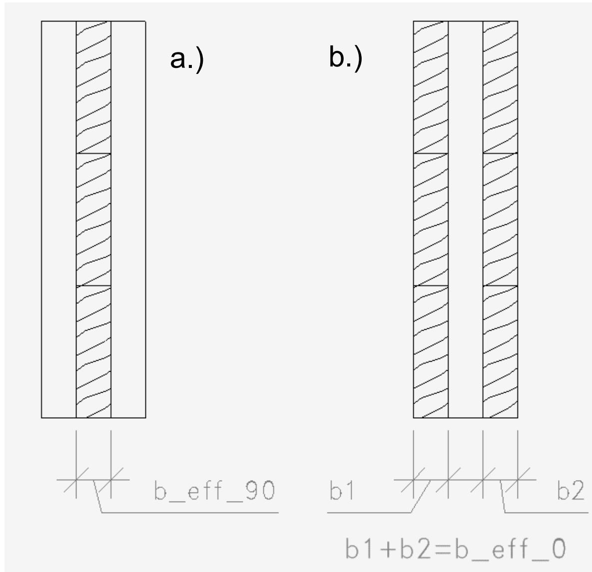


Figure 9. Lintel and effective width for bending shown for, a.) loading parallel to the outermost layers b.) loading perpendicular to outermost layers.

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.

$$I_{app} = EI_{app} \div E \qquad [1]$$

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

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.

Assembly	Table Load [1] (LL, SDL)	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

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.

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

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.

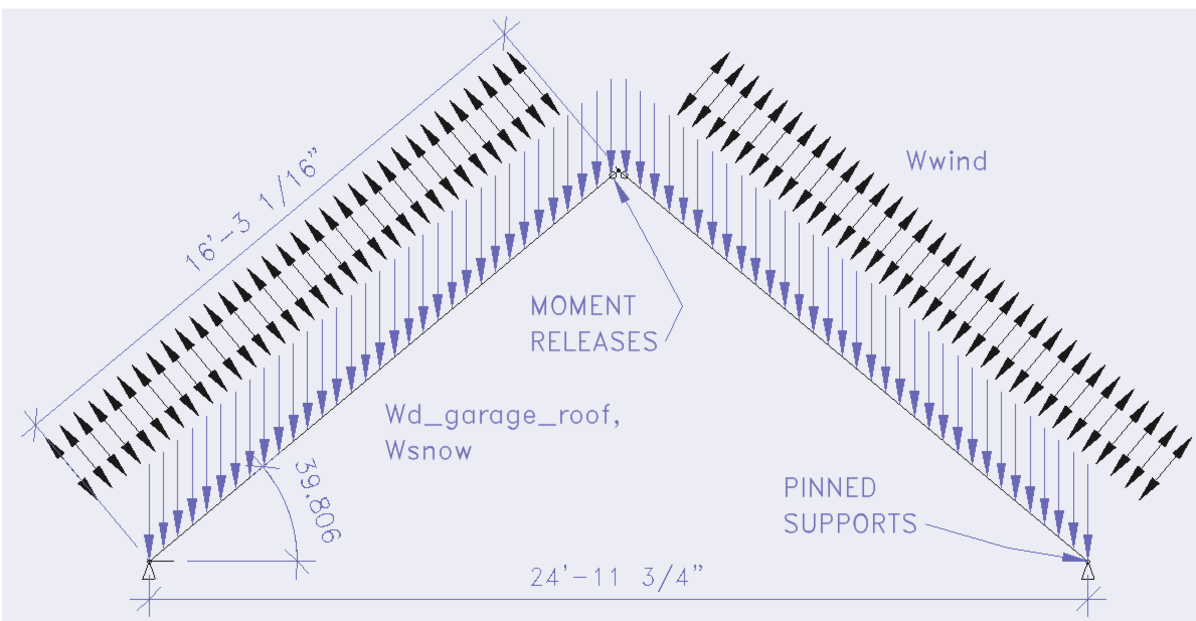


Figure 10. Garage roof free-body diagram.

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.

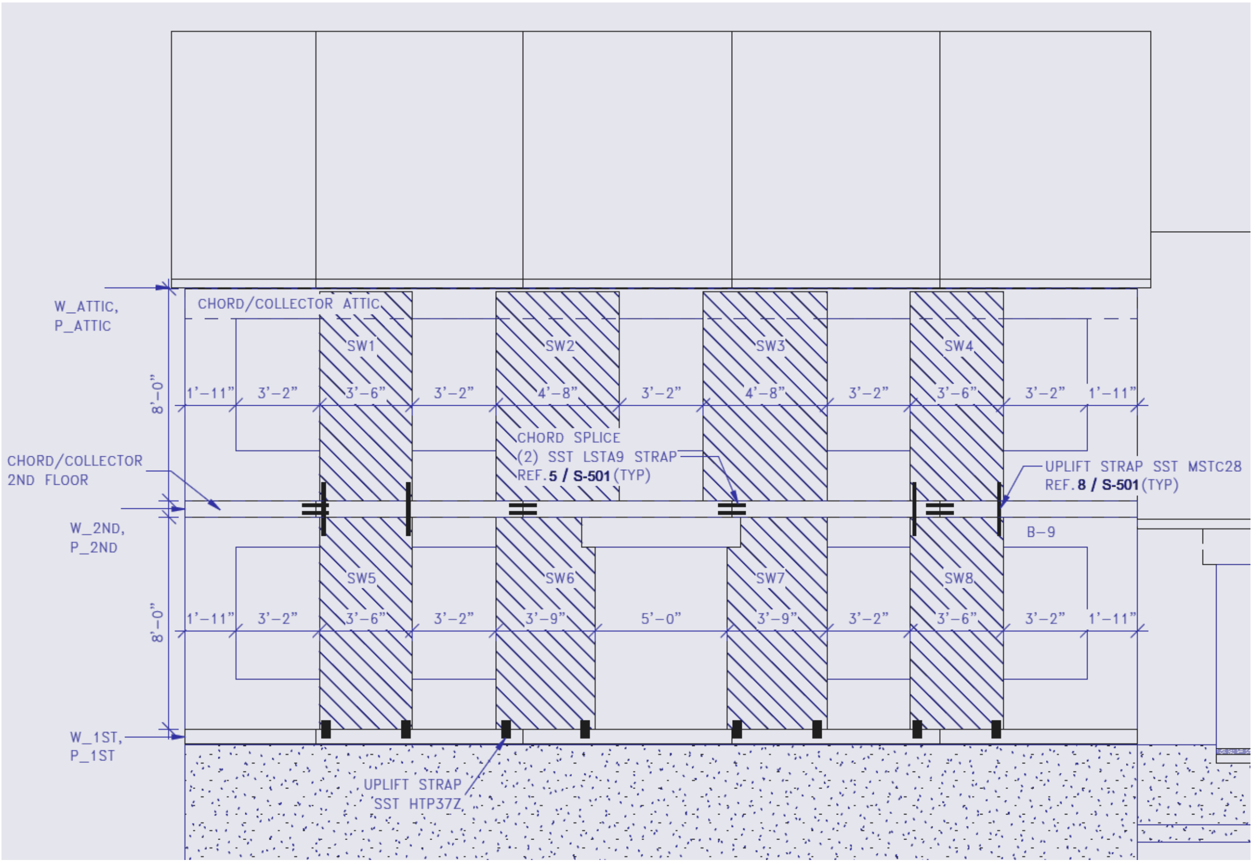


Figure 11. LFRS components, southern building elevation.

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 4.2.2 Maximum Diaphragm Aspect Ratios
(Flat or Sloped Diaphragms)

Sheathed Wood-Frame Diaphragm Assemblies	Maximum L/W Ratio
Wood structural panel, unblocked	3:1
Wood structural panel, blocked	4:1
Single-layer horizontally-sheathed lumber	2:1
Single-layer diagonally-sheathed lumber	3:1
Double-layer diagonally-sheathed lumber	4:1

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.

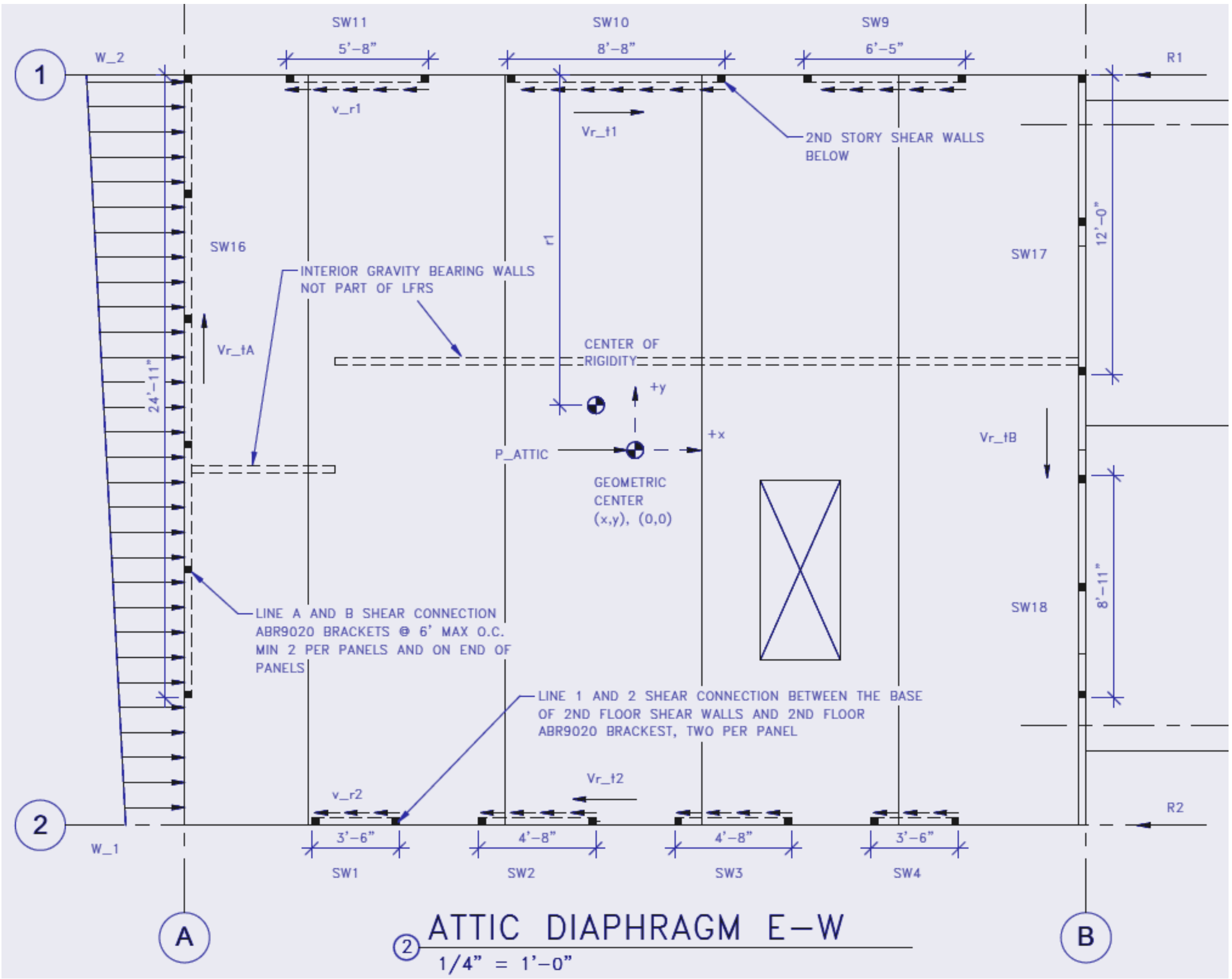


Figure 13. Attic diaphragm rigid diaphragm analysis.

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

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

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.

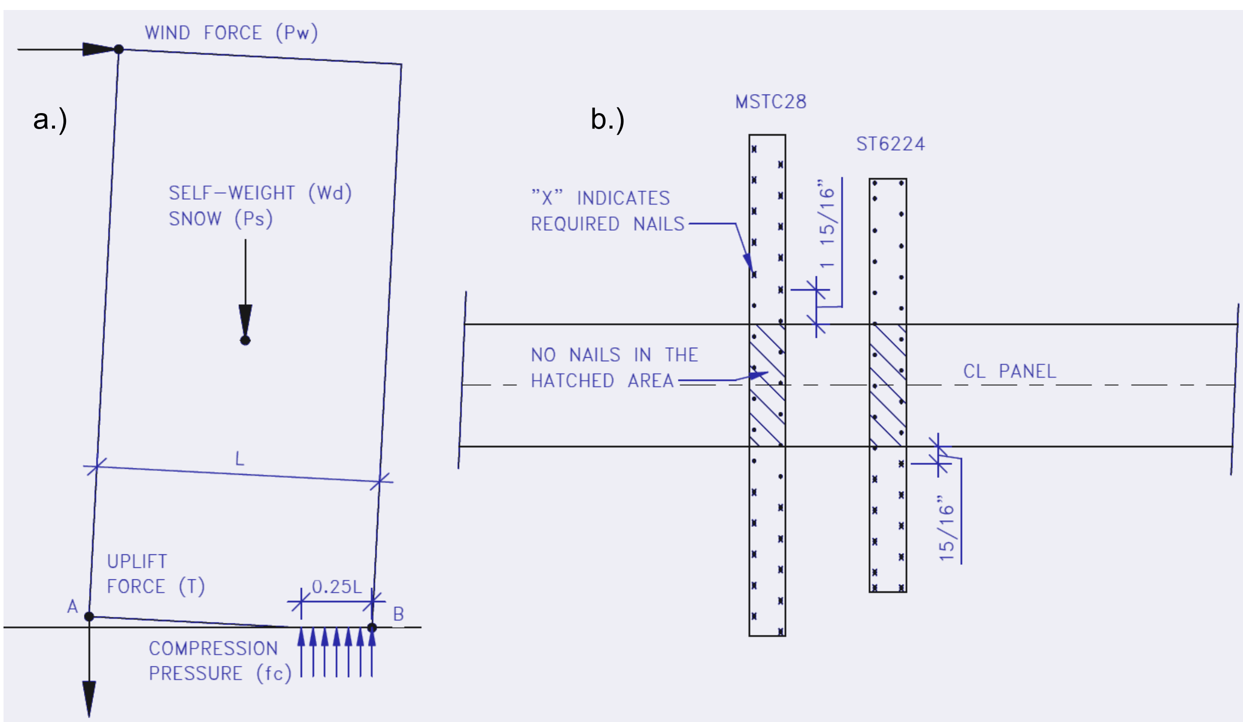


Figure 14. a.) Shear panel overturning free-body diagram b.) Shear panel tension strap.

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.

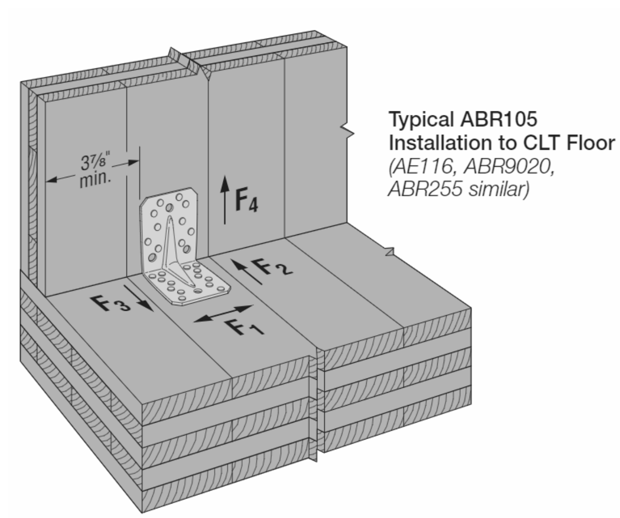


Figure 15. Simpson Strong-Tie shear transfer bracket (Strong-Tie 2020).

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 ft < 0.15 (36 ft) = 5.40 ft; therefore, not significant.
2. Length of the opening (L_o) no greater than 15% of the diaphragm length.

   a. L_o = 7.15 ft > 0.15 (30 ft) = 4.5 ft; therefore, significant.

3. Distance from diaphragm edge to opening edge (D_e) less than three (3) times the larger opening.
   a. D_e = 6.55 ft < 3 (7.15 ft) = 21.45 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.

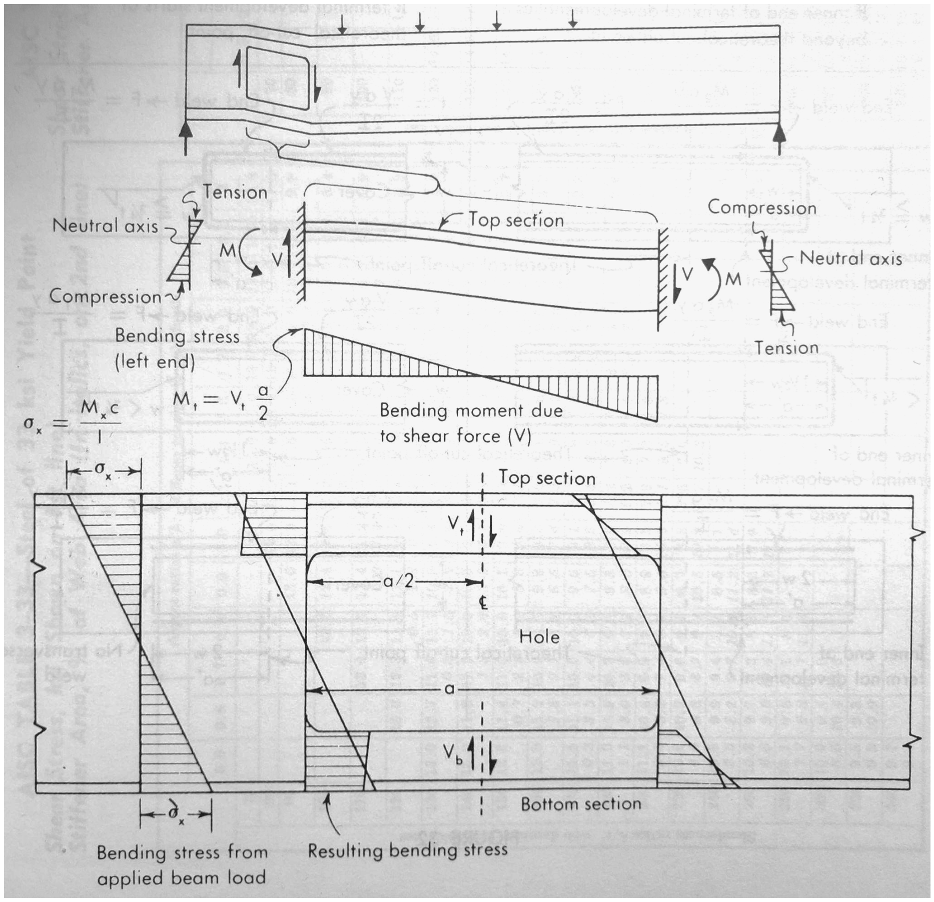


Figure 16. Description of internal forces around a steel girder opening (Blodgett 1966).

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 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 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.

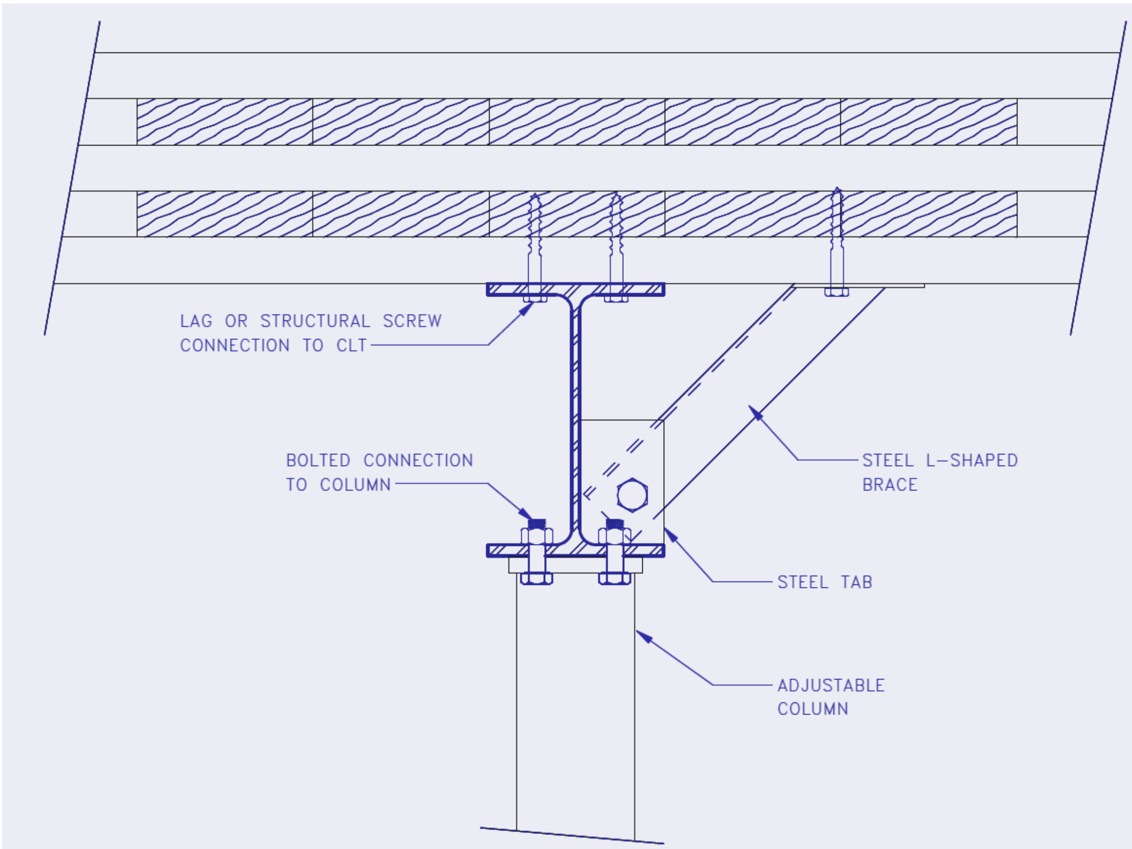


Figure 17. Steel girder rotational restraint detail.

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 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 L_R (Roof Live) + 0.75 (0.6 W).

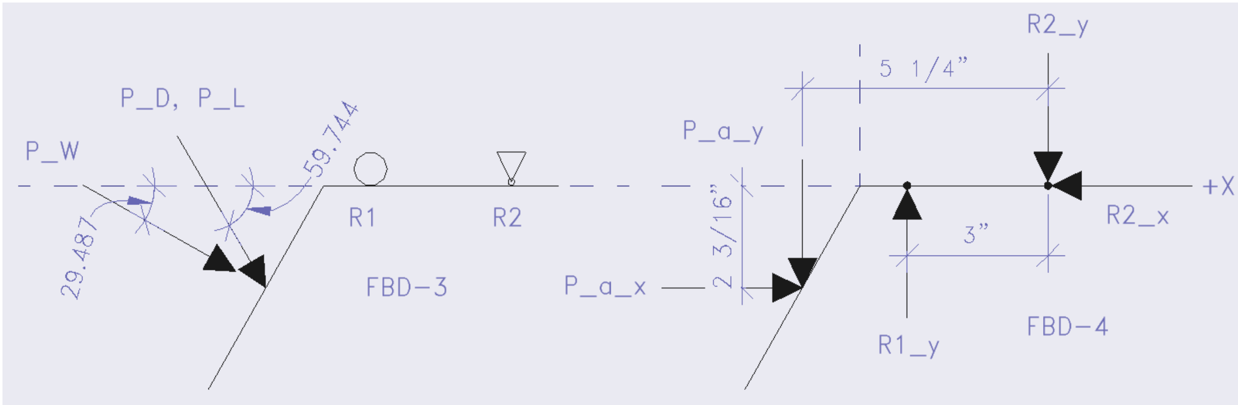


Figure 18. Roof peak connection free-body diagrams.

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 $F_{e\_parallel}$ / $F_{e\_perpindicular}$ 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 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.

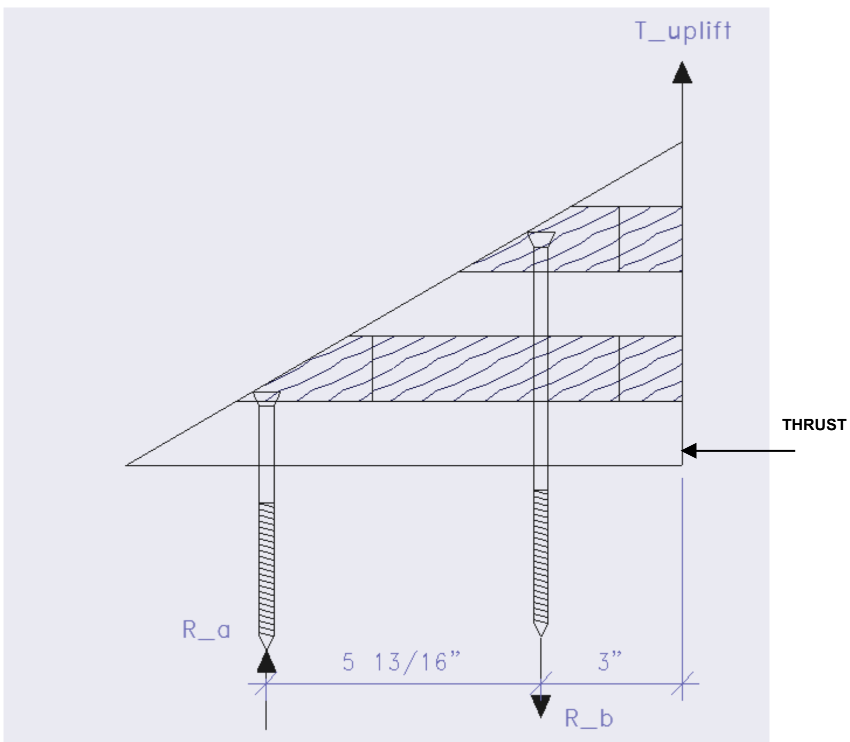


Figure 19. Wooden block eccentric force resolution.

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_{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 (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

2015 International Residential Code for One-and Two-Family Dwellings. (2014). International Code Council Country Club Hills, IL.

American Institute of Steel Construction (AISC). (2011). *Steel Construction Manual*. American Institute of Steel Construction.

American Wood Council. (2015). “Cross-Laminated Timber in the 2015 IBC and NDS (MAT242).”

American Wood Council. (2020). *Special Design Provisions for Wind and Seismic (SDPWS)*. American Wood Council.

American Wood Council, AWC. (2019). WoodWorks Sizer [Computer Software]. Ottawa, ON: Woodworks Software

American Wood Council, AWC. (2015). *National design specification for wood construction*. AWC Leesburg, VA.

APA. (2019). *Cross-Laminated Timber Selection and Specification*. The Engineered Wood Association.

APA. (2020). *ANSI/APA PRG 320: 2018: Standard for performance-rated cross-laminated timber*. APA - The Engineered Wood Association.

ATC. (2020). “ATC Hazard by Location.” *Applied Technology Council (ATC)*, <https://hazards.atcouncil.org/> (Dec. 9, 2020).

AWC. (2017). *2018 National Design Specification for Wood Construction*. American Wood Council Leesburg, VA, USA.

Blodgett, O. W. (1966). *Design of welded structures*.

Borgstrom, E., and Frobel, J. (Eds.). (2019). *The CLT Handbook, CLT Structures - Facts and Planning*. Swedish Wood.

Breneman, S., and Line, P. (2020). “CLT Diaphragm Design: New Code Provisions and Design Examples.”

Breneman, S., Timmers, M., and Richardson, D. (2019). *Tall Wood Buildings in the 2021 IBC Up to 18 Stories of Mass Timber*. WoodWorks.

Breyer, D. E., J., F. K., Pollock, D. J., and Cobeen, K. E. (Eds.). (2003). “Advanced Topics in Lateral Force Design.” *Design of Wood Structures ASD 5th Ed.*, McGraw-Hill, 16.39.

DeStafano, J., and Way, R. (2020). *Cross-Laminated Timber (CLT) Diaphragms*. Timber Frame Engineering Council.

Evans, L. (2013). *Cross Laminated Timber Taking Wood to the Next Level*. reThink Wood, American Wood Council, FPInnovations.

FEMA. (2020). *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures Volume I: Part 1 Provisions, Part 2 Commentary*.

Ford, C. (2020). *Cost of Constructing a Home*. NAHB Economics and Housing Policy Group.

Gräfe, M., Dietsch, P., and Winter, S. (2018). “CLT under in-plane loads: Investigation on stress distribution and creep.” *Meeting 51*.

International Code Council. (2014). *International Building Code 2015*. International Code Council.

Jellen, A., and Memari, A. (2019). “Structural Design of a Typical American Wood-Framed Single-Family Home.” *Timber Buildings and Sustainability*, G. Concu, ed., InTechOpen, 27–54.

Karacabeyli, E., and Douglas, B. (2013). “CLT handbook.” *US Edition, FPInnovations and Binational Softwood Lumber Council, Point-Claire, Quebec*.

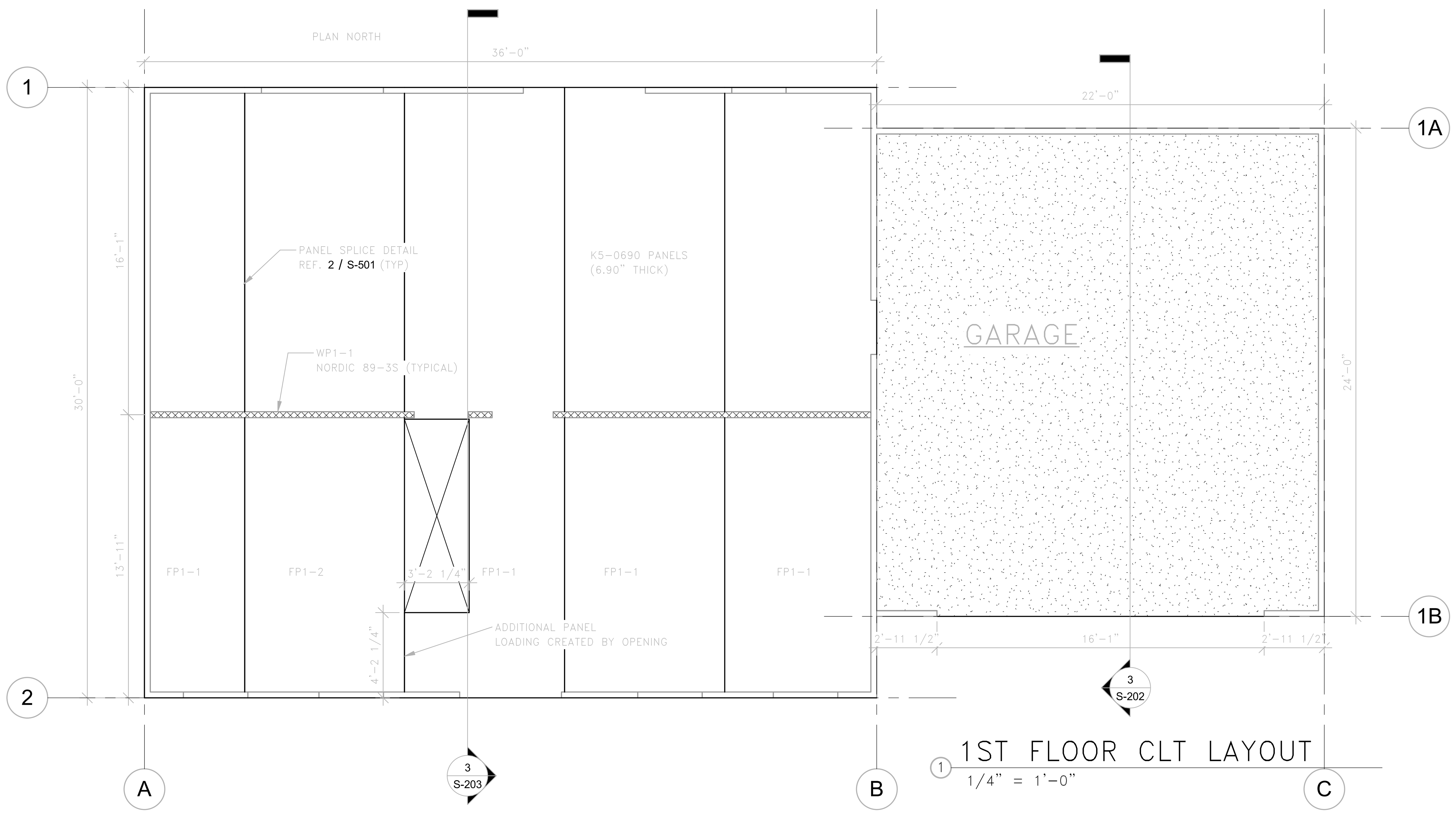
Katerra. (2020a). *Katerra CLT Product Definition*.

Katerra. (2020b). *Katerra CLT Pre-Analysis Span Tables*.

Kremer, P. D. (2018). “Design for Mass Customized Manufacturing and Assembly (DfMCMA): A

new Framework for Mass Timber Construction." *Mass Timber Construction Journal*, 1.
Mahamid, M. (2020). "Cross-Laminated Timber Design: Structural Properties, Standards and Safety." McGraw Hill, New York, New York.
Malone, T. R., and Rice, R. W. (2011). *The Analysis of Irregular Shaped Structures Diaphragms and Shear Walls*. McGraw Hill Professional.
"Manufacturer Directory." (2021). *apawood.org*, <https://www.apawood.org/manufacturer-directory> (September, 6 2021).
Mass Timber Design Manual. (2021). Think Wood, WoodWorks Wood Product Council.
MTC Solutions. (2020). *Structural Screw Design Guide*.
MyTiCon. (2019). *CLT Connections Design Guide*.
NAHB Research Center. (2001). *Model Guidelines for Design, Fabrication, and Installation of Engineered Panelized Walls*. Department of Housing and Urban Development (HUD).
Nordic. (2020). *Nordic X-LAM Technical Guide*. Nordic Structures.
SMARTLAM. (2020). *Cross-Laminated Timber Stairwell and Elevator Shafts*.
Solid Advantages. (2012). WoodWorks.
Spickler, K. (2020). .
Spickler, K., Closen, M., Line, P., and Pohll, M. (2015). *Cross Laminated Timber Horizontal Diaphragm Design Example*.
Strong-Tie, S. (2020). *Connectors & Fasteners for Mass Timber Construction*.
Taylor, A., Hamilton, T., III, and Nanni, A. (Eds.). (2015). "Chapter 8 - Diaphragms." *The Reinforce Concrete Design Handbook SP-17(14)*, American Concrete Institute (ACI), Farmington Hills, 306–322.
Wallner-Novak, M., Koppenlhuber, J., Pock, K., and Augustin, M. (2017). *Cross-Laminated Timber Structural Design Volume 2 Applications*. proHolz Austria.
Wikimedia Commons contributors. (2021). "File:Многослойная клееная деревянная панель Райн Александр Дмитриевич.jpg." Wikimedia Commons, the free media repository.
Woodworks. (2019). *Mass Timber Cost and Design Optimization Checklists*. Woodworks.

Appendix A – Structural Drawings



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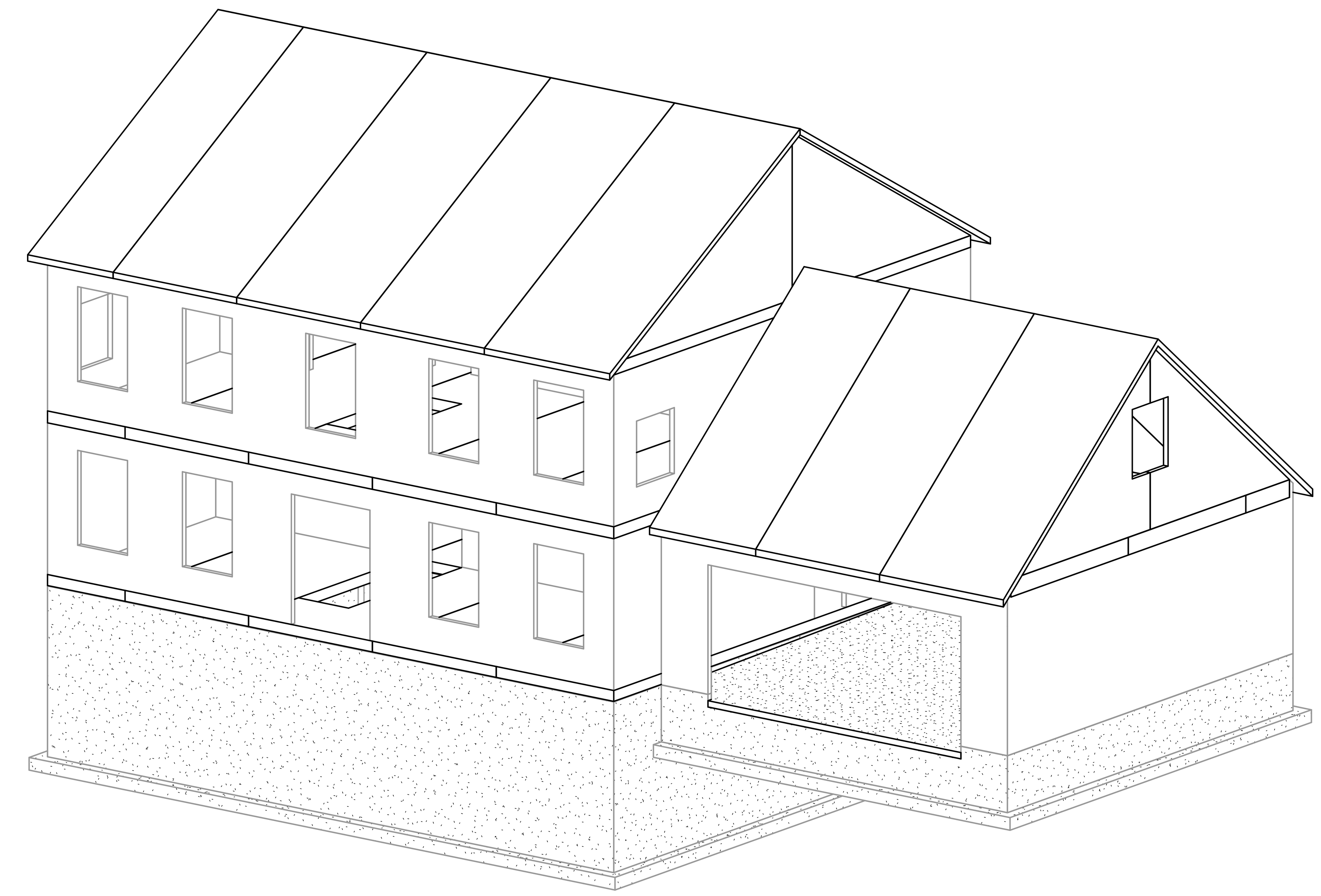

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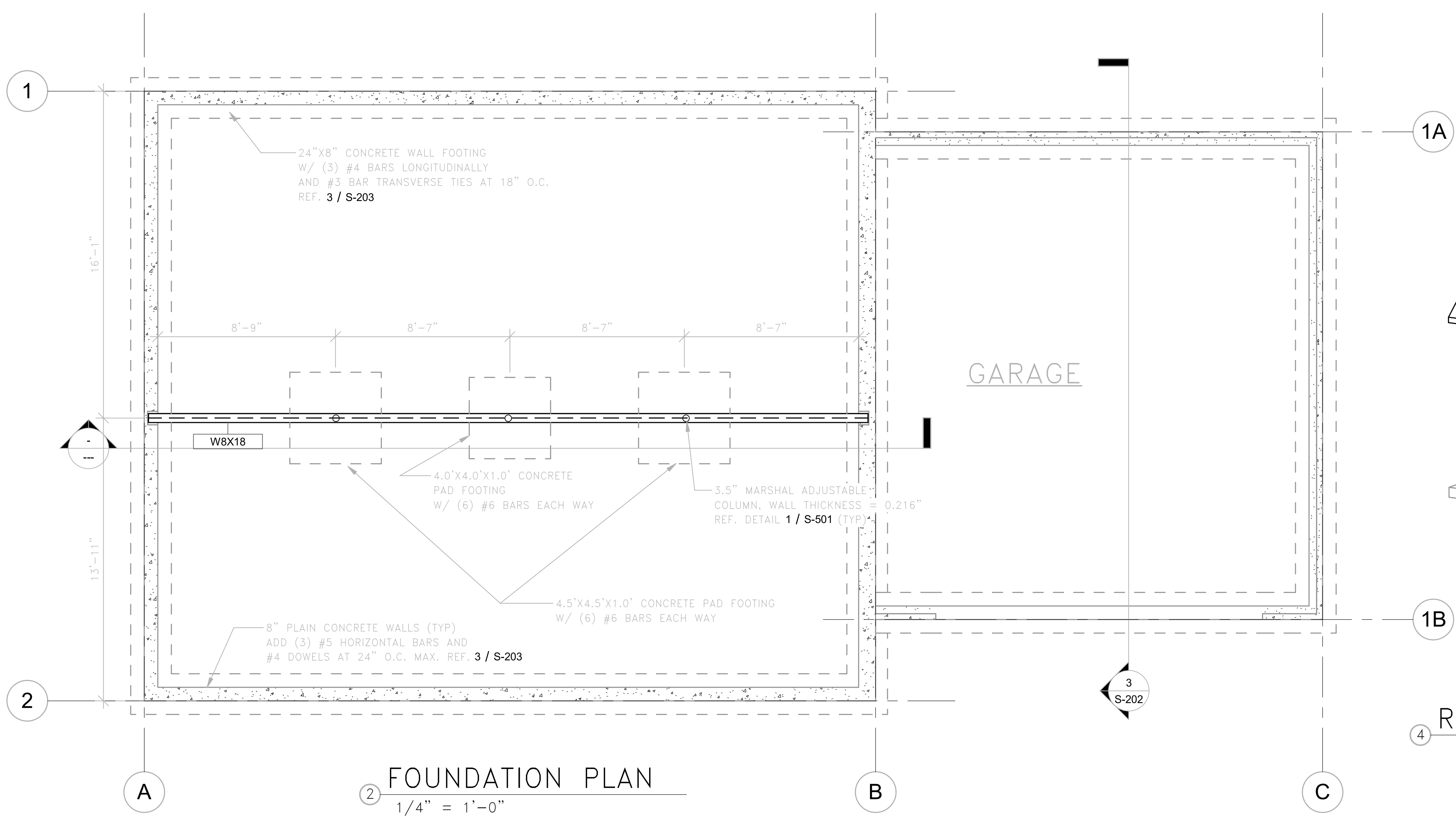

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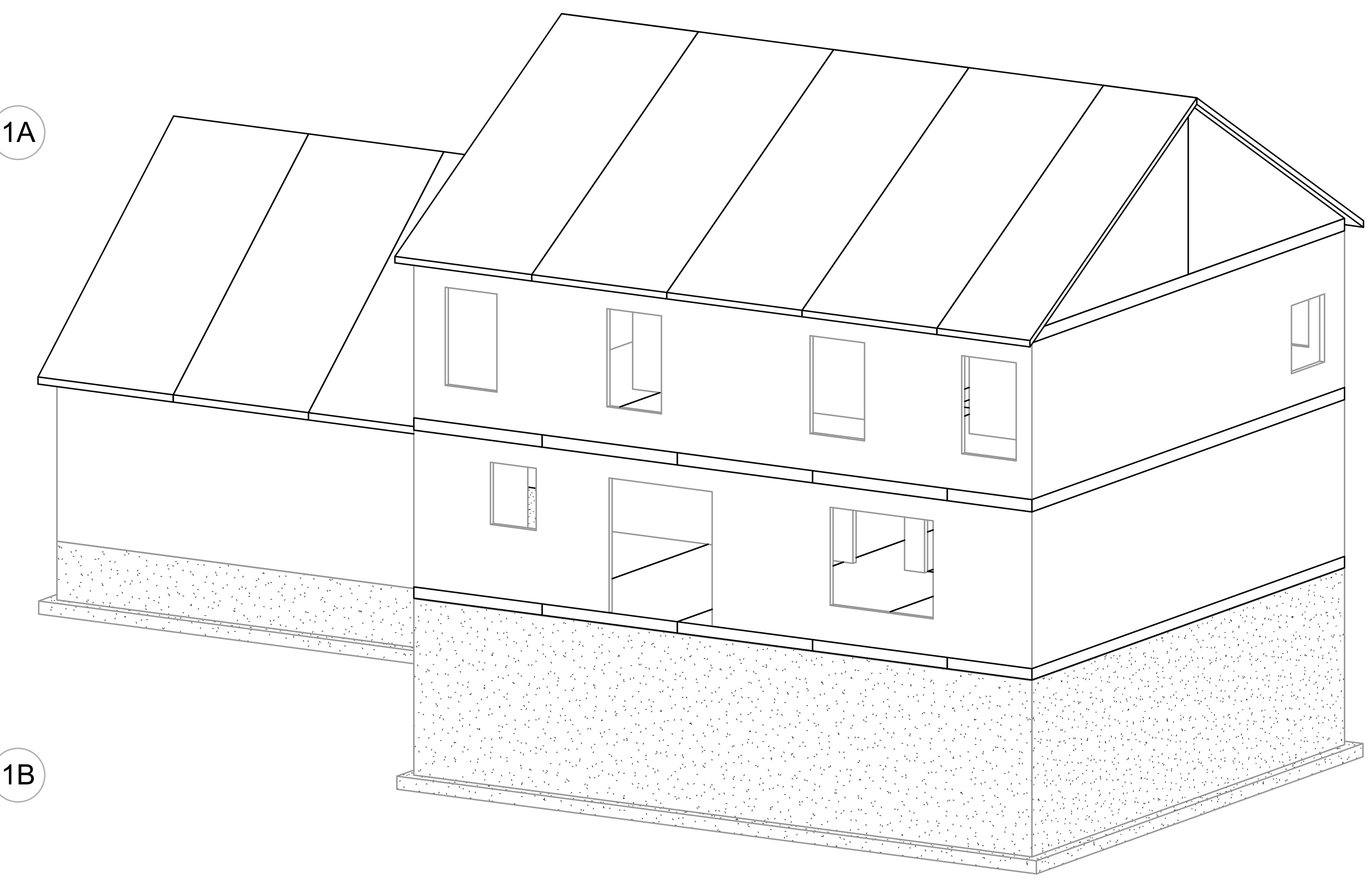

4 REAR ISOMETRIC

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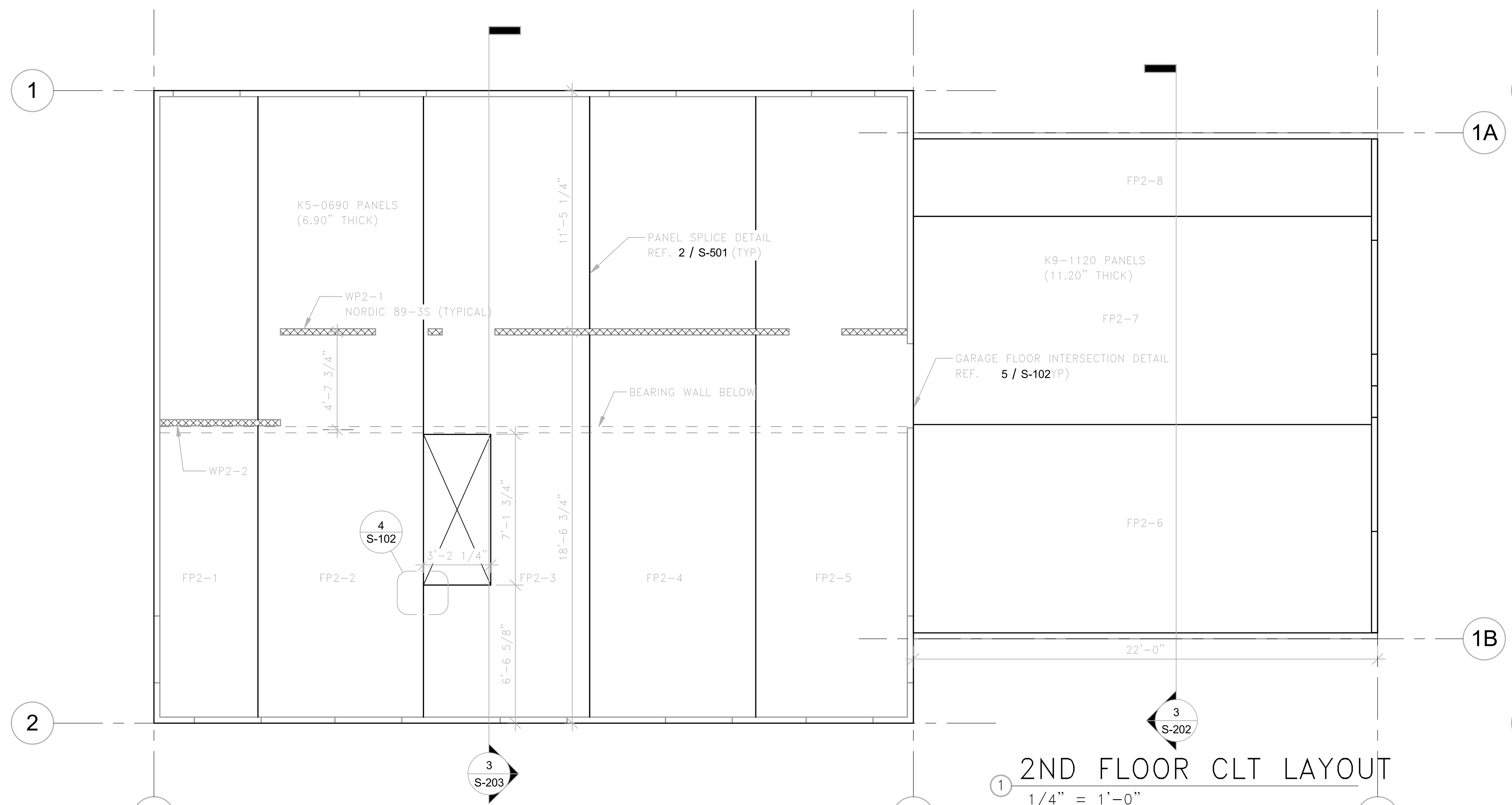

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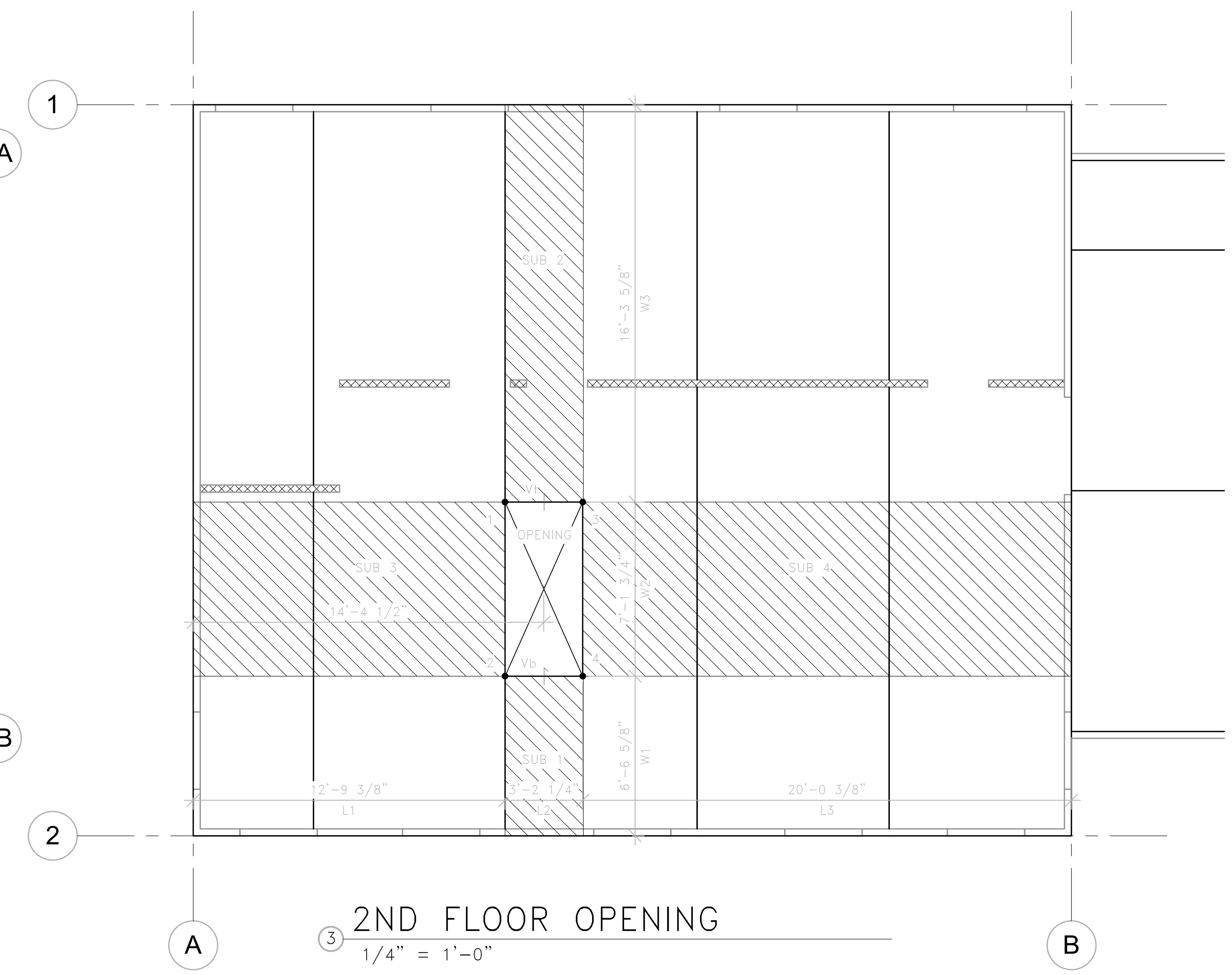

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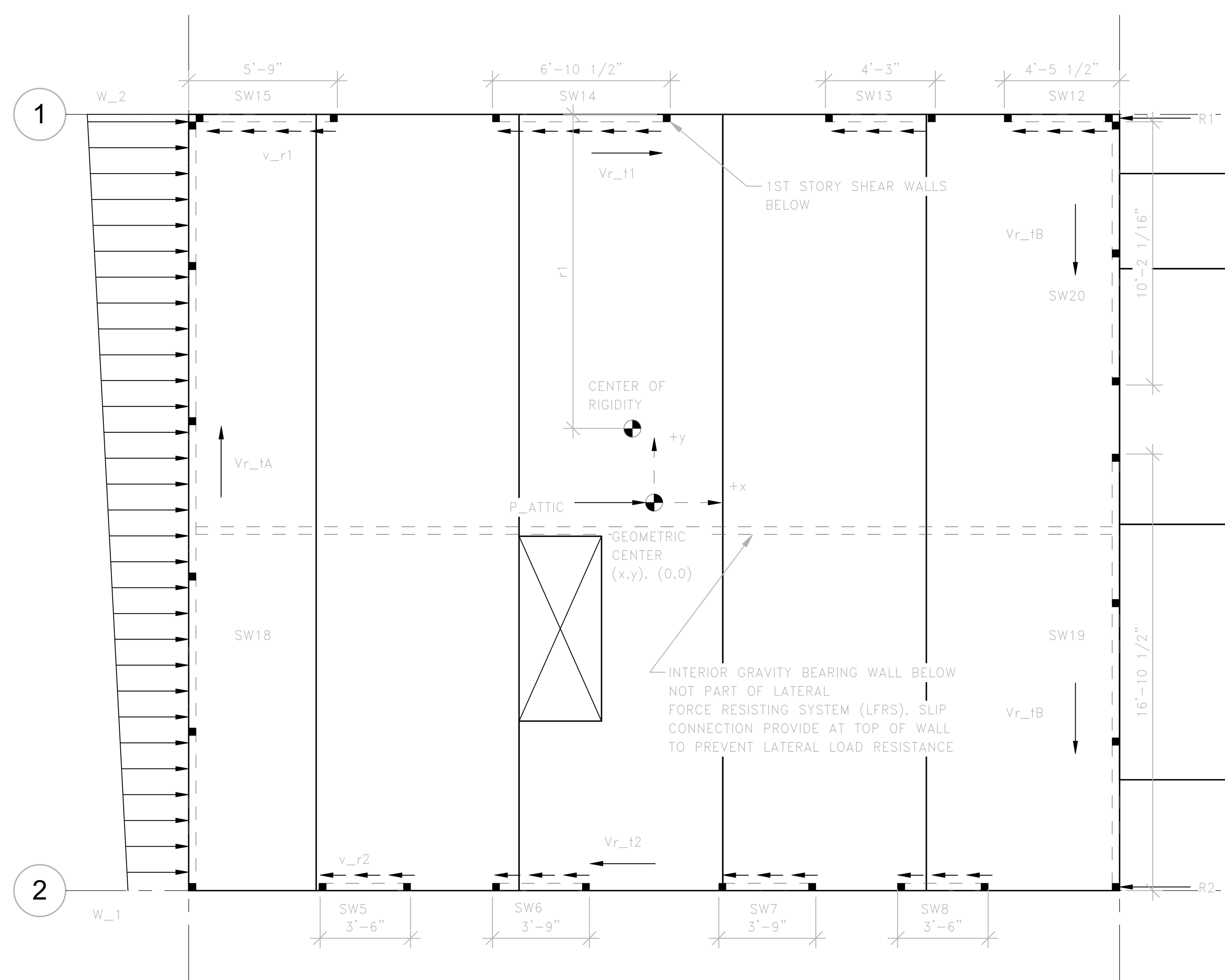

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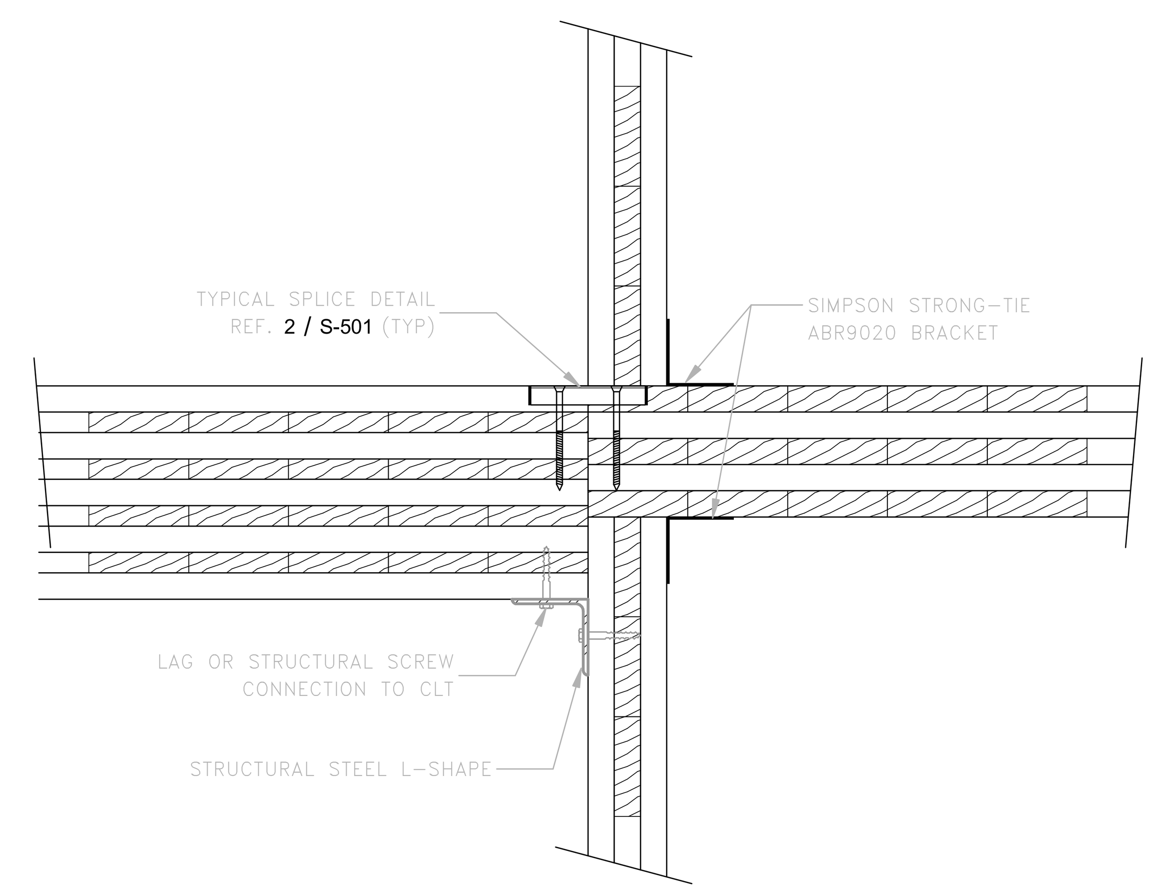

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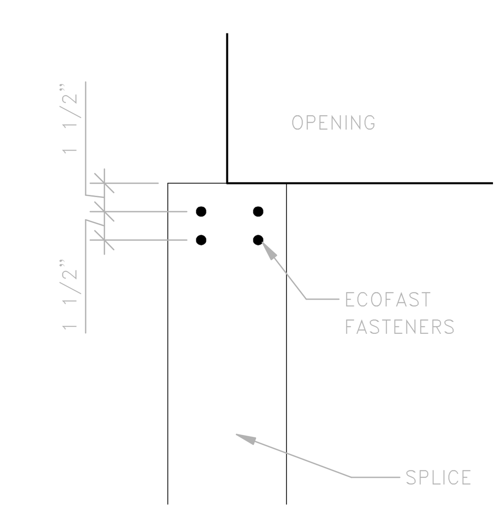

④ OPENING SPLICE DETAIL
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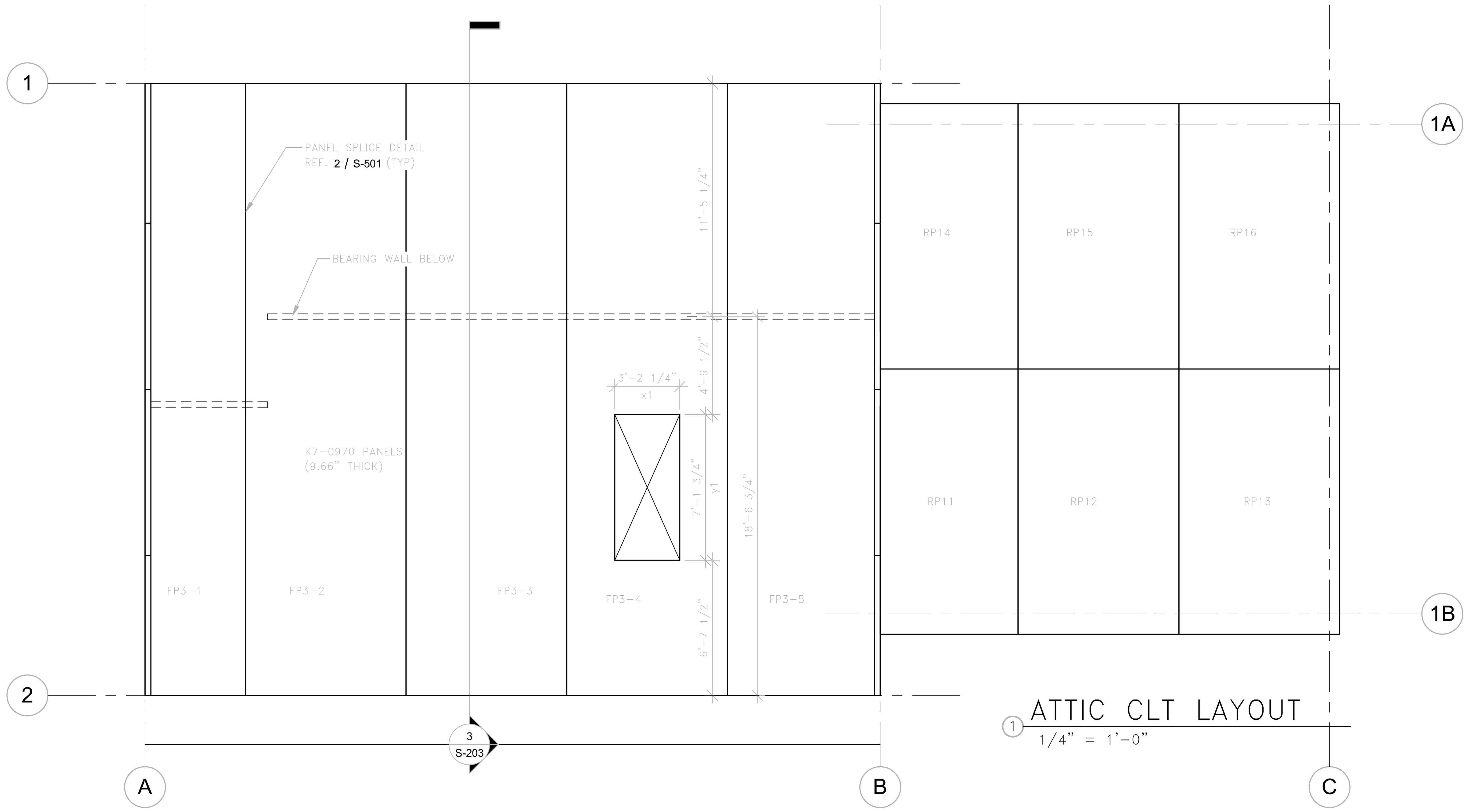

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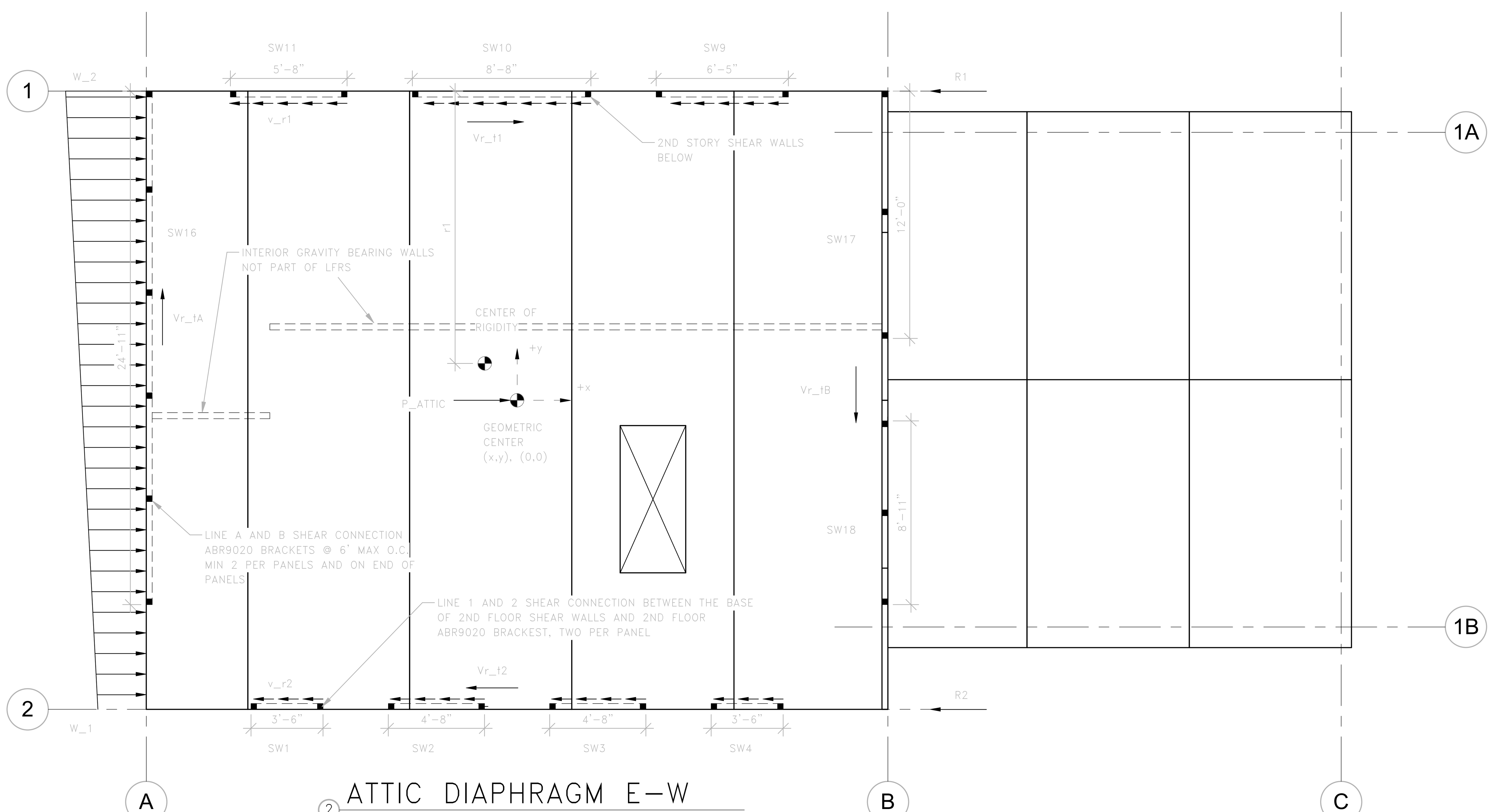

ATTIC DIAPHRAGM E-W
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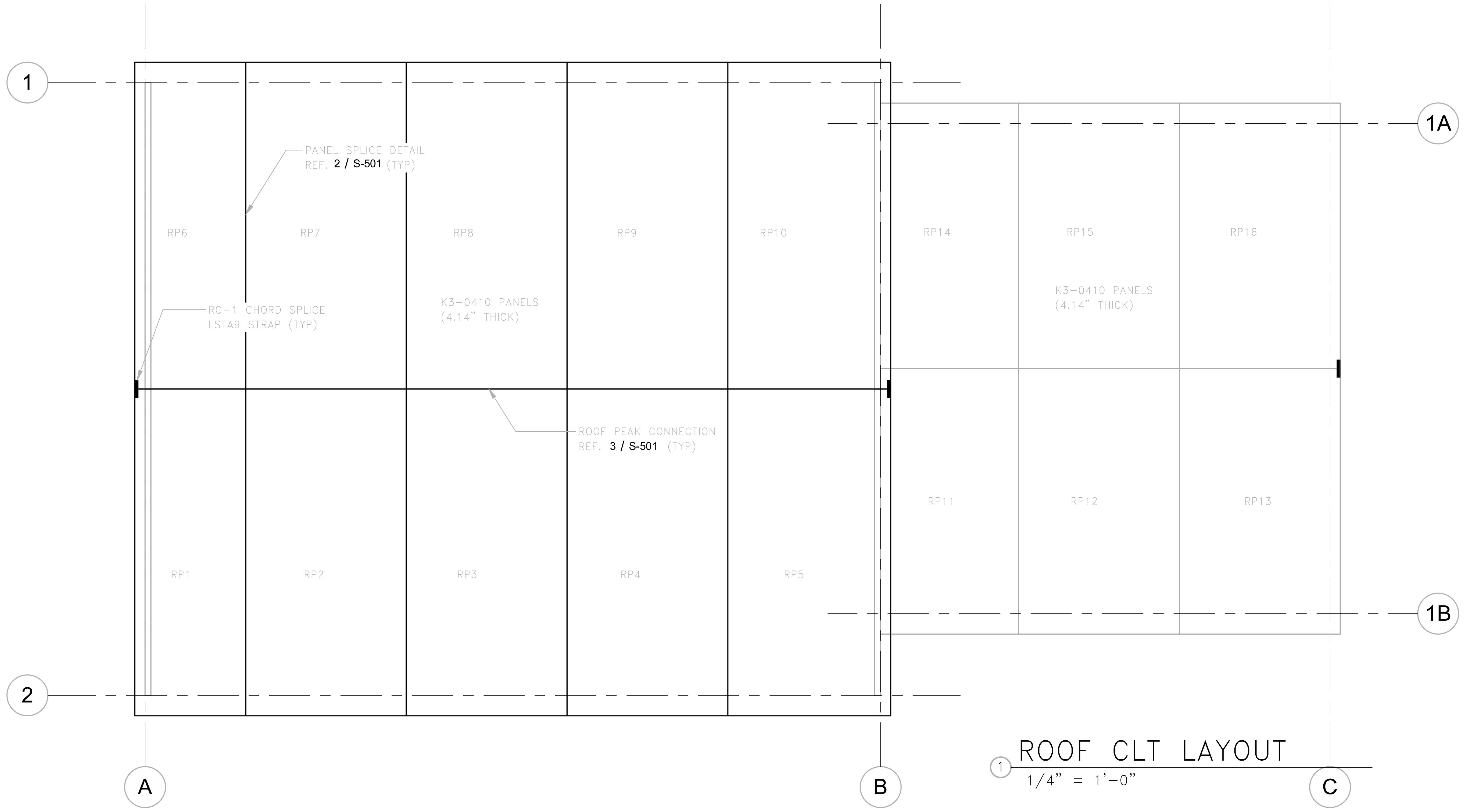

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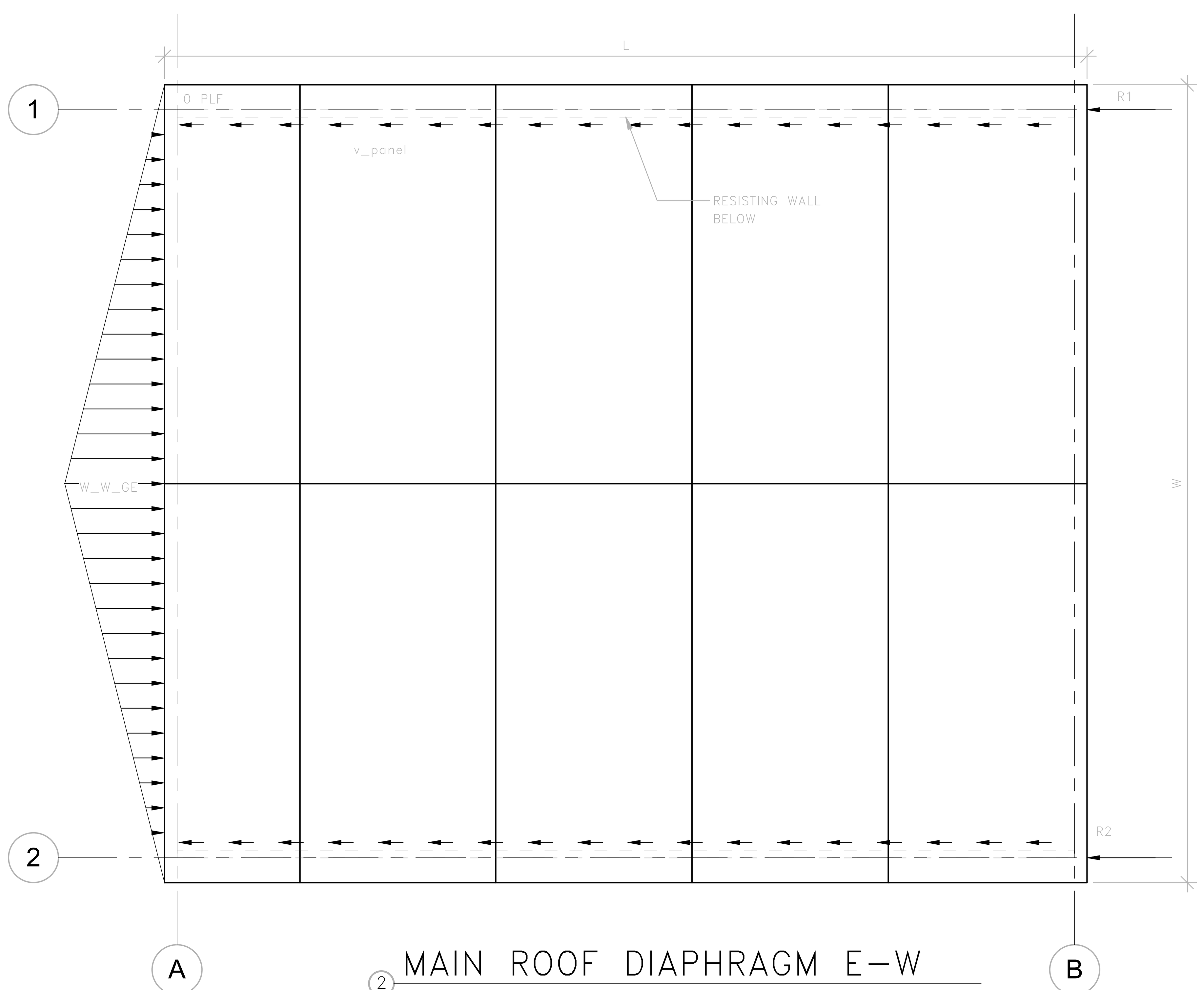

2 MAIN ROOF DIAPHRAGM E-W
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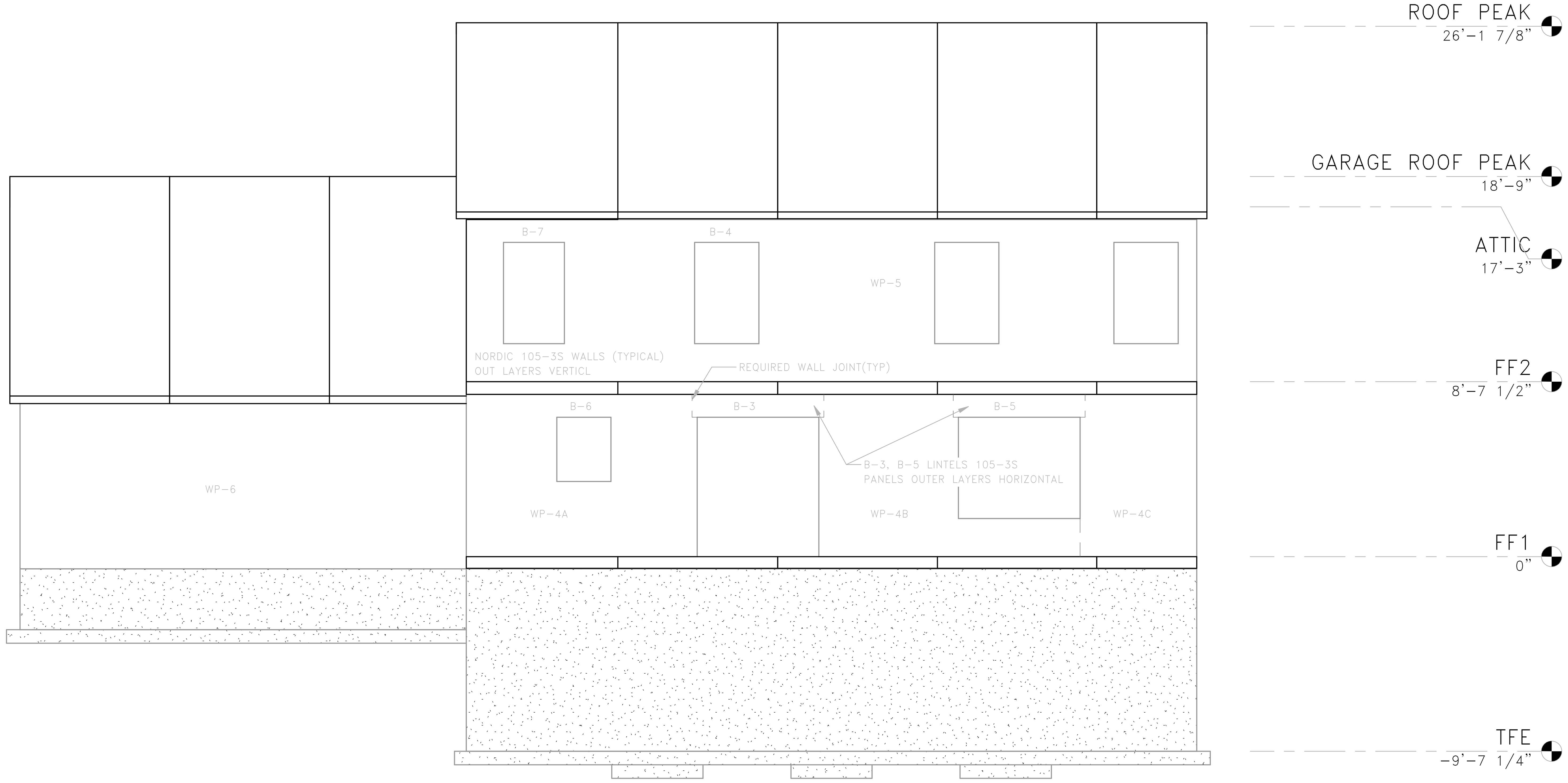

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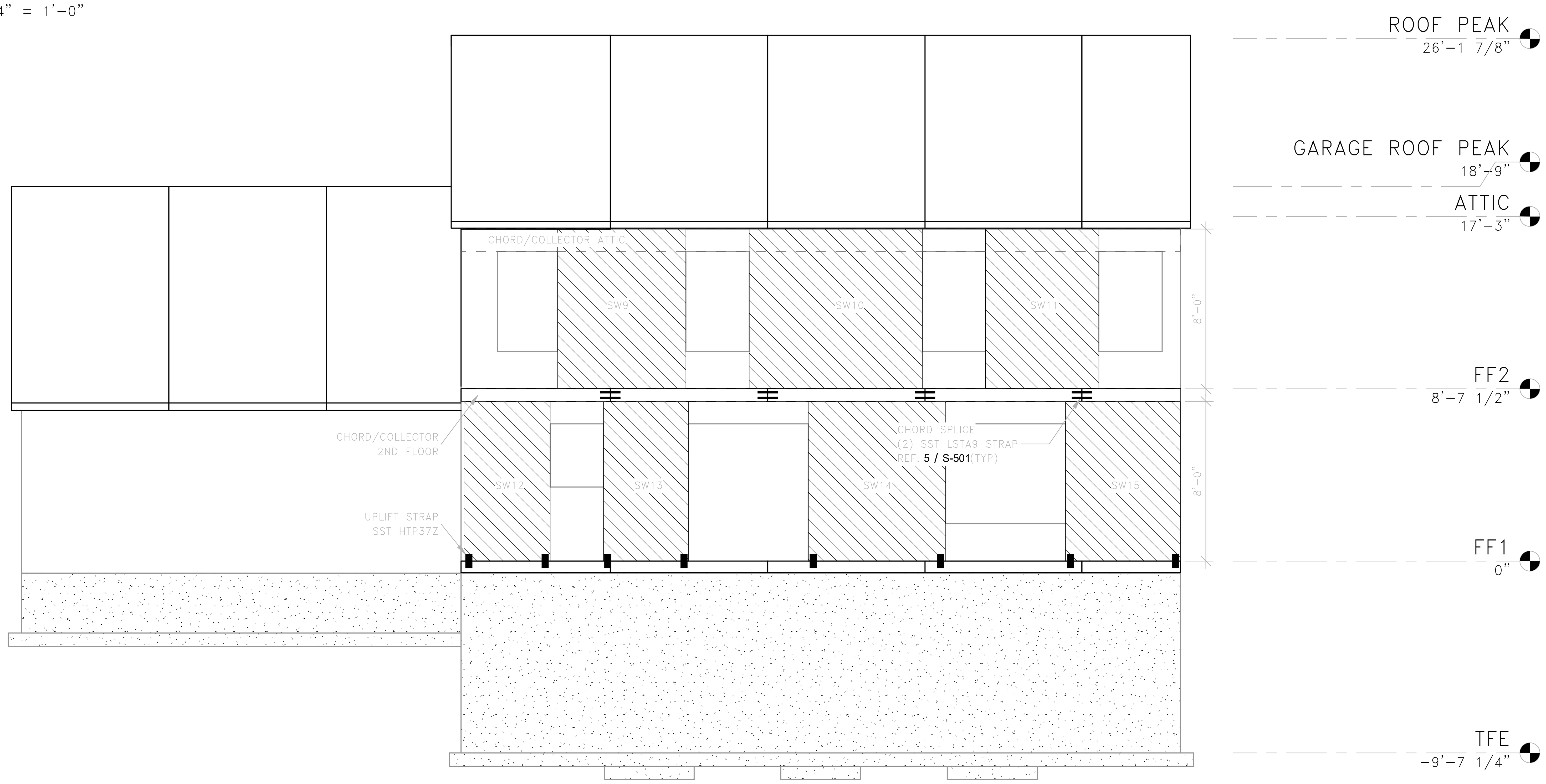

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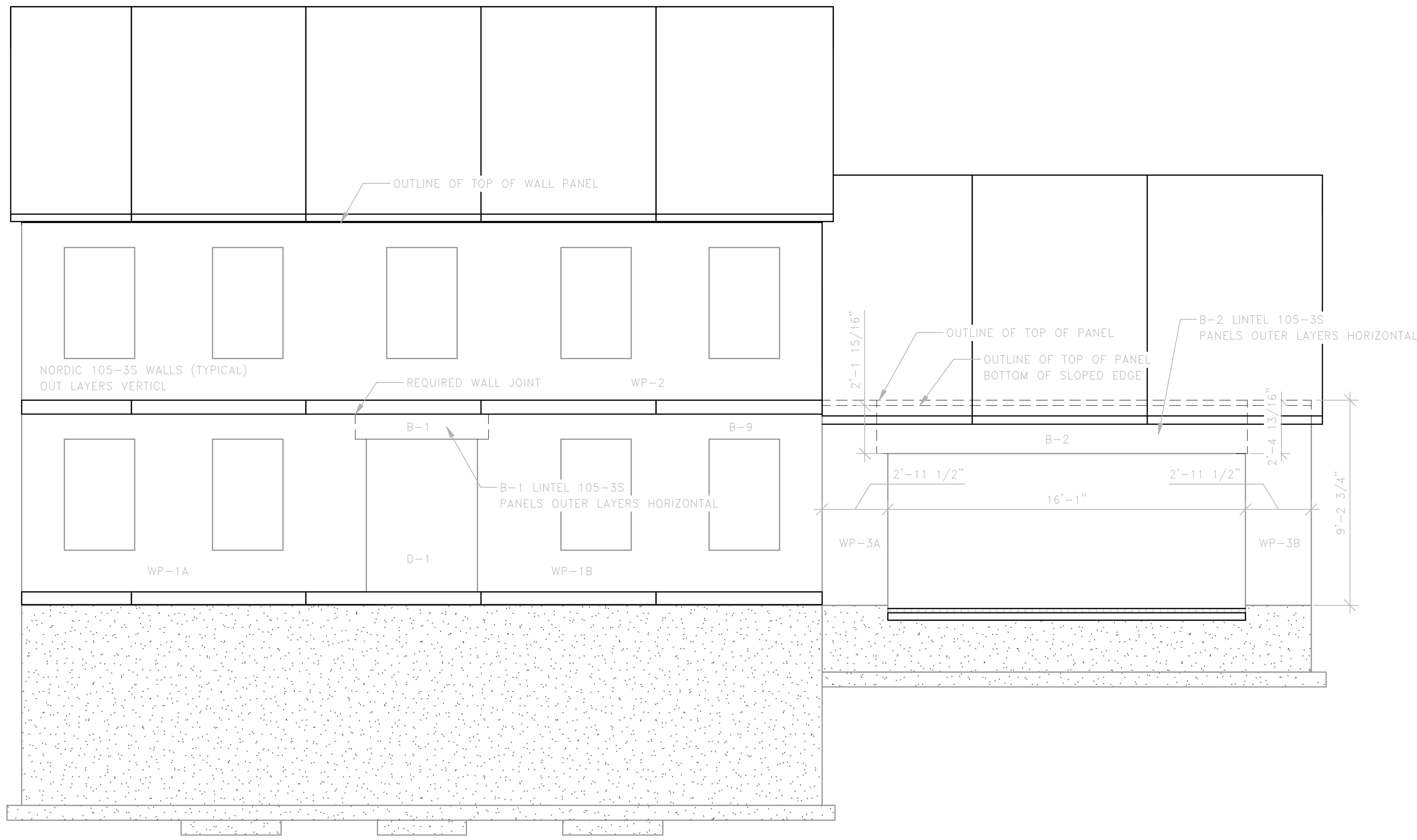

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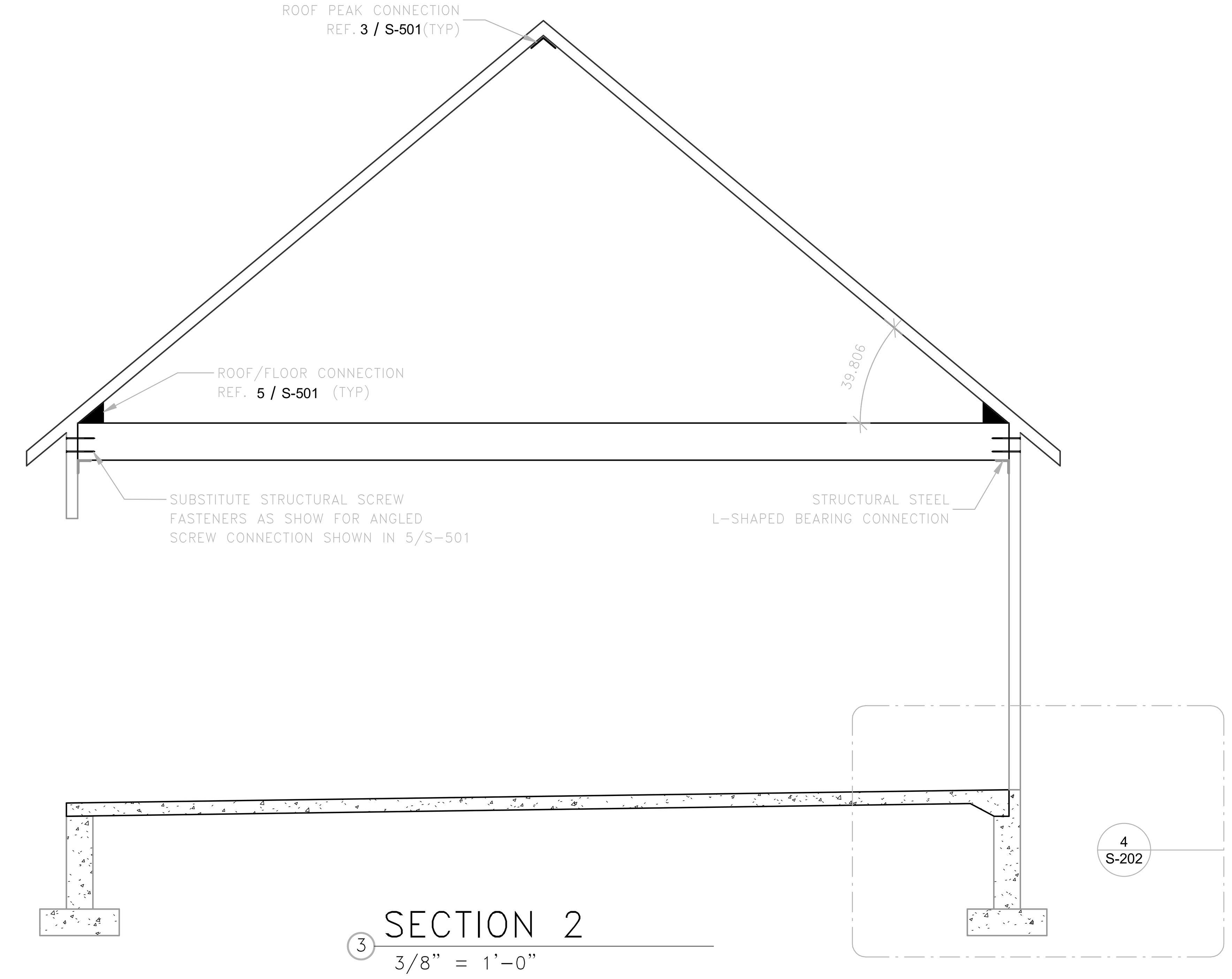

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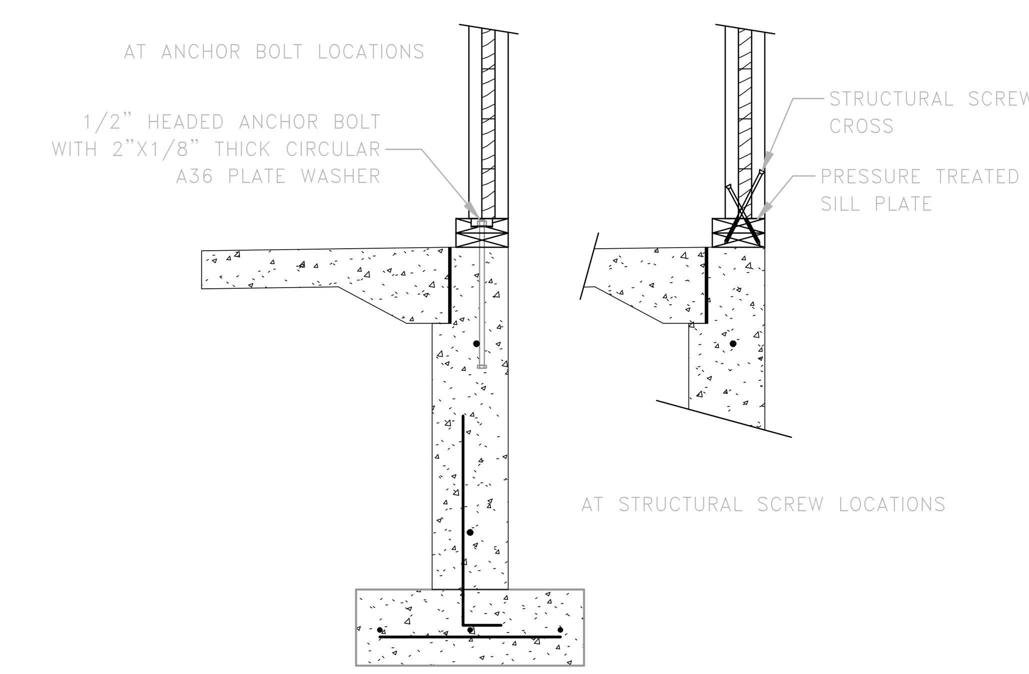

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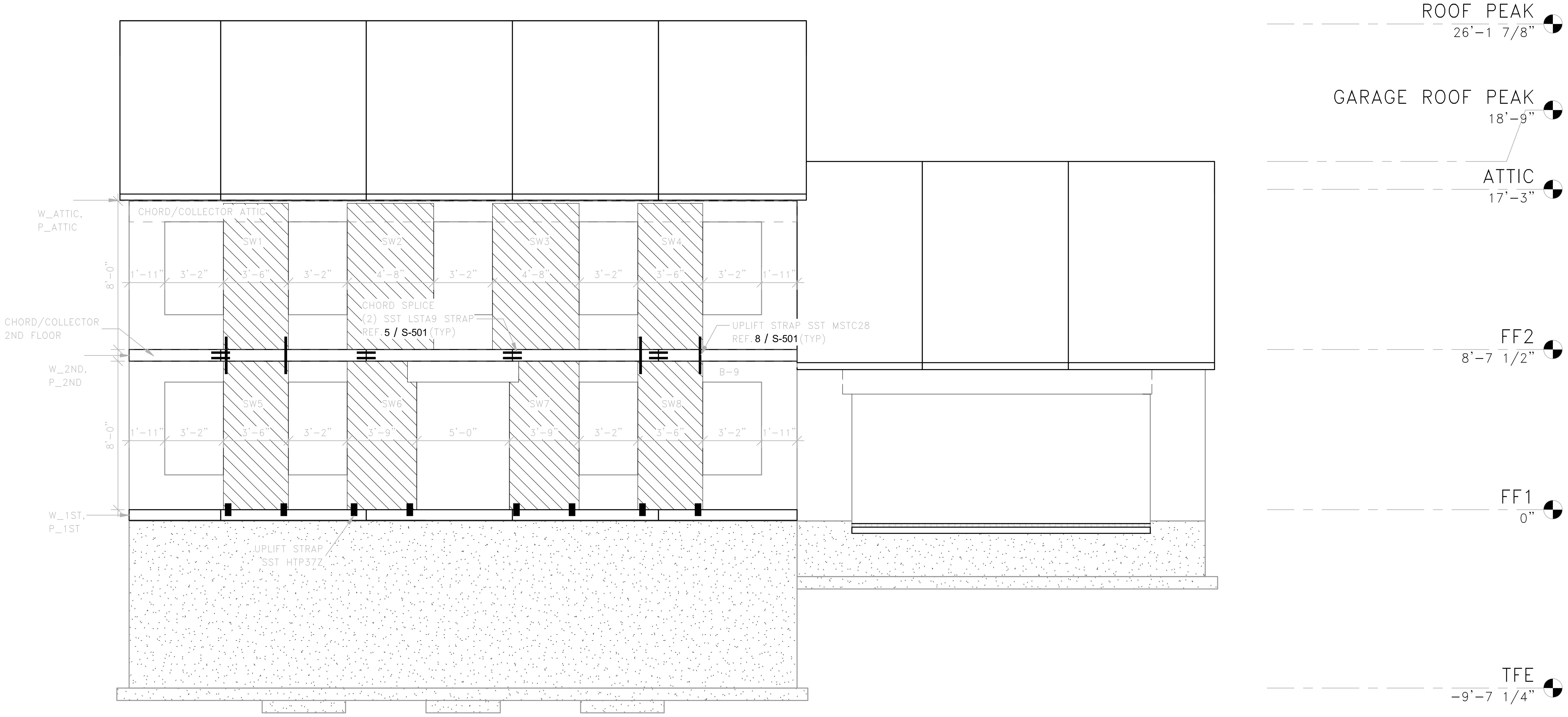

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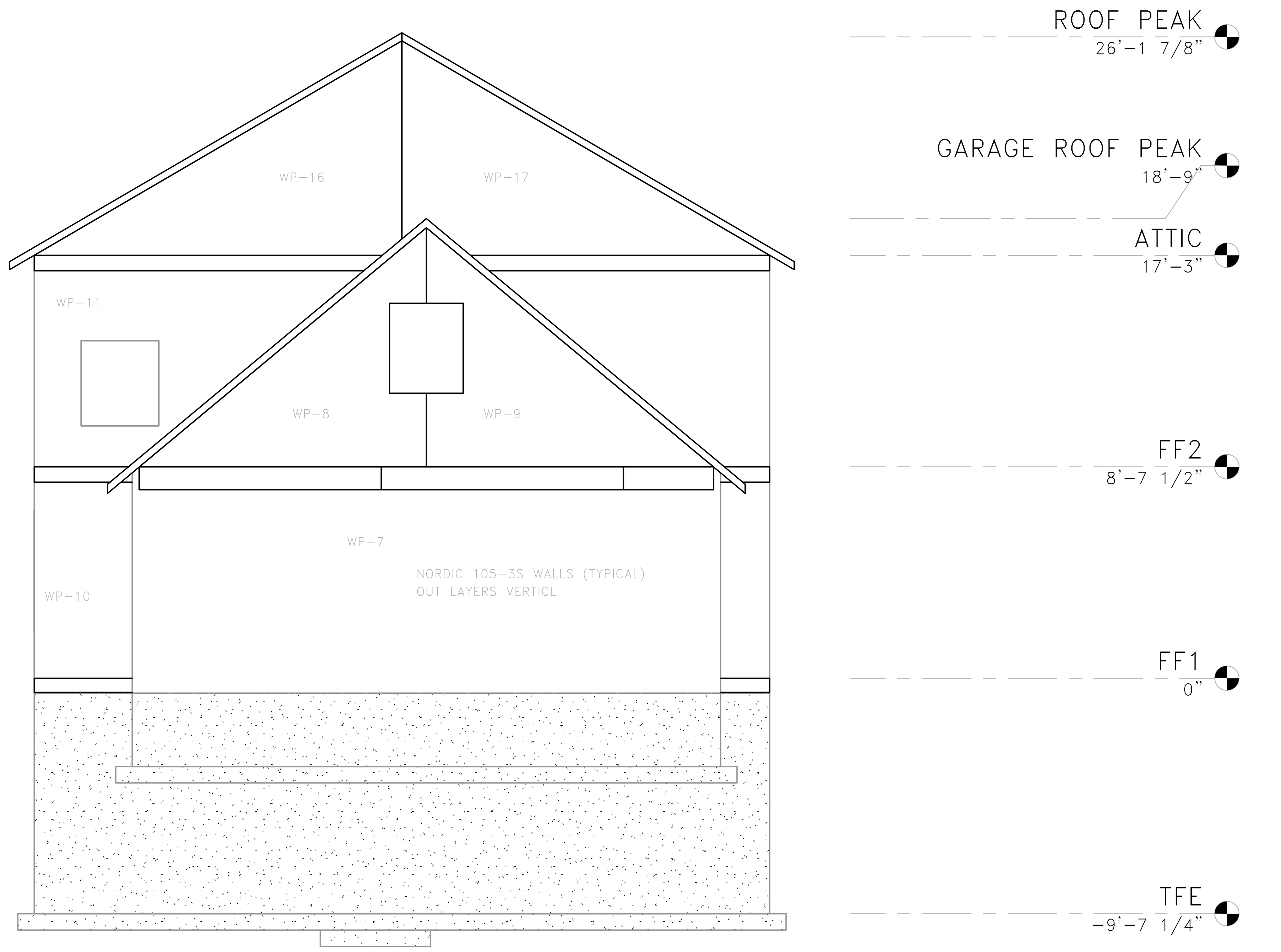

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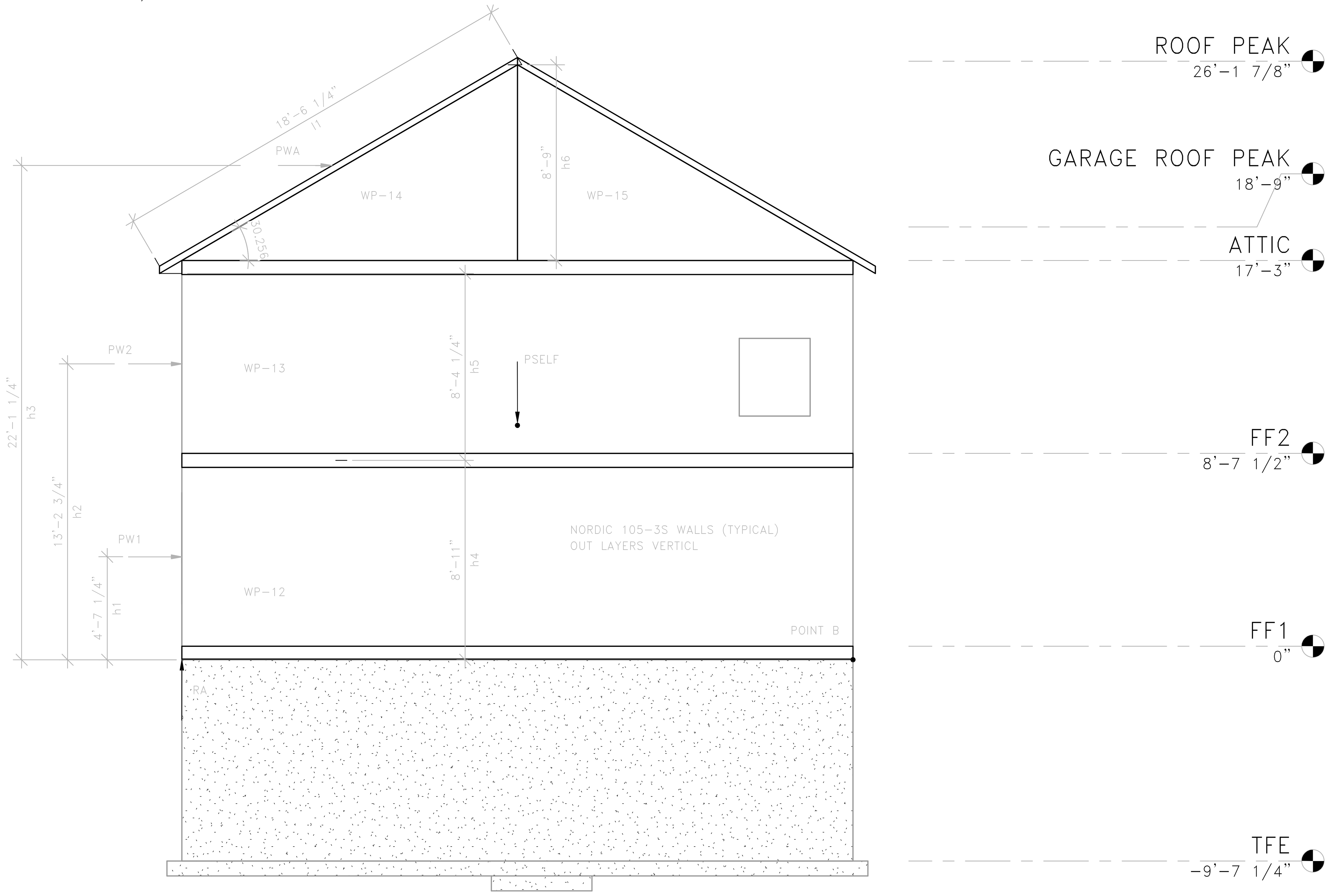

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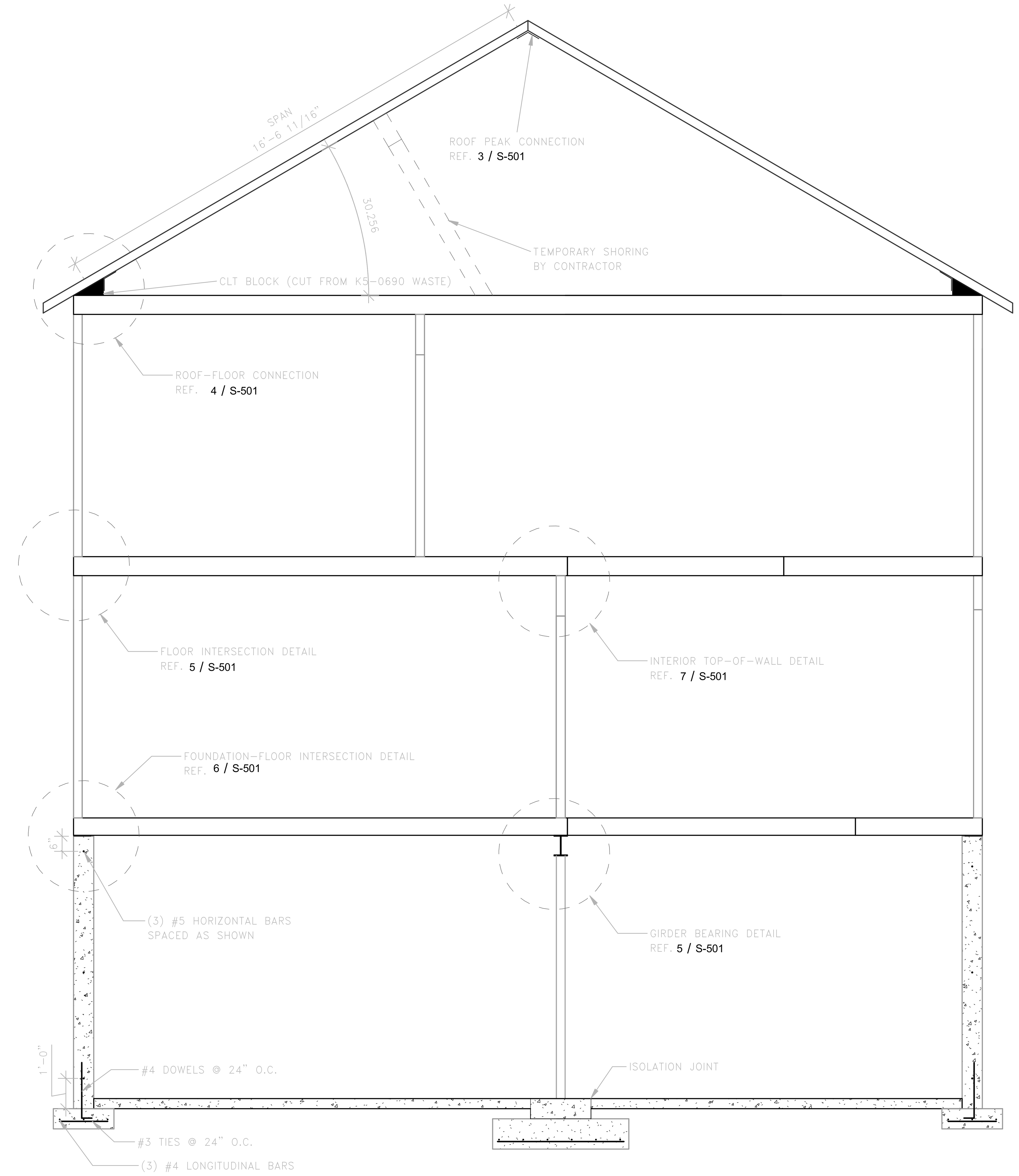

③ SECTION 1
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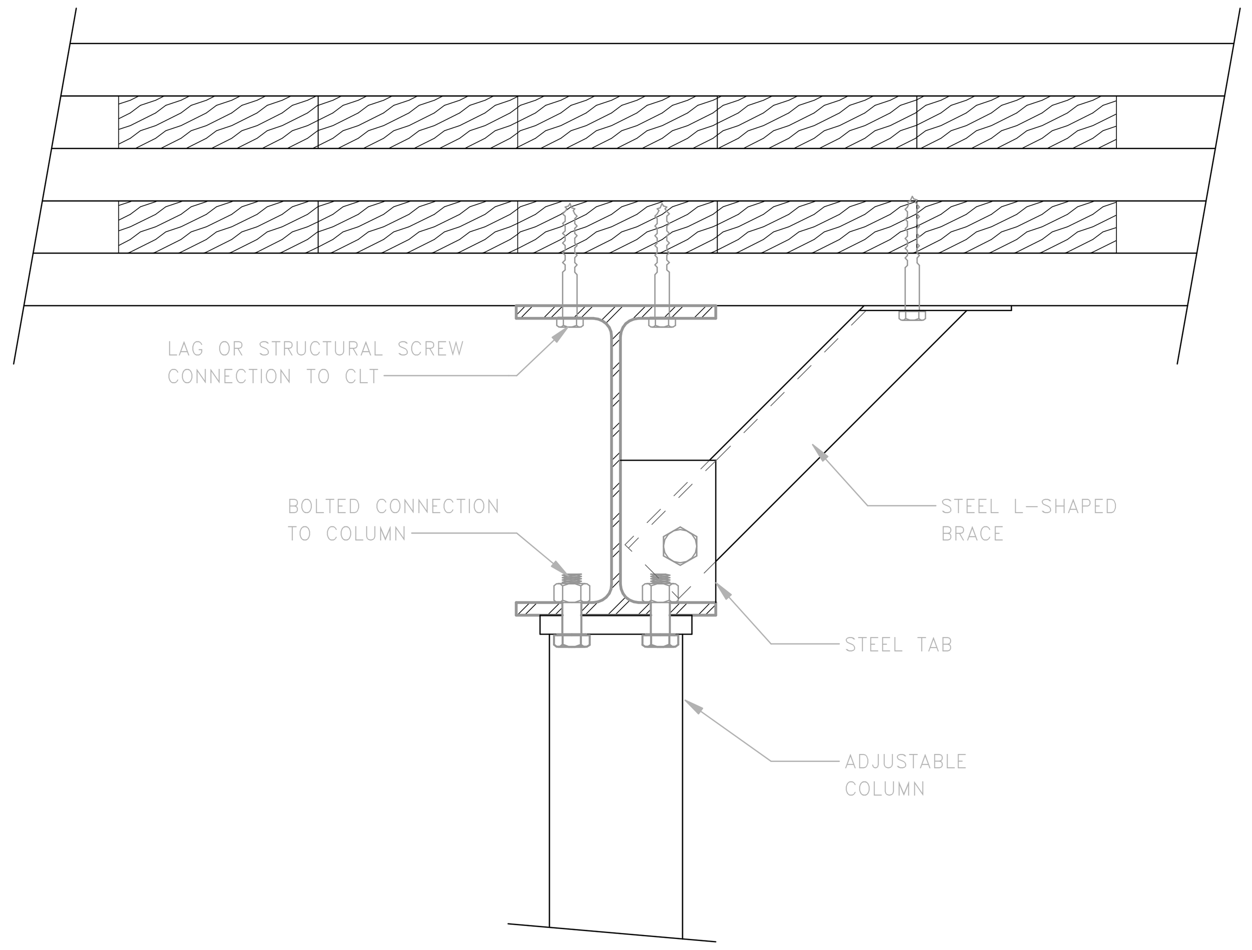
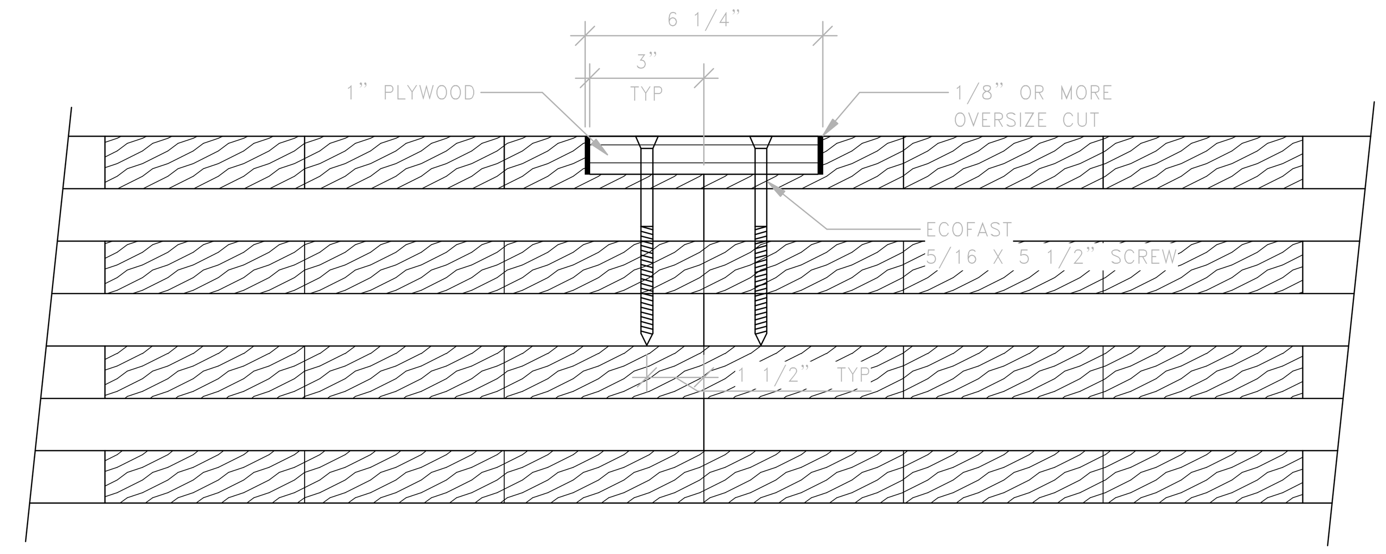
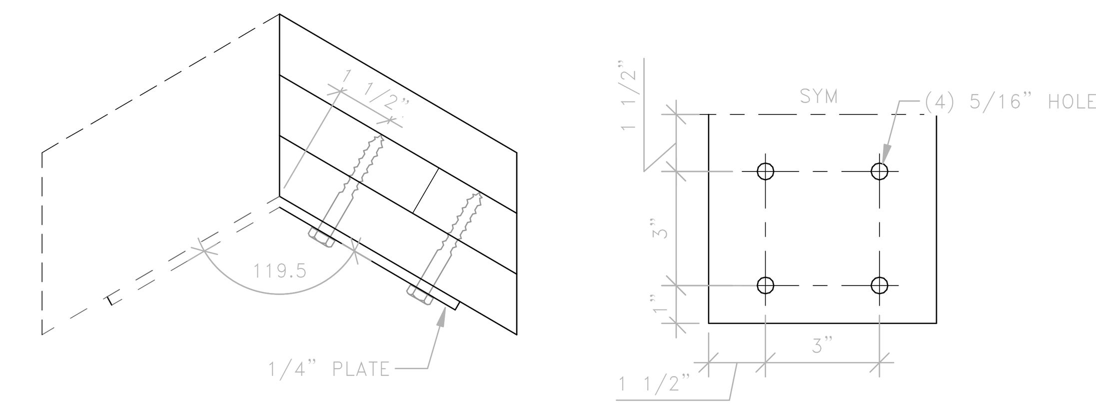


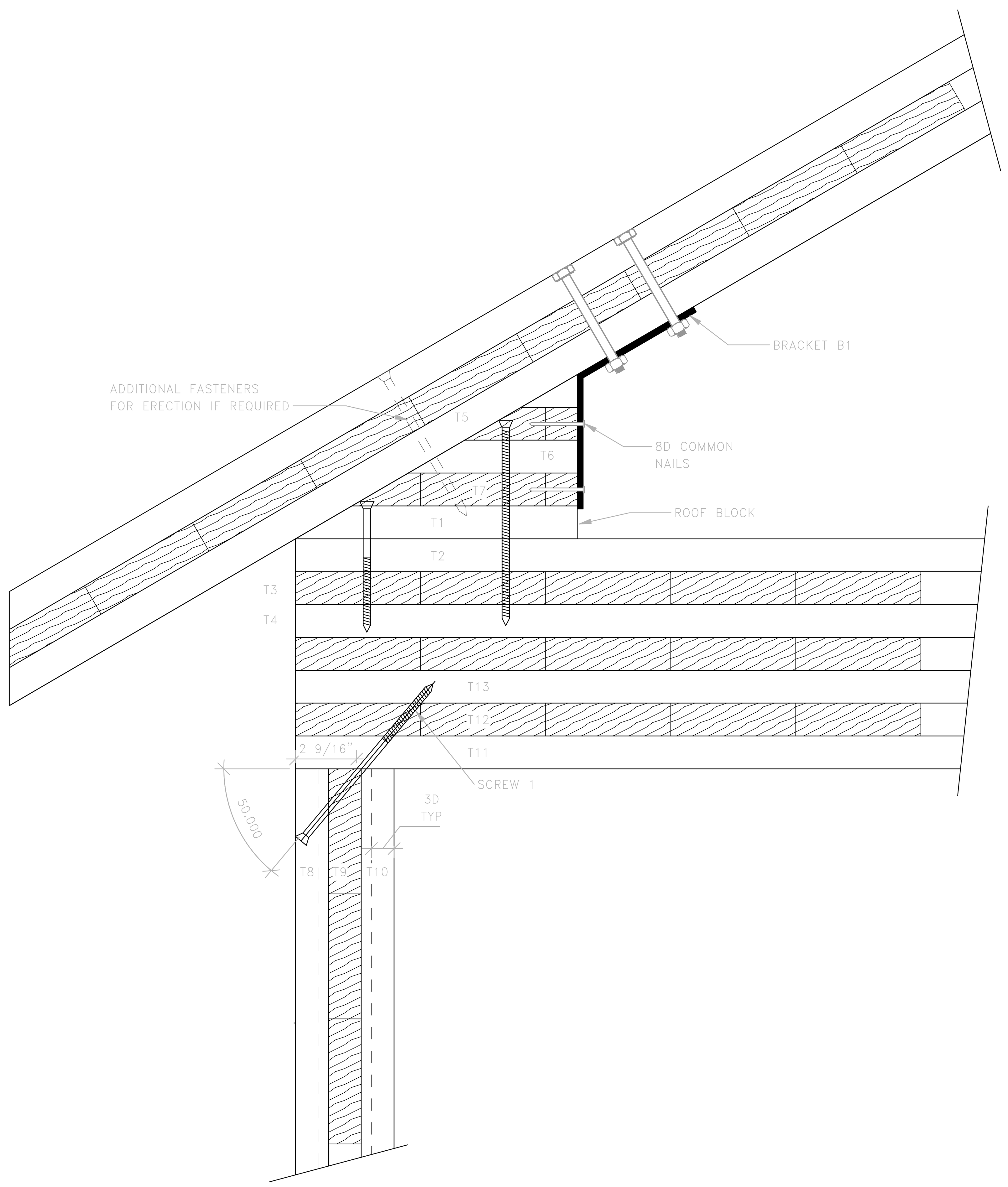
1 STEEL GIRDER CLT BEARING DETAIL
3" = 1'-0"



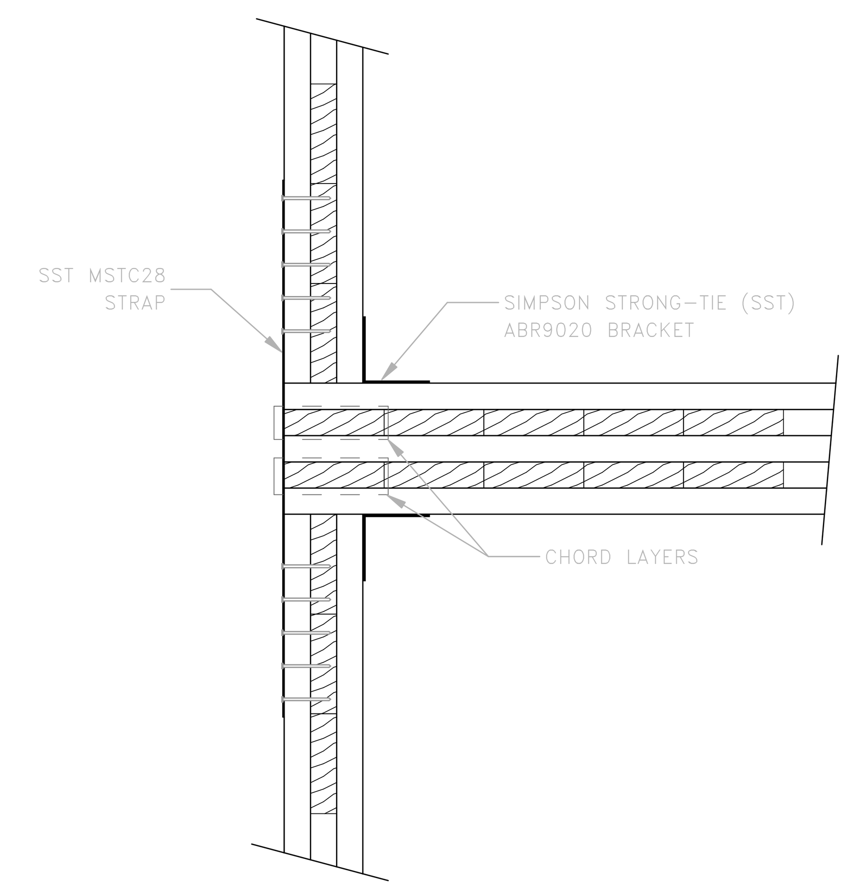
2 TYPICAL PANEL-TO-PANEL SPLICE DETAIL
3" = 1'-0"



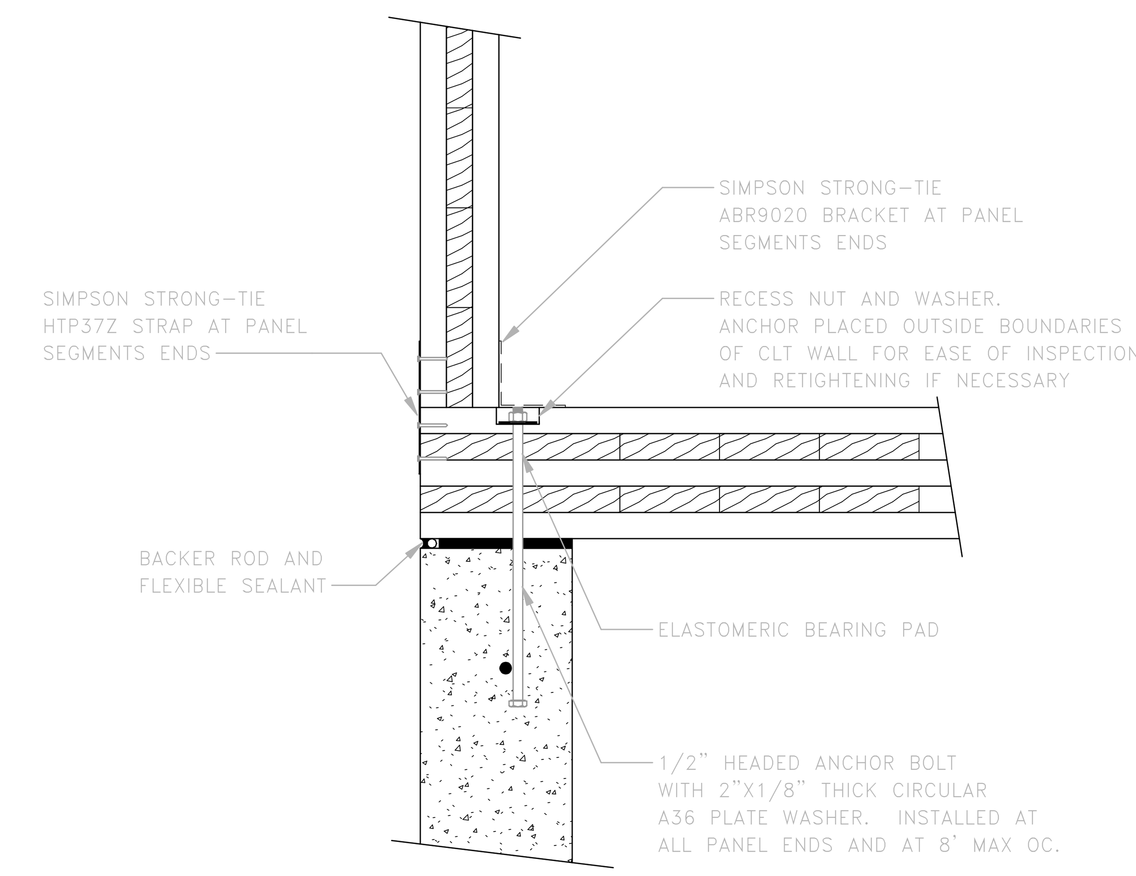
3 ROOF PEAK CONNECTION DETAIL
3" = 1'-0"



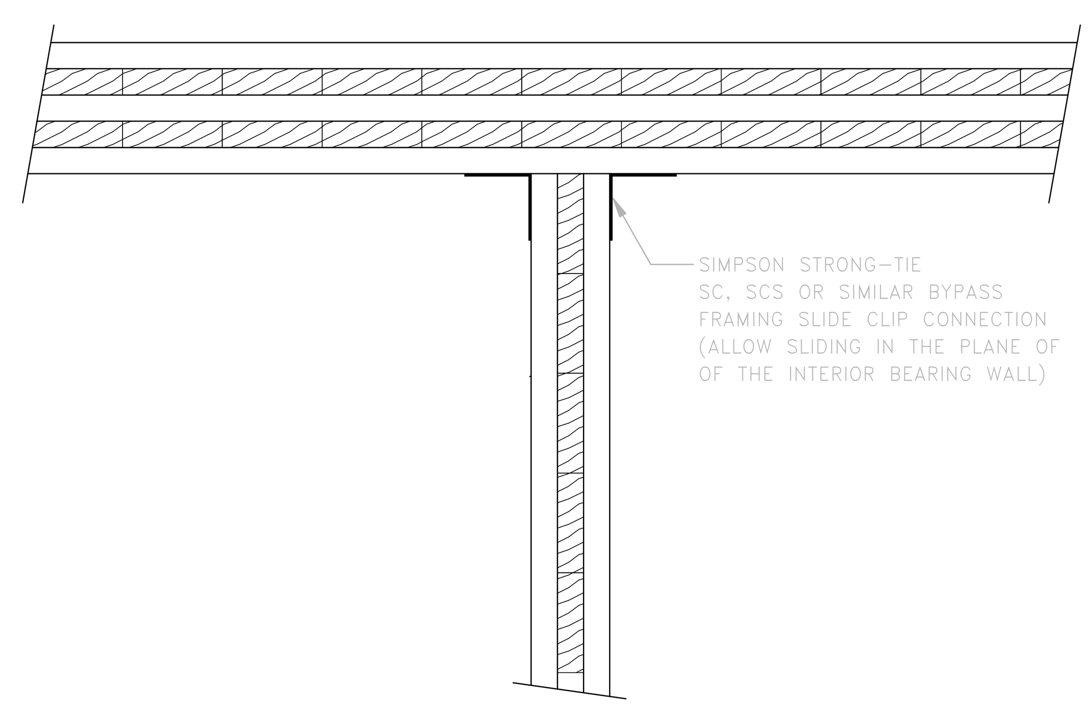
4 ROOF-FLOOR INTERSECTION DETAIL
3" = 1'-0"



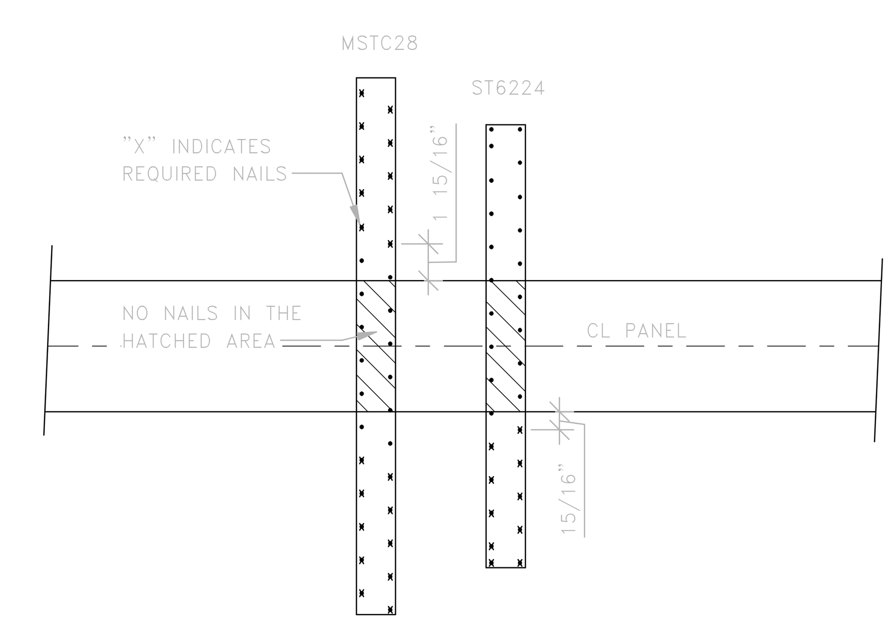
5 2ND FLOOR INTERSECTION DETAIL
1 1/2" = 1'-0"



6 FOUNDATION-FLOOR INTERSECTION DETAIL
1 1/2" = 1'-0"



7 INTERIOR TOP-OF-WALL DETAIL
1 1/2" = 1'-0"



8 SST TENSION STRAP DETAIL
1 1/2" = 1'-0"

JELLEN ENGINEERING SERVICES
tony@jellen.me

REVISIONS

No.	Description	Date

CLT HOME DESIGN

DETAILS

Date:	08/04/2021
Project Number:	202006.00
Scale:	AS NOTED
Drawn By:	Author

Sheet number:
S-501

8/5/2021 5:08:42 PM

Appendix B – Structural Design Calculations

ATC Hazards by Location

Search Information

Address:	state college pa
Coordinates:	40.7933949, -77.8600012
Elevation:	1151 ft
Timestamp:	2020-12-09T21:05:49.797Z
Hazard Type:	Wind

ASCE 7-16

MRI 10-Year	76 mph
MRI 25-Year	80 mph
MRI 50-Year	86 mph
MRI 100-Year	92 mph
Risk Category I	100 mph
Risk Category II	110 mph
Risk Category III	117 mph
Risk Category IV	124 mph

ASCE 7-10

MRI 10-Year	76 mph
MRI 25-Year	84 mph
MRI 50-Year	90 mph
MRI 100-Year	96 mph
Risk Category I	105 mph
Risk Category II	115 mph
Risk Category III-IV	120 mph

ASCE 7-05

ASCE 7-05 Wind Speed	90 mph

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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ATC Hazards by Location

Search Information

Address:	state college pa
Coordinates:	40.7933949, -77.8600012
Elevation:	1151 ft
Timestamp:	2020-12-09T21:06:46.159Z
Hazard Type:	Snow

ASCE 7-16

Ground Snow Load 25 lb/sqft

The reported ground snow load applies at the query location of 1151 feet up to a maximum elevation of 1200 feet.

ASCE 7-10

Ground Snow Load 25 lb/sqft

The reported ground snow load applies at the query location of 1151 feet up to a maximum elevation of 1200 feet.

ASCE 7-05

Ground Snow Load 25 lb/sqft

The reported ground snow load applies at the query location of 1151 feet up to a maximum elevation of 1200 feet.

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer.

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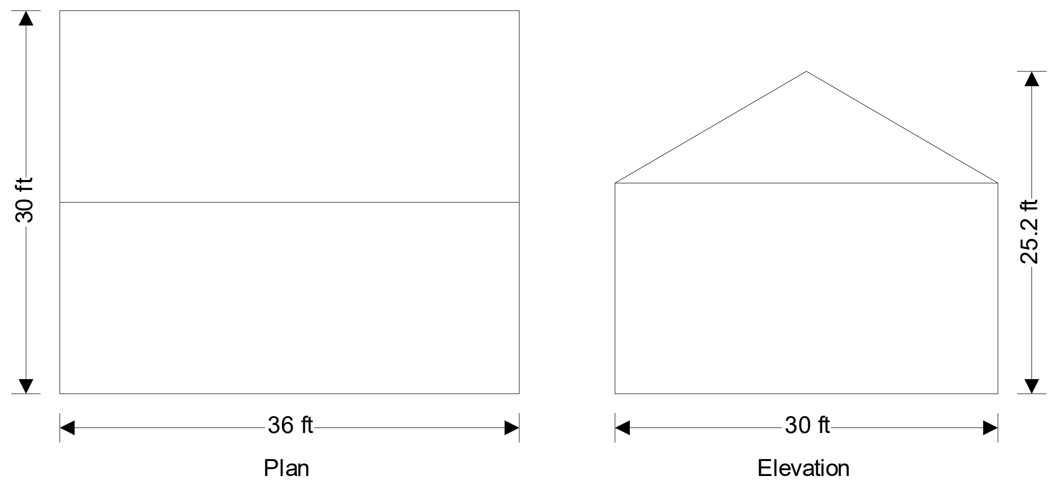
WIND LOADING

In accordance with ASCE7-10

Using the directional design method

Tedds calculation version 2.1.06



Building data

Type of roof	Gable
Length of building	b = **36.00** ft
Width of building	d = **30.00** ft
Height to eaves	H = **16.50** ft
Pitch of roof	α_0 = **30.3** deg
Mean height	h = **20.87** ft

General wind load requirements

Basic wind speed	V = **115.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Exposure category (cl 26.7.3)	B
Enclosure classification (cl.26.10)	Enclosed buildings
Internal pressure coef +ve (Table 26.11-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.11-1)	GC_{pi_n} = **-0.18**
Gust effect factor	G_f = **0.85**
Minimum design wind loading (cl.27.4.7)	p_{min_r} = **8** lb/ft^2

Topography

Topography factor not significant	K_{zt} = 1.0
Velocity pressure equation	$q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$

Velocity pressures table

z (ft)	K_z (Table 27.3-1)	q_z (psf)
15.00	0.57	16.40
15.00	0.57	16.40
16.50	0.59	16.83

z (ft)	K_z (Table 27.3-1)	q_z (psf)
20.87	0.63	18.04
25.25	0.66	19.05

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **18.04** psf

Pressures and forces

Net pressure $p = q \times G_f \times C_{pe} - q_i \times GC_{pi}$

Net force $F_w = p \times A_{ref}$

Roof load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	20.87	-0.24	18.04	-6.89	625.16	-4.30
B (-ve)	20.87	-0.60	18.04	-12.45	625.16	-7.78

Total vertical net force $F_{w,v}$ = **-10.44** kips

Total horizontal net force $F_{w,h}$ = **1.75** kips

Walls load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	16.40	7.91	540.00	4.27
A_2	16.50	0.80	16.83	8.20	54.00	0.44
B	20.87	-0.50	18.04	-10.92	594.00	-6.48
C	20.87	-0.70	18.04	-13.98	626.25	-8.76
D	20.87	-0.70	18.04	-13.98	626.25	-8.76

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times H =$ **594.00** ft^2

Projected vertical area of roof $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0) =$ **314.99** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} =$ **12.02** kips

Leeward net force $F_l = F_{w,wB} =$ **-6.5** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} =$ **4.7** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **12.9** kips

Roof load case 2 - Wind 0, GC_{pi} -0.18, $-0c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	20.87	0.20	18.04	6.36	625.16	3.98
B (+ve)	20.87	-0.60	18.04	-5.95	625.16	-3.72

Total vertical net force $F_{w,v}$ = **0.22** kips

Total horizontal net force $F_{w,h}$ = **3.88** kips

Walls load case 2 - Wind 0, GC_{pi} -0.18, -0c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	16.40	14.40	540.00	7.78
A_2	16.50	0.80	16.83	14.70	54.00	0.79
B	20.87	-0.50	18.04	-4.42	594.00	-2.63
C	20.87	-0.70	18.04	-7.49	626.25	-4.69
D	20.87	-0.70	18.04	-7.49	626.25	-4.69

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times H =$ **594.00** ft^2

Projected vertical area of roof $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0) =$ **314.99** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} =$ **12.02** kips

Leeward net force $F_l = F_{w,wB} =$ **-2.6** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} =$ **8.6** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **15.1** kips

Roof load case 3 - Wind 90, GC_{pi} 0.18, -c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	20.87	-0.94	18.04	-17.69	362.50	-6.41
B (-ve)	20.87	-0.87	18.04	-16.56	362.50	-6.00
C (-ve)	20.87	-0.53	18.04	-11.41	525.31	-5.99

Total vertical net force $F_{w,v} =$ **-15.90** kips

Total horizontal net force $F_{w,h} =$ **0.00** kips

Walls load case 3 - Wind 90, GC_{pi} 0.18, -c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	16.40	7.91	450.00	3.56
A_2	15.00	0.80	16.40	7.91	0.00	0.00
A_3	25.25	0.80	19.05	9.71	176.25	1.71
B	20.87	-0.46	18.04	-10.30	626.25	-6.45
C	20.87	-0.70	18.04	-13.98	594.00	-8.31
D	20.87	-0.70	18.04	-13.98	594.00	-8.31

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4 =$ **626.25** ft^2

Projected vertical area of roof $A_{vert\_r\_90} =$ **0.00** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} =$ **10.02** kips

Leeward net force $F_l = F_{w,wB} =$ **-6.5** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} =$ **5.3** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **11.7** kips

Roof load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (kips)
A (+ve)	20.87	-0.18	18.04	0.49	362.50	0.18
B (+ve)	20.87	-0.18	18.04	0.49	362.50	0.18
C (+ve)	20.87	-0.18	18.04	0.49	525.31	0.26

Total vertical net force $F_{w,v}$ = **0.53** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

Walls load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (kips)
A_1	15.00	0.80	16.40	14.40	450.00	6.48
A_2	15.00	0.80	16.40	14.40	0.00	0.00
A_3	25.25	0.80	19.05	16.20	176.25	2.86
B	20.87	-0.46	18.04	-3.81	626.25	-2.38
C	20.87	-0.70	18.04	-7.49	594.00	-4.45
D	20.87	-0.70	18.04	-7.49	594.00	-4.45

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4$ = **626.25** ft²

Projected vertical area of roof $A_{vert\_r\_90}$ = **0.00** ft²

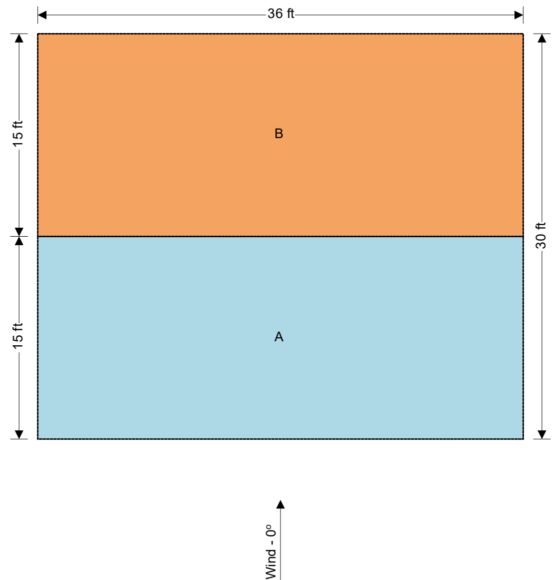
Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90}$ = **10.02** kips

Leeward net force $F_l = F_{w,wB}$ = **-2.4** kips

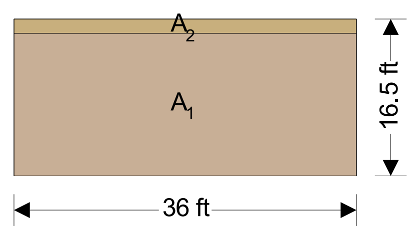
Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3}$ = **9.3** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **11.7** kips

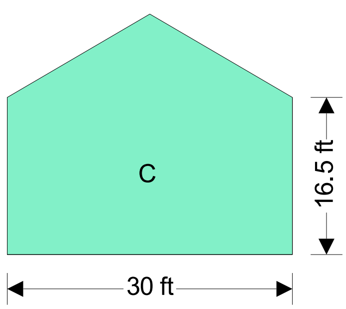
Tekla Tedds	Project CLT Residential Design				Job Ref.	
	Section Wind Loading MWFRS				Sheet no./rev. 5	
	Calc. by ACJ	Date 1/4/2021	Chk'd by	Date	App'd by	Date



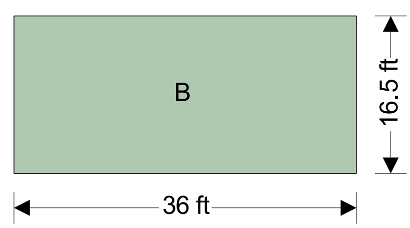
Plan view - Gable roof



Windward face

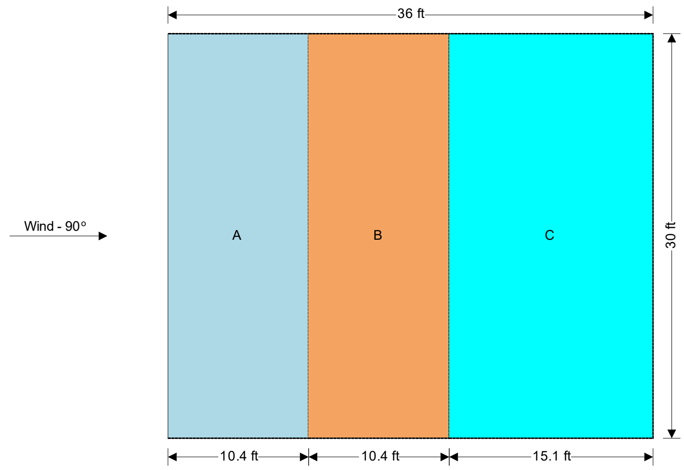


Side face

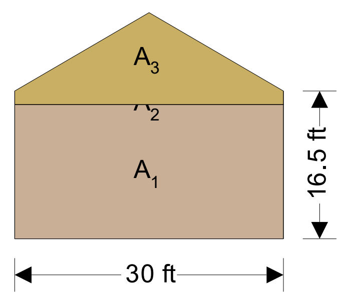


Leeward face

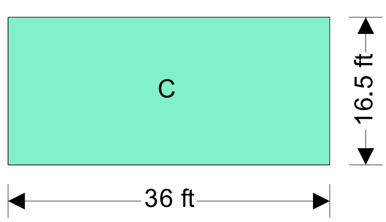
Tekla Tedds	Project CLT Residential Design				Job Ref.	
	Section Wind Loading MWFRS				Sheet no./rev. 6	
	Calc. by ACJ	Date 1/4/2021	Chk'd by	Date	App'd by	Date



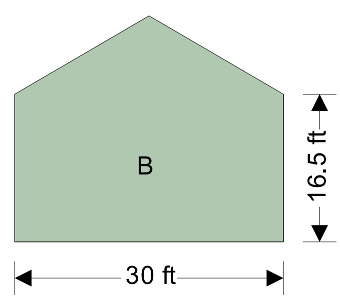
Plan view - Gable roof



Windward face



Side face



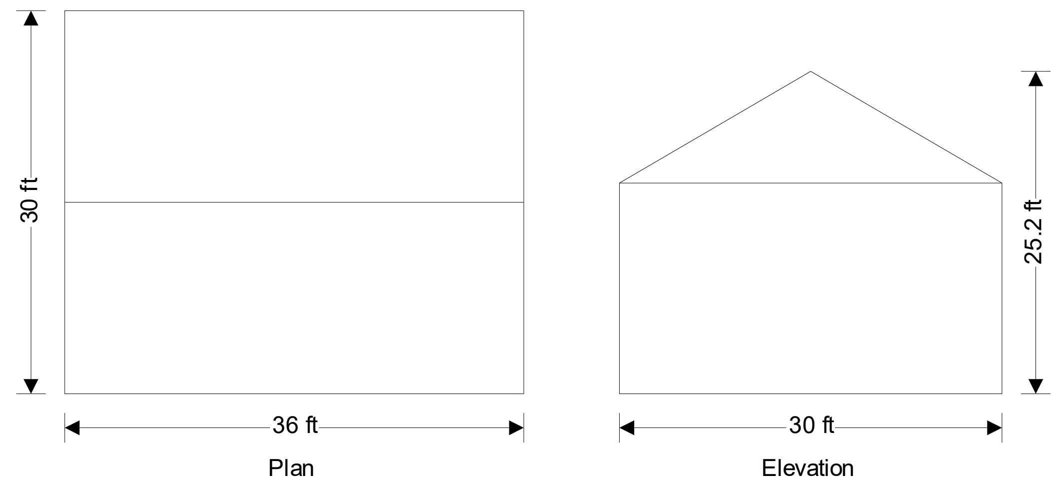
Leeward face

WIND LOADING

In accordance with ASCE7-10

Using the components and cladding design method

Tedds calculation version 2.1.06



Building data

Type of roof	Gable
Length of building	b = **36.00** ft
Width of building	d = **30.00** ft
Height to eaves	H = **16.50** ft
Pitch of roof	α_0 = **30.3** deg
Mean height	h = **20.87** ft

General wind load requirements

Basic wind speed	V = **115.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Exposure category (cl 26.7.3)	B
Enclosure classification (cl.26.10)	Enclosed buildings
Internal pressure coef +ve (Table 26.11-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.11-1)	GC_{pi_n} = **-0.18**
Gust effect factor	G_f = **0.85**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (T.30.3-1)	K_z = **0.70**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$ = **20.1** psf

Peak velocity pressure for internal pressure

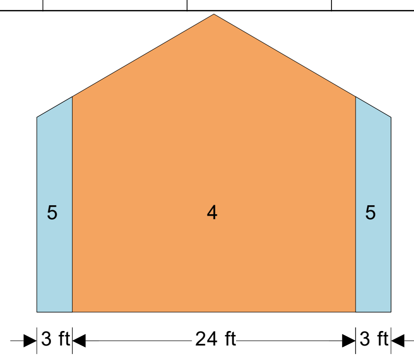
Peak velocity pressure – internal (as roof press.)	q_i = **20.14** psf

Equations used in tables

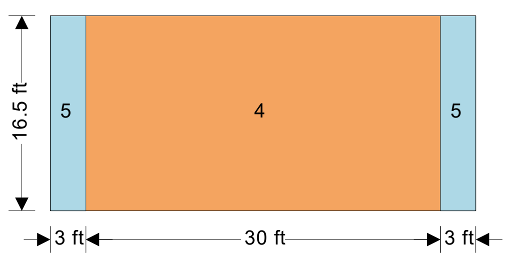
Net pressure $p = q_h \times [GC_p - GC_{pi}]$

Components and cladding pressures - Wall (Table 30.4-1)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	4	-	-	10.0	1.00	-1.10	23.8	-25.8
50 sf	4	-	-	50.0	0.88	-0.98	21.3	-23.3
200 sf	4	-	-	200.0	0.77	-0.87	19.1	-21.2
>500 sf	4	-	-	500.1	0.70	-0.80	17.7	-19.7
<=10 sf	5	-	-	10.0	1.00	-1.40	23.8	-31.8
50 sf	5	-	-	50.0	0.88	-1.15	21.3	-26.9
200 sf	5	-	-	200.0	0.77	-0.94	19.1	-22.6
>500 sf	5	-	-	500.1	0.70	-0.80	17.7	-19.7



Elevation of gable wall

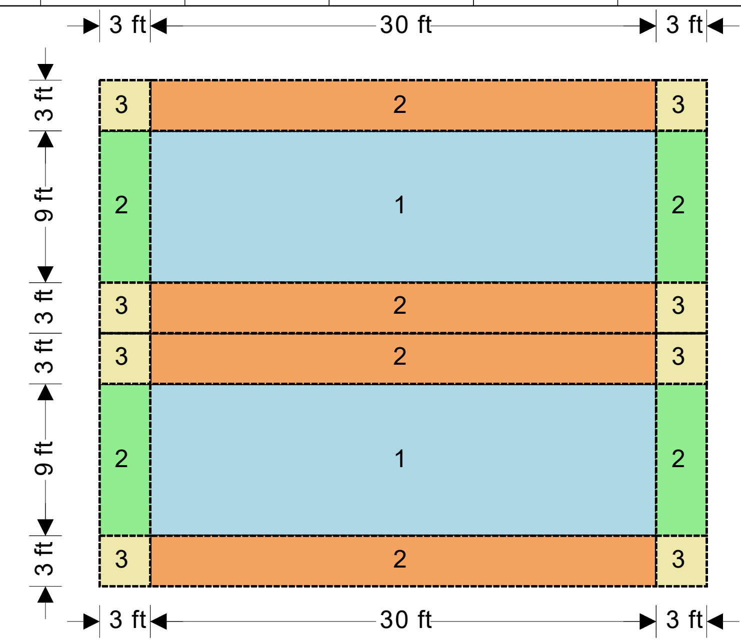


Elevation of side wall

Components and cladding pressures - Roof (Figure 30.4-2C)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	1	-	-	10.0	0.90	-1.00	21.8	-23.8
25 sf	1	-	-	25.0	0.86	-0.92	21.0	-22.2
50 sf	1	-	-	50.0	0.83	-0.86	20.3	-21.0
>100 sf	1	-	-	100.1	0.80	-0.80	19.7	-19.7
<=10 sf	2	-	-	10.0	0.90	-1.20	21.8	-27.8
25 sf	2	-	-	25.0	0.86	-1.12	21.0	-26.2

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
50 sf	2	-	-	50.0	0.83	-1.06	20.3	-25.0
>100 sf	2	-	-	100.1	0.80	-1.00	19.7	-23.8
<=10 sf	3	-	-	10.0	0.90	-1.20	21.8	-27.8
25 sf	3	-	-	25.0	0.86	-1.12	21.0	-26.2
50 sf	3	-	-	50.0	0.83	-1.06	20.3	-25.0
>100 sf	3	-	-	100.1	0.80	-1.00	19.7	-23.8



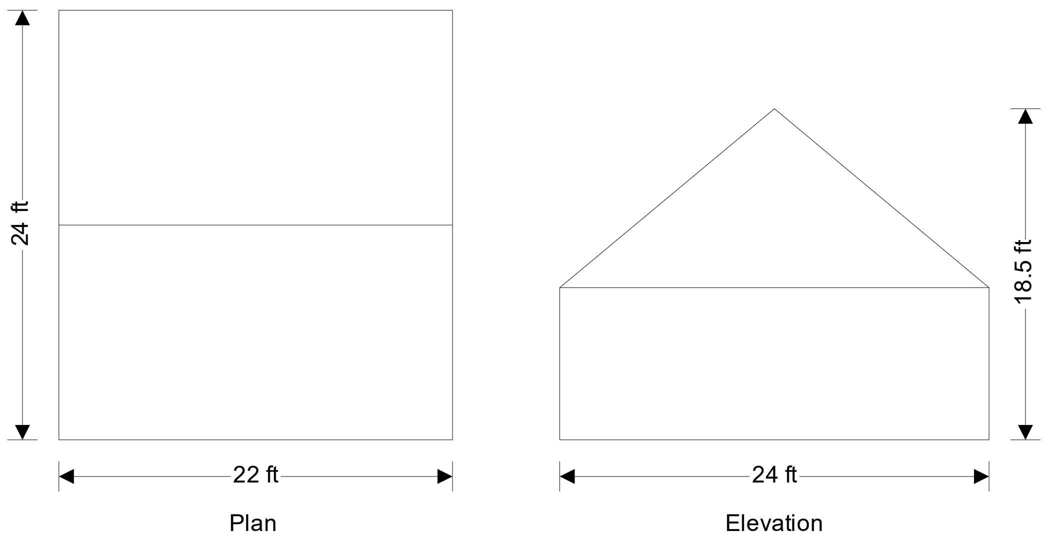
Plan on roof

WIND LOADING

In accordance with ASCE7-10

Using the components and cladding design method

Tedds calculation version 2.1.07



Building data

Type of roof	Gable
Length of building	b = **22.00** ft
Width of building	d = **24.00** ft
Height to eaves	H = **8.50** ft
Pitch of roof	α_0 = **39.8** deg
Mean height	h = **13.50** ft

General wind load requirements

Basic wind speed	V = **115.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Exposure category (cl 26.7.3)	B
Enclosure classification (cl.26.10)	Enclosed buildings
Internal pressure coef +ve (Table 26.11-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.11-1)	GC_{pi_n} = **-0.18**
Gust effect factor	G_f = **0.85**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (T.30.3-1)	K_z = **0.70**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$ = **20.1** psf

Peak velocity pressure for internal pressure

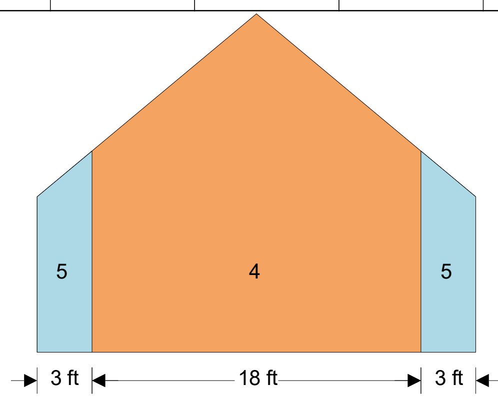
Peak velocity pressure – internal (as roof press.)	q_i = **20.14** psf

Equations used in tables

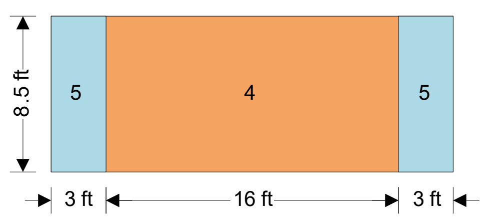
Net pressure $p = q_h \times [GC_p - GC_{pi}]$

Components and cladding pressures - Wall (Table 30.4-1)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft²)	+GCp	-GCp	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	4	-	-	10.0	1.00	-1.10	23.8	-25.8
50 sf	4	-	-	50.0	0.88	-0.98	21.3	-23.3
200 sf	4	-	-	200.0	0.77	-0.87	19.1	-21.2
>500 sf	4	-	-	500.1	0.70	-0.80	17.7	-19.7
<=10 sf	5	-	-	10.0	1.00	-1.40	23.8	-31.8
50 sf	5	-	-	50.0	0.88	-1.15	21.3	-26.9
200 sf	5	-	-	200.0	0.77	-0.94	19.1	-22.6
>500 sf	5	-	-	500.1	0.70	-0.80	17.7	-19.7



Elevation of gable wall

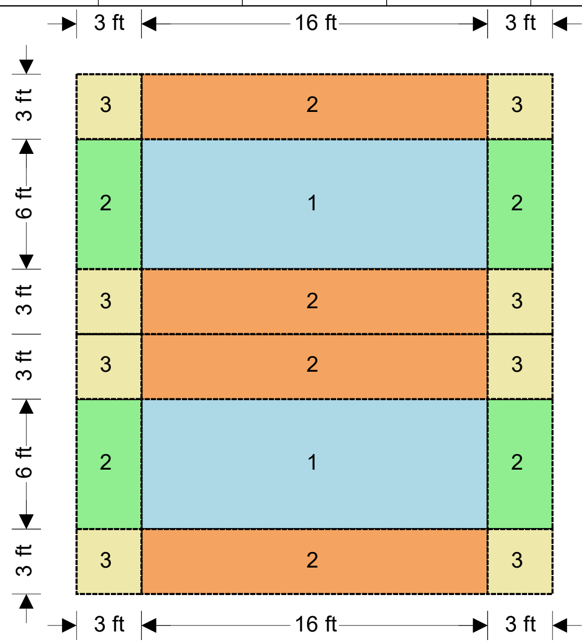


Elevation of side wall

Components and cladding pressures - Roof (Figure 30.4-2C)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft²)	+GCp	-GCp	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	1	-	-	10.0	0.90	-1.00	21.8	-23.8
25 sf	1	-	-	25.0	0.86	-0.92	21.0	-22.2
50 sf	1	-	-	50.0	0.83	-0.86	20.3	-21.0
>100 sf	1	-	-	100.1	0.80	-0.80	19.7	-19.7
<=10 sf	2	-	-	10.0	0.90	-1.20	21.8	-27.8
25 sf	2	-	-	25.0	0.86	-1.12	21.0	-26.2

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
50 sf	2	-	-	50.0	0.83	-1.06	20.3	-25.0
>100 sf	2	-	-	100.1	0.80	-1.00	19.7	-23.8
<=10 sf	3	-	-	10.0	0.90	-1.20	21.8	-27.8
25 sf	3	-	-	25.0	0.86	-1.12	21.0	-26.2
50 sf	3	-	-	50.0	0.83	-1.06	20.3	-25.0
>100 sf	3	-	-	100.1	0.80	-1.00	19.7	-23.8



Plan on roof

SNOW LOADING

In accordance with ASCE7-10

Tedds calculation version 1.0.09

Building details

Roof type	Hip and gable
Width of roof (left on elevation)	b_1 = **15.00** ft
Width of roof (right on elevation)	b_2 = **15.00** ft
Slope of roof (left on elevation)	α_1 = **30.26** deg
Slope of roof (right on elevation)	α_2 = **30.26** deg

Ground snow load

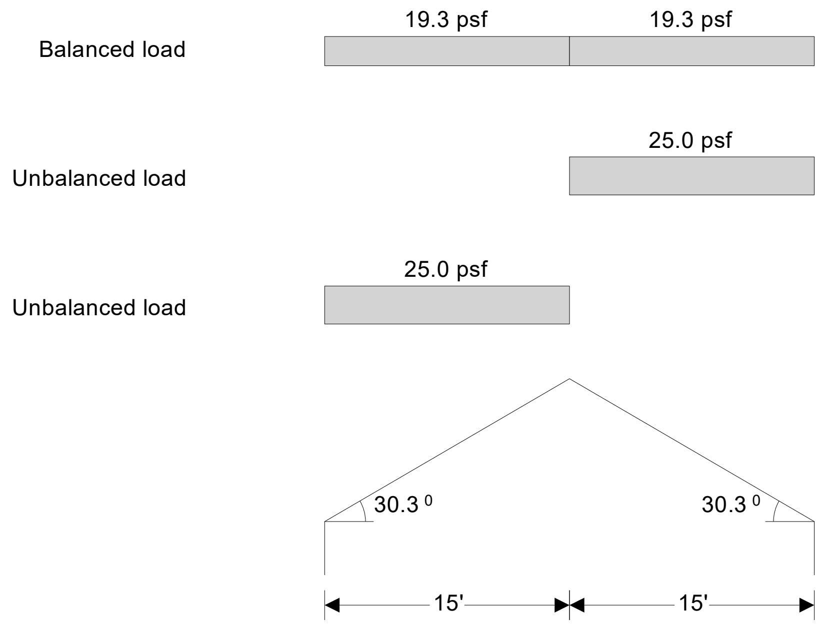
Ground snow load (Figure 7-1)	p_g = **25.00** lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1ft + 14lb/ft^3, 30lb/ft^3)$ = **17.25** lb/ft^3
Terrain typeSect. 26.7	B
Exposure condition (Table 7-2)	Partially exposed
Exposure factor (Table 7-2)	C_e = **1.00**
Thermal condition (Table 7-3)	Others with cold roofs
Thermal factor (Table 7-3)	C_t = **1.10**
Importance category (Table 1.5-1)	II
Importance factor (Table 1.5-2)	I_s = **1.00**
Flat roof snow load (Sect 7.3)	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g$ = **19.25** lb/ft^2

Cold roof slope factor (C_t > 1.0)

Roof surface type	Non slippery
Ventilation	Ventilated
Thermal resistance (R-value)	R = **30.00** ^{o}F h ft^2 / Btu
Roof slope factor - left Fig 7-2b (solid line)	C_{s_l} = **1.00**
Roof slope factor - right Fig 7-2b (solid line)	C_{s_r} = **1.00**

Hip and gable roof loads

Balanced sloped snow load - left (Cl.7.4)	$p_{s\_l} = C_{s\_l} \times p_f$ = **19.25** lb/ft^2
Balanced sloped snow load - right (Cl.7.4)	$p_{s\_r} = C_{s\_r} \times p_f$ = **19.25** lb/ft^2
Slope of left roof	$S_l = 1 / \tan(\alpha_1)$ = **1.71**
Slope of right roof	$S_r = 1 / \tan(\alpha_2)$ = **1.71**
Unbalanced load - left roof windward	p_{s_lw} = **0** lb/ft^2
Unbalanced load - right roof leeward	$p_{s\_rl} = I_s \times p_g$ = **25.00** lb/ft^2
Unbalanced load - left roof leeward	$p_{s\_ll} = I_s \times p_g$ = **25.00** lb/ft^2
Unbalanced load - right roof windward	p_{s_rw} = **0** lb/ft^2



Roof elevation

SNOW LOADING

In accordance with ASCE7-10

Tedds calculation version 1.0.09

Building details

Roof type	Hip and gable
Width of roof (left on elevation)	b_1 = **12.50** ft
Width of roof (right on elevation)	b_2 = **12.50** ft
Slope of roof (left on elevation)	α_1 = **39.81** deg
Slope of roof (right on elevation)	α_2 = **39.80** deg

Ground snow load

Ground snow load (Figure 7-1)	p_g = **25.00** lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3)$ = **17.25** lb/ft^3
Terrain typeSect. 26.7	B
Exposure condition (Table 7-2)	Partially exposed
Exposure factor (Table 7-2)	C_e = **1.00**
Thermal condition (Table 7-3)	Others with cold roofs
Thermal factor (Table 7-3)	C_t = **1.00**
Importance category (Table 1.5-1)	II
Importance factor (Table 1.5-2)	I_s = **1.00**
Flat roof snow load (Sect 7.3)	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g$ = **17.50** lb/ft^2

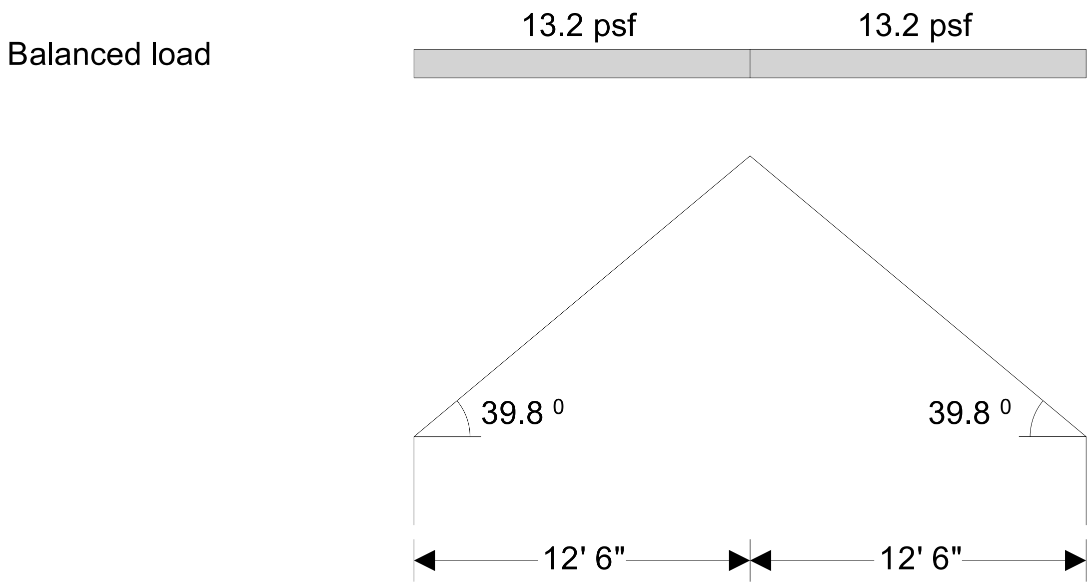
Warm roof slope factor (C_t <= 1.0)

Roof surface type	Non slippery
Ventilation	Ventilated
Thermal resistance (R-value)	R = **0.00** °F h ft^2 / Btu
Roof slope factor - left Fig 7-2a (solid line)	C_{s_l} = **0.75**
Roof slope factor - right Fig 7-2a (solid line)	C_{s_r} = **0.75**

Hip and gable roof loads

Balanced sloped snow load - left (Cl.7.4)	$p_{s\_l} = C_{s\_l} \times p_f$ = **13.21** lb/ft^2
Balanced sloped snow load - right (Cl.7.4)	$p_{s\_r} = C_{s\_r} \times p_f$ = **13.21** lb/ft^2
Slope of left roof	$S_l = 1 / \tan(\alpha_1)$ = **1.20**
Slope of right roof	$S_r = 1 / \tan(\alpha_2)$ = **1.20**

Tekla. Tedds	Project CLT Residential Design				Job Ref.	
	Section Snow Loads Garage Roof				Sheet no./rev. 2	
	Calc. by ACJ	Date 4/1/2021	Chk'd by	Date	App'd by	Date



Roof elevation

Drift calculations

Balanced snow load height — $h_b = \max(p_{s\_l}, p_{s\_r}) / \gamma =$ **0.77** ft

Length of upper roof — $l_u =$ **37.00** ft

Length of lower roof — $l_l =$ **22.50** ft

Height diff between uppper and lower roofs — $h_{diff} =$ **6.50** ft

Height from balance load to top of upper roof — $h_c = h_{diff} - h_b =$ **5.73** ft

Drift height leeward drift — $h_{d\_l} = 0.43 \times (\max(20\text{ ft}, l_u) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft} =$ **1.99** ft

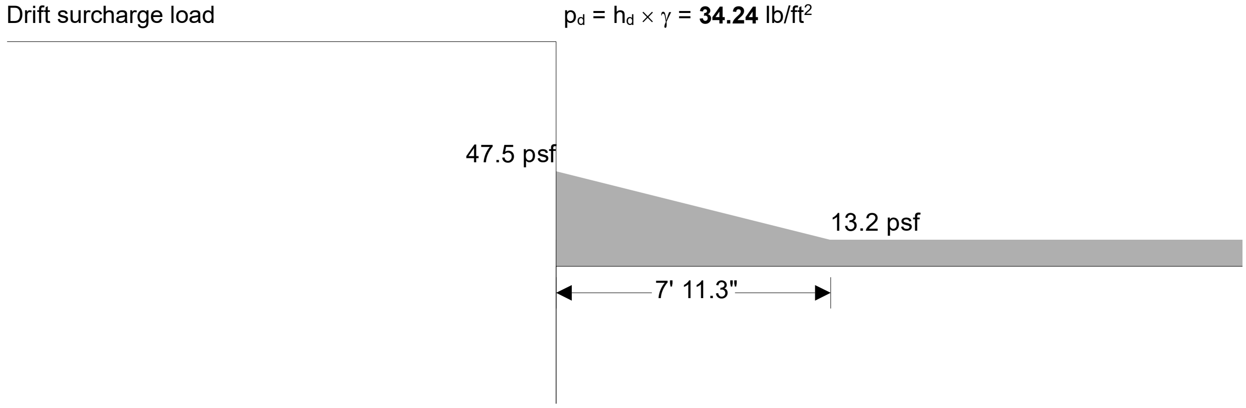
Drift height windward drift — $h_{d\_w} = 0.75 \times (0.43 \times (\max(20\text{ ft}, l_l) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft}) =$ **1.09** ft

Maximum lw/ww drift height — $h_{d\_max} = \max(h_{d\_w}, h_{d\_l}) =$ **1.99** ft

Drift height — $h_d = \min(h_{d\_max}, h_c) =$ **1.99** ft

Drift width — $W_d = \min(4 \times h_{d\_max}, 8 \times h_c) =$ **7.94** ft

Drift surcharge load — $p_d = h_d \times \gamma =$ **34.24** lb/ft^2



Elevation on snow drift

CLT Wall Panel Design

References:

1. 2018 National Design Specifications (NDS) for Wood Construction Supplement
2. 2018 NDS for Wood Construction
3. Nordic X-Lam Technical Guide NS-GT6-ASD; 2020-08-13
4. 2021 Special Design Provisions for Wind and Seismic (SDPWS)
5. Cross-Laminated Timber Structural Design Volume 2, pro:Holz 2017

Wall Panel Design (Ref. Appendix A Drawings for Panel Locations)

Size 2nd Floor Panel WP-5; North Elevation. Consider Nordic X-lam product. Initially consider the E1 Stress grade 89-3s layup with strong axis vertical.

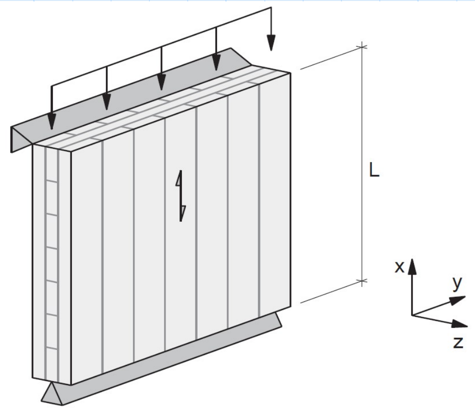


Figure 1. Wall panel typical orientation (Image from Ref.3)

Design properties per Ref. 3 (parallel to x axis on a per foot basis)

$t := 3.5\ in$ $\sigma_{self\_3.5} := 9.38\ psf$

Compressive properties parallel to the x axis on a per foot basis

$P_0 := 59000\ lbf$ Compressive Capacity $A_{eff\_0} := 33\ in^2$ Effective Area

$I_{eff\_0} := 42\ in^4$ Effective Moment of Inertia $r_{eff\_0} := 1.1\ in$ Radius of Gyration

Bending in major strength direction y-y

$M_0 := 3350\ lbf \cdot ft$ Bending Moment Capacity $V_0 := 1260\ lbf$ Rolling Shear Capacity

$EI_{eff\_0} := 72 \cdot 10^6\ lbf \cdot in^2$ Bending Stiffness $GA_{eff\_0} := 0.48 \cdot 10^6\ lbf$ Shear Rigidity

External Loading

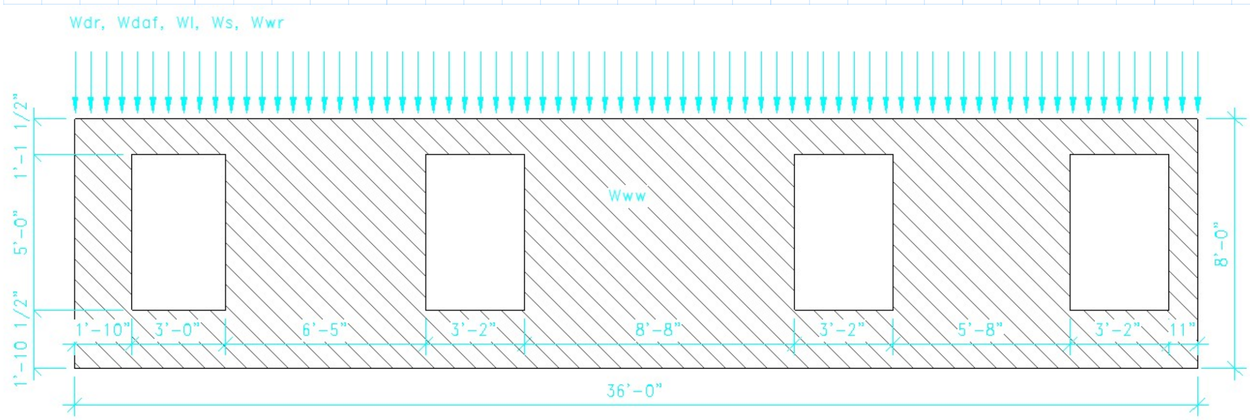


Figure 2. WP-5 Loading

Katerra Floor and Roof Panel Weight

$mc := 19$ Estimated Moisture Content (%) $G := 0.42$ Specific Gravity

$\gamma_{panel} := 62.4\ pcf \cdot \left(\frac{G}{1 + G \cdot 0.009 \cdot mc}\right) \cdot \left(1 + \frac{mc}{100}\right) = 29.1\ pcf$ Ref. 1 section 3.1.3

3.54" Roof Panels

$t_r := 3.54\ in$ $\sigma_{self\_3.54} := \gamma_{panel} \cdot t_r = 8.58\ psf$

$\sigma_{collateral\_r} := 10\ psf$

9.66" Attic Floor Panels

$t_{af} := 9.66\ in$ $\sigma_{self\_9.66} := \gamma_{panel} \cdot t_{af} = 23.42\ psf$

$\sigma_{collateral\_f} := 5\ psf$

Geometry

$W_{t\_af\_S} := 18.56\ ft$ Maximum Tributary Attic Floor Width, South Side

$L_{rp} := 18.5\ ft$ Roof Panel length

$W := 30\ ft$ Building Width

Uniformly Distributed Loading

$\omega_{d\_r} := \left(\sigma_{self\_3.54} + \sigma_{collateral\_r}\right) \cdot L_{rp} = 343.8\ plf$

$\omega_{d\_af} := \left(\sigma_{self\_9.66} + \sigma_{collateral\_f}\right) \cdot \frac{W_{t\_af\_S}}{2} = 263.77\ plf$

$\omega_l := 40\ psf \cdot \frac{W_{t\_af\_S}}{2} = 371.2\ plf$ Floor Live Load, assuming attic storage

$\omega_s := 25\ psf \cdot \frac{W}{2} = 375\ plf$ Snow load base on unbalanced condition see Tedds calculation in Appendix B

Wind Loading Roof

Loading calculated using the directional method for main wind force resisting systems (MWFRS). The wind load is not directly applied to the component, additionally the tributary area is large; therefore, the loading calculated by the components and cladding method would approach the MWFRS magnitudes. Consider Roof Load Case 2 (See Appendix B), which is the largest downward pressure. This case will be assumed to control.

$\sigma_{wr} := 6.36\ psf$ $\quad\theta_r := 30.256\ deg$ $\quad\omega_{wr} := \sigma_{wr} \cdot \cos\left(\theta_r\right) \cdot L_{rp} = 101.63\ plf$

Out-of-plane wind loading and wall panel

Use components and cladding loading

$L := 36\ ft$ $\quad H := 8\ ft$ $\quad A_t := H \cdot L = 288\ ft^2$ Wall Tributary Loading

Conservatively, use the negative wind pressure for tributary areas greater than 200 ft2 and wind zone 5 corner loading (See Appendix B).

$\sigma_{ww} := -22.6\ psf$ $\quad\omega_{ww} := \sigma_{ww} \cdot 1\ ft = -22.6\ plf$

Calculate Axial Capacity of Panels (See Ref. 2 3.7 and C3.7)

Check wall slenderness ratio (Ref. 2 3.7.1.4) $d := t = 3.5\ in$ $\quad K_e := 1.0$ Pin-Pin support conditions

$l_e := K_e \cdot H = 8\ ft$ $\quad\dfrac{l_e}{\left(\sqrt{12} \cdot r_{eff\_0}\right)} = 25.19$ < 50; OK for service

Adjustment Factors (Ref. 2 Table 10.3.1)

$C_{d\_Wind} := 1.6$ $\quad C_{d\_Dead} := 0.9$ $\quad C_m := 1.0$ $\quad C_L := 1.0$ $\quad C_t := 1.0$

Calculate Column Stability Factor (Cp), Ref. 2 3.7.1

$K_s := 11.8$ Pin-Pin supports Ref. 2 Equation 10.4-1

$$EI_{app} := \frac{EI_{eff\_0}}{1 + \dfrac{K_s \cdot EI_{eff\_0}}{GA_{eff\_0} \cdot H^2}} = 60399781.54\ lbf \cdot in^2 \qquad EI_{app\_min} := 0.518 \cdot EI_{app} = 31287086.84\ lbf \cdot in^2$$

$$EI_{app\_min} := EI_{app\_min} \cdot C_m \cdot C_t = 31287086.84\ lbf \cdot in^2$$

Slenderness controls for bending about the y-y axis since it is not braced in- and out-of-plane, CLT wall is fully braced for bending about the z-z axis. The following factors are calculated on a per foot basis.

$P_{ce} := \dfrac{\pi^2 \cdot EI_{app\_min}}{l_e^{\,2}} = 33505.99\ lbf$ $\quad P_{c\_star} := P_0 \cdot C_{d\_Dead} \cdot C_m \cdot C_t = 53100\ lbf$

$c := 0.9$ Empirical Parameter (See Ref. 2 C3.7.1.5) $\quad \alpha_c := \dfrac{P_{ce}}{P_{c\_star}} = 0.631$

$$C_p := \frac{1+\alpha_c}{2 \cdot c} - \sqrt{\left(\frac{1+\alpha_c}{2 \cdot c}\right)^2 - \left(\frac{\alpha_c}{c}\right)} = 0.56$$

$P'_s := P_{c\_star} \cdot C_p = 29725.69\ lbf$

Calculate Combined Axial Panel Load

1. ASD Load Combinations to check

    1. D+S
    2. D+0.75L+0.75S+(0.75)0.6W

$P_{load\_1} := \omega_{d\_r} + \omega_{d\_af} + \omega_s = 982.57\ plf$

$P_{load\_2} := \omega_{d\_r} + \omega_{d\_af} + 0.75 \cdot \omega_l + 0.75\ \omega_s + 0.75 \cdot 0.6 \cdot \omega_{wr} = 1212.96\ plf$

$P_{load} := \max\left(P_{load\_1}, P_{load\_2}\right) \cdot 1\ ft = 1212.96\ lbf$ $\ P'_s = 29725.69\ lbf >> P_{load} = 1212.96\ lbf$; therefore OK.

Review out-of-plane bending capacity of panels

1. ASD Load Combinations to check

    1. 0.6D+0.6W

$$M_a := 0.6 \cdot \left(\frac{\left(-1 \cdot \omega_{ww} \cdot H^2\right)}{8}\right) = 108.48\ lbf \cdot ft$$

$M_0' := M_0 \cdot C_{d\_Wind} \cdot C_m \cdot C_L \cdot C_t = 5360\ lbf \cdot ft$ $\quad >> \quad M_a = 108.48\ lbf \cdot ft$ $\quad$ Therefore, OK

Check Interaction (Ref. 2 C3.9.2-3)

$$\left(\frac{P_{load}}{P'_s}\right)^2+\frac{M_a}{M_0'\cdot\left(1-\frac{P_{load}}{P_{ce}}\right)}=0.023$$

<< 1.0 ;Therefore, OK

The 3.5" WP-5 Panel is more than adequate. No more axial/out-of-plane bending checks will be performed on the panels.

Next select openings will be reviewed to determine if the main building 3.5" panel material will be adequate to function as a beam/strut that spans openings. Initially, review B-3 (WP-4) on the North Elevation. Note it will be assumed that the reactions of B-4 (P on Figure 3) will be distributed downward through panel WP-5 at an approximate 25 degree angle (Ref. 5) The load will be applied as a distributed load to the 2nd Floor.

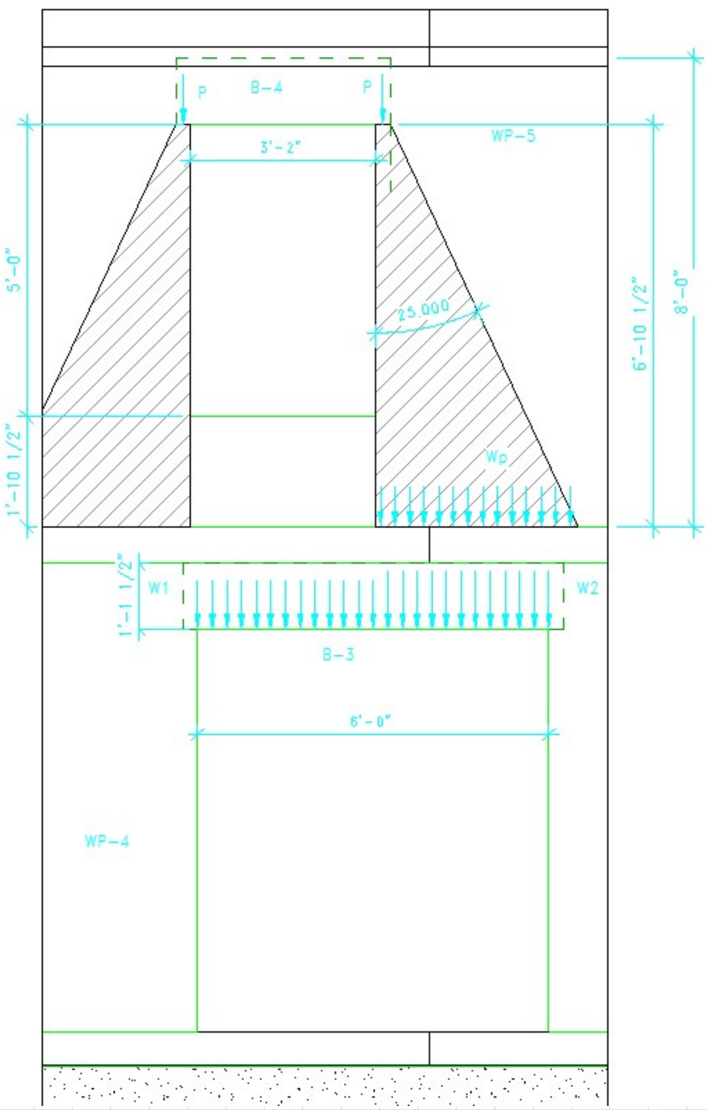


Figure 3. B-3 Loading

Determine Wp

$l_{B\_4} := 3.17\ ft$ Length of B-4 $H_{h\_B\_4} := 6.88\ ft$ Header Height B-4

$l_{wp} := \tan(25\ deg) \cdot H_{h\_B\_4} = 3.21\ ft$

$W_{t\_af\_N} := 11.44\ ft$ Maximum Tributary Attic Floor Width, North Side

$\omega_{d\_af\_N} := (\sigma_{self\_9.66} + \sigma_{collateral\_f}) \cdot \frac{W_{t\_af\_N}}{2} = 162.58\ plf$

$P_d := (\omega_{d\_r} + \omega_{d\_af\_N}) \cdot \frac{l_{B\_4}}{2} = 802.62\ lbf$ $\omega_{p\_d} := \frac{P_d}{l_{wp}} = 250.18\ plf$

$\omega_{l\_af} := 40\ psf \cdot \frac{W_{t\_af\_N}}{2} = 228.8\ plf$ Attic Floor Live Load, Assuming Storage

$P_L := \omega_{l\_af} \cdot \frac{l_{B\_4}}{2} = 362.65\ lbf$ $\omega_{p\_l} := \frac{P_L}{l_{wp}} = 113.04\ plf$

$P_s := \omega_s \cdot \frac{l_{B\_4}}{2} = 594.38\ lbf$ $\omega_{p\_s} := \frac{P_s}{l_{wp}} = 185.27\ plf$

$P_w := \omega_{wr} \cdot \frac{l_{B\_4}}{2} = 161.09\ lbf$ $\omega_{p\_w} := \frac{P_w}{l_{wp}} = 50.21\ plf$

Determine Uniformly Distributed Loads on B-4

6.90" 2nd Floor Panels

$t_{2\_f} := 6.9\ in$ $\sigma_{self\_6.9} := \gamma_{panel} \cdot t_{2\_f} = 16.73\ psf$

$W_{t\_2} := 16.08\ ft$ Maximum Tributary 2nd Floor Width, North Side

$\omega_{d\_2\_F\_N} := \left(\sigma_{self\_6.9} \cdot \frac{W_{t\_2}}{2}\right) = 134.52\ plf$

$\omega_{1\_d} := \omega_{d\_2\_F\_N} + \sigma_{self\_3.5} \cdot (1.13\ ft + 1.88\ ft) = 162.75\ plf$ $\omega_{1\_l} := 30\ psf \cdot \frac{W_{t\_2}}{2} = 241.2\ plf$

$\omega_{2\_d} := \omega_{d\_r} + \omega_{d\_af\_N} + \omega_{d\_2\_F\_N} + \sigma_{self\_3.5} \cdot (1.13\ ft + 8\ ft) + \omega_{p\_d} = 976.72\ plf$

$\omega_{2\_l} := \omega_{l\_af} + \omega_{1\_l} + \omega_{p\_l} = 583.04\ plf$ $\omega_{2\_s} := \omega_s + \omega_{p\_s} = 560.27\ plf$

$\omega_{2\_w} := \omega_{wr} + \omega_{p\_w} = 151.84\ plf$

Edgewise bending properties for lintel design (Ref. 3). Note that the Ref. 3 Lintel table notes instruct the user to use the effective width in table, the actual beam depth and the design values for S-P-F No. 3 lumber for loading parallel to outermost layer.

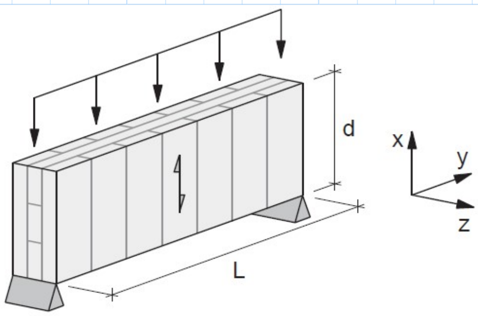


Figure 4. Edgewise bending layup (Ref. 3)

$b_{eff} := 0.75\ in$ $d := 13.5\ in$

$F_{v\_90} := 190\ psi$

$G_v t_{v\_90} := 1.52 \cdot 10^6\ \frac{lbf}{ft}$

$F_b := 500\ psi$

$E := 1.2 \cdot 10^6\ psi$

Determine design load effects in RISA3D (see Appendix C).

$M_a := 3.9\ kip \cdot ft$ $V_a := 4.4\ kip$ $S_{eff} := \frac{b_{eff} \cdot d^2}{6} = 22.78\ in^3$ $C_{d\_snow} := 1.15$

Determine beam stability factor

$l_u := 6\ ft$ Fixed end conditions, CL will not equal 1.0

Check Slenderness

$\frac{l_u}{d} = 5.33$ which is less than 7 therefore: $l_e := 2.06 \cdot l_u = 12.36\ ft$

$R_b := \sqrt{\frac{l_e \cdot d}{{b_{eff}}^2}} = 59.66$ > 50 therefore this member is too slender if considering the effective width of this member. Since there are many openings in the envelope it would be worth considering the 105-3s panel. This panel has a thicker middle layer, which increases the effective width of the panel.

$b_{eff} := 1.38\ in$ $R_b := \sqrt{\frac{l_e \cdot d}{{b_{eff}}^2}} = 32.43$ Slenderness is OK for the wider panel, proceed to investigate the effects of fixed end conditions on the beam stability factor CL.

Re-calculate CL

$COV_E := 0.10$ Ref. 2 Table F1, Structurally glued laminated timber

Ref. 2 Equation D-4

$$E_{min} := \frac{E \cdot (1 - 1.645 \cdot COV_E) \cdot 1.03}{1.66} = 622095.18\ psi$$

$C_i := 1.0$ Incising Factor

$$E'_{min} := E_{min} \cdot C_m \cdot C_t \cdot C_i = 622095.18\ psi$$

$$F_{be} := \frac{1.20 \cdot E'_{min}}{{R_b}^2} = 710.01\ psi \qquad F_{b\_star} := F_b \cdot C_{d\_snow} \cdot C_m \cdot C_t = 575\ psi$$

$$C_L := \frac{1 + \left(\frac{F_{be}}{F_{b\_star}}\right)}{1.9} - \sqrt{\left(\frac{1 + \left(\frac{F_{be}}{F_{b\_star}}\right)}{1.9}\right)^2 - \frac{\frac{F_{be}}{F_{b\_star}}}{0.95}} = 0.887 \qquad F_b' := F_{b\_star} \cdot C_L = 575\ psi$$

$$S_{eff} := \frac{b_{eff} \cdot d^2}{6} \qquad M_{90} := F_b' \cdot S_{eff} = 2.01\ kip \cdot ft$$ < $M_a = 3.9\ kip \cdot ft$; therefore no good

Since the axial and the strong axis capacity was so underutilized try rotating the panels 90 degrees such that the outer panels are parallel to the ground. See Figure 5. Try the 89-3s panel first.

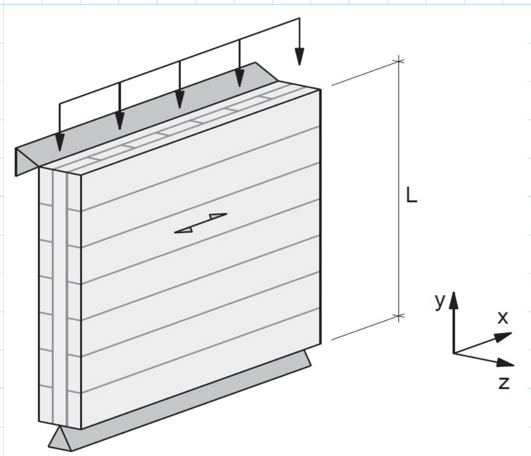


Figure 5. Panel axis rotated 90 degrees (Ref. 3)

Check slenderness first to determine if the panel would be OK in compression

$r_{eff\_90} := 0.22\ in$

$K_e := 1.0$ Pin-Pin support conditions, Ref. 2, Appendix G

$l_e := K_e \cdot H = 8\ ft$

$$\frac{l_e}{\left(\sqrt{12} \cdot r_{eff\_90}\right)} = 125.97$$ > 50; therefore, too slender.

Try the 105-3s

$r_{eff\_90} := 0.40\ in$

$$\frac{l_e}{\left(\sqrt{12} \cdot r_{eff\_90}\right)} = 69.28$$ > 50; therefore, too slender. Try the 143-5s

$r_{eff\_90} := 1.1\ in$

$\dfrac{l_e}{\left(\sqrt{12} \cdot r_{eff\_90}\right)} = 25.19$ < 50; This would be OK, but @ the cost of an additional laminate layer.

Move foward with the 105-3s panels oriented with x axis vertical. The 89-3s would work oriented vertically, but none of the lintels would work because they would be slender as identified previously. The 6-0 openings on the 1st floor, North and South elevation will require the installation of header members. The panels will have to be split at those locations. Try using a 105-3s CLT lintel with the x-axis horizontal (Figure 6). This will provide two layers to resist the bending.

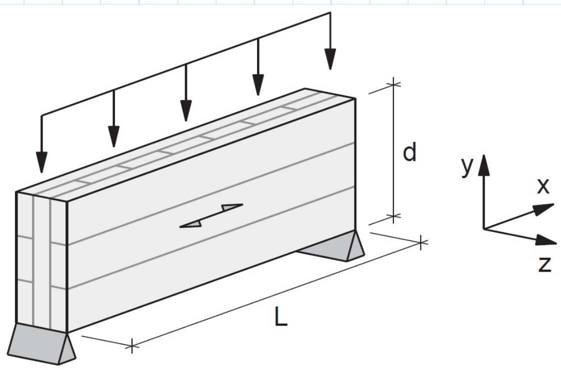


Figure 6. Lintel with outer layers horizontal (Ref. 3)

$b := 4.125\ in$ $\sigma_{self\_4.125} := 11.1\ psf$

$b_{eff} := 2.75\ in$ $d := 13.5\ in$

$F_{v\_0} := 155\ psi$

$G_v t_{v\_0} := 1.79 \cdot 10^6\ \dfrac{lbf}{ft}$

$F_b := 1950\ psi$

$E := 1.7 \cdot 10^6\ psi$ $S_{eff} := \dfrac{b_{eff} \cdot d^2}{6} = 83.53\ in^3$

Check Slenderness

$l_u := 6\ ft$ Note that this is conservative, this could be less (approx. 6") due to simple span condition

$\dfrac{l_u}{d} = 5.33$ which is less than 7 therefore: $l_e := 2.06 \cdot l_u = 12.36\ ft$ $R_b := \sqrt{\dfrac{l_e \cdot d}{{b_{eff}}^2}} = 16.27$ < 50 ; therefore OK.

Determine beam stability factor (CL)

$E_{min} := \dfrac{E \cdot \left(1 - 1.645 \cdot COV_E\right) \cdot 1.03}{1.66} = 881301.51\ psi$ $E'_{min} := E_{min} \cdot C_m \cdot C_t \cdot C_i = 881301.51\ psi$

$F_{be} := \dfrac{1.20 \cdot E'_{min}}{{R_b}^2} = 3994.27\ psi$ $F_{b\_star} := F_b \cdot C_{d\_snow} \cdot C_m \cdot C_t = 2242.5\ psi$

$M_a := 4.9\ kip \cdot ft$ Moment increase due to simple span condition

$$C_L := \frac{1+\left(\frac{F_{be}}{F_{b\_star}}\right)}{1.9} - \sqrt{\left(\frac{1+\left(\frac{F_{be}}{F_{b\_star}}\right)}{1.9}\right)^2 - \frac{\frac{F_{be}}{F_{b\_star}}}{0.95}} = 0.946 \qquad F_b' := F_{b\_star} \cdot C_L = 2122.21\ psi$$

$M_0 := F_b' \cdot S_{eff} = 14.77\ kip \cdot ft$ > $M_a = 4.9\ kip \cdot ft$ therefore OK

A 105-3s layup, with outer layers horizontal will be OK for the lintels over the lower level 6 foot openings. See if the outer layers vertical will work for the smaller openings on the first floor and 2nd floor openings. Investigate B-6, utilize loading from B-3 design. For simplification purposes assume that line load ω_2 act along entire length. ω_2 includes the ω_p line loading.

$\omega_{2\_d} = 976.72\ plf$ $\quad \omega_{2\_l} = 583.04\ plf$ $\quad \omega_{2\_s} = 560.27\ plf$ $\quad \omega_{2\_w} = 151.84\ plf$

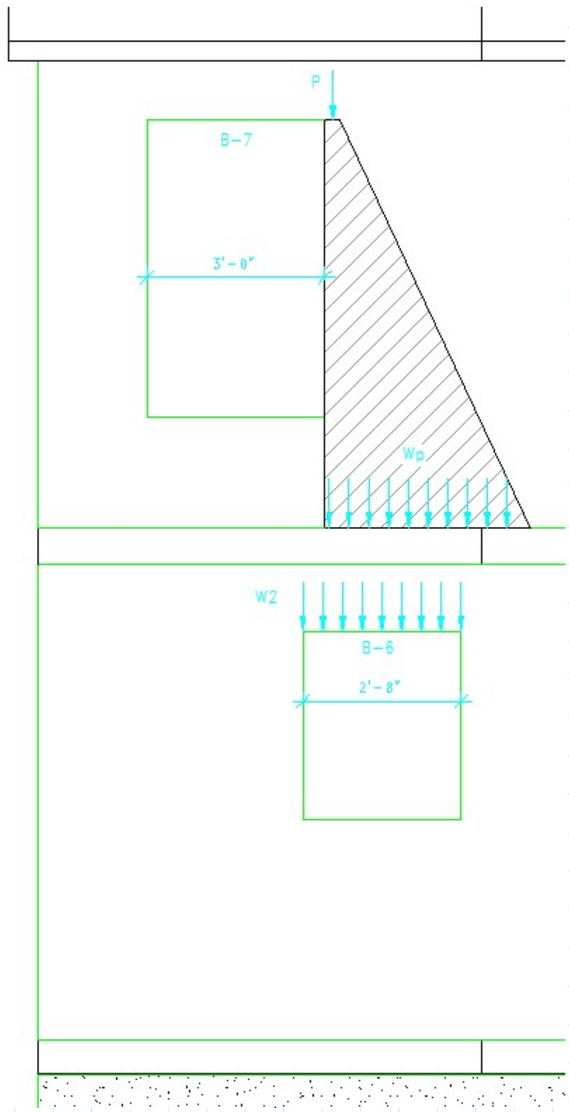


Figure 7. B-6 Loading

Shear Walls, Lintels, and Diaphragms – Design Properties

CLT stress grade	E1 (L = S-P-F 1950f MSR and T = S-P-F No. 3)								
Layup combination	89-3s	105-3s	143-5s	175-5s	197-7s	213-7l	244-7s	244-7l	267-9l
Loading parallel to outermost layers									
Edgewise bending (z-z)									
Effective width for bending, $b_{eff,90}$ (in.) [(a)]	0.75	1.38	1.50	2.75	2.25	1.50	4.13	2.75	2.25
Shear capacity, $F_{v,90}$ (psi) [(b)]	190	190	215	215	215	215	215	215	215
Shear rigidity, $G_v t_{v,90}$ (10^6 lbf/ft) [(c)]	1.52	1.79	2.44	2.99	3.37	3.64	4.18	4.18	4.56
Loading perpendicular to outermost layers									
Edgewise bending (z-z)									
Effective width for bending, $b_{eff,0}$ (in.) [(a)]	2.75	2.75	4.13	4.13	5.50	6.88	5.50	6.88	8.25
Shear capacity, $F_{v,0}$ (psi) [(b)]	155	155	185	185	155	185	185	185	155
Shear rigidity, $G_v t_{v,0}$ (10^6 lbf/ft) [(c)]	1.52	1.79	2.44	2.99	3.37	3.64	4.18	4.18	4.56

a) The bending moment capacity, M, and the effective bending stiffness, $(EI)_{eff}$, values shall be based on the effective width and depth of the panel, b_{eff} and d (see table above and figures below), respectively, and Section 5 of the NDS. Calculations shall be based on S-P-F No. 3 lumber (F_b = 500 psi, E = 1,200,000 psi) for loading parallel to outermost layer, or on S-P-F 1950f MSR lumber (F_b = 1,950 psi, E = 1,700,000 psi) for loading perpendicular to outermost layer.

b) The shear capacity values, V, shall be based on Section 5 of the NDS, taking into account the gross cross-sectional area of the panel and using the in-plane shear capacity, F_v.

c) G_v = 36,200 psi based on product performance testing. The shear rigidity, $(GA)_{eff}$, shall be calculated by multiplying $G_v t_v$ by the member depth, d, in feet (see figures below).

Figure 8. Ref. 3 lintel design properties

Calculate Edgewise Bending Resistance (Loading parallel to the outermost layer)

Check Slenderness

$l_u := 3\ ft$ Fixed end condition $d := 13.5\ in$ $b_{eff} := 1.38\ in$ $S_{eff} := \dfrac{b_{eff} \cdot d^2}{6} = 41.92\ in^3$

$\dfrac{l_u}{d} = 2.67$ which is less than 7 therefore: $l_e := 2.06 \cdot l_u = 6.18\ ft$ $R_b := \sqrt{\dfrac{l_e \cdot d}{{b_{eff}}^2}} = 22.93$ < 50 ; therefore OK.

$E := 1.2 \cdot 10^6\ psi$ $F_{v\_90} := 190\ psi$ $F_b := 500\ psi$

$COV_E := 0.25$ Ref. 2 Table F1, Sawn Lumber

$E_{min} := \dfrac{E \cdot (1 - 1.645 \cdot COV_E) \cdot 1.03}{1.66} = 438370.48\ psi$ $F_{be} := \dfrac{1.20 \cdot E'_{min}}{{R_b}^2} = 2011.69\ psi$

$F_{b\_star} := F_b \cdot C_{d\_snow} \cdot C_m \cdot C_t = 575\ psi$

$$C_L := \frac{1 + \left(\frac{F_{be}}{F_{b\_star}}\right)}{1.9} - \sqrt{\left(\frac{1 + \left(\frac{F_{be}}{F_{b\_star}}\right)}{1.9}\right)^2 - \frac{\frac{F_{be}}{F_{b\_star}}}{0.95}} = 0.981$$

$F_b' := F_{b\_star} \cdot C_L = 544.16\ psi$

$M_{90} := F_b' \cdot S_{eff} = 1.9\ kip \cdot ft$

Design load effects from Risa analysis

$M_a := 1.35\ kip \cdot ft$ $V_a := 2.74\ kip$ $R_a := 2.50\ kip$

Shear Check

$F_{v\_90} = 190\ psi$ $A_{gross} := b \cdot d = 55.69\ in^2$ $V_{90} := F_{v\_90} \cdot A_{gross} = 10.58\ kip$

$M_a = 1.35\ kip \cdot ft$ < $M_{90} = 1.9\ kip \cdot ft$ and $V_{90} = 10.58\ kip$ > $V_a = 2.74\ kip$; therefore OK for bending and shear.

Check bearing, assume 3 inches of bearing length per side. initially, assume that only the vertical members are effective in bearing.

$b_{bearing} := 1.375\ in \cdot 2 = 2.75\ in$ $b_{length} := 3\ in$ $b_{area} := b_{bearing} \cdot b_{length} = 8.25\ in^2$

Based on Ref. 3 lintel design info (pg 2.25) outer most layers are constructed from SPF MSR 1950f 1.7E lumber.

$F_{c\_parallel} := 1800\ psi$ $F_{c\_parallel\_}b_{area}' := F_{c\_parallel} \cdot b_{area} \cdot C_{d\_snow} \cdot C_m \cdot C_t = 17.08\ kip$

$F_{c\_parallel\_}b_{area}' = 17.08\ kip$ > $R_a = 2.5\ kip$; therefore bearing OK.

Bending, shear and bearing are OK; therefore the 105-3s panels are OK to span the remaining openings on the North elevations.

By inspection the remaining headers on the South elevation are OK; therefore use the 105-3s panels with the outer layers vertical for all the main building walls. The 1st floor walls longitudinal walls will need to be split at the locations indicated on the drawings to allow for the installaton of 105-3s lintels rotated such that the outer layers are horizontal (total of 3 beams). No splits or lintel rotation is necessary to span the remaining openings.

Assume that the 105-3s panels are OK for the garage; however a header design will be required to span the garage door opening B-2. Garage floor panels run parallel to header, therefore only roof load is present on header. Try the 105-3s panels mounted with the outer layers horizontal.

3.54" Garage Roof Panels

$t_r := 3.54\ in$ $\sigma_{self\_3.54} := \gamma_{panel} \cdot t_r = 8.58\ psf$ $L_{rp\_g} := 17\ ft$ Roof panel length garage

$\sigma_{collateral\_r} := 10\ psf$ $W_{garage} := 26\ ft$

$\omega_{d\_r} := \left(\sigma_{self\_3.54} + \sigma_{collateral\_r}\right) \cdot L_{rp\_g} = 315.93\ plf$ $\omega_{d\_s} := 25\ psf \cdot \dfrac{W_{garage}}{2} = 325\ plf$

$M_a := 22.8\ kip \cdot ft$ From Risa analysis

Calculate largest positive wind pressure based on components and cladding methodology (See Appendix B).

$effective\_area := 13\ ft \cdot 23\ ft = 299\ ft^2$ Therefore $\sigma_{wr} := 19.7\ psf$

$\theta_r := 39.806\ deg$ $\omega_{wr} := \sigma_{wr} \cdot \cos(\theta_r) \cdot L_{rp} = 279.98\ plf$ Vertical Component

Beam Section and Mechanical Properties

$l := 16.08\ ft$ $b := 4.125\ in$ $\sigma_{self\_4.125} := 11.1\ psf$ $b_{eff} := 2.75\ in$ $d := 2.4\ ft$

$F_{v\_0} := 155\ psi$ $F_b := 1950\ psi$

$E := 1.7 \cdot 10^6\ psi$ $S_{eff} := \dfrac{b_{eff} \cdot d^2}{6} = 380.16\ in^3$

Check Slenderness

$l_u := 0.5\ ft$ Less conservatively assume a simple span

$\dfrac{l_u}{d} = 0.21$ which is less than 7 therefore: $l_e := 2.06 \cdot l_u = 1.03\ ft$

Determine beam stability factor (CL)

$R_b := \sqrt{\dfrac{l_e \cdot d}{{b_{eff}}^2}} = 6.86$ < 50 ; therefore OK.

$COV_E := 0.11$ Ref. 2 Table F1, MSR Lumber

$E_{min} := \dfrac{E \cdot (1 - 1.645 \cdot COV_E) \cdot 1.03}{1.66} = 863949.73\ psi$ $E'_{min} := E_{min} \cdot C_m \cdot C_t \cdot C_i = 863949.73\ psi$

$F_{be} := \dfrac{1.20 \cdot E'_{min}}{{R_b}^2} = 22025.42\ psi$ $F_{b\_star} := F_b \cdot C_{d\_snow} \cdot C_m \cdot C_t = 2242.5\ psi$

$$C_L := \frac{1 + \left(\dfrac{F_{be}}{F_{b\_star}}\right)}{1.9} - \sqrt{\left(\frac{1 + \left(\dfrac{F_{be}}{F_{b\_star}}\right)}{1.9}\right)^2 - \frac{\dfrac{F_{be}}{F_{b\_star}}}{0.95}} = 0.994 \qquad F_b' := F_{b\_star} \cdot C_L = 2229.94\ psi$$

$M_0 := F_b' \cdot S_{eff} = 70.64\ kip \cdot ft$ > $M_a = 22.8\ kip \cdot ft$; therefore OK

Deflection

$\Delta_{SL} := 0.053\ in < \frac{l}{360} = 0.536\ in$; therefore OK

$\Delta_{TL} := 0.107\ in < \frac{l}{240} = 0.804\ in$; therefore OK

Bearing (perpindicular to grain)

$R_a := 5.7\ kip$

$b_{bearing} := b = 4.125\ in$ $b_{length} := 3\ in$ $b_{area} := b_{bearing} \cdot b_{length} = 12.38\ in^2$

Allowable bearing pressure based on SPF #3

$F_{c\_perp} := 425\ psi$ Ref.3 pg. 2.25 Note 2

$C_b := \frac{b_{length} + 0.375\ in}{b_{length}} = 1.13$ $F_{c\_perp\_}b_{area}' := F_{c\_perp} \cdot b_{area} \cdot C_{d\_snow} \cdot C_m \cdot C_t = 6.05\ kip$

$F_{c\_perp\_}b_{area}' = 6.05\ kip > R_a = 5.7\ kip$; therefore bearing OK.

Shear OK by inspection. 105-3s with outer layers horizontal OK for garage header.
Alternatively a glulam could be substituted if more cost effective.

CLT Floor Panel Design

References:

1. 2018 NDS Supplement
2. 2018 NDS
3. Katerra CLT Pre-Analysis Span Tables (Updated February 2020)
4. Katerra Product Definitions Technical Specifications (Updated January 2020)
5. 2021 SDPWS
6. Cross-Laminated Timber Structural Design Volume 2, pro:Holz 2017
7. CLT Handbook, FPInnovations and Binational Softwood Lumber Council, 2013
8. APA PRG 320-2019

1st Floor Wall Panel Design (Ref. Appendix A Drawings)

Consider Katerra CLT products. Size floor panels initially based on pre-analysis span tables (Ref. 3). See Figure 1 for panel estimates based on tables.

FLOOR | LL = 40psf | SDL=30psf

CLT Layup Designation		CLT Thickness (in)	Span Type	Major Direction			Minor Direction		
				Max Span	Controlling Criteria	Max Span 1hr Fire	Max Span	Controlling Criteria	Max Span 1hr Fire
3-Ply	K3-0320	3.24	SS	9' - 10"	Vibration	x	3' - 2"	Strength	x
			Cant	5' - 8"	Strength	x	1' - 7"	Strength	x
	K3-0350	3.54	SS	10' - 5"	Vibration	x	4' - 0"	Strength	x
			Cant	6' - 1"	Strength	x	2' - 0"	Strength	x
	K3-0380	3.84	SS	11' - 2"	Vibration	x	3' - 2"	Strength	x
			Cant	6' - 9"	Strength	x	1' - 7"	Strength	x
	K3-0410	4.14	SS	11' - 9"	Vibration	x	4' - 0"	Strength	x
			Cant	7' - 2"	Strength	x	2' - 0"	Strength	x
5-Ply	K5-0540	5.40	SS	13' - 8"	Vibration	13' - 8"	9' - 0"	Strength	5' - 8"
			Cant	8' - 4"	Strength	8' - 4"	4' - 6"	Strength	2' - 10"
	K5-0600	6.00	SS	14' - 6"	Vibration	14' - 6"	10' - 8"	Strength	10' - 0"
			Cant	8' - 11"	Strength	8' - 11"	5' - 4"	Strength	5' - 0"
	K5-0630	6.30	SS	15' - 7"	Vibration	15' - 7"	9' - 8"	Strength	9' - 8"
			Cant	9' - 10"	Strength	9' - 10"	4' - 10"	Strength	4' - 10"
	K5-0690	6.90	SS	16' - 8"	Vibration	16' - 8"	11' - 4"	Strength	11' - 4"
			Cant	10' - 5"	Strength	10' - 5"	5' - 8"	Strength	5' - 8"
7-Ply	K7-0970	9.66	SS	20' - 6"	Vibration	20' - 6"	15' - 11"	Vibration	15' - 11"
			Cant	13' - 5"	Strength	13' - 5"	8' - 3"	Strength	8' - 3"
9-Ply	K9-1120	11.22	SS	23' - 0"	Vibration	23' - 0"	17' - 7"	Vibration	17' - 7"
			Cant	15' - 4"	Strength	15' - 4"	9' - 1"	Strength	9' - 1"
	K9-1240	12.42	SS	24' - 3"	Vibration	24' - 3"	19' - 10"	Vibration	19' - 11"
			Cant	16' - 2"	Strength	16' - 2"	10' - 8"	Strength	10' - 8"

Notes
1. SS = Simply supported single span
2. Cant = Rigid end support cantilever
3. Table cells denoted 'x' indicate that a calculation for fire maximum span under fire conditions has not been performed.
4. LL = Live Load, Lr = Roof Live Load, S = Snow Load, SDL = Superimposed Dead Load
5. Self-weight of the CLT panel is included in the calculations and is in addition to the stated SDL.
6. Maximum Span can be considered the clear span between face of supports.
7. Minor direction maximum spans in excess of 12' - 0" may not be achievable due to manufacturing limitations. Please consult with the manufacturer.
8. Refer to stated limitations of use and notes on Page 2 of this document for additional information.

Figure 1. Katerra Floor Span Tables

Maximum Spans from Drawing

$L_{max\_1st} := 16.09\ ft$ $L_{max\_2nd} := 16.09\ ft$ $L_{max\_attic} := 18.56\ ft$ $L_{max\_garage} := 22\ ft$

Floor Trial Sizes

- 1st and 2nd Floor: Try K5-0690, L max = 16.67' > $L_{max\_1st} = 16.09\ ft$
- Attic Floor: Try K7-0970, L max = 20.5' > $L_{max\_attic} = 18.56\ ft$
- Garage Floor: Try K9-1120, L max = 23.0' > $L_{max\_garage} = 22\ ft$

Panel Widths and Weights

$G := 0.42$ Lamination specific gravity (Ref. 4) $mc := 15$ Estimated Moisture Content (%)

$\gamma_{panel} := 62.4\ pcf \cdot \left(\frac{G}{1 + G \cdot 0.009 \cdot mc}\right) \cdot \left(1 + \frac{mc}{100}\right) = 28.52\ pcf$ Ref. 1 section 3.1.3

$b_{690} := 6.90\ in$ $b_{970} := 9.70\ in$ $b_{1120} := 11.20\ in$

$\sigma_{self\_690} := \gamma_{panel} \cdot b_{690} = 16.4\ psf$ $\sigma_{self\_970} := \gamma_{panel} \cdot b_{970} = 23.06\ psf$ $\sigma_{self\_1120} := \gamma_{panel} \cdot b_{1120} = 26.62\ psf$

$\sigma_{collateral\_f} := 5\ psf$ $\sigma_{collateral\_r} := 10\ psf$ Estimated superimposed collateral dead load beyond self-weight.

$\sigma_{live} := 40\ psf$ Residential Live Load (Conservatively assume non-sleeping areas for all floor locations)

Check 1st floor preliminary panel selection

Use woodworks Sizer to verify panel sizes. Assume FP1-2 adjacent the stair opening will control panel selection. Calculate for continuous span.

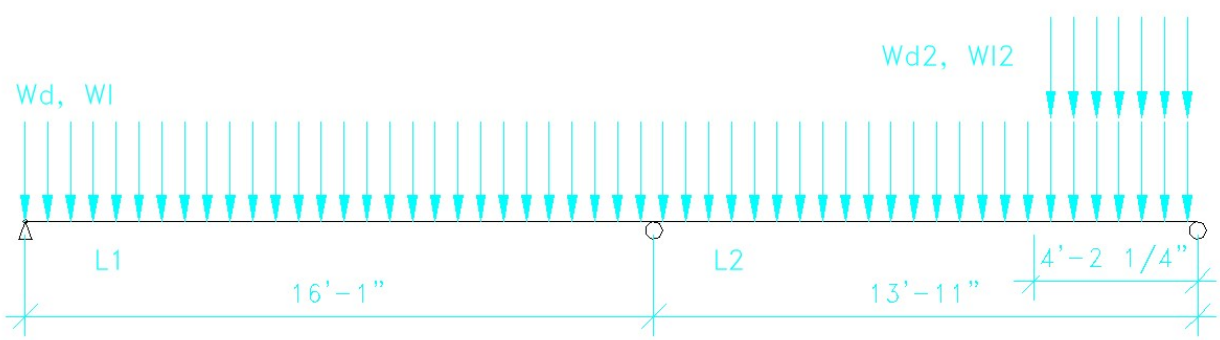


Figure 2. Loading diagram for controlling 1st floor panel strip

$\omega_d := (\sigma_{self\_690} + \sigma_{collateral\_f}) \cdot 1\ ft = 21.4\ plf$ $\omega_l := \sigma_{live} \cdot 1\ ft = 40\ plf$

$\omega_{d2} := (\sigma_{self\_690} + \sigma_{collateral\_f}) \cdot \frac{3.19}{2}\ ft = 34.13\ plf$ $\omega_{l2} := \sigma_{live} \cdot \frac{3.19}{2}\ ft = 63.8\ plf$

K5-0690 Panel OK (Ref. Appendix C for calculations) for strength and deflection, check results of wood works vibration calculations versus Ref. 7 Chapter 7 recommended simplified method, versus span table.

$EI_{eff\_0} := 367 \cdot 10^6\ lbf \cdot in^2$ $GA_{eff\_0} := 0.92 \cdot 10^6\ lbf$ MFG literature

$K_s := 11.5$ Ref. 2 Table 10.4.1.1 based on suggestion from Ref. 7, Chapter 7, Section 4.1

$L := 16.08\ ft$ Longest Span $A := b_{690} \cdot 12\ in = 82.8\ in^2$

$$EI_{app} := \frac{EI_{eff\_0}}{1 + \frac{K_s \cdot EI_{eff\_0}}{GA_{eff\_0} \cdot L^2}} = 326742473.4\ lbf \cdot in^2$$

$$L = 16.08\ ft \quad <= \quad \frac{1}{12.05} \cdot \frac{\left(\frac{EI_{app}}{lbf \cdot in^2}\right)^{0.293}}{\left(G \cdot \frac{A}{in^2}\right)^{0.122}} = 16.81 \text{ Ft.}$$ OK for vibration

Maximum Vibration Controlling Spans Comparison

- Span Tables Maximum Span = 16.67'
- Ref. 7 Chapter 7 Method = 16.81'
- WoodWorks Calculation = 16.94'

All three methods track fairly closely. The span tables appear to be the most conservtive, which would make sense since they are titled pre-analysis.

Perform hand calculations to compare with wood works results for just this first floor panel example

From Ref. 4, considering a K5-0690 Panel bending along the major axis

$F_bS_{eff\_0} := 4700\ lbf \cdot ft$ ASD Effective flatwise bending moment capacity (Ref. 8 defined)

$V_{s\_0} := 2480\ lbf$ ASD flatwise shear capacity (Ref. 8 defined)

Required NDS adjustment factors (Ref. 2)

$C_D := 1.0$ Occupancy live load duration factor, considering ASD load combination D+L

$C_M := 1.0$ Wet service factor, maximum moisture content = 15% which is less than the 16% limit prescribed in Section 10.1.5

$C_t := 1.0$ Temperature Factor, < 100 degrees (Table 2.3.3)

$C_L := 1.0$ Beam stability factor, d<b (Section 3.3.3.1)

$F_bS_{eff\_0}' := F_bS_{eff\_0} \cdot C_D \cdot C_M \cdot C_t \cdot C_L = 4700\ lbf \cdot ft$ $V_{s\_0}' := V_{s\_0} \cdot C_M \cdot C_t = 2480\ lbf$

Determine Internal Bending Moment and Shear (Model in RISA 3D)

Mechanical Properties for CLT (Ref. 4)

$EI_{eff\_0} := 367 \cdot 10^6\ lbf \cdot in^2$ Effective flatwise bending stiffness (Ref. 8)

$GA_{eff\_0} := 0.92 \cdot 10^6\ lbf$ Effective shear stiffness in flatwise bending (Ref. 8)

Calculate Ieff (only major axis layers considered)

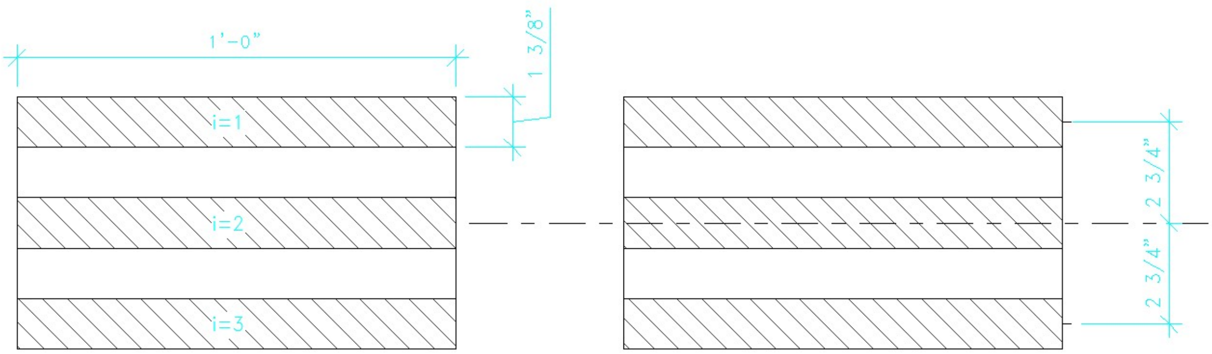


Figure 3. K5-0690 Panel contributing bending stiffness layers

$b := 12\ in$ $d := 1.375\ in$ $I_{layer} := \dfrac{b \cdot d^3}{12} = 2.6\ in^4$ $A := b \cdot d = 16.5\ in^2$

$d_c := 2.75\ in$ distance from neutral axis to centroid of layer 1 or 3

$I_{eff} := \left(I_{layer} + A \cdot d_c^{\,2}\right) + I_{layer} + \left(I_{layer} + A \cdot d_c^{\,2}\right) = 257.36\ in^4$

$E := 1.4 \cdot 10^6\ psi$ Modulus of elasticity major layer laminations

$E \cdot I_{eff} = 360305859.38\ lbf \cdot in^2$ which is roughly equivalent to the published $EI_{eff\_0} = 367000000\ lbf \cdot in^2$

The effects of shear deformation should be considered when calculation internal load effects and deformations; however, rather than having RISA calculate the shear deformation, the bending stiffness can be altered to account for shear deformation by calculating EI app (Ref.2 Section 10.4)

$EI_{app} = 326742473.4\ lbf \cdot in^2$

The material properties for the major axis lamination (Visual Grade SPF No.1/No. 2) can be used in the RISA model. Calculate an equivalent panel thickness based on EI app.

$I_{app} := \frac{EI_{app}}{E} = 233.39\ in^4$ $\quad d_{equiv} := \left(\frac{12 \cdot I_{app}}{b}\right)^{\frac{1}{3}} = 6.16\ in$

Results from RISA analysis

$M := 1.94\ kip \cdot ft$ $\quad V := 615\ lbf$

$\Delta_{D\_max} := 0.036\ in$ $\quad \Delta_{L\_max} := 0.125\ in$

Check Strength

$M = 1.94\ kip \cdot ft$ < $F_b S_{eff\_0}' = 4.7\ kip \cdot ft$; therefore OK.

$V = 615\ lbf$ < $V_{s\_0}' = 2480\ lbf$; therefore OK.

Check Deflection

* Note a pattern loading with live load applied to left span only controlled

1. Live load limit = $\frac{L}{360} = 0.54\ in$ > $\Delta_{L\_max} = 0.13\ in$; therefore OK. For comparison, Wod works calculated a live load deflection of 0.13 in., which is consistent with the RISA calculations.
2. $K_{cr} := 2.0$ Creep Factor (Ref. 2 Eq. 3.5-1)
3. Total load limit = $\frac{L}{240} = 0.8\ in$ > $K_{cr} \cdot \Delta_{D\_max} + \Delta_{L\_max} = 0.2\ in$; therefore OK. For comparison, Wood works calculated a total load deflection of 0.21 in, which is consistent with the RISA calculations.

Based on hand calculations the K5-0690 panel is OK. The mechanical properties, internal load effects, capacities and deflections calculated in Wood works track well with the hand calculated methods, therefore consider the software reliable and proceed with the use of it exclusively for the remainder of the floor and roof panels.

Check Attic floor preliminary panel selection

Use woodworks Sizer to verify panel sizes. Assume FP3-4 adjacent the stair opening will control panel selection. Calculate for continuous span.

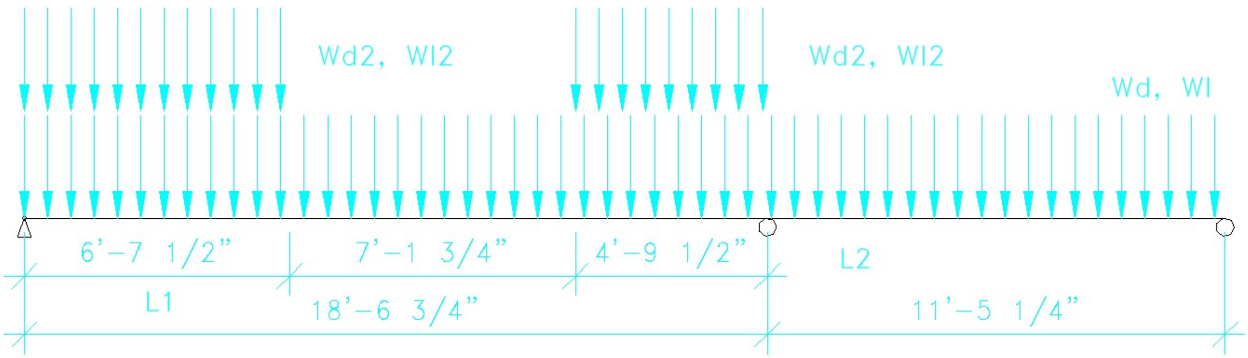


Figure 4. Loading diagram for controlling attic floor panel strip

K7-0970 Panel OK (Ref. Appendix C for calculations) for strength and deflection. Say OK to woodworks vibration calculations.

$\omega_d := (\sigma_{self\_970} + \sigma_{collateral\_f}) \cdot 1\ ft = 28.06\ plf$ $\qquad \omega_l := \sigma_{live} \cdot 1\ ft = 40\ plf$

$\omega_{d2} := (\sigma_{self\_970} + \sigma_{collateral\_f}) \cdot \dfrac{3.19}{2}\ ft = 44.75\ plf$ $\qquad \omega_{l2} := \sigma_{live} \cdot \dfrac{3.19}{2}\ ft = 63.8\ plf$

Check to determine if the K5-0690 is adequate for use on the second floor. Assume panel FP2-3 controls. Note that the attic panel was sized first because the attic bears on an interior wall which in turn bears within the span of the second floor panel (represented by P1 in Figure 5).

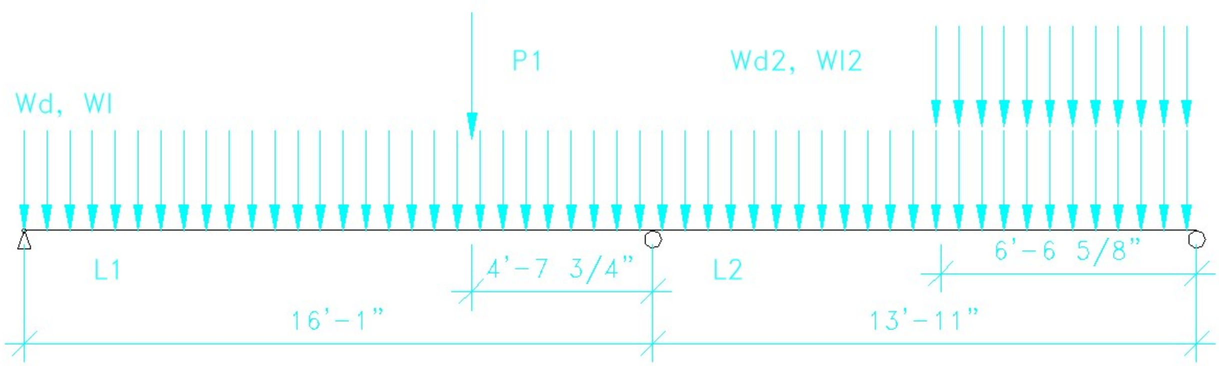


Figure 5. Loading diagram for controlling 2nd floor panel strip

$L_1 := L = 16.08\ ft$ $\qquad L_2 := 13.92\ ft$

$\omega_d := \left(\sigma_{self\_690} + \sigma_{collateral\_f}\right) \cdot 1\ ft = 21.4\ plf$ $\qquad \omega_l := \sigma_{live} \cdot 1\ ft = 40\ plf$

$\omega_{d2} := \left(\sigma_{self\_690} + \sigma_{collateral\_f}\right) \cdot \dfrac{3.19}{2}\ ft = 34.13\ plf$ $\qquad \omega_{l2} := \sigma_{live} \cdot \dfrac{3.19}{2}\ ft = 63.8\ plf$

Since Nordic walls were utilized for the exterior walls, utilize properties for the Nordic 89-3s panel for interior bearing walls.

$\sigma_{self\_350} := 9.38\ psf$ $\qquad \sigma_{collateral\_IW} := 6\ psf$

Interior wall collateral loading, assume 2 layers of gypsum and wood furring.

$h_{int\_wall} := 8\ ft$

$$P_{1\_D} := \left(\left(\sigma_{self\_970} + \sigma_{collateral\_f}\right) \cdot \left(\frac{L_1}{2} + \frac{L_2}{2}\right) \cdot 1\ ft\right) + \left(\sigma_{self\_350} + \sigma_{collateral\_IW}\right) \cdot h_{int\_wall} \cdot 1\ ft = 543.87\ lbf$$

$$P_{1\_L} := \omega_l \cdot \left(\frac{L_1}{2} + \frac{L_2}{2}\right) = 600\ lbf$$

K5-0690 Panel OK (Ref. App A for calculations) for strength, deflection and vibration.

Check to determine if the K9-1120 is adequate for use on the above garage floor.

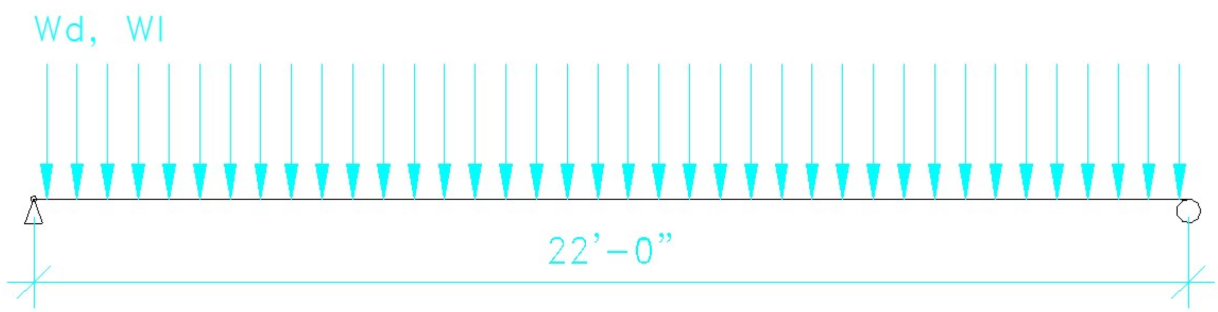


Figure 6. Loading diagram for controlling Garage panel strip

$\omega_d := \left(\sigma_{self\_1120} + \sigma_{collateral\_f}\right) \cdot 1\ ft = 31.62\ plf$ $\qquad \omega_l := \sigma_{live} \cdot 1\ ft = 40\ plf$

K9-1120 Panel OK (Ref. Appendix C for calculations) for strength, deflection and vibration.

Size Roof Panels

ROOF | S = 20psf | SDL=15psf									
CLT Layup Designation		CLT Thickness (in)	Span Type	Major Direction			Minor Direction		
				Max Span	Controlling Criteria	Max Span 1hr Fire	Max Span	Controlling Criteria	Max Span 1hr Fire
3-Ply	K3-0320	3.24	SS	16′ - 6″	Strength	x	4′ - 7″	Strength	x
			Cant	8′ - 3″	Strength	x	2′ - 3″	Strength	x
	K3-0350	3.54	SS	17′ - 8″	Strength	x	5′ - 10″	Strength	x
			Cant	8′ - 10″	Strength	x	2′ - 11″	Strength	x
	K3-0380	3.84	SS	19′ - 4″	Strength	x	4′ - 6″	Strength	x
			Cant	9′ - 8″	Strength	x	2′ - 3″	Strength	x
	K3-0410	4.14	SS	20′ - 6″	Strength	x	5′ - 9″	Strength	x
			Cant	10′ - 3″	Strength	x	2′ - 10″	Strength	x
5-Ply	K5-0540	5.40	SS	23′ - 7″	Strength	23′ - 7″	12′ - 9″	Strength	7′ - 6″
			Cant	11′ - 10″	Strength	11′ - 10″	6′ - 5″	Strength	3′ - 9″
	K5-0600	6.00	SS	25′ - 1″	Strength	25′ - 1″	15′ - 0″	Strength	13′ - 2″
			Cant	12′ - 6″	Strength	12′ - 6″	7′ - 6″	Strength	6′ - 7″
	K5-0630	6.30	SS	27′ - 7″	Strength	27′ - 7″	13′ - 6″	Strength	13′ - 6″
			Cant	13′ - 10″	Strength	13′ - 10″	6′ - 9″	Strength	6′ - 9″
	K5-0690	6.90	SS	29′ - 1″	Strength	29′ - 1″	15′ - 9″	Strength	15′ - 9″
			Cant	14′ - 7″	Strength	14′ - 7″	7′ - 10″	Strength	7′ - 10″
7-Ply	K7-0970	9.66	SS	36′ - 5″	Strength	36′ - 5″	22′ - 6″	Strength	22′ - 6″
			Cant	18′ - 3″	Strength	18′ - 3″	11′ - 3″	Strength	11′ - 3″
9-Ply	K9-1120	11.22	SS	41′ - 3″	Strength	41′ - 3″	24′ - 6″	Strength	24′ - 6″
			Cant	20′ - 7″	Strength	20′ - 7″	12′ - 3″	Strength	12′ - 3″
	K9-1240	12.42	SS	43′ - 0″	Strength	43′ - 0″	28′ - 5″	Strength	28′ - 5″
			Cant	21′ - 6″	Strength	21′ - 6″	14′ - 2″	Strength	14′ - 2″

Figure 7. Katerra pre-analysis span tables for roofs (Ref. 3)

Maximum Roof Spans from Drawing

$L_{max\_main} := 17.97\ ft$ $\quad L_{max\_garage} := 16.26\ ft$

Roof Panel Trial Sizes (From Figure 7)

- Main: Try K3-0380, L max = 19.33' > $L_{max\_main} = 17.97\ ft$
- Garage: Try K3-0350, L max = 17.67' > $L_{max\_garage} = 16.26\ ft$

Size Garage Panels First

Model in RISA 3D, Wood works can design roof panels but the loading and bearing options are limited. Due to the wind loading and lack of beam at the ridge, RISA 3D is a better tool. Panels RP14 and RP-11 would likely control design due potential drifted snow loads in this region.

K3-0350 Panel Properties (Ref. 4)

$b_{350} := 3.54\ in$ $\sigma_{self\_350} := \gamma_{panel} \cdot b_{350} = 8.41\ psf$

$F_b S_{eff\_0} := 1460\ lbf \cdot ft$ $V_{s\_0} := 1270\ lbf$ $EI_{eff\_0} := 59 \cdot 10^6\ lbf \cdot in^2$ $GA_{eff\_0} := 0.37 \cdot 10^6\ lbf$

$E_0 := 1.4 \cdot 10^6\ psi$

Capacities

$C_D := 1.15$ Load duration factor based on snow

$F_b S_{eff\_0}' := F_b S_{eff\_0} \cdot C_D \cdot C_M \cdot C_t \cdot C_L = 1679\ lbf \cdot ft$ $V_{s\_0}' := V_{s\_0} \cdot C_M \cdot C_t = 1270\ lbf$

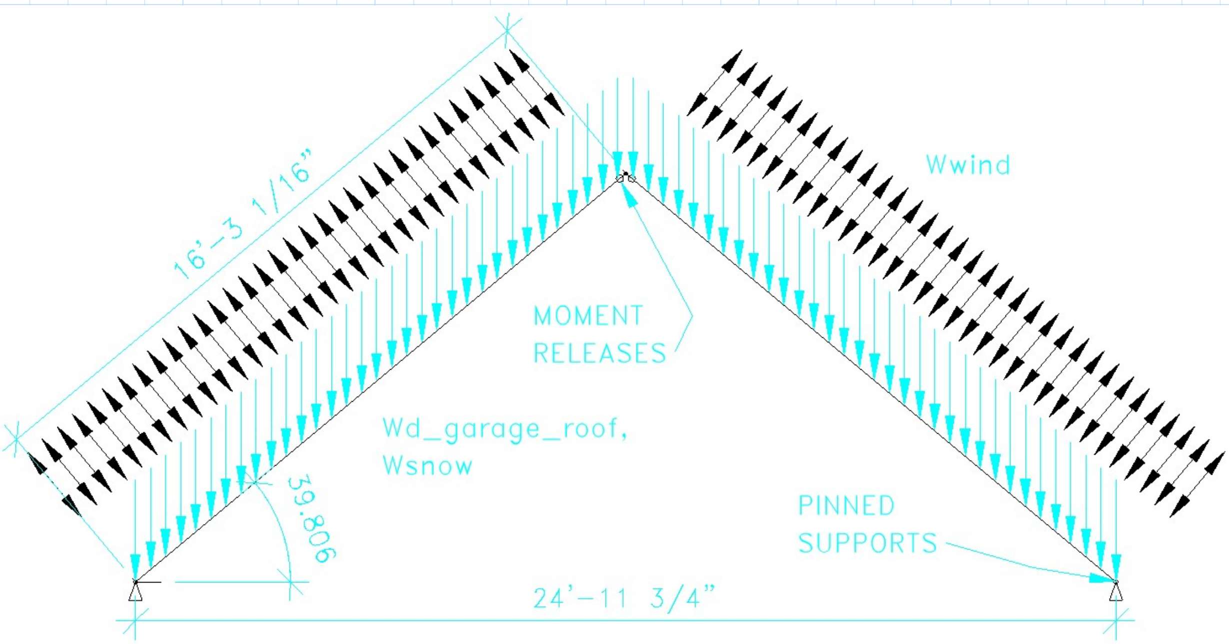


Figure 8. Garage roof free-body diagram

Apparent Stiffness

$L := 16.26\ ft$ Roof Span $K_s := 11.5$ $EI_{app} := \dfrac{EI_{eff\_0}}{1 + \dfrac{K_s \cdot EI_{eff\_0}}{GA_{eff\_0} \cdot L^2}} = 56288770.29\ lbf \cdot in^2$

Determine equivalent panel thickness for RISA model

$I_{app} := \frac{EI_{app}}{E_0} = 40.21\ in^4$ $\quad d_{equiv} := \left(\frac{12 \cdot I_{app}}{b}\right)^{\frac{1}{3}} = 3.43\ in$

Loading

$\sigma_{collateral\_r} = 10\ psf$ $\quad \omega_{d\_garage} := (\sigma_{self\_350} + \sigma_{collateral\_r}) \cdot 1\ ft = 18.41\ plf$

$\sigma_{snow} := 47.5\ psf$ Drifted snow load maginitude (See Appendix B for calculations)

$\omega_{snow} := \sigma_{snow} \cdot 1\ ft = 47.5\ plf$

Wind Load Based on Components and Cladding Method (See Appendix B for calculations)

$W_{panel} := 6.75\ ft$ $\quad A_{panel} := W_{panel} \cdot L = 109.76\ ft^2$ > 100 ft^2

Zone 2 and 3 applies to roof edges. Zone 2 and 3 wind magnitude is the same in Tedds calculation table.

$\sigma_{wind} := -23.8\ psf$ $\quad \omega_{wind} := \sigma_{wind} \cdot 1\ ft = -23.8\ plf$

Results From RISA

$M := 1.674\ kip \cdot ft$ $\quad V := 412\ lbf$

$\Delta_{D\_max} := 0.394\ in$ $\quad \Delta_{S\_max} := 1.018\ in$

Check Deflection

1. Live load limit = $\frac{L}{240} = 0.81\ in$ > $\Delta_{S\_max} = 1.02\ in$; therefore NG.
2. $K_{cr} := 2.0$ Creep Factor (Ref. 2 Eq. 3.5-1)
3. Total load limit = $\frac{L}{180} = 1.08\ in$ > $K_{cr} \cdot \Delta_{D\_max} + \Delta_{S\_max} = 1.81\ in$; therefore NG.

Upsize panel to K3-0380

New Panel Properties (K3-0380)

$b_{380} := 3.84\ in$ $\sigma_{self\_380} := \gamma_{panel} \cdot b_{380} = 9.13\ psf$

$F_bS_{eff\_0} := 1790\ lbf \cdot ft$ $V_{s\_0} := 1380\ lbf$ $EI_{eff\_0} := 78 \cdot 10^6\ lbf \cdot in^2$ $GA_{eff\_0} := 0.45 \cdot 10^6\ lbf$

Capacities

$F_bS_{eff\_0}' := F_bS_{eff\_0} \cdot C_D \cdot C_M \cdot C_t \cdot C_L = 2058.5\ lbf \cdot ft$ $V_{s\_0}' := V_{s\_0} \cdot C_M \cdot C_t = 1380\ lbf$

$$EI_{app} := \frac{EI_{eff\_0}}{1 + \dfrac{K_s \cdot EI_{eff\_0}}{GA_{eff\_0} \cdot L^2}} = 74119320.44\ lbf \cdot in^2 \qquad I_{app} := \frac{EI_{app}}{E_0} = 52.94\ in^4$$

$$d_{equiv} := \left(\frac{12 \cdot I_{app}}{b}\right)^{\frac{1}{3}} = 3.75\ in \qquad \omega_{d\_garage} := \left(\sigma_{self\_380} + \sigma_{collateral\_r}\right) \cdot 1\ ft = 19.13\ plf$$

New Analysis Results From RISA

$\Delta_{D\_max} := 0.313\ in$ $\Delta_{S\_max} := 0.779\ in$

Check Deflection

1. Live load limit = $\dfrac{L}{240} = 0.81\ in$ > $\Delta_{S\_max} = 0.78\ in$; therefore NG.
2. $K_{cr} := 2.0$ Creep Factor (Ref. 2 Eq. 3.5-1)
3. Total load limit = $\dfrac{L}{180} = 1.08\ in$ > $K_{cr} \cdot \Delta_{D\_max} + \Delta_{S\_max} = 1.41\ in$; therefore NG.

Still no good. Upsize one more time.

New Panel Properties (K3-0410)

$b_{410} := 4.14\ in$ $\sigma_{self\_410} := \gamma_{panel} \cdot b_{410} = 9.84\ psf$

$F_bS_{eff\_0} := 2050\ lbf \cdot ft$ $V_{s\_0} := 1490\ lbf$ $EI_{eff\_0} := 96 \cdot 10^6\ lbf \cdot in^2$ $GA_{eff\_0} := 0.46 \cdot 10^6\ lbf$

Capacities

$F_bS_{eff\_0}' := F_bS_{eff\_0} \cdot C_D \cdot C_M \cdot C_t \cdot C_L = 2357.5\ lbf \cdot ft$ $V_{s\_0}' := V_{s\_0} \cdot C_M \cdot C_t = 1490\ lbf$

$$EI_{app} := \frac{EI_{eff\_0}}{1 + \dfrac{K_s \cdot EI_{eff\_0}}{GA_{eff\_0} \cdot L^2}} = 90307149.22\ lbf \cdot in^2 \qquad I_{app} := \frac{EI_{app}}{E_0} = 64.51\ in^4$$

$$d_{equiv} := \left(\frac{12 \cdot I_{app}}{b}\right)^{\frac{1}{3}} = 4.01\ in \qquad \omega_{d\_garage} := \left(\sigma_{self\_410} + \sigma_{collateral\_r}\right) \cdot 1\ ft = 19.84\ plf$$

New Analysis Results From RISA

$M := 1.70\ kip \cdot ft$ $V := 419\ lbf$

$\Delta_{D\_max} := 0.266\ in$ $\Delta_{S\_max} := 0.637\ in$

Check Deflection

1. Live load limit = $\dfrac{L}{240} = 0.81\ in$ > $\Delta_{S\_max} = 0.64\ in$; therefore OK.
2. $K_{cr} := 2.0$ Creep Factor (Ref. 2 Eq. 3.5-1)
3. Total load limit = $\dfrac{L}{180} = 1.08\ in$ > $K_{cr} \cdot \Delta_{D\_max} + \Delta_{S\_max} = 1.17\ in$; therefore NG.

Say OK for K3-0410 Panel. The total load deflection is a bit higher than the long term limit; however, the panel edge will have support on the wall-roof panel connection and also some load sharing should occur between this panel and the adjacent panel which is more lightly loaded due to non-drift conditions.

$M = 1.7\ kip \cdot ft$ < $F_bS_{eff\_0}' = 2.358\ kip \cdot ft$; therefore OK.

$V = 419\ lbf$ < $V_{s\_0}' = 1490\ lbf$; therefore OK.

Size Main Roof Panel (Try K3-0380 panels based on span tables)

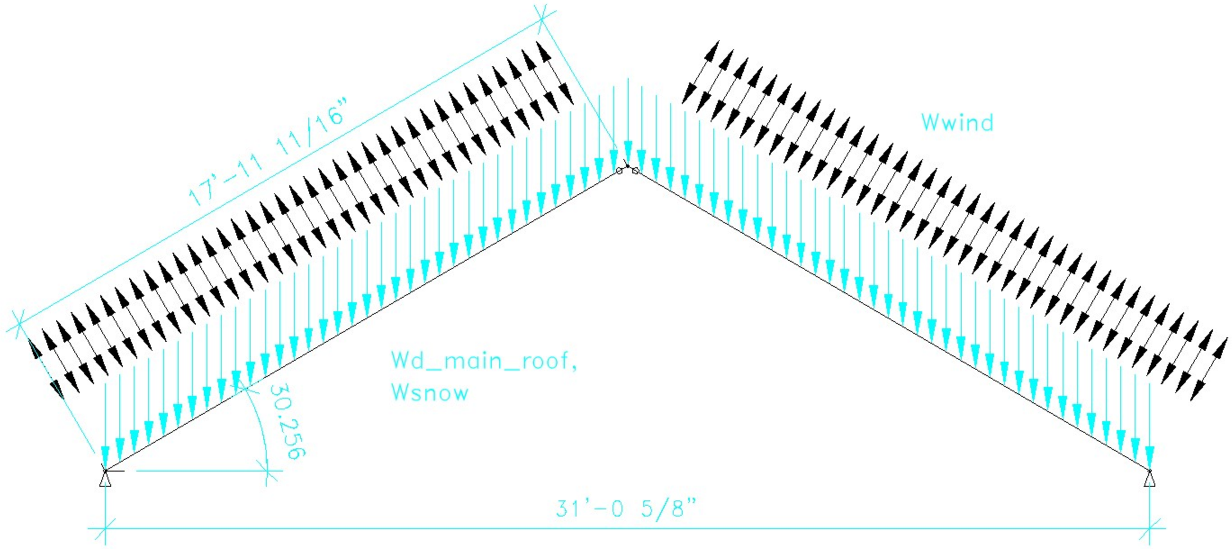


Figure 9. Main roof free-body diagram

Panel Properties (K3-0380)

$b_{380} := 3.84\ in$ $\sigma_{self\_380} := \gamma_{panel} \cdot b_{380} = 9.13\ psf$ $L := 17.97\ ft$

$F_bS_{eff\_0} := 1790\ lbf \cdot ft$ $V_{s\_0} := 1380\ lbf$ $EI_{eff\_0} := 78 \cdot 10^6\ lbf \cdot in^2$ $GA_{eff\_0} := 0.45 \cdot 10^6\ lbf$

Capacities

$F_bS_{eff\_0}' := F_bS_{eff\_0} \cdot C_D \cdot C_M \cdot C_t \cdot C_L = 2058.5\ lbf \cdot ft$ $V_{s\_0}' := V_{s\_0} \cdot C_M \cdot C_t = 1380\ lbf$

$$EI_{app} := \frac{EI_{eff\_0}}{1 + \frac{K_s \cdot EI_{eff\_0}}{GA_{eff\_0} \cdot L^2}} = 74793826.39\ lbf \cdot in^2 \qquad I_{app} := \frac{EI_{app}}{E_0} = 53.42\ in^4$$

$$d_{equiv} := \left(\frac{12 \cdot I_{app}}{b}\right)^{\frac{1}{3}} = 3.77\ in \qquad \omega_{d\_garage} := \left(\sigma_{self\_380} + \sigma_{collateral\_r}\right) \cdot 1\ ft = 19.13\ plf$$

Loading

$\sigma_{snow} := 25.0\ psf$ Unbalanced Snow Load (See Appendix B for calculations)

Note: Some means of load sharing mechanism should be put in place in case an unbalanced condition would occur. In conventional joist framing a collar tie would provide this functionality.

$\omega_{snow} := \sigma_{snow} \cdot 1\ ft = 25\ plf$

Wind Load Based on Components and Cladding Method (See Appendix B for calculations)

Zone 2 and 3 applies to roof edges. Zone 2 and 3 wind magnitude is the same in Tedds calculation table.

$\sigma_{wind} := -23.8\ psf$ $\omega_{wind} := \sigma_{wind} \cdot 1\ ft = -23.8\ plf$

New Analysis Results From RISA

$M := 1.533$ $V := 341\ lbf$

$\Delta_{D\_max} := 0.514\ in$ $\Delta_{S\_max} := 0.674\ in$

Check Deflection

Live load limit = $\frac{L}{240} = 0.9\ in$ > $\Delta_{S\_max} = 0.67\ in$; therefore NG.

1. $K_{cr} := 2.0$ Creep Factor (Ref. 2 Eq. 3.5-1)
2. Total load limit = $\frac{L}{180} = 1.2\ in$ > $K_{cr} \cdot \Delta_{D\_max} + \Delta_{S\_max} = 1.7\ in$; therefore NG.

No good try the K3-0410 Panels

New Panel Properties (K3-0410)

$b_{410} := 4.14\ in$ $\sigma_{self\_410} := \gamma_{panel} \cdot b_{410} = 9.84\ psf$

$F_b S_{eff\_0} := 2050\ lbf \cdot ft$ $V_{s\_0} := 1490\ lbf$ $EI_{eff\_0} := 96 \cdot 10^6\ lbf \cdot in^2$ $GA_{eff\_0} := 0.46 \cdot 10^6\ lbf$

Capacities

$F_b S_{eff\_0}' := F_b S_{eff\_0} \cdot C_D \cdot C_M \cdot C_t \cdot C_L = 2357.5\ lbf \cdot ft$ $V_{s\_0}' := V_{s\_0} \cdot C_M \cdot C_t = 1490\ lbf$

$$EI_{app} := \frac{EI_{eff\_0}}{1 + \frac{K_s \cdot EI_{eff\_0}}{GA_{eff\_0} \cdot L^2}} = 91288402.29\ lbf \cdot in^2 \qquad I_{app} := \frac{EI_{app}}{E_0} = 65.21\ in^4$$

$$d_{equiv} := \left(\frac{12 \cdot I_{app}}{b}\right)^{\frac{1}{3}} = 4.02\ in \qquad \omega_{d\_garage} := \left(\sigma_{self\_410} + \sigma_{collateral\_r}\right) \cdot 1\ ft = 19.84\ plf$$

New Analysis Results From RISA

$M := 1.56\ kip \cdot ft$ $V := 347\ lbf$

$\Delta_{D\_max} := 0.314\ in$ $\Delta_{S\_max} := 0.556\ in$

Check Deflection

* Note a pattern loading with snow load applied to left span only controlled

1. Live load limit = $\frac{L}{240} = 0.9\ in$ > $\Delta_{S\_max} = 0.56\ in$; therefore OK.
2. $K_{cr} := 2.0$ Creep Factor (Ref. 2 Eq. 3.5-1)
3. Total load limit = $\frac{L}{180} = 1.2\ in$ > $K_{cr} \cdot \Delta_{D\_max} + \Delta_{S\_max} = 1.18\ in$; therefore OK.

$M = 1.56\ kip \cdot ft$ < $F_b S_{eff\_0}' = 2.358\ kip \cdot ft$; therefore OK.

$V = 347\ lbf$ < $V_{s\_0}' = 1490\ lbf$; therefore OK.

K3-0410 Panel OK. This panel can be used on both the garage and roof.

CLT Lateral System Review

References:

1. 2018 NDS Supplement
2. 2018 NDS
3. Nordic X-Lam Technical Guide NS-GT6-ASD; 2020-08-13
4. 2021 SDPWS
5. Cross-Laminated Timber Structural Design Volume 2, pro:Holz 2017
6. Cross-Laminated Timber Horizontal Diaphragm Design Example (White Paper); Spickler
7. Structural SYSTEMS; Cross-Laminated Timber Diaphragms, DeStefano, P.E.
8. Katerra Product Definitions Technical Specifications (Updated January 2020)
9. PRG 320-2019
10. 2018 IBC
11. Analysis of irregular shaped structures: Diaphragms and Shearwalls; Malone
12. Simpson Strong-Tie Wood Construction Connectors Catalog C-C-2019
13. AISC Steel Construction Manual; 14th ed.
14. CLT Handbook
15. Determintation of Seismic Performance Factors for CLT Shear Wall Systems; Amini, WCTE 2016
16. ASCE 7-10
17. Design of Wood Structures ASD (5th ed.); Breyer
18. The swedish CLT handbook
19. CLT Connection Design Guide, MyTiCon Timber Connectors 2019
20. AWC TR12
21. Connectors and Fasteners for Mass Timber Construction C-C-MASSTIMBER20, Simpson Strong-Tie
22. MTC Solutions Structural Screw Design Guide
23. 2018 Manual for Engineered Wood Construction

Lateral system design procedure not well documented at this point and largely based on basic principles of engineering mechanics. Ref. 4 has some design guidance. The following are excerts from Ref 4.: Wind design based on linear elastic structural response.

1. 4.1.2 Design of shear walls and diaphragms in accordance with 4.5 and 4.6. Approved alternate procedures that are in accordance with principles of engineering mechanics are permitted.
2. 4.1.4 For wind design of diaphragms and shear walls the ASD allowable shear capacity shall be determined by dividing the nominal shear capacity in 4.1.2 by an ASD reduction factor of 2.0.
3. 4.5, 4.6 states requirements for CLT diaphragms and shear walls respectively.
4. Appendix B states mandatory requirments for CLT shear walls.

Roof Diaphragm Design (Ref. Appendix A Drawings)

Wind perpindicular to gable end: Note that only a small amount of windload is present at the roof membrane. The majority of the lateral loading will be resisted by the attic floor; however, a few connections should be considered for this loading condition:

1. Chord splice RC-1.
2. Panel-Panel splice.
3. Peak connection

From MWFRS load calculations (Ref. Appendix B)

Consider load case 4 and combine windward and leeward pressures.

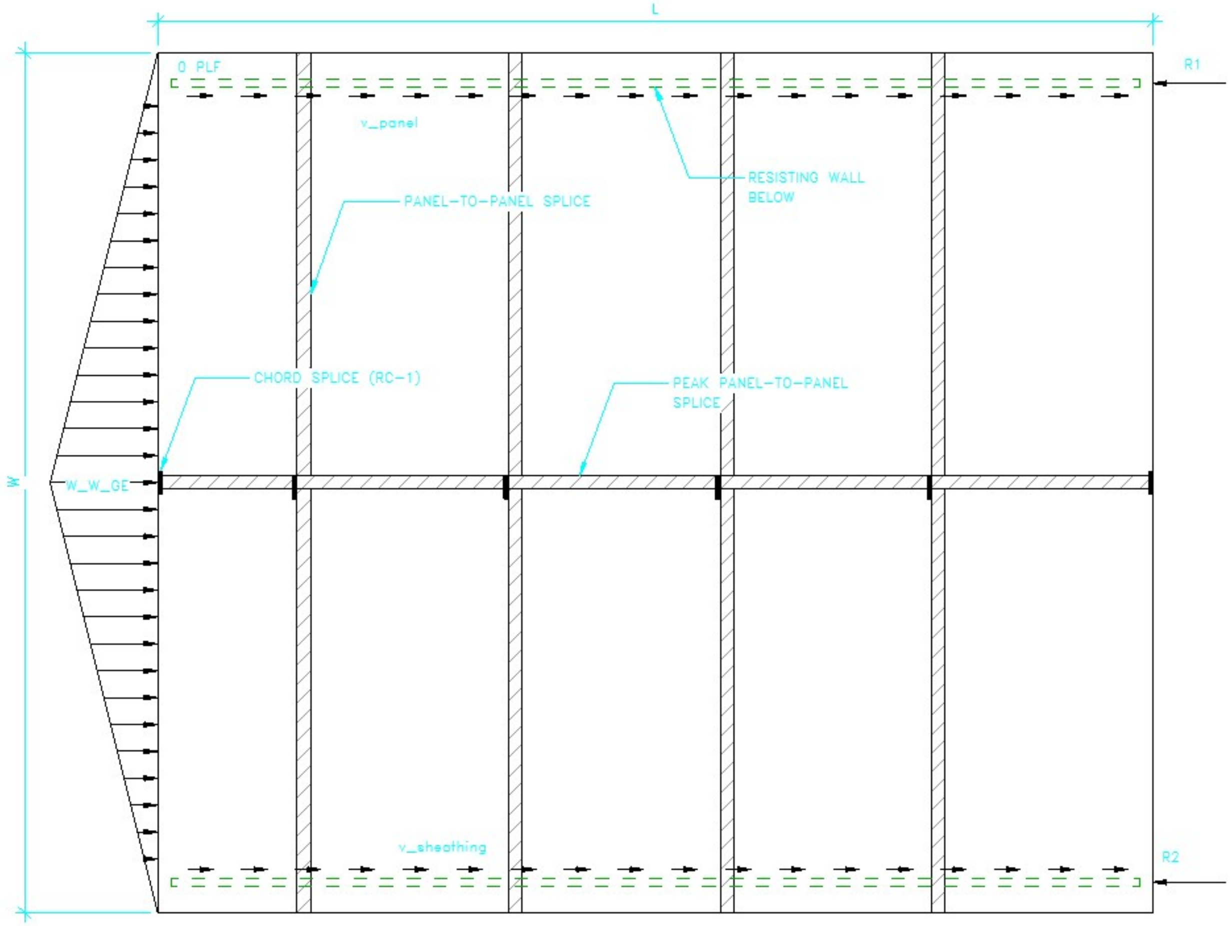


Figure 1. Roof diaphragm diagram

$\sigma_{W\_GE} := 16.20\ psf + 3.81\ psf = 20.01\ psf$

$\omega_{W\_GE} := \sigma_{W\_GE} \cdot 4.62\ ft = 92.45\ plf$

$W := 30\ ft$ $L := 36\ ft$ $LF_{wind} := 0.6$ Load factor for wind

$R_1 := LF_{wind} \cdot \left(0.5 \left(\frac{W}{2}\right) \cdot \omega_{W\_GE}\right) = 416.01\ lbf$ $R_2 := R_1$

$v_{panel} := \frac{R_1}{L} = 11.56\ plf$

CLT Panel Shear Strength

Design values for the roof panel (K3-0410) from Ref. 8

$b_{410} = 4.14\ in$ $\qquad F_{v\_0} := 190\ psi$ Minimum edgewise shear stress from Table 3 Ref. 8

The 2.0 reduction factor required in Ref. 4, 4.1.4 is assumed to be accounted for in the table values. From Ref. 9, 8.5.6.2 edgewise shear capacity published values for Fv include an adjustment factor of 2.1.

NDS adjustment Factors (Ref. 2)

$C_d := 1.6$ Load Duration Factor Wind $\qquad C_m := 1.0 \qquad C_t := 1.0$

$\Omega_w := 1.5$ Overstrength factor for wind (Ref. 4, 4.5.4.3.1)

Design Strength

$v_r := \dfrac{F_{v\_0} \cdot b_{410} \cdot C_d \cdot C_m \cdot C_t}{\Omega_w} = 10068.48\ plf$ >>>> $v_{panel} = 11.56\ plf$; therefore OK

Size chord splice RC-1

From Risa analysis:

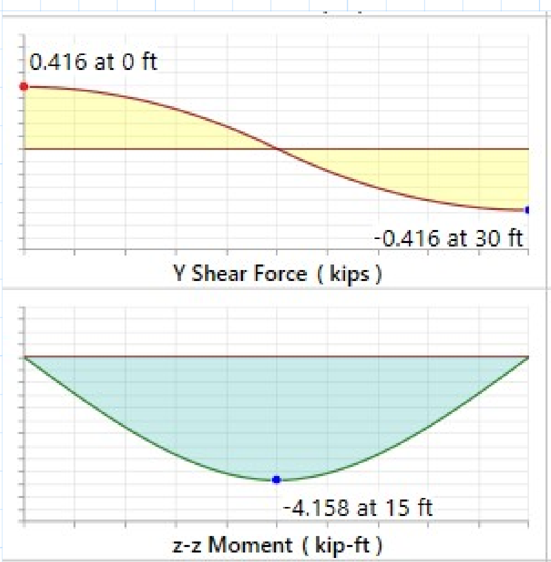


Figure 2. Shear and moment diagram for roof diaphragm

$M_{max} := 4.16\ kip \cdot ft$

$T_{RC1} := \dfrac{M_{max}}{L} = 115.56\ lbf$

This is a very low force. The roof splice can be designed to accomodate this force; however, there is no ridge beam in this design and it would be prudent to add hardware to decrease the chance of horizontal movement at the ridge panel joints.

- Select Simpson Strong Tie LSTA9 strap across the exterior side of the ridge. Install as shown on roof loading plan.
- $P_r := 635\ lbf$ Ref. 12 for SPF considering wind loading.
- $P_r = 635\ lbf > T_{RC1} = 115.56\ lbf$; OK
- Use (8) 0.148x2 1/2 nails.

Roof Attachment (See Figure 3)

Utilize bent plates to act as erection aids as well as permanent connections. The intent is for the CLT fabricator to cut blocks from the scraps left over from the floor panels. The blocks are fastened to the attic floor with structural screws at intervals to act as stops. Bent steel plates will be attached to the base and peak as shown on one panel. This first panel is craned into position and the base bent plate rests against the 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. Make all connections.

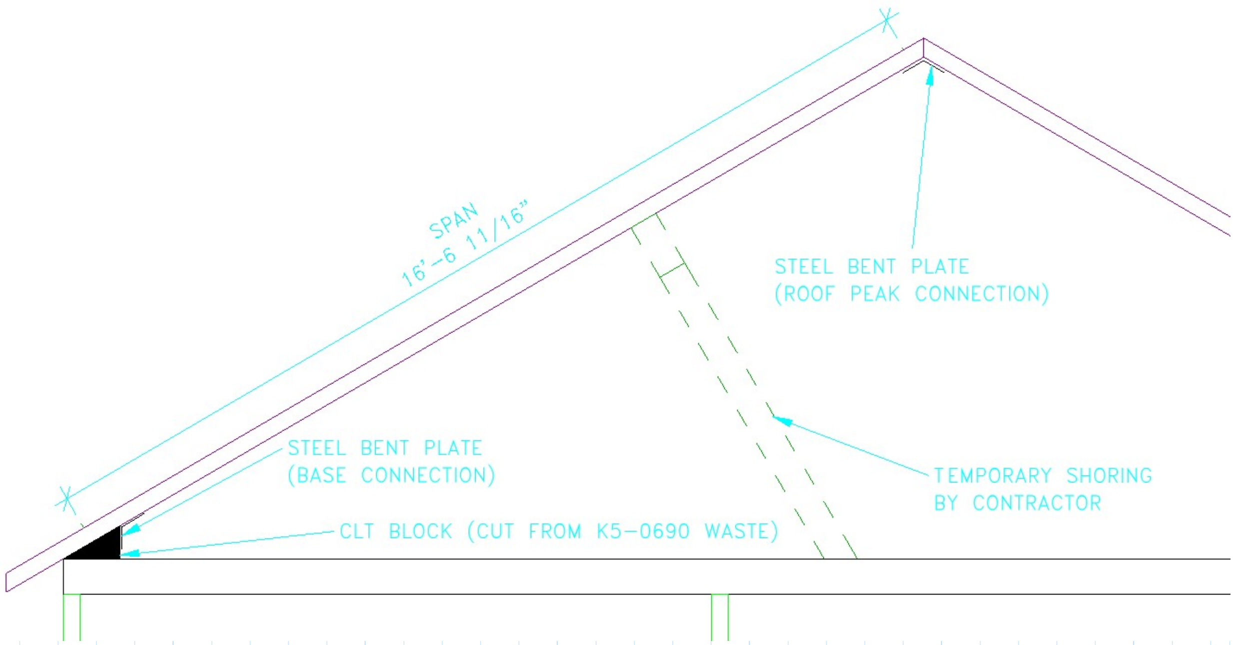


Figure 3. Roof panel connection strategy

Design bent plate for roof peak (See Figure 4)

Loading

$\sigma_{self\_410} = 9.84\ psf$ $\quad \sigma_{live} := 20\ psf$

Consider components and cladding wind loading for connections (See Appendix B)

$W_{panel\_min} := 5.5\ ft$ $\quad L_{span} := 16.56\ ft$ $\quad Tributary := \frac{L_{span}}{2} \cdot W_{panel\_min} = 45.54\ ft^2$

Therefore considering a tributary of approximately 50 ft2 and zone 2,3 the wind loading is:

$\sigma_{wind\_positive} := 20.3\ psf$ $\quad \sigma_{wind\_negative} := 25\ psf$

Design a fixed length connector based on the typical roof panel width:

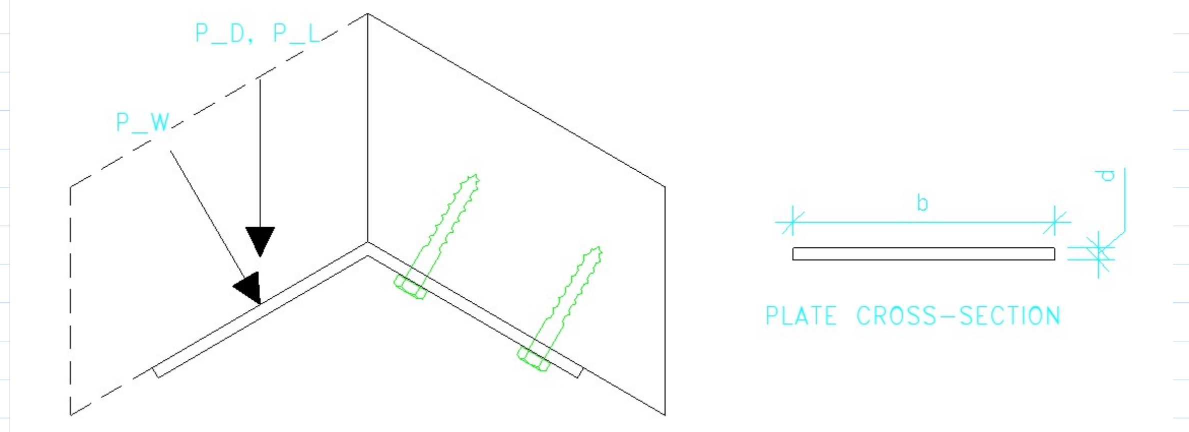


Figure 4. Roof peak connector sketch

$W_{typ} := 7.88\ ft$ Typical roof panel length

$P_D := \sigma_{self\_410} \cdot \frac{L_{span}}{2} \cdot W_{typ} = 642.03\ lbf$ $P_L := \sigma_{live} \cdot \frac{L_{span}}{2} \cdot W_{typ} = 1304.93\ lbf$

$P_{W\_Pos} := \sigma_{wind\_positive} \cdot \frac{L_{span}}{2} \cdot W_{typ} = 1324.5\ lbf$ $P_{W\_Neg} := \sigma_{wind\_negative} \cdot \frac{L_{span}}{2} \cdot W_{typ} = 1631.16\ lbf$

Review load case 1, considering wind in the positive direction

Design plate first (Ref Figure 5). Calculate loads perpidicular and parallel to leg 1

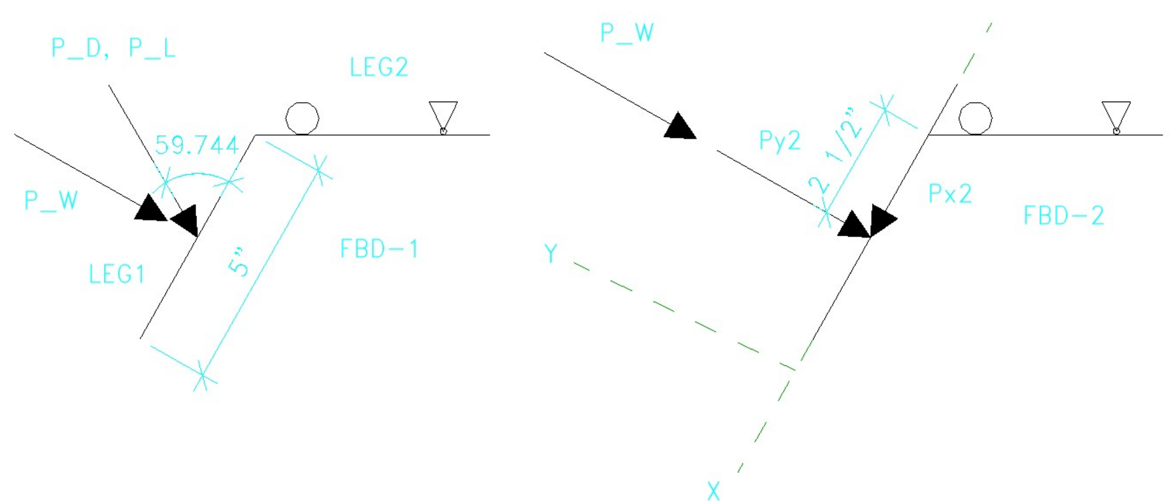


Figure 5. Free body diagrams of bent plate for steel design

$\phi := 60\ deg$

$P_{y\_2\_D} := \sin(\phi) \cdot P_D = 556.01\ lbf$ $\quad P_{x\_2\_D} := \cos(\phi) \cdot P_D = 321.02\ lbf$

$P_{y\_2\_L} := \sin(\phi) \cdot P_L = 1130.1\ lbf$ $\quad P_{x\_2\_L} := \cos(\phi) \cdot P_L = 652.46\ lbf$ $\quad P_{W\_Pos} = 1324.5\ lbf$

Design plate based on leg 1 moment and shear capacity

$M_D := P_{y\_2\_D} \cdot 2.5\ in = 0.116\ kip \cdot ft$ $\quad M_L := P_{y\_2\_L} \cdot 2.5\ in = 0.235\ kip \cdot ft$

$M_W := P_{W\_Pos} \cdot 2.5\ in = 0.276\ kip \cdot ft$

$M_{u\_1} := 1.2\ M_D + 1.6\ M_L + 0.5\ M_W = 0.654\ kip \cdot ft$

$M_{u\_2} := 1.2\ M_D + 1.0 \cdot M_W + 0.5\ M_L = 0.533\ kip \cdot ft$

$M_u := \max\left(M_{u\_1}, M_{u\_2}\right) = 0.654\ kip \cdot ft$

$F_y := 36\ ksi$ Say $d := 0.25\ in$ $\quad \phi_b := 0.9$ $\quad b_{min} := \dfrac{\phi_b \cdot 4 \cdot M_u}{F_y \cdot d^2} = 12.551\ in$

Check Shear (Ref. 13 J4.2)

$A_{gv} := b_{min} \cdot d = 3.14\ in^2$ $\quad \phi_v := 1.0$

$V := 1.2\ P_{y\_2\_D} + 1.6\ P_{y\_2\_L} + 0.5\ P_{W\_Pos} = 3137.63\ lbf$

$\phi R_n := \phi_v \cdot 0.60 \cdot F_y \cdot A_{gv} = 67.77\ kip > \ V = 3137.63\ lbf$; therefore, OK

Design lag connection (Ref. Figure 6)

Consider load combination D+0.75Lr+0.75(0.60)W

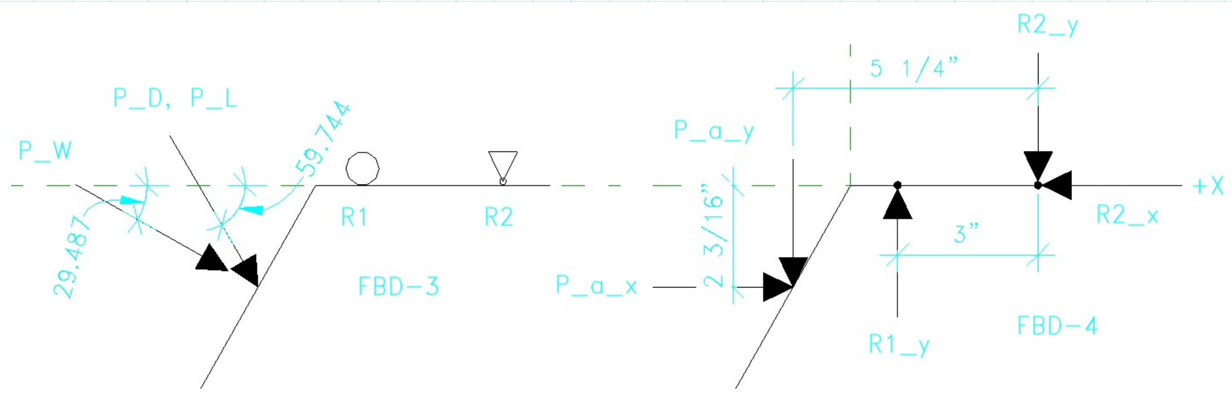


Figure 6. Free body diagrams of bent plate for lag design

$\theta_1 := 29.487\ deg \qquad \theta_2 := 59.744\ deg$

$P_{a\_x} := \cos(\theta_1) \cdot P_D + 0.75 \cdot \cos(\theta_1) \cdot P_L + 0.75 \cdot 0.60 \cdot \cos(\theta_2) \cdot P_{W\_Pos} = 1711.11\ lbf$

$P_{a\_y} := \sin(\theta_1) \cdot P_D + 0.75 \cdot \sin(\theta_1) \cdot P_L + 0.75 \cdot 0.60 \cdot \sin(\theta_2) \cdot P_{W\_Pos} = 1312.6\ lbf$

Determine Reactions (Lag Bolt Forces)

Sum of the forces in the X Direction = 0 lbf

$R_{2\_x} := P_{a\_y} = 1312.6\ lbf$

Summation of the moments about R2 (counterclockwise positive)

$$R_{1\_y} := \frac{P_{a\_y} \cdot 5.25\ in + P_{a\_x} \cdot 2.19\ in}{3\ in} = 3546.16\ lbf$$

Summation of forces in the Y direction = 0 lbf

$R_{2\_y} := -P_{a\_y} + R_{1\_y} = 2233.56\ lbf$

Size lag screw

Base on the withdrawal load R1_y and half of the shear load R2_x

$P := R_{1\_y} = 3546.16\ lbf \qquad V := 0.5 \cdot R_{2\_x} = 656.3\ lbf \qquad \alpha := \operatorname{atan}\left(\frac{P}{V}\right) = 79.51\ deg$

$R := \sqrt{P^2 + V^2} = 3606.38\ lbf \qquad t_p := d = 0.25\ in$

Adjustment Factors

$C_d := 1.6$ Wind $\quad C_m := 1.0 \quad C_t := 1.0$

Try 3/8"x3" lag

$d := \frac{3}{8}\ in \qquad d_{edge} := 1.5 \cdot d = 0.563\ in \qquad l_{thread} := 1.78\ in \qquad W := 235\ \frac{lbf}{in}$ Table 12.2.A

$d_{edge} := 1.5 \cdot d = 0.563\ in$ < 1.25" Provided; therefore OK (Ref. 2 Table 12.5.1E)

$W' := W \cdot C_d \cdot l_{thread} = 669.28\ lbf$

Estimate the required amount of lags based on only withdrawal force

$$n_{lags} := \frac{P}{W'} = 5.298$$

Try 3 connectors per panel each having 2 rows of 2 lags per connector (4 total lags per connector)

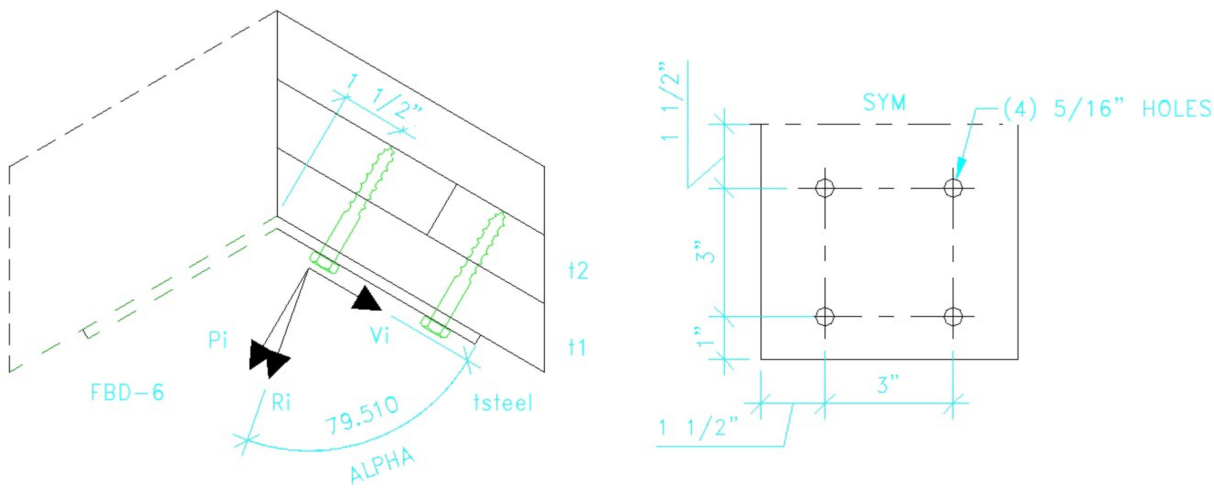


Figure 7. Resultant force on lag screw on left. Individual connector hole spacing on right.

Check combine withdrawal and lateral load

Recalculate individual lag load based on the 3 connectors. Check an individual lag near the peak since that lag will be in tension. The lag loads are:

$n_r := 6$ $\quad P_i := \frac{P}{n_r} = 591.03\ lbf$ $\quad V_i := \frac{V}{n_r} = 109.38\ lbf$ $\quad R_i := \frac{R}{n_r} = 601.06\ lbf$

Calculate Lateral Load Capacity of Lag

Calculate group factor

$E_s := 29000\ ksi$ $\quad E_{long} := 1400\ ksi$ $\quad E_{trans} := 1200\ ksi$ Ref. 8 Table 2.5

$E_m := \frac{E_{long} + E_{trans}}{2} = 1300\ ksi$ $\quad t_{steel} := 0.25\ in$ $\quad t_{main} := b_{410} = 4.14\ in$ $\quad W_{side} := 6\ in$

$A_m := W_{typ} \cdot t_{main} = 391.48\ in^2$ $\quad A_s := t_{steel} \cdot W_{side} = 1.5\ in^2$ $\quad s := 3\ in$ $\quad D := \frac{3}{8}\ in$

$R_{EA} := min\left(\frac{E_s \cdot A_s}{E_m \cdot A_m}, \frac{E_m \cdot A_m}{E_s \cdot A_s}\right) = 0.09$ $\quad \gamma := 270000 \cdot \left(\frac{D}{in}\right)^{1.5} \cdot \frac{lbf}{in} = 62002.71\ \frac{lbf}{in}$

$u := 1 + \gamma \cdot \frac{s}{2} \cdot \left(\frac{1}{E_m \cdot A_m} + \frac{1}{E_s \cdot A_s} \right) = 1.002$ $m := u - \sqrt{u^2 - 1} = 0.934$ $n := 2$

$$C_g := \left(\frac{m \cdot \left(1 - m^{2 \cdot n}\right)}{n \cdot \left(\left(1 + R_{EA} \cdot m^n\right) \cdot (1 + m) - 1 + m^{2 \cdot n} \right)} \right) \cdot \left(\frac{1 + R_{EA}}{1 - m} \right) = 0.998$$

Calculate the geometry factor (Ref. 2 Section 12.5)

From Table 12.5.1A minimum end distance for full capacity is $4\ D = 1.5\ in$ which is greater than the 1 1/4" previously estimated; therefore, increase end distance to 1 1/2". This parameter would apply to the CLT layer closest to the steel plate. The loading in this ply would be compression parallel to the grain.

From Table 12.5.1B minimum required spacing (for full capacity) for fasteners in a row is $4\ D = 1.5\ in$ which is less than the provided 3"; therefore OK.

From Table 12.5.1C, all edge spacings greater than or equal to $4\ D = 1.5\ in$; therefore OK.

From Table 12.5.1D

$p := 3\ in - \frac{7}{32}\ in - t_{steel} = 2.53\ in$ $\frac{p}{D} = 6.75$ > 6; therefore, min spacing between rows = $5\ D = 1.88\ in$ < the 3" provided, so OK for full capacity.

Also, the perpindicular to grain spacing = 3" <= 5"; therefore OK.

All components of 12.5.1 have been satisfied; therefore: $C_{\Delta} := 1.0$

Compute lateral capacity of single lag (Ref. 2 Section 12.3)

From Table 12.3.3

$G = 0.42$ $F_{e\_parallel} := 4700\ psi$ $F_{e\_perp} := 2850\ psi$

$F_u := 58\ ksi$ A36 Steel $F_{es} := \frac{2.4}{1.6} \cdot F_u = 87000\ psi$ Ref. 2 App I.2

Adjusted bearing length in CLT member (Ref. 14 Section 6.2.1). The approach is to reduce the capacity of the parallel to grain portion based on the respective bearing length within the layers.

CLT K3-0410 panel layer thickness

$t_1 := 1.38\ in$ (Parallel to Grain) $t_1 := 1.38\ in$ (Perpindicular to Grain)

Yield limit equation variables

$l_m := t_1 + (p - t_1) \cdot \frac{F_{e\_perp}}{F_{e\_parallel}} = 2.08\ in$ $D_r := 0.265\ in$ $l_s := t_{steel} = 0.25\ in$

$p_{min} := 4 \cdot D = 1.5\ in$ < $p = 2.53\ in$ therefore OK. (Section 12.1.4.7)

$F_{yb} := 45\ ksi$ Ref. 2 App I, table I1 $F_{em} := F_{e\_parallel} = 4700\ psi$

$\theta := 90\ deg$ t2 layer grain is oriented 90 degrees to the load $K_\theta := 1 + 0.25 \cdot \left(\frac{\theta}{90\ deg}\right) = 1.25$

$R_{d\_1m} := 4 \cdot K_\theta = 5$ $R_{d\_1s} := R_{d\_1m}$ $R_{d\_2} := 3.6 \cdot K_\theta = 4.5$ $R_{d\_3m} := 3.2 \cdot K_\theta = 4$

$R_{d\_3s} := R_{d\_3m}$ $R_{d\_4} := R_{d\_3m}$ $R_e := \frac{F_{em}}{F_{es}} = 0.05$ $R_t := \frac{l_m}{l_s} = 8.31$

$$k_1 := \frac{\sqrt{R_e + 2 \cdot R_e^{\,2}\ \left(1 + R_t + R_t^{\,2}\right) + R_t^{\,2} \cdot R_e^{\,3}} - R_e \cdot \left(1 + R_t\right)}{\left(1 + R_e\right)} = 0.209$$

$$k_2 := -1 + \sqrt{2 \cdot \left(1 + R_e\right) + \frac{2 \cdot F_{yb} \cdot \left(1 + 2 \cdot R_e\right) \cdot D_r^{\,2}}{3 \cdot F_{em} \cdot l_m^{\,2}}} = 0.491$$

$$k_3 := -1 + \sqrt{\frac{2 \cdot \left(1 + R_e\right)}{R_e} + \frac{2 \cdot F_{yb} \cdot \left(2 + R_e\right) \cdot D_r^{\,2}}{3 \cdot F_{em} \cdot l_s^{\,2}}} = 6.332$$

Yield limit equations

$Z_{1m} := \frac{D_r \cdot l_m \cdot F_{em}}{R_{d\_1m}} = 517.65\ lbf$ $Z_{1s} := \frac{D_r \cdot l_s \cdot F_{es}}{R_{d\_1s}} = 1152.75\ lbf$

$Z_2 := \frac{k_1 \cdot D_r \cdot l_s \cdot F_{es}}{R_{d\_2}} = 267.12\ lbf$ $Z_{3m} := \frac{k_2 \cdot D_r \cdot l_m \cdot F_{em}}{\left(1 + 2 \cdot R_e\right) \cdot R_{d\_3m}} = 286.73\ lbf$

$Z_{3s} := \frac{k_3 \cdot D_r \cdot l_s \cdot F_{em}}{\left(2 + R_e\right) \cdot R_{d\_3s}} = 239.96\ lbf$ $Z_4 := \frac{D_r^{\,2}}{R_{d\_4}} \cdot \sqrt{\frac{2 \cdot F_{em} \cdot F_{yb}}{3 \cdot \left(1 + R_e\right)}} = 203.06\ lbf$

$Z' := min\left(Z_{1m}, Z_{1s}, Z_2, Z_{3m}, Z_{3s}, Z_4\right) \cdot C_d \cdot C_m \cdot C_t \cdot C_g \cdot C_\Delta = 324.26\ lbf$ Mode IV yielding per Ref. 4 Section B.3.6

Combined Lateral and Withdrawal (Ref 2. Section 12.4)

$\alpha = 79.51\ deg$ $Z_\alpha' := \frac{W' \cdot Z'}{W' \cdot (\cos(\alpha))^2 + Z' \cdot (\sin(\alpha))^2} = 646.5\ lbf$ > $R_i = 601.06\ lbf$; therefore OK

$W_{plate} := 6\ in$ $n_{connectors} := 3$

$W_{plate} \cdot n_{connectors} = 18\ in$ > $b_{min} = 12.55\ in$; therefore OK for plate bending

OK to use three connectors per panel with (4) 3/8"x3" lag screws. 1/4" A36 steel OK for plate material. Assume a similar connection will suffice for the base connection. Note that uplift resistance will be required from the wood block to the attic floor. This can be accomplished with timber connectors (structural wood screws). The panel-to-panel splice that will be designed for the attic floor can be utilized for the roof splice.

Design Roof Base Connection

Review maximum shear transfer case. The shear from the roof will be transferred through bracket B1, into the CLT roof block and into the CLT attic floor panel via structural wood screws. The magnitude of shear load will be ascertained from the RISA model. Maximum shear at the base occurs as thrust during the the load combination Dead + Snow. Assume that the connection designed for this load combination will be adequate to resist shear generated by gable end wind loading. The magnitude of the shear ($v_{panel}=11.56\ plf$) that will be transferred from roof to attic floor diaphragm is low.

Design the structural wood screw shear connection

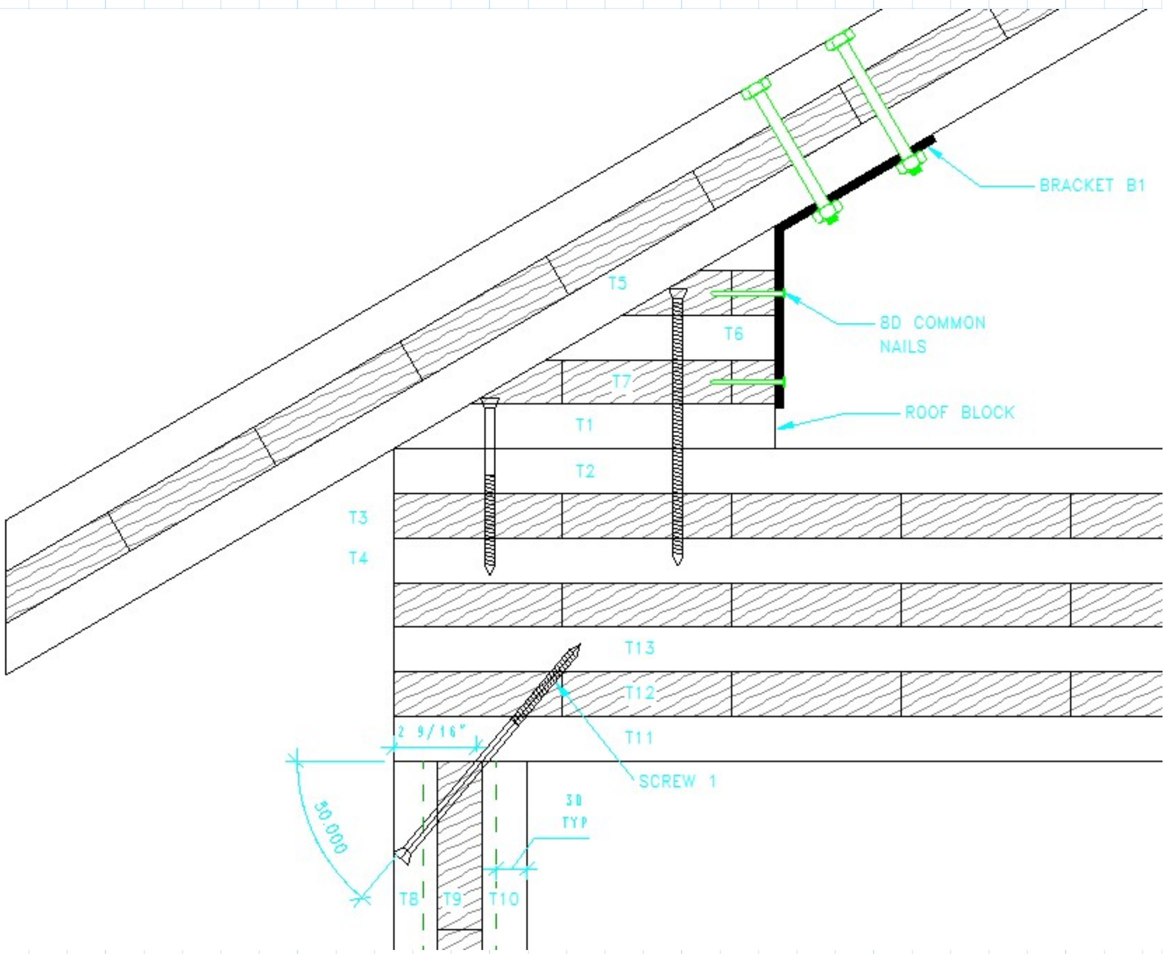


Figure 8. Roof-Attic wall/floor joint

$v_{ASD} := 700\ plf$ Factored shear due to thrust

Panel Dimensions (P=Parallel to grain, PE=Perpindicular to grain)

$t_1 := 1.38\ in$ (P) $t_2 := 1.38\ in$ (P) $t_3 := 1.38\ in$ (PE) $t_4 := 1.38\ in$ (P)

Two different screw lengths are required due to the block taper. For the shorter screw consider a MyTiCon countersunk 3/8"x5 1/2" ASSY Ecofast Screw and for the longer screw consider a 3/8"x8 5/8". Compute strength of shorter screw. From Ref. 19 Table PP.5.3 and their ESR 3179 report:

$L_f := 5.5\ in$ $D_r := 0.248\ in$ $E := 0.394\ in$ Tip length

$D := 0.394\ in$ $F_{yb} := 136.6\ ksi$ $G = 0.42$ $l_s := t_1 = 1.38\ in$

Geometry Requirements, based on predrilled hole installation (Ref. 19 Table S.1.2)

Min edge distance (a) = $3 \cdot D = 1.18\ in$

Min end distance (eL) = $7 \cdot D = 2.76\ in$ < 3.0 in; therefore OK

Min screw spacing (Sp) = $4 \cdot D = 1.58\ in$

Min screw penetration (p) = $6 \cdot D = 2.36\ in$ < $3.92\ in - E = 3.53\ in$ OK

Geometry Requirements met therefore $C_\Delta := 1.0$, other adjustment factors are

$C_d := 1.6$ $C_{di} := 1.0$ Not a nail or spike $C_g := 1.0$ $C_\Delta := 1.0$

Compute lateral capacity of fastener (Ref. 2 Section 12.3)

From Table 12.3.3

$G = 0.42$ $F_{e\_parallel} := 4700\ psi$ $F_{e\_perp} := 2850\ psi$ $F_{es} := 4700\ psi$

$F_{em} := F_{e\_parallel} = 4700\ psi$ At the shear interface

Primary loading direction parallel to grain at shear plane therefore adjust penetration through perpindicular layers (Ref. 14 Section 6.2.1)

$$l_m := t_2 + t_3 \cdot \frac{F_{e\_perp}}{F_{e\_parallel}} + 1.16\ in - E = 2.98\ in$$

Yield limit equation variables

$\theta := 90\ deg$ $\quad K_\theta := 1 + 0.25 \cdot \left(\frac{\theta}{90\ deg}\right) = 1.25$

$R_{d\_1m} := 4 \cdot K_\theta = 5$ $\quad R_{d\_1s} := R_{d\_1m}$ $\quad R_{d\_2} := 3.6 \cdot K_\theta = 4.5$ $\quad R_{d\_3m} := 3.2 \cdot K_\theta = 4$

$R_{d\_3s} := R_{d\_3m}$ $\quad R_{d\_4} := R_{d\_3m}$ $\quad R_e := \frac{F_{em}}{F_{es}} = 1$ $\quad R_t := \frac{l_m}{l_s} = 2.16$

$$k_1 := \frac{\sqrt{R_e + 2 \cdot R_e^{\ 2}\ \left(1 + R_t + R_t^{\ 2}\right) + R_t^{\ 2} \cdot R_e^{\ 3}} - R_e \cdot \left(1 + R_t\right)}{\left(1 + R_e\right)} = 0.729$$

$$k_2 := -1 + \sqrt{2 \cdot \left(1 + R_e\right) + \frac{2 \cdot F_{yb} \cdot \left(1 + 2 \cdot R_e\right) \cdot D_r^{\ 2}}{3 \cdot F_{em} \cdot l_m^{\ 2}}} = 1.098$$

$$k_3 := -1 + \sqrt{\frac{2 \cdot \left(1 + R_e\right)}{R_e} + \frac{2 \cdot F_{yb} \cdot \left(2 + R_e\right) \cdot D_r^{\ 2}}{3 \cdot F_{em} \cdot l_s^{\ 2}}} = 1.424$$

Yield limit equations

$Z_{1m} := \frac{D_r \cdot l_m \cdot F_{em}}{R_{d\_1m}} = 695.35\ lbf$ $\quad Z_{1s} := \frac{D_r \cdot l_s \cdot F_{es}}{R_{d\_1s}} = 321.71\ lbf$

$Z_2 := \frac{k_1 \cdot D_r \cdot l_s \cdot F_{es}}{R_{d\_2}} = 260.57\ lbf$ $\quad Z_{3m} := \frac{k_2 \cdot D_r \cdot l_m \cdot F_{em}}{\left(1 + 2 \cdot R_e\right) \cdot R_{d\_3m}} = 318.14\ lbf$

$Z_{3s} := \frac{k_3 \cdot D_r \cdot l_s \cdot F_{em}}{\left(2 + R_e\right) \cdot R_{d\_3s}} = 190.92\ lbf$ $\quad Z_4 := \frac{D_r^{\ 2}}{R_{d\_4}} \cdot \sqrt{\frac{2 \cdot F_{em} \cdot F_{yb}}{3 \cdot \left(1 + R_e\right)}} = 224.93\ lbf$

$$Z'_{short} := min\left(Z_{1m}, Z_{1s}, Z_2, Z_{3m}, Z_{3s}, Z_4\right) \cdot C_d \cdot C_{di} \cdot C_m \cdot C_t \cdot C_g \cdot C_\Delta = 305.47\ lbf$$

Check the longer Fastener

$t_5 := 1.38\ in$ (PE) $\quad t_6 := 1.38\ in$ (P) $\quad t_7 := 1.38\ in$ (PE) $\quad L_f := 8.625\ in$

$l_s := t_1 + t_6 + t_7 \cdot \frac{F_{e\_perp}}{F_{e\_parallel}} = 3.6\ in$ $l_m := t_2 + t_3 \cdot \frac{F_{e\_perp}}{F_{e\_parallel}} + 0.887\ in - E = 2.71\ in$

$R_e := \frac{F_{em}}{F_{es}} = 1$ $R_t := \frac{l_m}{l_s} = 0.75$

$$k_1 := \frac{\sqrt{R_e + 2 \cdot R_e^{2}\ \left(1 + R_t + R_t^{2}\right) + R_t^{2} \cdot R_e^{3}} - R_e \cdot (1 + R_t)}{(1 + R_e)} = 0.369$$

$$k_2 := -1 + \sqrt{2 \cdot (1 + R_e) + \frac{2 \cdot F_{yb} \cdot (1 + 2 \cdot R_e) \cdot D_r^{2}}{3 \cdot F_{em} \cdot l_m^{2}}} = 1.118$$

$$k_3 := -1 + \sqrt{\frac{2 \cdot (1 + R_e)}{R_e} + \frac{2 \cdot F_{yb} \cdot (2 + R_e) \cdot D_r^{2}}{3 \cdot F_{em} \cdot l_s^{2}}} = 1.068$$

Yield limit equations

$Z_{1m} := \frac{D_r \cdot l_m \cdot F_{em}}{R_{d\_1m}} = 631.71\ lbf$ $Z_{1s} := \frac{D_r \cdot l_s \cdot F_{es}}{R_{d\_1s}} = 838.49\ lbf$

$Z_2 := \frac{k_1 \cdot D_r \cdot l_s \cdot F_{es}}{R_{d\_2}} = 344.02\ lbf$ $Z_{3m} := \frac{k_2 \cdot D_r \cdot l_m \cdot F_{em}}{(1 + 2 \cdot R_e) \cdot R_{d\_3m}} = 294.33\ lbf$

$Z_{3s} := \frac{k_3 \cdot D_r \cdot l_s \cdot F_{em}}{(2 + R_e) \cdot R_{d\_3s}} = 373.1\ lbf$ $Z_4 := \frac{D_r^{2}}{R_{d\_4}} \cdot \sqrt{\frac{2 \cdot F_{em} \cdot F_{yb}}{3 \cdot (1 + R_e)}} = 224.93\ lbf$

$$Z'_{long} := min\left(Z_{1m}, Z_{1s}, Z_2, Z_{3m}, Z_{3s}, Z_4\right) \cdot C_d \cdot C_{di} \cdot C_m \cdot C_t \cdot C_g \cdot C_{\Delta} = 359.9\ lbf$$

$$Z' := Z'_{short} + Z'_{long} = 665.37\ lbf$$

$S_{req} := \frac{Z'}{v_{ASD}} = 11.41\ in$ Space fastener rows @ 11" Maximum on center

Design uplift resistance

From RISA analysis:

$\omega_{uplift} := 100\ plf$ As with the roof peak connectors, assume three per panel, spaced at maximum of 36 inches on center.

$$T_{uplift} := \omega_{uplift} \cdot 3\ ft = 300\ lbf$$

Design Bracket B1 nailed connection to roof block.

$D := 0.131\ in$

From Ref. 2 Table 12.5.1G

Min edge distance = $3 \cdot D = 0.393\ in$ < 0.902 in at min location; therefore, OK.
Min fastener spacing = $4 \cdot D = 0.52\ in$ < 3"; therefore, OK.
Min row spacing = $4 \cdot D = 0.524\ in$ < 3"; therefore, OK

Compute lateral capacity of single 8D nail

$l_s := 0.25\ in$ $\qquad G = 0.42$

$F_e := (3350\ psi) \cdot 0.67 = 2244.5\ psi$ Reduction factor based on the recommendation for end grain installation for dowel type fasteners (Ref. 14 Section 6.2.2)

$F_{em} := F_e = 2244.5\ psi$ $\qquad K_d := 2.2$ $\qquad D_r := D$ $\qquad F_{es} := 36\ ksi$ $\qquad l_m := 2.00\ in$

Yield limit equation variables

$R_{d\_1m} := K_d$ $\qquad R_{d\_1s} := K_d$ $\qquad R_{d\_2} := K_d$

$R_{d\_3s} := K_d$ $\qquad R_{d\_4} := K_d$ $\qquad R_{d\_3m} := K_d$

$R_e := \frac{F_{em}}{F_{es}} = 0.06$ $\qquad R_t := \frac{l_m}{l_s} = 8$

$$k_1 := \frac{\sqrt{R_e + 2 \cdot R_e^2 \left(1 + R_t + R_t^2\right) + R_t^2 \cdot R_e^3} - R_e \cdot (1 + R_t)}{(1 + R_e)} = 0.228$$

$$k_2 := -1 + \sqrt{2 \cdot (1 + R_e) + \frac{2 \cdot F_{yb} \cdot (1 + 2 \cdot R_e) \cdot D_r^2}{3 \cdot F_{em} \cdot l_m^2}} = 0.523$$

$$k_3 := -1 + \sqrt{\frac{2 \cdot (1 + R_e)}{R_e} + \frac{2 \cdot F_{yb} \cdot (2 + R_e) \cdot D_r^2}{3 \cdot F_{em} \cdot l_s^2}} = 6.553$$

Yield limit equations

$$Z_{1m} := \frac{D_r \cdot l_m \cdot F_{em}}{R_{d\_1m}} = 267.3\ lbf \qquad Z_{1s} := \frac{D_r \cdot l_s \cdot F_{es}}{R_{d\_1s}} = 535.91\ lbf$$

$$Z_2 := \frac{k_1 \cdot D_r \cdot l_s \cdot F_{es}}{R_{d\_2}} = 122.2\ lbf \qquad Z_{3m} := \frac{k_2 \cdot D_r \cdot l_m \cdot F_{em}}{(1 + 2 \cdot R_e) \cdot R_{d\_3m}} = 124.37\ lbf$$

$$Z_{3s} := \frac{k_3 \cdot D_r \cdot l_s \cdot F_{em}}{(2 + R_e) \cdot R_{d\_3s}} = 106.17\ lbf \qquad Z_4 := \frac{D_r^2}{R_{d\_4}} \cdot \sqrt{\frac{2 \cdot F_{em} \cdot F_{yb}}{3 \cdot (1 + R_e)}} = 108.2\ lbf$$

$C_d := 1.6$ $C_{eg} := 1.0$ $C_\Delta := 1.0$ $C_g := 1.0$ $n := 4$

$Z' := min\left(Z_{1m}, Z_{1s}, Z_2, Z_{3m}, Z_{3s}, Z_4\right) \cdot C_d \cdot C_{eg} \cdot C_g \cdot C_\Delta \cdot n = 679.51\ lbf$

$Z' = 679.51\ lbf$ > $T_{uplift} = 300\ lbf$; therefore, OK use (4) 8D nails.

Review tension placed on lag bolts due to uplift and eccentricity. To simplify, assume the tension and compression reactions due to the uplift eccentricity act at the fasteners.

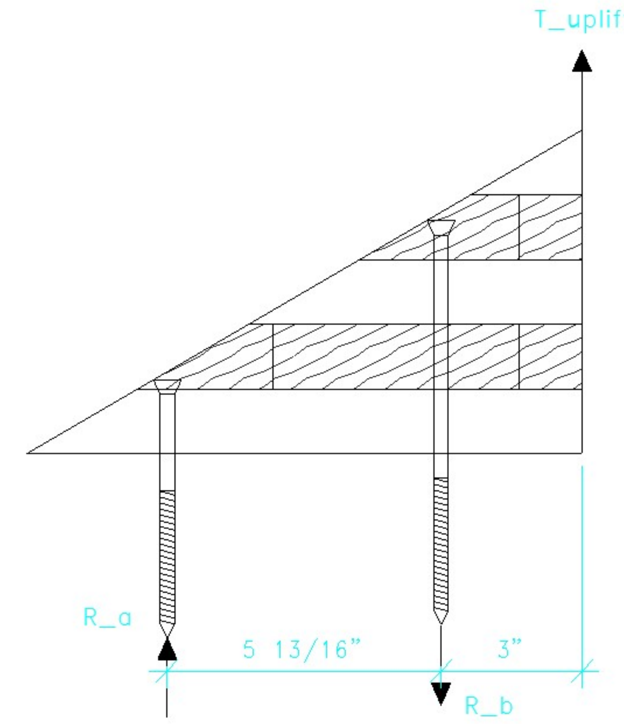


Figure 9. FBD Roof Block

Sum the moments about the shorter lag

$R_b := \dfrac{\omega_{uplift} \cdot 8.8125\ in}{5.8125\ in} = 151.61\ plf$ $R_a := R_b - \omega_{uplift} = 51.61\ plf$

Calculate Withdrawal Capacity of 3/8x8 5/8" Ecofast Screw

$l_{thread} := 3.64\ in$ $p_t := l_{thread} - E = 3.246\ in$ $W := 237\ \dfrac{lbf}{in}$ Ref. 22 Table RDV.1.1

$W'_{main} := W \cdot C_d \cdot p_t = 1230.88\ lbf$

Calculate pull-through resistance

$W_H := 319\ lbf$ From Ref. 22 Tables RDV.2.1 $W_H' := W_H \cdot C_d = 510.4\ lbf$

Pull-through limit state controls

Check Combined Loading for 3/8x8 5/8 screw @ 11" O.C. as originally specified

$v_{ASD} = 700\ plf$ Shear Loading $Z'_{long} = 359.9\ lbf$ $Z'_{short} = 305.47\ lbf$

$R_z := \frac{v_{ASD}}{2} \cdot 11\ in = 320.83\ lbf$ $R_w := R_b \cdot 11\ in = 138.98\ lbf$

From Ref. 2 Commentary C12.4.2-2

$\frac{R_w}{W_H'} + \frac{R_z}{Z'_{long}} = 1.164$ No good, try switch to all thread screw ASSY VG CSK and recalculate withdrawal only values. Note pull through does not need to be considered with the all threaded screws.

Calculate Withdrawal in side member

$l_{thread} := 8.125\ in - 3.64\ in$ $p_t := l_{thread} - E = 4.091\ in$ $W := 237\ \frac{lbf}{in}$ Ref. 22 Table RDV.1.1

$W'_{side} := W \cdot C_d \cdot p_t = 1551.31\ lbf$

$W' := min\left(W'_{side}, W'_{main}\right) = 1230.88\ lbf$

Recheck Combined Loading

$\frac{R_w}{W'} + \frac{R_z}{Z'_{long}} = 1.004$ No good reduce spacing to 10" O.C.

$R_z := \frac{v_{ASD}}{2} \cdot 10\ in = 291.67\ lbf$ $R_w := R_b \cdot 10\ in = 126.34\ lbf$

$\frac{R_w}{W'} + \frac{R_z}{Z'_{long}} = 0.913$ OK, Space the VG CSK screws @ 10" O.C.

The compression reaction was conservatively assumed to occur at the smaller fastner; however, in reality the screw would not likely transfer much compression into the attic floor because of the smooth shank and the small countersunk head. The compression will be transferred largely by bearing of the block edge, therefore no compression force should act on the small screw. Check for shear resistance only.

$R_z := \frac{v_{ASD}}{2} \cdot 10\ in = 291.67\ lbf$ $\frac{R_z}{Z'_{short}} = 0.95$ OK

Design an angled screw connection (Screw 1) to transfer uplift between floor and wall. This is a conservative step. The weight of the attic panel is more than adequate to resist the uplift force; however, adding hardware at this joint helps to maintain alignment as well provide positive connection between the components. The shear from longitudinal windforce will also be evaluated through the screw as a safeguard against inadvertent shear loading due to variations in connection stiffness between the interior and exterior connection. The manufacturer may recommend additional fasteners at the joints to help ensure stabilty during construction.

$\omega_{uplift} = 100\ plf$ $v_r := 145\ plf$ Shear from longitudinal wind loading, derived in the subsequent attic diaphram check.

Assume the joint is pinned. Design for direct tension and longitudinal shear. The second floor walls are continuous and it's reasonable to assume, by inspection, that the upper portion above the window openings can act as chord and collector.
Design Screw 1 connection based on longitudinal wind shear and uplift first then verify that the connection would be adequate to resist chord forces.

Angled Fastener Distance Through Layer (P=Parallel to grain, PE=Perpindicular to grain)
Distance includes the head and is measured along the centerline of the fastener

$t_8 := 1.78\ in$ (PE) $t_9 := 2.15\ in$ (P) $t_{10} := 0\ in$ (PE) $t_{11} := 1.81\ in$ (PE)

$t_{12} := 1.81\ in$ (P) $t_{13} := 0\ in$ (PE)

Try a MyTiCon countersunk 5/16"x8 5/8" ASSY Ecofast Screw. From Ref. 19 Table PP.5.3 and their ESR 3179 report:

$L_f := 8.625\ in$ $D_r := 0.209\ in$ $E := 0.315\ in$ Tip length $l_{thread} := 3.875\ in$

$D := 0.315\ in$ $F_{yb} := 150.2\ ksi$ $G = 0.42$ $p_t := l_{thread} - E = 3.56\ in$

Geometry Requirements, based on predrilled hole (Ref. 19 Table S.1.2)

Min edge distance, narrow edge side member (e) = $3 \cdot D = 0.95\ in$ only count the portion of the screw within this band to maintain a geometry factor of 1.0.

Min edge distance, main member (a) = $4 \cdot D = 1.26\ in$ < 2.56 in; therefore OK

Min end distance (aL) = $7 \cdot D = 2.21\ in$ restrict to 2.5" min from end

Min screw spacing (Sp) = $4 \cdot D = 1.26\ in$

Min screw penetration (p) = $6 \cdot D = 1.89\ in$ < $p_t = 3.56\ in$ OK

Geometry Requirements met therefore $C_\Delta := 1.0$, other adjustment factors are

$C_d := 1.6$ $C_{di} := 1.0$ Not a nail or spike $C_g := 1.0$ $C_\Delta := 1.0$

Compute lateral capacity of fastener (Ref. 2 Section 12.3)

From Table 12.3.3

$G = 0.42$ $F_{e\_parallel} := 4700\ psi$ $F_{e\_perp} := 3100\ psi$

$F_{em} := F_{e\_perp} = 3100\ psi$ $F_{es} := F_{e\_parallel} = 4700\ psi$ At the shear interface

Adjust penetration length through layers where the fastener shear loading is perpindicular to grain (Ref. 14 Section 6.2.1)

$l_m := t_{11} \cdot \frac{F_{e\_perp}}{F_{e\_parallel}} + t_{12} = 3\ in$ $l_s := t_8 \cdot \frac{F_{e\_perp}}{F_{e\_parallel}} + t_9 = 3.32\ in$

Yield limit equation variables

$\theta := 90\ deg$ $K_\theta := 1 + 0.25 \cdot \left(\frac{\theta}{90\ deg}\right) = 1.25$

$R_{d\_1m} := 4 \cdot K_\theta = 5$ $R_{d\_1s} := R_{d\_1m}$ $R_{d\_2} := 3.6 \cdot K_\theta = 4.5$ $R_{d\_3m} := 3.2 \cdot K_\theta = 4$

$R_{d\_3s} := R_{d\_3m}$ $R_{d\_4} := R_{d\_3m}$ $R_e := \frac{F_{em}}{F_{es}} = 0.66$ $R_t := \frac{l_m}{l_s} = 0.9$

$$k_1 := \frac{\sqrt{R_e + 2 \cdot R_e^{\,2} \left(1 + R_t + R_t^{\,2}\right) + R_t^{\,2} \cdot R_e^{\,3}} - R_e \cdot (1 + R_t)}{(1 + R_e)} = 0.331$$

$$k_2 := -1 + \sqrt{2 \cdot (1 + R_e) + \frac{2 \cdot F_{yb} \cdot (1 + 2 \cdot R_e) \cdot D_r^{\,2}}{3 \cdot F_{em} \cdot l_m^{\,2}}} = 0.919$$

$$k_3 := -1 + \sqrt{\frac{2 \cdot (1 + R_e)}{R_e} + \frac{2 \cdot F_{yb} \cdot (2 + R_e) \cdot D_r^{\,2}}{3 \cdot F_{em} \cdot l_s^{\,2}}} = 1.318$$

Yield limit equations

$Z_{1m} := \frac{D_r \cdot l_m \cdot F_{em}}{R_{d\_1m}} = 389.24\ lbf$ $Z_{1s} := \frac{D_r \cdot l_s \cdot F_{es}}{R_{d\_1s}} = 653.04\ lbf$

$Z_2 := \frac{k_1 \cdot D_r \cdot l_s \cdot F_{es}}{R_{d\_2}} = 240.53\ lbf$ $Z_{3m} := \frac{k_2 \cdot D_r \cdot l_m \cdot F_{em}}{(1 + 2 \cdot R_e) \cdot R_{d\_3m}} = 192.76\ lbf$

$Z_{3s} := \frac{k_3 \cdot D_r \cdot l_s \cdot F_{em}}{(2 + R_e) \cdot R_{d\_3s}} = 266.77\ lbf$ $Z_4 := \frac{D_r^{\,2}}{R_{d\_4}} \cdot \sqrt{\frac{2 \cdot F_{em} \cdot F_{yb}}{3 \cdot (1 + R_e)}} = 149.35\ lbf$

Per Ref. 4 Section 4.5.4.1

$Z_{star} := min\left(Z_{1m}, Z_{1s}, Z_2, Z_{3m}, Z_{3s}, Z_4\right) \cdot C_{di} \cdot C_m \cdot C_t \cdot C_g \cdot C_\Delta = 149.35\ lbf$

Failure Mode IV controls therefore, OK per Ref. 4 Section 4.5.4.1

$Z_n := 4.5 \cdot Z_{star} = 672.07\ lbf$ $RF := 2.0$ Ref. 4 Section 4.1.4 $S := 12\ in$ Fastener Spacing

$Z' := min\left(Z_{1m}, Z_{1s}, Z_2, Z_{3m}, Z_{3s}, Z_4\right) \cdot C_d \cdot C_{di} \cdot C_m \cdot C_t \cdot C_g \cdot C_\Delta = 238.96\ lbf$

$v_{n1} := \dfrac{Z_n}{S \cdot RF} = 336.04\ plf$ $v_{n2} := \dfrac{Z'}{S} = 238.96\ plf$

Per Ref. 4 Section 4.5.4.1 Per Ref. 2 Table 11.3.1

Calculate Withdrawal Capacity of 5/16x8 5/8" Ecofast Screw

$W_{90} := 212\ \dfrac{lbf}{in}$ $R_\propto := 0.879$ From Ref. 22 Tables RDV.1.1 and RDV. 1.2, angle 50 degrees

$W' := W \cdot R_\propto \cdot C_d \cdot p_t = 1186.61\ lbf$

Note that Ref. 4 Section 4.5.4.2 does not allow combined shear and tension connections for diaphragm connections; however, by engineering judgement in this situation the risk of combined loading failure is very low considering this is a very conservative connection design. In fact, most if not all, of the light shear loading would be actually be transferred via friction between the attic panel and the 2nd story walls. Additionally, the weight of the panel is adequate to resist the applied tensile force. This connection is more of a safe-guard or general stability measure. Due to the combined loading NDS (Ref. 2) shear capacity will be utilized in lieu of the SPDWS (Ref. 4) diaphragm strength. The attic panel has excess shear capacity therefore the limit state would still be the ductile fastener failure .

Calculate pull-through resistance

$W_H := 232\ lbf$ From Ref. 22 Tables RDV.2.1 $W_H' := W_H \cdot C_d = 371.2\ lbf$

Pull-through limit state controls

$t_n := \dfrac{W_H'}{S} = 371.2\ plf$

Check Combined Loading

$R_z := v_r = 145\ plf$ $R_w := \omega_{uplift} = 100\ plf$

From Ref. 2 Commentary C12.4.2-2

$\frac{R_w}{t_n} + \frac{R_z}{v_{n2}} = 0.876$ Fasteners spaced at 12" O.C. OK.

Check whether connection is adequate to resist chord force

Wind Loading (MWFRS - Appendix B)

Roof Pitch: $\theta := 30.256\ deg$ Roof Length: $l_r := 37\ ft$ Roof Width: $W_r := 18.52\ ft$

Building Length: $l_b := 36\ ft$ 2nd Story Height (center-to-center): $h_{2nd} := 8.625\ ft$

Building width: $W_b := 30\ ft$

Case I

$p_A := 6.89\ psf$ Windward Roof Pressure (Away from and perpindicular to the Structure)

$p_{A\_H} := -p_A \cdot \sin(\theta) = -3.47\ psf$ Horizontal component of Windward pressure

$p_B := 12.45\ psf$ Leeward Roof Pressure (Away from and perpindicular to the Structure)

$p_{B\_H} := p_B \cdot \sin(\theta) = 6.27\ psf$ Horizontal component of Leeward pressure

$$\omega_{roof\_1} := \frac{\left(p_{A\_H} \cdot l_r \cdot W_r\right) + \left(p_{B\_H} \cdot l_r \cdot W_r\right)}{l_b} = 53.32\ plf$$

Case II

$p_A := 6.36\ psf$ Windward Roof Pressure (Away from and perpindicular to the Structure)

$p_{A\_H} := p_A \cdot \sin(\theta) = 3.2\ psf$ Horizontal component of Windward pressure

$p_B := 5.95\ psf$ Leeward Roof Pressure (Away from and perpindicular to the Structure)

$p_{B\_H} := p_B \cdot \sin(\theta) = 3\ psf$ Horizontal component of Leeward pressure

$$\omega_{roof\_2} := \frac{\left(p_{A\_H} \cdot l_r \cdot W_r\right) + \left(p_{B\_H} \cdot l_r \cdot W_r\right)}{l_b} = 118.06\ plf$$

$\omega_{roof} := \max\left(\omega_{roof\_1}, \omega_{roof\_2}\right) = 118.06\ plf$ Case II controls

$p_{AW} := 14.7\ psf$ Windward Wall Pressure $p_{BW} := 4.42\ psf$ Leeward Wall Pressure

$\omega_{attic\_NS} := \omega_{roof} + \left(p_{AW} \cdot 0.5 \cdot h_{2nd} + p_{BW} \cdot 0.5 \cdot h_{2nd}\right) = 200.52\ plf$

Max moment @ mid floor

$LF_{wind} := 0.6$ $M_{max} := LF_{wind} \cdot \left(\frac{\omega_{attic\_NS} \cdot l_b^{\ 2}}{8}\right) = 19.49\ kip \cdot ft$ $T_{chord} := \frac{M_{max}}{W_b} = 649.68\ lbf$

Assuming $n := \frac{l_b}{2\ ft} - 1 = 17$ fasteners per side of centerline

$V_{per\_fastener} := \frac{T_{chord}}{n} = 38.22\ lbf$

Therefore; $V_{per\_fastener} := \frac{T_{chord}}{n} = 38.22\ lbf$ < $v_r \cdot 1\ ft = 145\ lbf$. OK, Screw 1 design OK

Design the longitudinal shear walls (Design based on Ref. 4)

The design of shear walls is not well established yet. Ref. 4 will be used as a guide to conservatively design the shear walls in this building. The 2:1 lower bound aspect ratio required in Ref. 4 will be relaxed due to the low load conditions. Tests summarized in Ref 15. concluded that in relation to seismic performance walls with aspect ratios lower than 2:1 ceased to have any beneficial effect on wall behavior (strength, stiffness, deformation capacity, energy dissapation) likely due to a transition from a rocking behaviour to sliding behavior. Cyclic testing (hysteresis) was performed observing non-linear behaviour. This is not directly relevant to wind design. Wind design is based on linear elastic behaviour (Ref. 14 Chapter 4). The lateral system will be designed based on a mechanics of materials and linear elastic approach. Slip connections or weaker connections will be utilized for the interior bearing wall/diaphragm connections to ensure lateral load transfers to the exterior shear walls and not the interior walls.

Determine Lateral Loading to Diaphragms

Figure 9. Gable end wind loading

Review Winds From Plan East-to-West Direction (Ref. MWFRS Main Building wind calculations in Appendix B)

1. Wind from 90 degrees (Windward and Leeward Added Together)
2. Case 4

$p_1 := 16.20\ psf + 3.81\ psf = 20.01\ psf$ $\quad p_2 := 14.40\ psf + 3.81\ psf = 18.21\ psf$

$W := 30\ ft$ $\quad h_1 := 9.22\ ft$ $\quad h_2 := 4.31\ ft$ $\quad h_3 := 8.76\ ft$ $\quad h_4 := 4.44\ ft$

$A_{t1} := 0.5\ W \cdot h_1 = 138.3\ ft^2$

$\omega_{attic} := \frac{p_1 \cdot A_{t1}}{W} + p_2 \cdot h_2 = 170.73\ plf$ $\quad \omega_{2nd} := p_2 \cdot h_3 = 159.52\ plf$ $\quad \omega_{1st} := p_2 \cdot h_4 = 80.85\ plf$

$P_{attic} := \omega_{attic} \cdot W = 5.12\ kip$ $\quad P_{2nd} := \omega_{2nd} \cdot W = 4.79\ kip$ $\quad P_{1st} := \omega_{1st} \cdot W = 2.43\ kip$

Diaphragm Flexibility (Ref. 10, 1604.4, Ref. 4, 4.1.7.2)

Assume that the diaphragm in this direction is rigid. A diaphragm is considered rigid for the purposes of distributing story shear and torsional moment when the lateral deformation of the diaphragm is less than 2.0 times that of the average story drift.

Aspect Ratio

There's no particular restrictions on CLT diaphragms in Ref. 4; however for reference sheathed blocked diaphragms are limited to a 4:1 L/W Ratio per table 4.2.2.

$L := 36\ ft$ $\quad W := 30\ ft$ $\quad \frac{L}{W} = 1.2$ < 4; Therefore no immediate concerns.

Attic Rigid Diaphragm Analysis

Notes:

1. The attic stair opening hole will be assumed to have negligible affect on the diaphragm due to it's location within the interior of the panel. It is assumed the strength of the CLT panel is adequate to distribute loading around the opening. The 2nd floor stair opening will be checked; however, due to it's location at the edge of the panel.
2. Assume the building meets the criteria establish in Ref. 16 App D1.3 for torsionally regular building under winds load and therefore does not require torsional load case checks.
3. Accidental eccentricity will not be applied in this example. Check local design codes for applicability in wind design.
4. Wall are all the same construction; therefore, stiffness will be proportional to wall length.
5. Wind loads applied through the geometric center of the windward and leeward face (center of pressure).
6. Since this is a linear elastic analysis, assume the full length of shear wall may utilized for stiffness calculations. In other words assume shear wall (thin plate) buckling is not a concern. This should be verified; however for the purposes of this project, no resources have been identified that addresses the subject of low aspect ratio CLT shear wall response and performance when subject to wind loading or plate buckling of CLT panels subjected to in-plane shear loading.

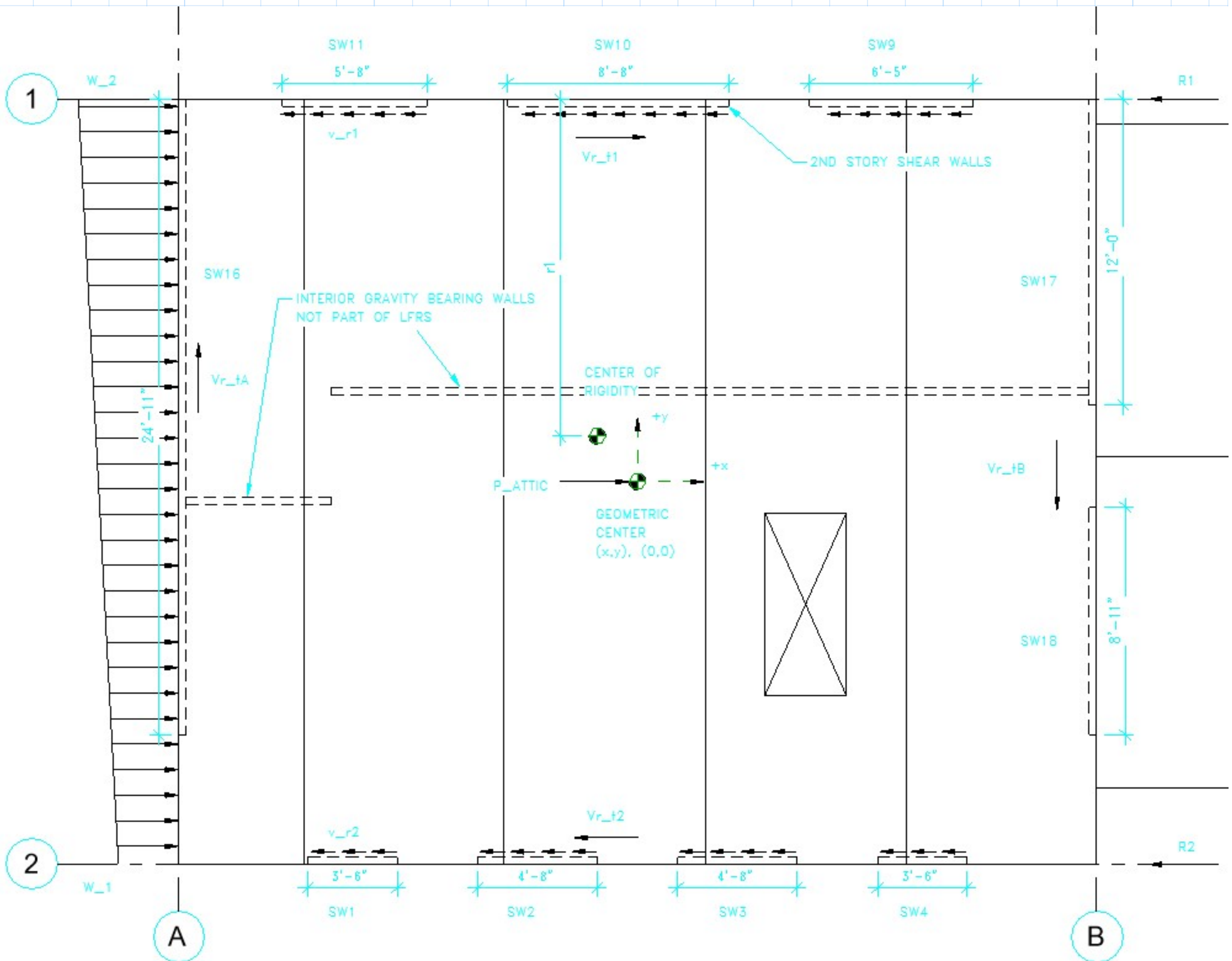


Figure 10. Attic diaphragm force diagram E-W directon

Check aspect ratio (Ref. 4 App B.3.1.2)

$l_{sw1} := 3.5\ ft$ $l_{sw2} := 4.67\ ft$ $l_{sw3} := l_{sw2}$ $l_{sw4} := l_{sw1}$ $l_{sw9} := 6.42\ ft$

$l_{sw10} := 8.67\ ft$ $l_{sw11} := 5.67\ ft$ $l_{sw16} := 24.92\ ft$ $l_{sw17} := 12\ ft$ $l_{sw18} := 8.92\ ft$

$l_{min} := min\left(l_{sw1}, l_{sw2}, l_{sw3}, l_{sw4}, l_{sw9}, l_{sw10}, l_{sw11}, l_{sw16}, l_{sw17}, l_{sw18}\right) = 3.5\ ft$

$H = 8\ ft$ $\frac{H}{l_{min}} = 2.29$ < 4; therefore OK, all walls can be utilized.

Determine Center of Rigidity (Ref. 17 Section 16.9)

$l_{wall\_1} := l_{sw9} + l_{sw10} + l_{sw11} = 20.76\ ft$ $l_{wall\_2} := l_{sw1} + l_{sw2} + l_{sw3} + l_{sw4} = 16.34\ ft$

$l_{wall\_A} := l_{sw16} = 24.92\ ft$ $l_{wall\_B} := l_{sw17} + l_{sw18} = 20.92\ ft$

$$e_x := \frac{-l_{wall\_A} \cdot \frac{L}{2} + l_{wall\_B} \cdot \frac{L}{2}}{l_{wall\_A} + l_{wall\_B}} = -1.57\ ft \qquad e_y := \frac{-l_{wall\_2} \cdot \frac{W}{2} + l_{wall\_1} \cdot \frac{W}{2}}{l_{wall\_1} + l_{wall\_2}} = 1.79\ ft$$

$$r_1 := \frac{W}{2} - e_y = 13.21\ ft \qquad r_2 := \frac{W}{2} + e_y = 16.79\ ft \qquad r_A := \frac{L}{2} + e_x = 16.43\ ft \qquad r_B := \frac{L}{2} - e_x = 19.57\ ft$$

Distribute Direct Shear Forces

$$v_r := \frac{P_{attic}}{l_{wall\_1} + l_{wall\_2}} = 138.06\ plf \qquad R_1 := v_r \cdot l_{wall\_1} = 2866.08\ lbf \qquad R_2 := v_r \cdot l_{wall\_2} = 2255.86\ lbf$$

Distribute Torsional Shear Forces

k = Stiffness, which in this case is equivalent to wall length.
r = Linear distance from wall line to center of rigidity.
Line = Wall line

$$A := \begin{bmatrix} \text{“Line”} & \text{“k”} & \text{“r”} \\ \text{“1”} & l_{wall\_1} & r_1 \\ \text{“2”} & l_{wall\_2} & r_2 \\ \text{“A”} & l_{wall\_A} & r_A \\ \text{“B”} & l_{wall\_B} & r_B \end{bmatrix} \qquad B := \begin{bmatrix} \text{“line”} & \text{“k(r)”} & \text{“k(r^2)”} \\ \text{“1”} & A_{1,1} \cdot A_{1,2} & A_{1,1} \cdot \left(A_{1,2}\right)^2 \\ \text{“2”} & A_{2,1} \cdot A_{2,2} & A_{2,1} \cdot \left(A_{2,2}\right)^2 \\ \text{“A”} & A_{3,1} \cdot A_{3,2} & A_{3,1} \cdot \left(A_{3,2}\right)^2 \\ \text{“B”} & A_{4,1} \cdot A_{4,2} & A_{4,1} \cdot \left(A_{4,2}\right)^2 \end{bmatrix}$$

$$J := \sum \left(\text{submatrix}\left(B, 1, 4, 2, 2\right)\right) = 22968.09\ ft^3 \qquad V_r(kr) := \frac{P_{attic} \cdot e_y \cdot kr}{J}$$

$V_{r\_t1} := V_r\left(B_{1,1}\right) = 109.31\ lbf$ $V_{r\_t2} := V_r\left(B_{2,1}\right) = 109.31\ lbf$

$V_{r\_tA} := V_r\left(B_{3,1}\right) = 163.16\ lbf$ $V_{r\_tB} := V_r\left(B_{4,1}\right) = 163.16\ lbf$

Neglect torsional forces on wall A and B. The wind loading from the N-S direction will produce large shear loadings. Add torsional shear loads to direct shear loads for Wall lines 1 and 2.

$R_1 := R_1 - V_{r\_t1} = 2756.76\ lbf$ $R_2 := R_2 + V_{r\_t2} = 2365.17\ lbf$

$v_{r1} := \dfrac{R_1}{l_{wall\_1}} = 132.79\ plf$ $v_{r2} := \dfrac{R_2}{l_{wall\_2}} = 144.75\ plf$

For comparison, what would the panel resisting shear be if the diaphragm was assumed to be flexible.

$v_{r1\_flexible} := \dfrac{\left(\dfrac{P_{attic}}{2}\right)}{l_{wall\_1}} = 123.36\ plf$ $v_{r2\_flexible} := \dfrac{\left(\dfrac{P_{attic}}{2}\right)}{l_{wall\_2}} = 156.73\ plf$

$Difference\_1 := \dfrac{v_{r1} - v_{r1\_flexible}}{v_{r1}} = 0.07$ $Difference\_2 := \dfrac{v_{r2} - v_{r2\_flexible}}{v_{r2}} = -0.08$

Almost 10% difference. It could be significant in some instances. Continue with the rigid diaphragm analysis; however verify assumption.

Design Shear Panel Anchorage and Check Compresssion Bearing

The approach in this design for transferring lateral load to the foundation and providing load path continuity is as follows. Shear loading will be transferred from the diaphragm to the shear panel and to the floor below via proprietary angle bracket connectors. Straps at shear wall ends will be provided to transfer overturning tensile forces to panels below (if required). Compressive forces caused by overturning will be resisted by bearing. Anchorage will be provided at the foundation level to resist aggregate overturning tensile forces.

Define resisting dead load, conservatively consider the weight of the panels alone.

$\sigma_{self\_410} = 9.84\ psf$ $\sigma_{self\_970} = 23.06\ psf$ $\sigma_{self\_4.125} := 11.1\ psf$

$\omega_{d\_line\_1} := 18.5\ ft \cdot \sigma_{self\_410} + \dfrac{11.44}{2}\ ft \cdot \sigma_{self\_970} + H \cdot \sigma_{self\_4.125} = 402.72\ plf$

$$\omega_{d\_line\_2} := 18.5\ ft \cdot \sigma_{self\_410} + \frac{18.56}{2}\ ft \cdot \sigma_{self\_970} + H \cdot \sigma_{self\_4.125} = 484.79\ plf$$

Review overturning forces for wall lines 1 and 2

Summation of the moments about point B
ASD design level forces (0.6D+0.6W)
Negative would indicate no tension

$$W_{d\_11} := 0.6 \cdot \omega_{d\_line\_1} \cdot l_{sw11} = 1370.05\ lbf$$

$$P_{w\_sw11} := 0.6 \cdot v_{r1} \cdot l_{sw11} = 451.76\ lbf$$

$$T_{11} := \frac{P_{w\_sw11} \cdot H - W_{d\_11} \cdot \frac{l_{sw11}}{2}}{l_{sw11}} = -47.62\ lbf$$

$$W_{d\_10} := 0.6 \cdot \omega_{d\_line\_1} \cdot l_{sw10} = 2094.94\ lbf$$

$$P_{w\_sw10} := 0.6 \cdot v_{r1} \cdot l_{sw10} = 690.78\ lbf$$

$$T_{10} := \frac{P_{w\_sw10} \cdot H - W_{d\_10} \cdot \frac{l_{sw10}}{2}}{l_{sw10}} = -410.07\ lbf$$

$$W_{d\_9} := 0.6 \cdot \omega_{d\_line\_1} \cdot l_{sw9} = 1551.27\ lbf$$

$$P_{w\_sw9} := 0.6 \cdot v_{r1} \cdot l_{sw9} = 511.51\ lbf$$

$$T_9 := \frac{P_{w\_sw9} \cdot H - W_{d\_9} \cdot \frac{l_{sw9}}{2}}{l_{sw9}} = -138.23\ lbf$$

$$W_{d\_1} := 0.6 \cdot \omega_{d\_line\_2} \cdot l_{sw1} = 1018.07\ lbf$$

$$P_{w\_sw1} := 0.6 \cdot v_{r2} \cdot l_{sw1} = 303.97\ lbf$$

$$T_1 := \frac{P_{w\_sw1} \cdot H - W_{d\_1} \cdot \frac{l_{sw1}}{2}}{l_{sw1}} = 185.75\ lbf$$

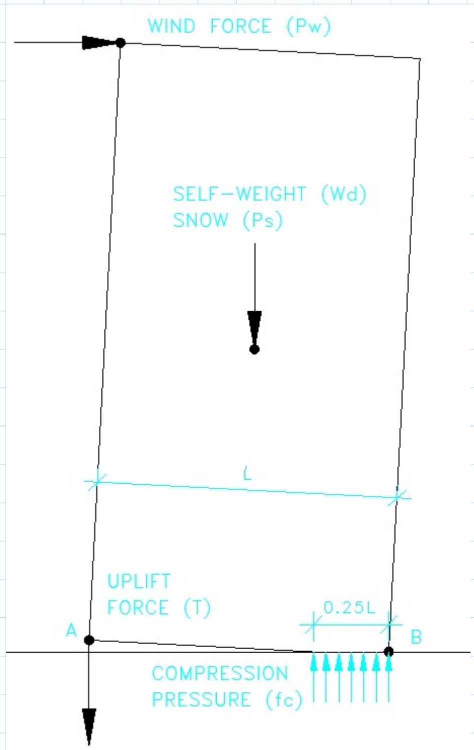


Figure 11. Shear wall overturning

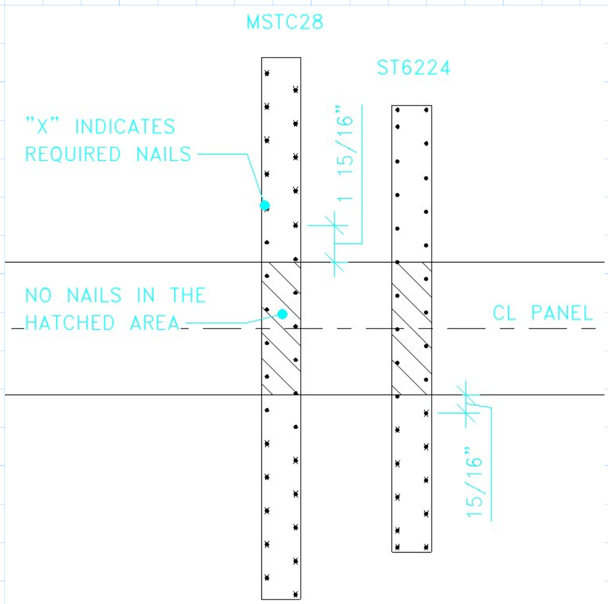
$$W_{d\_2} := 0.6 \cdot \omega_{d\_line\_2} \cdot l_{sw2} = 1358.39\ lbf$$

$$P_{w\_sw2} := 0.6 \cdot v_{r2} \cdot l_{sw2} = 405.58\ lbf$$

$$T_2 := \frac{P_{w\_sw2} \cdot H - W_{d\_2} \cdot \frac{l_{sw2}}{2}}{l_{sw2}} = 15.59\ lbf$$

$$P_{c\_11} := W_{d\_11} + T_{11} = 1322.42\ lbf$$

Notice most panel segments do not have uplift force present. Shear wall panels along line 2 will have some minimal uplift forces. T2 tensile force can be neglected; however, add straps to T1 panel. Consider Simpson Strong-Tie strap MSTC28 for the application. See Figure 12. More than enough capacity is available; however, due to the geometry requirements the smaller ST6215 and ST6224 will not work out. Note that the nails are to be installed in the wall panels only. Maintain adequate edge and end distance to prevent splitting as required in Ref. 2. For steel side members, 2015 NDS commentary recommends a minimum edge distance of 2.5D. A 16D common nail has a diameter of 0.162"; therefore 2.5 x 0.162" = 0.405". Minimum end distance is 10D, which would equal 10 x 0.162" = 1.62".



Allowable Tension Loads

Model No.	Ga.	Dimensions (in.) W	Dimensions (in.) L	Fasteners (Total)	Allowable Tension Loads (lb.) (DF/SP) (160)	Allowable Tension Loads (lb.) (SPF/HF) (160)	Code Ref.
ST6215	16	2 1/16	16 5/16	(20) 0.162" x 2½"	2,090	1,910	IBC, FL, LA
ST6224		2 1/16	23 5/16	(28) 0.162" x 2½"	2,535	2,535	
MSTC28		3	28¼	(36) 0.148" x 2½"	3,460	2,990	
MSTC40		3	40¼	(52) 0.148" x 2½"	4,735	4,315	
MSTC52		3	52¼	(62) 0.148" x 2½"	4,735	4,735	
MSTC66	14	3	65¾	(76) 0.148" x 2½"	5,850	5,850	
MSTC78		3	77¾	(76) 0.148" x 2½"	5,850	5,850	
ST6236		2 1/16	33 13/16	(40) 0.162" x 2½"	3,845	3,845	
MDCST48		5¾	47½	(36) ¼" x 3" SDS	11,905	10,560	
MDCST48 (Doubled/Overlapped)		5¾	47½	(72) ¼" x 3" SDS	23,810	21,120	—
HRS416Z	12	3¼	16	(16) ¼" x 1½" SDS	2,835	2,305	
MST27		2 1/16	27	(30) 0.162" x 2½"	3,700	3,210	IBC, FL, LA
MST37		2 1/16	37½	(42) 0.162" x 2½"	5,070	4,495	
MST48		2 1/16	48	(50) 0.162" x 2½"	5,310	5,190	
MST60	10	2 1/16	60	(68) 0.162" x 2½"	6,730	6 ,475	
MST72		2 1/16	72	(68) 0.162" x 2½"	6,730	6,475	

Figure 12. Simpson Strong-Tie strap detail and allowable loads

Try and reduce the number of nails required to 10 since we don't need full capacity. Use the same nail type listed in the table for the strap.

Compute lateral capacity of single nail, 0.148" diameter, 2 1/2" length (Ref. 2 Section 12.3)

From Table 12.3.3

$t_1 := 1.38\ in$ (Parallel to Grain) $t_2 := 1.38\ in$ (Perp to Grain) $t_3 := 1.38\ in$ (Parallel to Grain)

$D := 0.148\ in$ $l := 2.5\ in$ $l_s := 0.0625\ in$ $E := 2 \cdot D = 0.296\ in$

$G = 0.42$ $F_e := 3350\ psi$ $F_{es} := 33\ ksi$ $l_m := t_1 + (l - l_s - t_1 - E) = 2.14\ in$

$F_{em} := F_e = 3350\ psi$ $K_d := 2.2$ $D_r := D$

Yield limit equation variables

$R_{d\_1m} := K_d$ $R_{d\_1s} := K_d$ $R_{d\_2} := K_d$ $R_e := \dfrac{F_{em}}{F_{es}} = 0.1$ $R_t := \dfrac{l_m}{l_s} = 34.26$

$R_{d\_3s} := K_d$ $R_{d\_4} := K_d$ $R_{d\_3m} := K_d$

$$k_1 := \frac{\sqrt{R_e + 2 \cdot R_e^2 \left(1 + R_t + R_t^2\right) + R_t^2 \cdot R_e^3} - R_e \cdot (1 + R_t)}{(1 + R_e)} = 1.402$$

$$k_2 := -1 + \sqrt{2 \cdot (1 + R_e) + \frac{2 \cdot F_{yb} \cdot (1 + 2 \cdot R_e) \cdot D_r^2}{3 \cdot F_{em} \cdot l_m^2}} = 0.541$$

$$k_3 := -1 + \sqrt{\frac{2 \cdot (1 + R_e)}{R_e} + \frac{2 \cdot F_{yb} \cdot (2 + R_e) \cdot D_r^2}{3 \cdot F_{em} \cdot l_s^2}} = 18.337$$

Yield limit equations

$$Z_{1m} := \frac{D_r \cdot l_m \cdot F_{em}}{R_{d\_1m}} = 482.62\ lbf \qquad Z_{1s} := \frac{D_r \cdot l_s \cdot F_{es}}{R_{d\_1s}} = 138.75\ lbf$$

$$Z_2 := \frac{k_1 \cdot D_r \cdot l_s \cdot F_{es}}{R_{d\_2}} = 194.49\ lbf \qquad Z_{3m} := \frac{k_2 \cdot D_r \cdot l_m \cdot F_{em}}{(1 + 2 \cdot R_e) \cdot R_{d\_3m}} = 217.04\ lbf$$

$$Z_{3s} := \frac{k_3 \cdot D_r \cdot l_s \cdot F_{em}}{(2 + R_e) \cdot R_{d\_3s}} = 122.9\ lbf \qquad Z_4 := \frac{D_r^2}{R_{d\_4}} \cdot \sqrt{\frac{2 \cdot F_{em} \cdot F_{yb}}{3 \cdot (1 + R_e)}} = 173.75\ lbf$$

$C_d := 1.6$ $n := 10$ $Z' := min\left(Z_{1m}, Z_{1s}, Z_2, Z_{3m}, Z_{3s}, Z_4\right) \cdot C_d \cdot n = 1966.48\ lbf$

Apply overstrength factor per Ref. 4 Section B.3.4.3

$Z' = 1966.48\ lbf$ > $\Omega_w \cdot T_1 = 278.63\ lbf$; therefore, OK to reduce nails to 10 per side.

Net section tension rupture, row tear out, and group tear out per Ref. 2 Appendix E (Ref. 4 Section B.3.4.3) should be considered; however, by inspection this connection does not look as if it would be suceptible to these failure modes. The steel side member and the thicker main member should prevent these limit states from occuring.

Check the compression leg for bearing

Consider two alternative load combinations for the bearing check (D+0.6W and D+0.75(0.6W)+0.75S), consider the collateral dead load as well. Neglect uplift roof panel wind pressures.

$\sigma_{snow} = 25\ psf$ $\sigma_{collateral\_r} = 10\ psf$ $\sigma_{collateral\_f} = 5\ psf$ $\sigma_{collateral\_ew} := 5\ psf$

$\sigma_{roof} := \left(\sigma_{self\_410} + \sigma_{collateral\_r}\right) = 19.84\ psf$ $\sigma_{attic} := \left(\sigma_{self\_970} + \sigma_{collateral\_f}\right) = 28.06\ psf$

$\sigma_{ew} := \left(\sigma_{self\_4.125} + \sigma_{collateral\_ew}\right) = 16.1\ psf$

$$\omega_{d\_line\_1} := 18.5\ ft \cdot \left(\sigma_{roof}\right) + \frac{11.44}{2}\ ft \cdot \left(\sigma_{attic}\right) + H \cdot \left(\sigma_{ew}\right) = 656.32\ plf$$

$$\omega_{d\_line\_2} := 18.5\ ft \cdot \left(\sigma_{roof}\right) + \frac{18.56}{2}\ ft \cdot \left(\sigma_{attic}\right) + H \cdot \left(\sigma_{ew}\right) = 756.19\ plf$$

$$\omega_s := \sigma_{snow} \cdot \left(\frac{W}{2} + 1\ ft\right) = 400\ plf$$

LC 1 Review D+0.6W

$W_{d\_11} := \omega_{d\_line\_1} \cdot l_{sw11} = 3721.32\ lbf$

$P_{w\_sw11} := 0.6 \cdot v_{r1} \cdot l_{sw11} = 451.76\ lbf$

$$C_{11} := \frac{P_{w\_sw11} \cdot H + W_{d\_11} \cdot \frac{l_{sw11}}{2}}{l_{sw11}} = 2498.06\ lbf$$

LC 2 Review D+0.75(0.6W)+0.75S

$W_{d\_11} := \omega_{d\_line\_1} \cdot l_{sw11} = 3721.32\ lbf$

$P_{w\_sw11} := 0.45 \cdot v_{r1} \cdot l_{sw11} = 338.82\ lbf$

$P_{s\_sw11} := 0.75 \cdot \omega_s \cdot l_{sw11} = 1701\ lbf$

$$C_{11} := \frac{P_{w\_sw11} \cdot H + W_{d\_11} \cdot \frac{l_{sw11}}{2} + P_{s\_sw11} \cdot \frac{l_{sw11}}{2}}{l_{sw11}} = 3189.21\ lbf$$

Note that LC2 controls. Assume LC2 controls for remaining SW segments. Determine maximum force and then determine actual bearing pressure (fc). For determination of bearing pressure only consider the stiffer longitudinal layers (those parallel to the compressive force). Assume the bearing failure will occur perpindicular to the grain in the underlying CLT panel (Ref. 18 formula 6.11)

Estimate Bearing Pressure SW11 Based on Ref. 18 method of distributing the compressive force

$t_{layer} := 1.375\ in$ $\quad A_{sw11} := t_{layer} \cdot 2 \cdot \frac{l_{sw11}}{4} = 46.78\ in^2$ $\quad f_{c\_sw11} := \frac{C_{11}}{A_{sw11}} = 68.18\ psi$

$W_{d\_10} := \omega_{d\_line\_1} \cdot l_{sw10} = 5690.28\ lbf$ $\quad P_{w\_sw10} := 0.45 \cdot v_{r1} \cdot l_{sw10} = 518.09\ lbf$

$P_{s\_sw10} := 0.75 \cdot \omega_s \cdot l_{sw10} = 2601\ lbf$ $\quad C_{10} := \frac{P_{w\_sw10} \cdot H + W_{d\_10} \cdot \frac{l_{sw10}}{2} + P_{s\_sw10} \cdot \frac{l_{sw10}}{2}}{l_{sw10}} = 4623.69\ lbf$

$A_{sw10} := t_{layer} \cdot 2 \cdot \frac{l_{sw10}}{4} = 71.53\ in^2$ $\quad f_{c\_sw10} := \frac{C_{10}}{A_{sw10}} = 64.64\ psi$

$W_{d\_9} := \omega_{d\_line\_1} \cdot l_{sw9} = 4213.56\ lbf$ $\quad P_{w\_sw9} := 0.45 \cdot v_{r1} \cdot l_{sw9} = 383.64\ lbf$

$P_{s\_sw9} := 0.75 \cdot \omega_s \cdot l_{sw9} = 1926\ lbf$ $\quad C_9 := \frac{P_{w\_sw9} \cdot H + W_{d\_9} \cdot \frac{l_{sw9}}{2} + P_{s\_sw9} \cdot \frac{l_{sw9}}{2}}{l_{sw9}} = 3547.83\ lbf$

$A_{sw9} := t_{layer} \cdot 2 \cdot \frac{l_{sw9}}{4} = 52.97\ in^2$ $\quad f_{c\_sw9} := \frac{C_9}{A_{sw9}} = 66.98\ psi$

$W_{d\_1} := \omega_{d\_line\_2} \cdot l_{sw1} = 2646.68\ lbf$ $\quad P_{w\_sw1} := 0.45 \cdot v_{r2} \cdot l_{sw1} = 227.98\ lbf$

$P_{s\_sw1} := 0.75 \cdot \omega_s \cdot l_{sw1} = 1050\ lbf$ $\quad C_1 := \frac{P_{w\_sw1} \cdot H + W_{d\_1} \cdot \frac{l_{sw1}}{2} + P_{s\_sw1} \cdot \frac{l_{sw1}}{2}}{l_{sw1}} = 2369.43\ lbf$

$A_{sw1} := t_{layer} \cdot 2 \cdot \frac{l_{sw1}}{4} = 28.88\ in^2$ $\quad f_{c\_sw1} := \frac{C_1}{A_{sw1}} = 82.06\ psi$

$W_{d\_2} := \omega_{d\_line\_2} \cdot l_{sw2} = 3531.43\ lbf$ $\quad P_{w\_sw2} := 0.45 \cdot v_{r2} \cdot l_{sw2} = 304.19\ lbf$

$P_{s\_sw2} := 0.75 \cdot \omega_s \cdot l_{sw2} = 1401\ lbf$ $\quad C_2 := \frac{P_{w\_sw2} \cdot H + W_{d\_2} \cdot \frac{l_{sw2}}{2} + P_{s\_sw2} \cdot \frac{l_{sw2}}{2}}{l_{sw2}} = 2987.31\ lbf$

$A_{sw2} := t_{layer} \cdot 2 \cdot \frac{l_{sw2}}{4} = 38.53\ in^2$ $\quad f_{c\_sw2} := \frac{C_2}{A_{sw2}} = 77.54\ psi$

Determine Panel Bearing Capacity

$C_m := 1.0$ $C_t := 1.0$ $C_b := 1.0$ Conservative

$F_{c\_perp}' := 425\ psi \cdot C_m \cdot C_t \cdot C_b = 425\ psi$ > $f_{c\_sw1} = 82.06\ psi$; there OK for bearing

Design shear transfer brackets for base

$v_r := \max\left(v_{r1}, v_{r2}\right) = 144.75\ plf$

Review available friction. Consider only the panel weight along wall line 2. ASD design level frictional resistance will be:

$$\omega_{d\_line\_2} := 18.5\ ft \cdot \sigma_{self\_410} + \frac{18.56}{2}\ ft \cdot \sigma_{self\_970} + H \cdot \sigma_{self\_4.125} = 484.79\ plf$$

$\mu_{s\_low} := 0.25$ $\mu_{s\_high} := 0.50$

$F_{f\_low} := 0.6 \cdot \omega_{d\_line\_2} \cdot \mu_{s\_low} = 72.72\ plf$ $F_{f\_high} := 0.6 \cdot \omega_{d\_line\_2} \cdot \mu_{s\_high} = 145.44\ plf$

At the high end of the estimated frictional resistance range the friction would be adequate to transfer the shear; however, at the low range it would be not. In any regards, it would be conservative and good practice to ignore the frictional resistance in this instance due to it's unreliability and design for full shear. In addition fasteners are required for general stability of the CLT panels. For the base connections consider Simpson Strong-Tie connector ABR9020 (Ref. 21) see Figure 13.

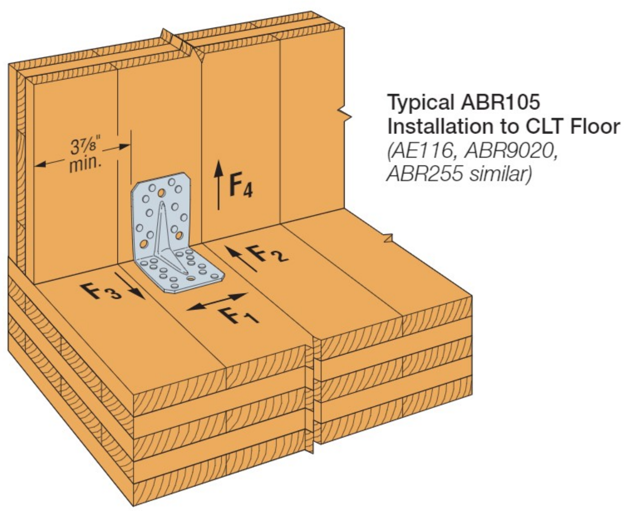


Figure 13. Wall-to-Floor connection

Allowable Loads — CLT Floor Values

Model No.	Gauge	Dimensions (in.)			Fastener Schedule		Allowable Load (lb.) C_D = 1.60				Code Ref.
		W_1	W_2	L	Horizontal Leg	Vertical Leg	F_1	F_2	F_3	F_4	
ABR9020	14	3 7/16	3 7/16	2 9/16	(10) CNA 4 x 60	(10) CNA 4 x 60	1,085	780	1,330	590	—
					(10) SD #10 x 2½"	(10) SD #10 x 2½"	1,480	1,200	1,330	1,010	
					(10) 0.162" x 2½"	(10) 0.162" x 2½"	980	425	1,330	510	
ABR105	11	4 1/8	4 1/8	3 9/16	(14) CNA 4 x 60	(10) CNA 4 x 60	1,350	835	2,300	1,020	
					(14) SD #10 x 2½"	(10) SD #10 x 2½"	1,880	1,235	2,300	1,475	
					(14) 0.162" x 2½"	(10) 0.162" x 2½"	1,220	580	2,020	415	
AE116	11	3 9/16	1 7/8	4 9/16	(7) CNA 4 x 60	(18) CNA 4 x 60	1,720	1,225	1,550	650	
					(7) SD #10 x 2½"	(18) SD #10 x 2½"	1,850	1,445	1,850	1,035	
					(7) 0.162" x 2½"	(18) 0.162" x 2½"	1,440	840	1,440	395	
ABR255	11	4¾	3 15/16	10	(41) CNA 4 x 60	(52) CNA 4 x 60	3,530	2,370	4,080	2,385	
					(41) SD #10 x 2½"	(52) SD #10 x 2½"	3,805	4,430	3,165	3,970	
					(41) 0.162" x 2½"	(52) 0.162" x 2½"	3,800	2,715	4,315	2,080	

Figure 14. Allowable connector loads

$v_{asd} := LF_{wind} \cdot v_r = 86.85\ plf$ $\qquad v_{allow} := 980\ lbf$

$v_{asd} \cdot l_{sw10} = 752.98\ lbf < 2 \cdot v_{allow} = 1960\ lbf$; therefore OK

Two brackets are OK for the most heavily load shear wall on the floor; therefore OK for all. Use two ABR9020 brackets per shear panel (one on each end) fastened as circled in Figure 14 check out-of-plane wind loading. Consider the North wall, which will have the least fasteners.

$h_{2nd} = 8.63\ ft$ $\qquad p_{AW} = 14.7\ psf$ $\qquad n := 6$

$\omega := p_{AW} \cdot 0.5 \cdot h_{2nd} \cdot l_b = 2282.18\ lbf$ $\qquad v_{allow\_F3} := 1330\ lbf$

$\omega = 2282.18\ lbf < n \cdot v_{allow\_F3} = 7980\ lbf$; therefore OK for out-of-plane shear load transfer to 2nd floor diaphragm.

Note that the Simpson strong brackets allowable loads include the load duration factor of 1.6. Because this is diaphragm shear connection it should really be considered without the load duration factor and should consider Ref. 4 sections 4.1.4 and 4.5.4. It's currently unclear how this should be resolved with tested assemblies such as the bracket. For the purposes of this report the table values will be used. Simpson-Strong-Tie was contacted regarding this discrepancy. They are currently investigating this question and expect to have clarity on the subject in their next mass timber catalog publication.

Determine an equivalent lateral loading based on the reactions calculated for the purposes of checking shear wall and diaphragm deflection

From sum of the forces equilibrium

$V := R_1 + R_2 = 5121.94\ lbf$

From sum of the moments equilibrium

$x_1 := \dfrac{R_1 \cdot W}{V} = 16.15\ ft$

Decompose the resultant force

$V_1 := \dfrac{-6 \cdot V \cdot x_1 + 4 \cdot V \cdot W}{W} = 3947.18\ lbf$ $\quad V_2 := V - V_1 = 1174.76\ lbf$

$w_1 := \dfrac{V_1}{W} = 131.57\ plf$ $\quad w_2 := w_1 + \dfrac{2 \cdot V_2}{W} = 209.89\ plf$

Calculate shear wall deflection (Ref. 4 Section B.4)

Wall panels, Nordic 105-3s layup, loading perpindicular to outermost layer

Review deflection in SW11, all shear wall panels will have equal deformations within the wall line due to distribution by stiffness method. Check wall line 1.

$b_{eff} := 2.75\ in$ $\quad E := 1700\ ksi$ $\quad d := l_{sw11} = 5.67\ ft$ $\quad I_{eff} := \dfrac{b_{eff} \cdot d^3}{12} = 72184.57\ in^4$

$EI_{eff} := E \cdot I_{eff} = 122713765851.6\ lbf \cdot in^2$ $\quad G_v t_v := 1.79 \cdot 10^6\ \dfrac{lbf}{ft}$

$GA_{eff} := G_v t_v \cdot d = 10149300\ lbf$

$v := v_{r1} = 11.07\ \dfrac{lbf}{in}$ $\quad b_s := d = 68.04\ in$ $\quad h := H = 96\ in$

Estimate nail shear load, assume 8 total nails at base

$n_{nails} := 8$ $\quad V_{nail\_load} := \dfrac{v \cdot b_s}{n_{nails}} = 94.12\ lbf$

$\Delta_{nail\_slip\_h} := \dfrac{V_{nail\_load}}{6700\ \dfrac{lbf}{in}} = 0.014\ in$ $\quad \Delta_{nail\_slip\_v} := 0\ in$ (= 0 inches for single panel)

$\Delta_a := 0.125\ in$ (Assumption based on max anchor deformation requirement in Section B.3.4)

$\delta_{sw\_11\_a} := \dfrac{576 \cdot v \cdot b_s \cdot h^3}{EI_{eff}} = 3.13\ in$

This is not a very good representation of deflection due to bending use equation 6.8 Ref. 18 which is a strictly mechanics based derivation to compute bending deflection.

$$\delta_{sw\_11\_a\_2} := \frac{v \cdot b_s \cdot h^3}{3 \cdot EI_{eff}} = 0.002\ in$$

$$\delta_{sw\_11\_b} := \frac{v \cdot h}{G_v t_v} = 0.007\ in$$ This shear bending component is the same in both Ref. 18 and Section B.4

$$\delta_{sw\_11\_c} := 3 \cdot \Delta_{nail\_slip\_h} = 0.042\ in$$ From Section B.4

$$\delta_{sw\_11\_d} := \Delta_a \cdot \frac{h}{b_s} = 0.176\ in$$

$$\delta_{sw\_11\_tot} := \delta_{sw\_11\_a\_2} + \delta_{sw\_11\_b} + \delta_{sw\_11\_c} + \delta_{sw\_11\_d} = 0.227\ in$$

Calculate deflection for SW2

$d := l_{sw1} = 3.5\ ft$ $I_{eff} := \frac{b_{eff} \cdot d^3}{12} = 16978.5\ in^4$ $EI_{eff} := E \cdot I_{eff} = 28863450000\ lbf \cdot in^2$

$v := v_{r2} = 12.06\ \frac{lbf}{in}$ $b_s := d = 42\ in$ $\delta_{sw\_1\_a} := \frac{v \cdot b_s \cdot h^3}{3 \cdot EI_{eff}} = 0.005\ in$

$\delta_{sw\_1\_b} := \frac{v \cdot h}{G_v t_v} = 0.008\ in$ $\delta_{sw\_1\_c} := 3 \cdot \Delta_{nail\_slip\_h} = 0.042\ in$ $\delta_{sw\_1\_d} := \Delta_a \cdot \frac{h}{b_s} = 0.286\ in$

$$\delta_{sw\_1\_tot} := \delta_{sw\_1\_a} + \delta_{sw\_1\_b} + \delta_{sw\_1\_c} + \delta_{sw\_1\_d} = 0.341\ in$$

Average deflection of Both Wall Lines

$$\delta_{sw\_attic\_avg} := \frac{\delta_{sw\_11\_tot} + \delta_{sw\_1\_tot}}{2} = 0.284\ in$$

Design Attic Diaphragm in East-to-West Direction (Ref. 6)

Check aspect ratio

$\frac{W}{L} = 0.83$ < 4.0 therefore OK.

Check Panel Capacity (Ref. Figure 9.)

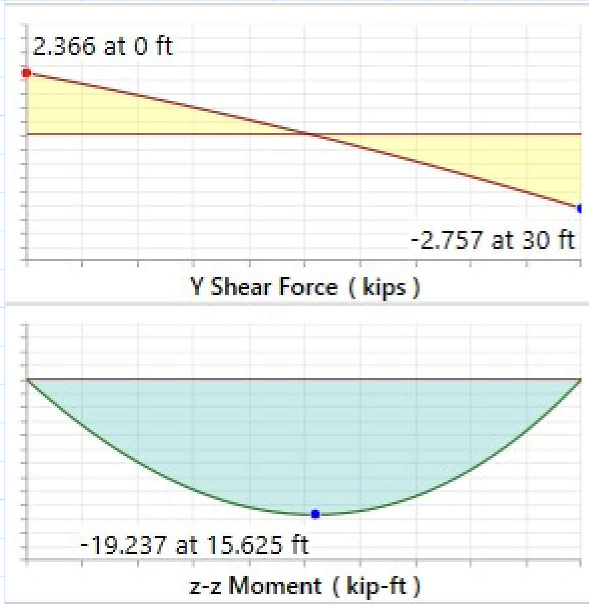


Figure 15. Attic diaphragm force diagrams (Ref. Appendix C)

CLT Panel Shear Strength

Design values for the roof panel (K7-0970) from Ref. 8

$b_{970} := 9.66\ in$ $\quad F_{v\_0} := 240\ psi$ Minimum edgewise shear stress from Table 3 Ref. 8

The 2.0 reduction factor required in Ref. 4, 4.1.4 is assumed to be accounted for in the table values. From Ref. 9, 8.5.6.2 edgewise shear capacity published values for Fv include an adjustment factor of 2.1.

NDS adjustment Factors (Ref. 2)

$C_d := 1.6$ Load Duration Factor Wind $\quad C_m := 1.0 \quad C_t := 1.0$

$\Omega_w := 1.5$ Overstrength factor for wind (Ref. 4, 4.5.4.3.1)

Design Strength

ASD Design level diaphragm load

$v_a := \dfrac{F_{v\_0} \cdot b_{970} \cdot C_d \cdot C_m \cdot C_t}{\Omega_w} = 29675.52\ plf$ >>>> $v_{ASD} := 0.6\ \dfrac{R_1}{L} = 45.95\ plf$; therefore OK

Design Panel Splice

Panel-to-Panel Connection (Consider the MyTiCon ASSY Ecofast Timber Screw for connection. See Ref. 19 and Figure 10)

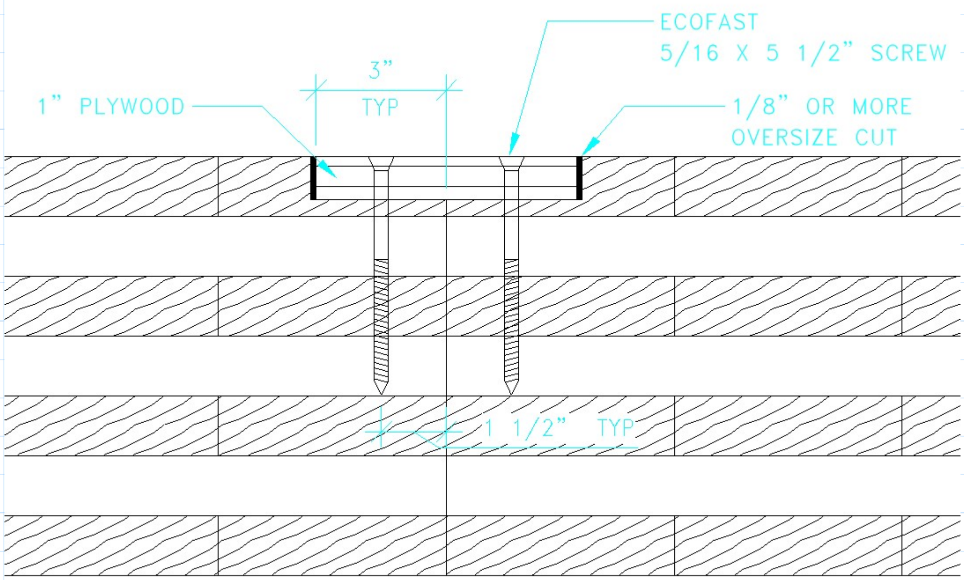


Figure 16. Panel-Panel Splice

From Ref. 19 Table PP.5.3, consider a 5/16"x5 1/2" Screw

$Z_{parallel} := 172\ lbf$ $Z_{perpindicular} := 138\ lbf$ $L_f := 5.5\ in$

$T := 3.125\ in$ $D_H := 0.591\ in$

From ESR-3179

$D_r := 0.209\ in$ $E := 0.315\ in$ Tip length $F_{yb} := 150.2\ ksi$ $D := 0.315\ in$

$D_{shank} := 0.228\ in$

From Ref. 19 Table S.1.1 and ESR-3179

$G = 0.42$ $l_s := 1\ in$

Min screw spacing (Sp) = $4 \cdot D = 1.26\ in$

Min screw penetration (p) = $6 \cdot D = 1.89\ in$ < $L_f - l_s = 4.5\ in$ OK

Min edge distance(e parallel) = $2.5 \cdot D = 0.79\ in$ < 1.5 in; therefore OK

Geometry Requirements met therefore $C_{\Delta} := 1.0$, other adjustment factors are

$C_d := 1.0$ Not included in Z star calculation $C_{di} := 1.0$ Not a nail or spike

Compute lateral capacity of single lag (Ref. 2 Section 12.3)

From Table 12.3.3

$G = 0.42$ $F_{e\_parallel} := 4700\ psi$ $F_{e\_perp} := 3100\ psi$ $F_{es} := 5600\ psi$ Ref. 20 Table A1

$F_{em} := F_{e\_perp} = 3100\ psi$ At the shear interface

Adjusted bearing length in CLT member (Ref. 14 Section 6.2.1). The approach is to reduce the capacity of the parallel to grain portion based on the respective bearing length within the layers.

CLT K7-0970 panel layer thickness (neglect top layer because it's notched)

$t_1 := 1.38\ in$ (Perp to Grain) $t_2 := 1.38\ in$ (Parallel to Grain) $t_3 := 1.38\ in$ (Perp to Grain)

Yield limit equation variables

$$l_m := t_1 \frac{F_{e\_perp}}{F_{e\_parallel}} + t_2 + \left(t_3 - E - \left(4 \cdot t_3 - L_f\right)\right) \cdot \frac{F_{e\_perp}}{F_{e\_parallel}} = 2.98\ in$$

$\theta := 90\ deg$ $K_\theta := 1 + 0.25 \cdot \left(\frac{\theta}{90\ deg}\right) = 1.25$

$R_{d\_1m} := 4 \cdot K_\theta = 5$ $R_{d\_1s} := R_{d\_1m}$ $R_{d\_2} := 3.6 \cdot K_\theta = 4.5$ $R_{d\_3m} := 3.2 \cdot K_\theta = 4$

$R_{d\_3s} := R_{d\_3m}$ $R_{d\_4} := R_{d\_3m}$ $R_e := \frac{F_{em}}{F_{es}} = 0.55$ $R_t := \frac{l_m}{l_s} = 2.98$

$$k_1 := \frac{\sqrt{R_e + 2 \cdot R_e^2 \left(1 + R_t + R_t^2\right) + R_t^2 \cdot R_e^3} - R_e \cdot \left(1 + R_t\right)}{\left(1 + R_e\right)} = 0.611$$

$$k_2 := -1 + \sqrt{2 \cdot \left(1 + R_e\right) + \frac{2 \cdot F_{yb} \cdot \left(1 + 2 \cdot R_e\right) \cdot D_r^2}{3 \cdot F_{em} \cdot l_m^2}} = 0.855$$

$$k_3 := -1 + \sqrt{\frac{2 \cdot \left(1 + R_e\right)}{R_e} + \frac{2 \cdot F_{yb} \cdot \left(2 + R_e\right) \cdot D_r^2}{3 \cdot F_{em} \cdot l_s^2}} = 2.036$$

Yield limit equations

$Z_{1m} := \frac{D_r \cdot l_m \cdot F_{em}}{R_{d\_1m}} = 386.08\ lbf$ $Z_{1s} := \frac{D_r \cdot l_s \cdot F_{es}}{R_{d\_1s}} = 234.08\ lbf$

$Z_2 := \frac{k_1 \cdot D_r \cdot l_s \cdot F_{es}}{R_{d\_2}} = 159\ lbf$ $Z_{3m} := \frac{k_2 \cdot D_r \cdot l_m \cdot F_{em}}{\left(1 + 2 \cdot R_e\right) \cdot R_{d\_3m}} = 195.88\ lbf$

$Z_{3s} := \frac{k_3 \cdot D_r \cdot l_s \cdot F_{em}}{(2+R_e) \cdot R_{d\_3s}} = 129.13\ lbf$ $\quad Z_4 := \frac{D_r^{\ 2}}{R_{d\_4}} \cdot \sqrt{\frac{2 \cdot F_{em} \cdot F_{yb}}{3 \cdot (1+R_e)}} = 154.36\ lbf$

$Z_{star} := min(Z_{1m}, Z_{1s}, Z_2, Z_{3m}, Z_{3s}, Z_4) \cdot C_{di} \cdot C_m \cdot C_t \cdot C_g \cdot C_\Delta = 129.13\ lbf$

Note the Z is limited by failure mode IIIs which meets the requirements of Ref. 4 Section 4.5.4

$\Omega_w = 1.5$ Overstrength Factor Ref. 4 4.5.4.3

$RF := 2.0$ Reduction Factor Ref. 4 4.1.4.2

$v_{ASD} = 45.95\ plf$ Really low shear requirement, could be satisfied with...

$S_{required} := \left(\frac{4.5 \cdot Z_{star}}{RF \cdot v_{ASD}} \right) = 6.32\ ft$

Not a reasonable spacing for the splice or the panel to chord/wall connection in the E-W direction. Let's assume that splice connection will not be controlled by loading in the E-W direction. Additionally, it should be safe to conclude that, due to the low level of lateral loading, using the longitudinal laminations of the CLT panels as chords to resist bending moment such as in the example presented in Ref. 6 should be adequate. The tension and compression in the chords will be minimal.

$M_{ASD} := 0.6 \cdot 19237\ lbf \cdot ft = 11542.2\ lbf \cdot ft$ Diaphragm design moment Ref. Figure x

$T := \frac{M_{ASD}}{L} = 320.62\ lbf$ Trivial, for the continuous K7-0970 panel (not chord splice in this direction)

Estimate diaphragm deflection in this direction (Ref. 6), Note that DeStefano (Ref. 7) remarks that it's reasonable to assume a ridgid diaphragm when aspect ratios are less than 2:1; otherwise a semi-ridgid diaphagm should be considered. In this case the L/W is less than 2:1 in both cardinal directions.

$v := \frac{R_1}{L \cdot plf} = 76.58$ plf $\quad L := 30$ ft. $\quad W := 36$ ft. Redefine W and L for the equations

Define the diaphragm chord. Say the chord is the entire width of FP3-1, longitudinal layers only.

$d_{chord} := 4.92 \cdot 12 = 59.04$ in. $\quad t_{chord} := 4 \cdot \frac{t_1}{in} = 5.52$ in. $\quad A_{chord} := d_{chord} \cdot t_{chord} = 325.9$ in.^2

$E := 1400$ ksi

Deflection Due to Bending

$\delta_b := \frac{5 \cdot v \cdot L^3}{8 \cdot E \cdot A_{chord} \cdot W} = 0.079$ in.

Deflection Due to Shear

Estimate shear stiffness based on Nordic panels 244-7s

$G_v t_v := 4.18 \cdot 10^6 \frac{lbf}{ft}$ $v := v \cdot \frac{lbf}{ft}$ $L := L \cdot ft$ $\delta_v := \frac{v \cdot L}{4 \cdot G_v t_v} = 0.002\ in$

Deflection Due to Fastener Slip at Panel to Panel Joints

$W := 7.875$ ft. $L := 30$ ft. $C := 0.5 \cdot \left(\frac{1}{L} + \frac{1}{W}\right) = 0.08$

$S := 12\ in$ Assumed fastener spacing $F_{fastener} := v \cdot S = 76.58\ lbf$

$\gamma := \left(\frac{180000}{2}\right) \cdot \left(\frac{D}{in}\right)^{1.5} = 15911.4$ $\gamma := \gamma \cdot \frac{lbf}{in}$ Note 1/2 of load slip modulus used (Ref. 6)

$e_n := \frac{F_{fastener}}{\gamma} = 0.005\ in$ $\delta_{fs} := C \cdot L \cdot \frac{e_n}{in} = 0.012$ in.

$\delta_{total\_dia} := \delta_b + \frac{\delta_v}{in} + \delta_{fs} = 0.092$ Very stiff in this direction

So from Ref. 4 Section 4.1.7.2 a diaphram may be consider ridgid if the diaphragm deflection is less than or equal to 2 times the average story deflection of the shear walls.

$\delta_{total\_dia} = 0.092$ in. is <<<< $2 \cdot \delta_{sw\_attic\_avg} = 0.568\ in$ therefore the diaphragm can be considered ridgid in the E-W direction

Design the 2nd floor diaphragm in North-South direction

Note that the wall lines A and B shear connection, and panel-to-panel splices designed in this section will be utilized for the attic, first floor and garage 2nd floor as well as the 2nd floor.

Although based on the attic diaphragm analysis the 2nd floor can be considered rigid, there's little benefit in putting the effort into a rigid diaphragm analysis for the North-South direction because the East and West walls are similar in stiffness. The lateral wind load will distribute fairly evenly between the two walls due to the lack of torsion; therefore, the loads can be distributed as they would with a flexible diaphragm approach.

Additionally, note that the stair opening detailing will be reviewed in this section.

Determine shear loads to walls

Wind Loading (MWFRS - See Tedds output in Appendix B)

Building Length: $l_b := 36\ ft$ Building width: $W_b := 30\ ft$

1st Story Height (center-to-center): $h_{1st} := 8.60\ ft$

2nd Story Height (center-to-center): $h_{2nd} := 8.625\ ft$

Case I

$p_A := 14.70\ psf$ Windward Wall Pressure $p_B := 4.42\ psf$ Leeward Wall Pressure

$\omega_{2nd\_NS} := \left(\left(0.5 \cdot h_{1st} + 0.5 \cdot h_{2nd}\right) \cdot \left(p_A + p_B\right)\right) = 164.67\ plf$ $R_{2nd} := \omega_{2nd\_NS} \cdot l_b \cdot 0.5 = 2964.08\ lbf$

$l_{sw18} := 29.71\ ft$ $l_{sw19} := 16.87\ ft$ $l_{sw20} := 10.17\ ft$

Specify the bracket fasteners along Wall lines A and B connecting the 1st story shear walls to the bottom of the 2nd story floor panels.

$\omega_{attic\_NS} = 200.52\ plf$ From previous calculations

$R_{attic} := \omega_{attic\_NS} \cdot l_b \cdot 0.5 = 3609.31\ lbf$

$v_{r\_A} := LF_{wind} \cdot \left(\dfrac{R_{2nd} + R_{attic}}{l_{sw18}}\right) = 132.75\ plf$ $v_{r\_B} := LF_{wind} \cdot \left(\dfrac{R_{2nd} + R_{attic}}{l_{sw19} + l_{sw20}}\right) = 145.86\ plf$

$P_{allow} := 980\ lbf$ Ref. 21 Pg 31

$n := \dfrac{P_{allow}}{v_{r\_B}} = 6.72\ ft$

Say brackets spaced at 6' O.C maximum, minimum 2 per panel and on ends of panel. Same applies for the 2nd story wall base connection to the 2nd story floor and the garage walls to ceiling connections. The same fastener and pattern can be utilized for the attic gable end walls as well.

Check attic splice design and modify if necessary.

Note that the wind magnitude calculated for the attic diaphragm in the N-S direction is actually greater than that of the 2nd floor; therefore, the attic diaphragm loading will be checked.

$v_{diaphragm} := LF_{wind} \cdot \left(\dfrac{R_{attic}}{W_b}\right) = 72.19\ plf$ $Z_{star} = 129.13\ lbf$ From previous calculations

$S_{required} := \left(\frac{4.5 \cdot Z_{star}}{RF \cdot v_{diaphragm}} \right) = 4.02\ ft$

Space 5/16x5 1/2" Ecofast fasteners at a maximum of 48" O.C. both sides of splice. This splice can be used for all splices. Note that the length of the fastener can most likely be shortened if needed; however, the shear strength will need to be recomputed. Also in addition to the larger Ecofast screws,consider using smaller erection screws at tighter spacing to better align and assemble panel splices.

Design the 2nd Floor Chords

Because of the 1st floor wall joints, the panel edges along the perimeter of the 2nd floor will be utilized as chords for this floor. Assume that the continuous edges along wall lines A and B are sufficient to act as chords. Design the chords and the chord splices along lines 1 and 2.

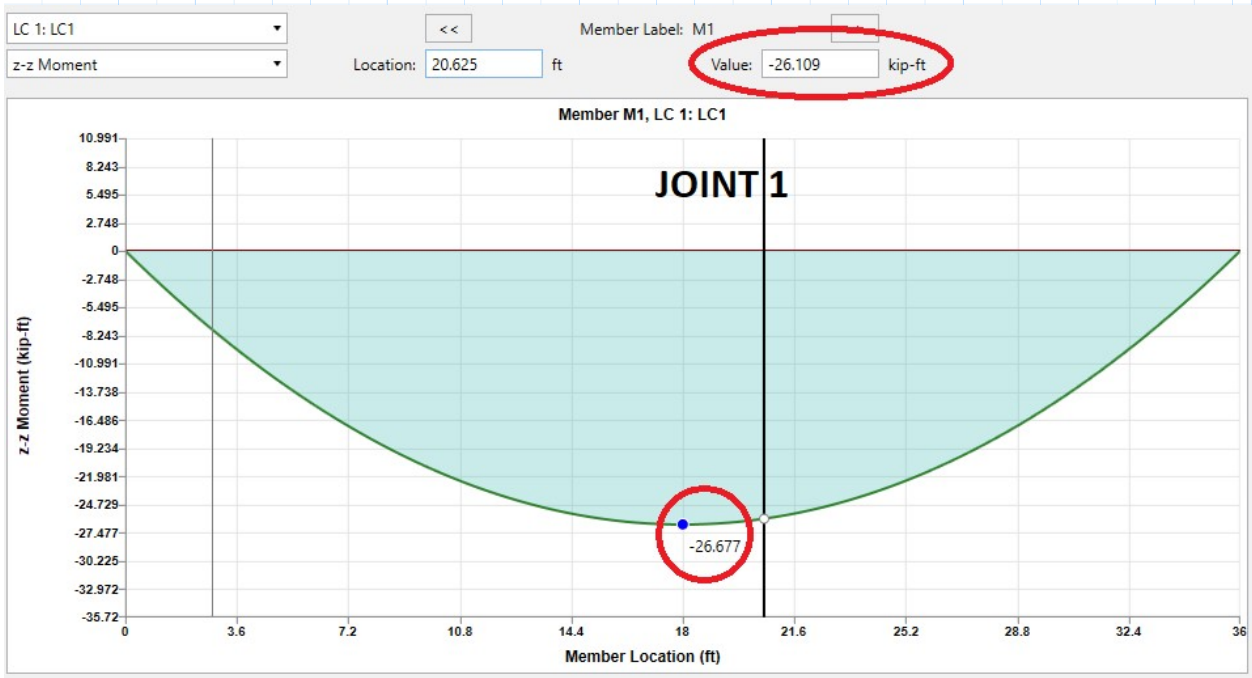


Figure 17. 2nd Story Diaphragm Moment Diagram, wind in N-S direction

$M_{max} := 26.7\ kip \cdot ft$ $\quad P_{max} := LF_{wind} \cdot \Omega_w \cdot \frac{M_{max}}{W_b} = 801\ lbf$

Use the two edge plies of the middle two layers running parallel to lines 1 and 2 as the chord

Check Tension For the K5-0690 Panels:

$t := 1.38\ in$ $\quad b := 5.25\ in$ $\quad A_{parallel} := 2\ t \cdot b = 14.49\ in^2$ $\quad F_t := 250\ psi$ $\quad C_d := 1.6$

$F_tA_{parallel}' := C_d \cdot F_t \cdot A_{parallel} = 5796\ lbf$ $\quad > P_{max} = 801\ lbf$; therefore OK

Design a plywood tension splice to be installed along the exterior edge of the floor panels. Rabbit a recess for the plywood such that it can be installed flush with the edge of the CLT panel. Alternatively, steel prefabricated straps can be design to transfer the tension forces across the joint; however in this case it would be difficult to inspect the installation becauase the wall above will obscure the installation.

$C_s := 0.5$ Ref. 2 Section 9.3.4 $d_{splice} := t \cdot 3 = 4.14\ in$ $t_{layer} := 1.38\ in$

$t := \frac{15}{32}\ in$ 32/16 span rating, 3-ply panel

$F_tA := 1250\ \frac{lbf}{ft}$ Ref. 23 Table M9.2-2, stress perpindicular to the strength axis

$F_tA' := F_tA \cdot d_{splice} \cdot C_s \cdot C_d = 345\ lbf$ < $P_{max} = 801\ lbf$, NO GOOD for stress perpindicular to the strength axis installation. The 4 and 5-ply strengths are not significantly greater, try installation for stress parallel to strength axis.

$F_tA := 3400\ \frac{lbf}{ft}$ Ref. 23 Table M9.2-2, 24/16 (7/16") panel

$F_tA' := F_tA \cdot d_{splice} \cdot C_s \cdot C_d = 938.4\ lbf$ > $P_{max} = 801\ lbf$

OK; however, the strenth of the connection would be dependent on the contractor knowing and installing the plywood in the proper orientation. This is an opportunity for error. Investigate steel straps installed on the edge of the panels.

The straps could be investigated for installation on the panel walking surface; however, the strap would have to be inspected prior to 2nd story wall installation. The walls would obscure the installation. Additionally, it may be difficult to transfer tension reliably from the lower ply to the strap. Essentially, reliance on the adhesive bond between the CLT layers would be required.

Use (2) Simpson Strong-Tie LSTA9 straps,one per chord ply. $T_{allow} := 635\ lbf$

$P_{max} = 801\ lbf$ < $2 \cdot T_{allow} = 1270\ lbf$; therefore OK. Use (8) total 0.148"x2.5" nails.

Minimum edge distance is: $D := 0.148\ in$, $Edge_{min} := 2.5 \cdot D = 0.37\ in$

$Edge_{min} = 0.37\ in$ < $\frac{t_{layer}}{2} = 0.69\ in$; therefore, OK.

Check Compression of Chord Member

Compression member fully braced in-plane by floor stiffness and out-of-plane by walls above and below.

$C_d = 1.6$ $F'_c := 650\ psi$ Ref. 8, Transverse member

$C_p := 1.0$ Fully braced in all directions

$f_c := \frac{P_{max}}{A_{parallel}} = 55.28\ psi$ <<< $F'_c = 650\ psi$; therefore OK.

Check Effects of Opening in 2nd Floor Diaphragm (Ref. Plans 2nd Floor Opening Drawing)

Ref. 11 provides guidance on determining whether an diaphargm opening size is significant and warrants analysis.

Detailed analysis typically not required if all four of the following are true (Ref. 11):

1. Depth of opening no greater than 15% of diaphragm depth.
2. Length of opening no greater than 15% of diaphragm length.
3. Distance from diaphragm edge to opening edge less than 3 times the larger opening dimension.
4. Diaphragm portion on all sides of opening satisfies the maximum aspect ratio requirements.

Verify that the opening is significant and warrants further analysis

Diaphragm Dimensions

$W := 30\ ft$ $L := 36\ ft$ $L_1 := 12.78\ ft$ $L_2 := 3.19\ ft$ $L_3 := 20.03\ ft$

$W_1 := 6.55\ ft$ $W_2 := 7.15\ ft$ $W_3 := 16.30\ ft$

Investigate Point 1

Is $L_2 = 3.19\ ft$ < $0.15 \cdot L = 5.4\ ft$, No, therefore not significant

Investigate Point 2

Is $W_2 = 7.15\ ft$ > $0.15 \cdot W = 4.5\ ft$, Yes, therefore significant

Investigate Point 3

Is $W_1 = 6.55\ ft$ < $3 \cdot W_2 = 21.45\ ft$, No, therefore significant

Investigate Point 4 (Ref. Appendix A 3/S-102)

$AR_1 := \frac{L_2}{W_1} = 0.49$ $AR_2 := \frac{L_2}{W_3} = 0.2$ $AR_3 := \frac{W_2}{L_1} = 0.56$ $AR_4 := \frac{W_2}{L_3} = 0.36$

Does all 4 diaphram portions satisfy the assumed maximum aspect ration of 4:1? Yes, therefore, not significant.

Conclusion is that 2 of the 4 points are untrue; therefore it is assumed that the opening is large enough to have an affect on the diaphagm performance. Analysis of the stresses in the diaphragm near the opening is beyond the scope of this project; however a simplified analysis follows that provides an estimate of the tension forces at the corners that needs to be transferred. between sub-diaphragms.

Assume that the reentrant FP2-3 panel corners 3 and 4 are OK to resist the internal diaphragm transfer force at the opening; however, corners 1 and 2, which are at the joint will require tension straps to transfer the load between the panels. Determine the tension forces and design the straps.

Utilize a simplified method used for evaluating openings in concrete diaphragms and in steel beam webs to determine the secondary chord forces present at the top and bottom of the opening. Assumptions are:

1. Rigid diaphragm behavior.
2. Unbalance shear forces in subpanels adjacent the opening create a secondary moment which is resloved into tension and compression chord forces.
3. Point of contraflexure is estimated at midpoint of opening.

Shear at the center of the opening identified in Figure 18.

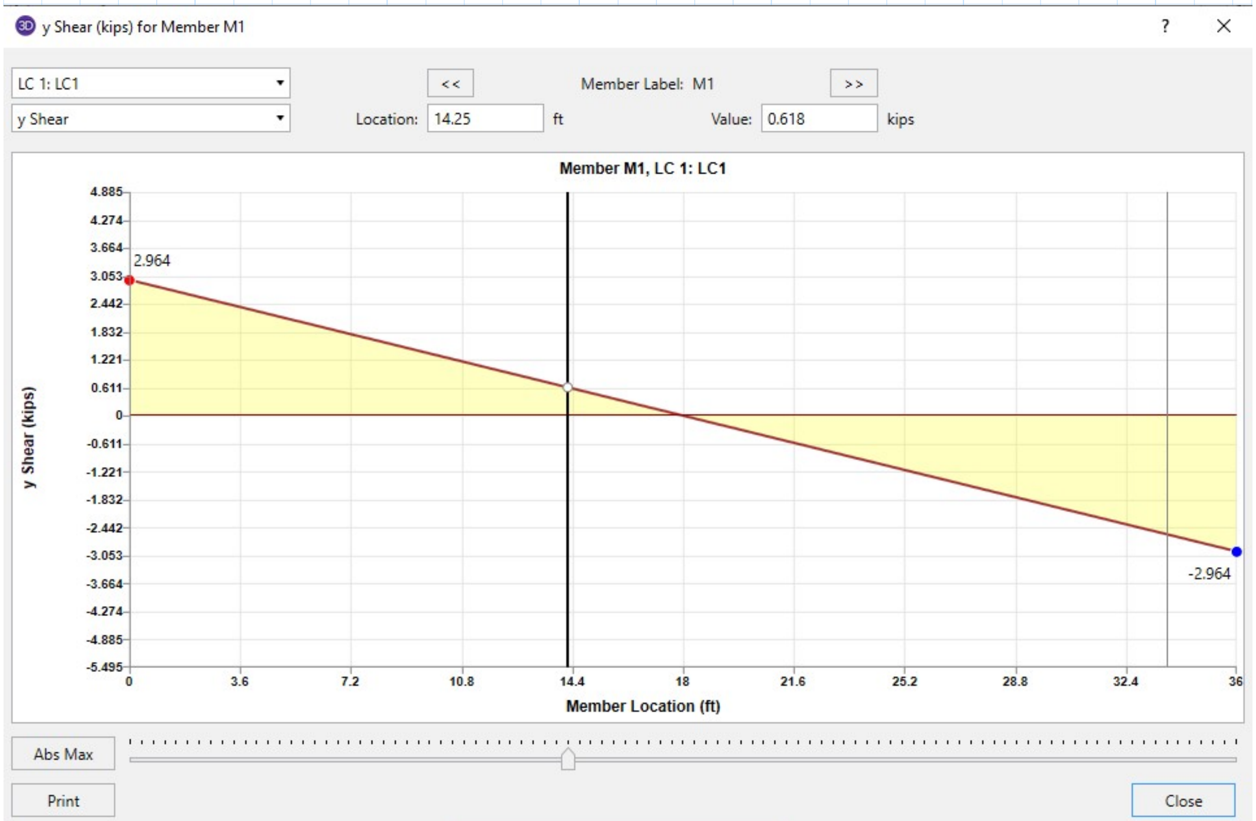


Figure 18. 2nd Story Diaphragm shear at opening centerline

$V_x := \Omega_w \cdot LF_{wind} \cdot 0.618\ kip = 556.2\ lbf$

Divide shear porportional to panel depth above and below the opening.

$V_b := V_x \cdot \frac{W_1}{W_1 + W_3} = 159.44\ lbf$ $\quad V_t := V_x \cdot \frac{W_3}{W_1 + W_3} = 396.76\ lbf$ $\quad V_t + V_b = 556.2\ lbf$

Maximum moment at end of subpanels resolved into a Tension force

$$T_{sub\_chord} := \max\left(\frac{V_t \cdot 0.5 \cdot L_2}{W_3}, \frac{V_b \cdot 0.5 \cdot L_2}{W_1}\right) = 38.82\ lbf$$

Very minimal tension resistance is required at the opening. The splice connection should be able to transfer the shear adequately. Add two additional ecofast fasteners to the joint at corners 1 and 2 to improve connection. The same detailing can be used for the 1st floor opening as well. See Figure 19 .

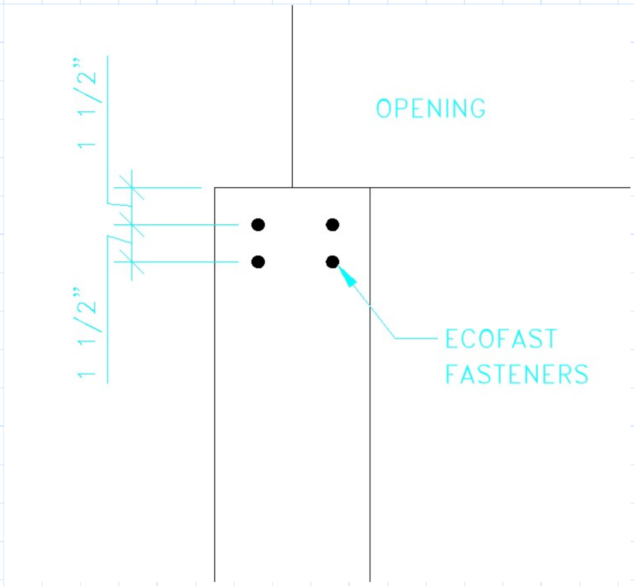


Figure 19. 2nd-Story diaphragm splice at opening detail

Check aspect ratio (Ref. 4 App B.3.1.2)

$l_{sw5} := 3.5\ ft$ $\quad l_{sw6} := 3.75\ ft$ $\quad l_{sw7} := l_{sw6}$ $\quad l_{sw8} := l_{sw5}$ $\quad l_{sw12} := 4.46\ ft$ $\quad l_{sw13} := 4.25\ ft$

$l_{sw14} := 6.88\ ft$ $\quad l_{sw15} := 5.75\ ft$ $\quad l_{sw18} := 29.71\ ft$ $\quad l_{sw19} := 16.88\ ft$ $\quad l_{sw20} := 10.17\ ft$

$l_{min} := min\left(l_{sw5}, l_{sw6}, l_{sw7}, l_{sw8}, l_{sw12}, l_{sw13}, l_{sw14}, l_{sw15}, l_{sw18}, l_{sw19}, l_{sw20}\right) = 3.5\ ft$

$H = 8\ ft$ $\quad \frac{H}{l_{min}} = 2.29$ < 4; therefore OK, all walls can be utilized.

Determine Center of Rigidity (Ref. 17 Section 16.9)

$l_{wall\_1} := l_{sw12} + l_{sw13} + l_{sw14} + l_{sw15} = 21.34\ ft$ $\quad l_{wall\_2} := l_{sw5} + l_{sw6} + l_{sw7} + l_{sw8} = 14.5\ ft$

$l_{wall\_A} := l_{sw18} = 29.71\ ft$ $\quad l_{wall\_B} := l_{sw19} + l_{sw20} = 27.05\ ft$

$$e_x := \frac{-l_{wall\_A} \cdot \frac{L}{2} + l_{wall\_B} \cdot \frac{L}{2}}{l_{wall\_A} + l_{wall\_B}} = -0.84\ ft \qquad e_y := \frac{-l_{wall\_2} \cdot \frac{W}{2} + l_{wall\_1} \cdot \frac{W}{2}}{l_{wall\_1} + l_{wall\_2}} = 2.86\ ft$$

$$r_1 := \frac{W}{2} - e_y = 12.14\ ft \qquad r_2 := \frac{W}{2} + e_y = 17.86\ ft \qquad r_A := \frac{L}{2} + e_x = 17.16\ ft \qquad r_B := \frac{L}{2} - e_x = 18.84\ ft$$

Distribute Direct Shear Forces

$$v_r := \frac{P_{2nd}}{l_{wall\_1} + l_{wall\_2}} = 133.53\ plf \qquad R_1 := v_r \cdot l_{wall\_1} = 2849.45\ lbf \qquad R_2 := v_r \cdot l_{wall\_2} = 1936.13\ lbf$$

Distribute Torsional Shear Forces

k = Stiffness, which in this case is equivalent to wall length.
r = Linear distance from wall line to center of rigidity.
Line = Wall line

$$A := \begin{bmatrix} \text{“Line”} & \text{“k”} & \text{“r”} \\ \text{“1”} & l_{wall\_1} & r_1 \\ \text{“2”} & l_{wall\_2} & r_2 \\ \text{“A”} & l_{wall\_A} & r_A \\ \text{“B”} & l_{wall\_B} & r_B \end{bmatrix} \qquad B := \begin{bmatrix} \text{“line”} & \text{“k(r)”} & \text{“k(r^2)”} \\ \text{“1”} & A_{1,1} \cdot A_{1,2} & A_{1,1} \cdot \left(A_{1,2}\right)^2 \\ \text{“2”} & A_{2,1} \cdot A_{2,2} & A_{2,1} \cdot \left(A_{2,2}\right)^2 \\ \text{“A”} & A_{3,1} \cdot A_{3,2} & A_{3,1} \cdot \left(A_{3,2}\right)^2 \\ \text{“B”} & A_{4,1} \cdot A_{4,2} & A_{4,1} \cdot \left(A_{4,2}\right)^2 \end{bmatrix}$$

$$J := \sum \left(\text{submatrix}\left(B, 1, 4, 2, 2\right)\right) = 26120.14\ ft^3 \qquad V_r(kr) := \frac{P_{2nd} \cdot e_y \cdot kr}{J}$$

$V_{r\_t1} := V_r\left(B_{1,1}\right) = 135.85\ lbf$ $V_{r\_t2} := V_r\left(B_{2,1}\right) = 135.85\ lbf$

$V_{r\_tA} := V_r\left(B_{3,1}\right) = 267.34\ lbf$ $V_{r\_tB} := V_r\left(B_{4,1}\right) = 267.34\ lbf$

Neglect torsional forces on wall A and B. The wind loading from the N-S direction will produce large shear loadings. Add torsional shear loads to direct shear loads for Wall lines 1 and 2.

$R_1 := R_1 - V_{r\_t1} = 2713.61\ lbf$ $R_2 := R_2 + V_{r\_t2} = 2071.98\ lbf$

$R_{1\_attic} := 2756.76\ lbf$ $R_{2\_attic} := 2365.17\ lbf$

$v_{r1} := \dfrac{R_1 + R_{1\_attic}}{l_{wall\_1}} = 256.34\ plf$ $v_{r2} := \dfrac{R_2 + R_{2\_attic}}{l_{wall\_2}} = 306.01\ plf$

Based on previous calculations it will be assumed that the remaining CLT elements have adequate strength.

Check shear bracket requirements for the top and bottom 1st story shearwalls

Determine maximum bracket load

$V_{sw6} := v_{r2} \cdot l_{sw6} = 1147.54\ lbf$ $V_{sw14} := v_{r1} \cdot l_{sw14} = 1763.64\ lbf$

$V_{asd} := LF_{wind} \cdot \max\left(V_{sw6}, V_{sw14}\right) = 1058.19\ lbf$ $V_{allow} := 980\ lbf$

$V_{asd} = 1058.19\ lbf < 2 \cdot V_{allow} = 1960\ lbf$; therefore OK

Two brackets are OK for the most heavily load shear wall on the floor; therefore OK for all. Use two ABR9020 brackets per shear panel (one on each end).

Assume panel compressive resistance for overturning is adequate based on the results of the 2nd floor panel calculations. Check to see if uplift resistance required at the ground flooor. Consider the dead load of the panels only (construction load case). Consider load combination 0.6D+0.6W.

$\sigma_{self\_410} = 9.84\ psf$ $\sigma_{self\_690} = 16.4\ psf$ $\sigma_{self\_970} = 23.06\ psf$ $\sigma_{self\_4.125} := 11.1\ psf$

Check line 2. Summation of the moments about point B. Negative indicates no tension present.

$$\omega_{d\_line\_2} := 18.5\ ft \cdot \sigma_{self\_410} + \frac{18.56}{2}\ ft \cdot \sigma_{self\_970} + \frac{13.92}{2}\ ft \cdot \sigma_{self\_690} + 2 \cdot H \cdot \sigma_{self\_4.125} = 687.74\ plf$$

$$W_{d\_5} := 0.6 \cdot \omega_{d\_line\_2} \cdot l_{sw5} = 1444.25\ lbf \qquad W_{d\_6} := 0.6 \cdot \omega_{d\_line\_2} \cdot l_{sw6} = 1547.41\ lbf$$

$$P_{w\_sw5} := 0.6 \cdot v_{r2} \cdot l_{sw5} = 642.62\ lbf \qquad P_{w\_sw6} := 0.6 \cdot v_{r2} \cdot l_{sw6} = 688.52\ lbf$$

$$T_5 := \frac{P_{w\_sw5} \cdot H - W_{d\_5} \cdot \frac{l_{sw5}}{2}}{l_{sw5}} = 746.72\ lbf \qquad T_6 := \frac{P_{w\_sw6} \cdot H - W_{d\_6} \cdot \frac{l_{sw6}}{2}}{l_{sw6}} = 695.14\ lbf$$

Check Wall Line 1.

Use the lever rule to estimate the amount of attic floor load transmitted through the interior bearing partition and the 2nd floor system to the 1st floor walls.

$$\omega_{int\_attic} := \left(\sigma_{self\_970} \cdot \left(\frac{11.44}{2}\ ft + \frac{18.56}{2}\ ft\right) + H \cdot \sigma_{self\_4.125}\right) \cdot \frac{4.64\ ft}{16.08\ ft} + \sigma_{self\_690} \cdot \frac{18.56}{2}\ ft = 277.61\ plf$$

$$\omega_{d\_line\_1} := 18.5\ ft \cdot \sigma_{self\_410} + \frac{11.44}{2}\ ft \cdot \sigma_{self\_970} + \omega_{int\_attic} + 2 \cdot H \cdot \sigma_{self\_4.125} = 769.13\ plf$$

$$W_{d\_12} := 0.6 \cdot \omega_{d\_line\_1} \cdot l_{sw12} = 2058.18\ lbf \qquad W_{d\_13} := 0.6 \cdot \omega_{d\_line\_1} \cdot l_{sw13} = 1961.27\ lbf$$

$$P_{w\_sw12} := 0.6 \cdot v_{r1} \cdot l_{sw12} = 685.97\ lbf \qquad P_{w\_sw13} := 0.6 \cdot v_{r1} \cdot l_{sw13} = 653.68\ lbf$$

$$T_{12} := \frac{P_{w\_sw12} \cdot H - W_{d\_12} \cdot \frac{l_{sw12}}{2}}{l_{sw12}} = 201.36\ lbf \qquad T_{13} := \frac{P_{w\_sw13} \cdot H - W_{d\_13} \cdot \frac{l_{sw13}}{2}}{l_{sw13}} = 249.81\ lbf$$

$$W_{d\_14} := 0.6 \cdot \omega_{d\_line\_1} \cdot l_{sw14} = 3174.96\ lbf \qquad W_{d\_15} := 0.6 \cdot \omega_{d\_line\_1} \cdot l_{sw15} = 2653.49\ lbf$$

$$P_{w\_sw14} := 0.6 \cdot v_{r1} \cdot l_{sw14} = 1058.19\ lbf \qquad P_{w\_sw15} := 0.6 \cdot v_{r1} \cdot l_{sw15} = 884.38\ lbf$$

$$T_{14} := \frac{P_{w\_sw14} \cdot H - W_{d\_14} \cdot \frac{l_{sw14}}{2}}{l_{sw14}} = -357.03\ lbf \qquad T_{15} := \frac{P_{w\_sw15} \cdot H - W_{d\_15} \cdot \frac{l_{sw15}}{2}}{l_{sw15}} = -96.3\ lbf$$

Nearly all panel segments indicate uplift. Design anchorage of segment ends.

Note, however, that these calculations are conservative. Firstly, the panels are rigid bodies and likely transfer a bigger tributary loading to the individual panel segments than just the tributary directly above the segments. Secondly, the load combination under evaluation, reduces the dead load by a factor of 0.60. This is conservative since we can accurately calculate the dead load. A higher load factor could be justified. Additionally, evaluating the panel dead load only is a condition that would only occur during construction. The design wind speed is based on a 700-YR MRI. For the temporary construction case, the MRI can likely be lowered (at the discretion of the engineer) and the wind speed reduced, thus reducing the magnitude of the lateral loading.

Determine the amount of nails required for the maximum shear load = the maximum uplift force.

Compute lateral capacity of single nail, 0.148" diameter, 1 1/2" length (Ref. 2 Section 12.3)

From Table 12.3.3

$t_1 := 1.38\ in$ (Parallel to Grain) $t_2 := 1.38\ in$ (Perp to Grain) $t_3 := 1.38\ in$ (Parallel to Grain)

$D := 0.148\ in$ $l := 1.5\ in$ $l_s := 0.0625\ in$ $E := 2 \cdot D = 0.296\ in$

$G = 0.42$ $F_e := 0.67 \cdot 3350\ psi = 2244.5\ psi$ Ref. 14 6.2.2

$F_{es} := 33\ ksi$ $l_m := t_1 + (l - l_s - t_1 - E) = 1.14\ in$ $F_{em} := F_e = 2244.5\ psi$ $K_d := 2.2$ $D_r := D$

Yield limit equation variables

$R_{d\_1m} := K_d$ $R_{d\_1s} := K_d$ $R_{d\_2} := K_d$

$R_{d\_3s} := K_d$ $R_{d\_4} := K_d$ $R_{d\_3m} := K_d$

$R_e := \frac{F_{em}}{F_{es}} = 0.07$ $R_t := \frac{l_m}{l_s} = 18.26$

$$k_1 := \frac{\sqrt{R_e + 2 \cdot R_e^{\,2}\left(1 + R_t + R_t^{\,2}\right) + R_t^{\,2} \cdot R_e^{\,3}} - R_e \cdot (1 + R_t)}{(1 + R_e)} = 0.509$$

$$k_2 := -1 + \sqrt{2 \cdot (1 + R_e) + \frac{2 \cdot F_{yb} \cdot (1 + 2 \cdot R_e) \cdot D_r^{\,2}}{3 \cdot F_{em} \cdot l_m^{\,2}}} = 0.729$$

$$k_3 := -1 + \sqrt{\frac{2 \cdot (1 + R_e)}{R_e} + \frac{2 \cdot F_{yb} \cdot (2 + R_e) \cdot D_r^{\,2}}{3 \cdot F_{em} \cdot l_s^{\,2}}} = 22.425$$

Yield limit equations

$Z_{1m} := \frac{D_r \cdot l_m \cdot F_{em}}{R_{d\_1m}} = 172.36\ lbf$ $Z_{1s} := \frac{D_r \cdot l_s \cdot F_{es}}{R_{d\_1s}} = 138.75\ lbf$

$Z_2 := \frac{k_1 \cdot D_r \cdot l_s \cdot F_{es}}{R_{d\_2}} = 70.65\ lbf$ $Z_{3m} := \frac{k_2 \cdot D_r \cdot l_m \cdot F_{em}}{(1 + 2 \cdot R_e) \cdot R_{d\_3m}} = 110.54\ lbf$

$$Z_{3s} := \frac{k_3 \cdot D_r \cdot l_s \cdot F_{em}}{(2+R_e) \cdot R_{d\_3s}} = 102.33 \ lbf \qquad Z_4 := \frac{D_r^{\ 2}}{R_{d\_4}} \cdot \sqrt{\frac{2 \cdot F_{em} \cdot F_{yb}}{3 \cdot (1+R_e)}} = 144.43 \ lbf$$

$C_d := 1.6$

$n := 10 \qquad Z' := min(Z_{1m}, Z_{1s}, Z_2, Z_{3m}, Z_{3s}, Z_4) \cdot C_d \cdot n = 1130.35 \ lbf$

Apply overstrength factor per Ref. 4 Section B.3.4.3

$Z' = 1130.35 \ lbf$ > $\Omega_w \cdot T_5 = 1120.09 \ lbf$; therefore, OK.

Use Simpson Strong Tie HTP37Z strap using 10 0.148"x1 1/2" nails per side.
Minimum edge distance per Ref. 2 Fig. 12I = $3 \cdot D = 0.44 \ in$

Review global overturning (See Figure 20). Consider 0.6D+0.6W load combination and panel dead load only. Check for N-S wind direction. Estimate grade at the top of foundation wall. From Appendix Z wind calculations:

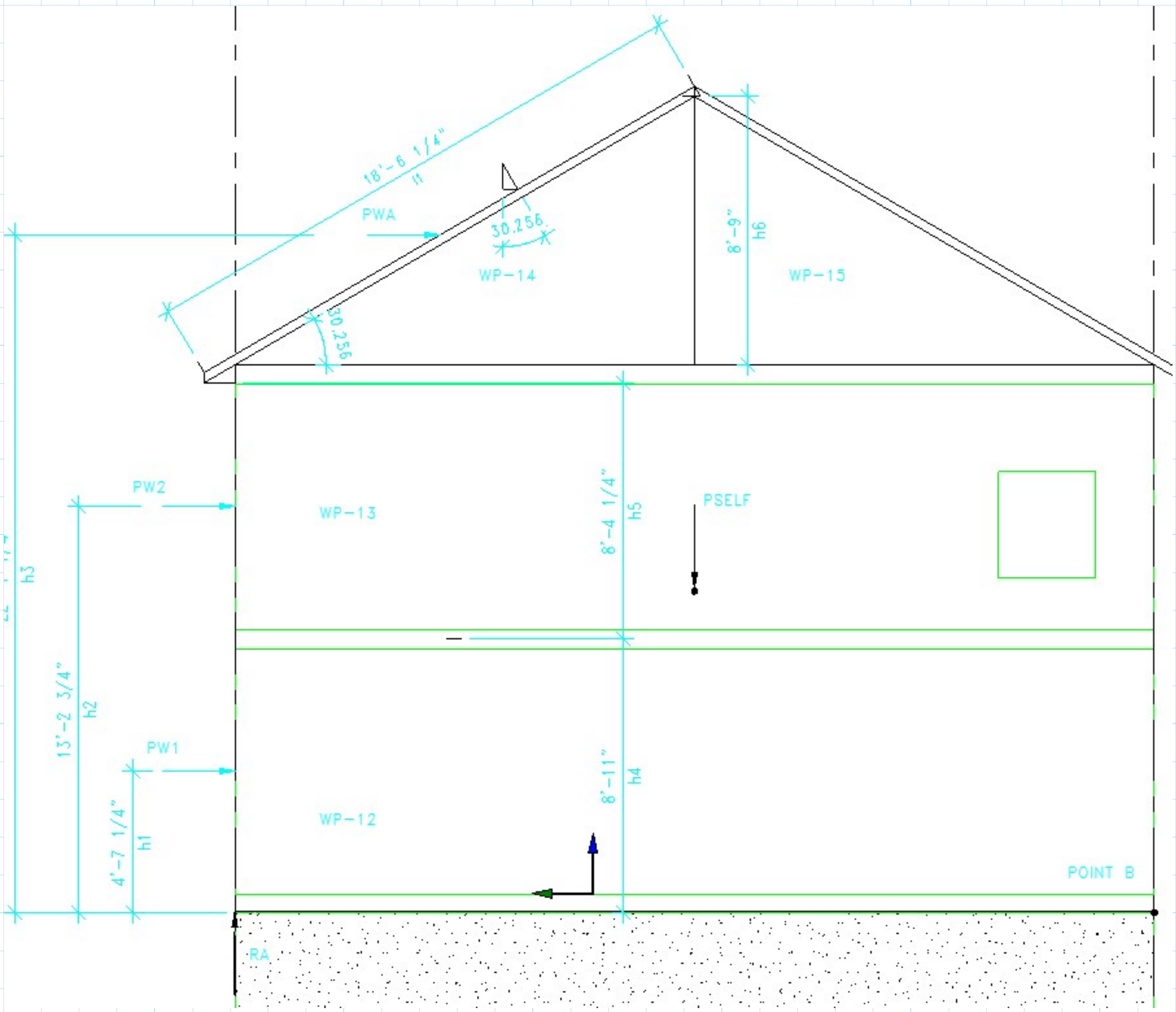


Figure 20. West Elevation; wind loading for global overturning analyis

Geometry

$L=36\ ft$ $W=30\ ft$ $\theta:=30.256\ deg$ $l_1:=18.52\ ft$ $h_1:=4.60\ ft$ $h_2:=13.23\ ft$

$h_3:=22.10\ ft$ $h_4:=8.92\ ft$ $h_5:=8.35\ ft$ $h_6:=8.75\ ft$ $l_{oh}:=1\ ft$

Roof Wind Load (Case II)

$p_{windward}:=6.36\ psf$ $p_{leeward}:=-5.95\ psf$ $p_{horizontal}:=\sin(\theta)\cdot p_{windward}-\sin(\theta)\cdot p_{leeward}=6.2\ psf$

$P_{WA}:=LF_{wind}\cdot p_{horizontal}\cdot(L+2\cdot l_{oh})\cdot l_1=2619.07\ lbf$

Wall Load (Case II)

$p_{windward}:=14.70\ psf$ $p_{leeward}:=-4.42\ psf$ $p_{total}:=p_{windward}-p_{leeward}=19.12\ psf$

$P_{W1}:=LF_{wind}\cdot p_{total}\cdot L\cdot h_4=3683.89\ lbf$ $P_{W2}:=LF_{wind}\cdot p_{total}\cdot L\cdot h_5=3448.48\ lbf$

Weight of Panels

$P_{walls}:=(6\cdot H\cdot L+4\cdot H\cdot W+W\cdot h_6)\cdot\sigma_{self\_4.125}=32750.55\ lbf$

$P_{roof}:=2\cdot(L+2\cdot l_{oh})\cdot l_1\cdot\sigma_{self\_410}=13850.12\ lbf$

$P_{floor}:=2\cdot L\cdot W\cdot\sigma_{self\_690}+L\cdot W\cdot\sigma_{self\_970}=60324.04\ lbf$

$P_{self}:=(P_{walls}+P_{roof}+P_{floor})\cdot 0.6=64154.83\ lbf$

Summation of Forces about Point B (Couterclockwise is Positive)

$$R_A:=\frac{P_{self}\cdot\frac{W}{2}-P_{W1}\cdot h_1-P_{W2}\cdot h_2-P_{WA}\cdot h_3}{W}=28062.39\ lbf$$

The overturning estimate indicates no overturning. Note this is only an estimate to determine if further analysis is warranted. The a more detailed analysis would require an interior bearing point. The reaction calculated in this estimate is large and compresseive, no further analyis is required. Estimate available frictional shear resistance.

$\mu_s:=0.50$ $F_f:=\mu_s\cdot P_{self}=32077.41\ lbf$

$F_f=32077.41\ lbf$ >> $P_{WA}+P_{W1}+P_{W2}=9751.44\ lbf$; No further analysis required. Assume frictional resistance is adequate to transfer shear to foudation. Provide minimum foundation anchorage for the purposes of providing positive connection and to prevent accidental displacement of the framing.

Garage Component Specification

For the garage, assume that wind loading in the E-W direction is transferred from windward/ leeward walls and roof panels to the main house 2nd floor diaphragm through the floor/ceiling panels above the garage. The wind loading in the N-S direction is transferred through the windward/leeward garage roof an wall panels through the garage floor/ceiling assembly to shear walls along wall lines B and C.

Diaphragm splice details and shear wall connections specified for the main house can be applied to the garage with minor modification if necessary. Two additional details are required. The connection of the floor/ceiling assembly to the wall along line B and wall connection to the foundation along wall lines 1A, 1B and C.

Ref. Appendix A for details.

Foundation System Review

The goal of this section is estimate foundation element sizes so they can be compared to the foundation design in Ref.1 for identification of difference that could affect cost.

References:

1. Timber Buildings and Sustainability; Intech Open, edited by Giovanna Concu
2. AISC Steel Manual 14th ed.
3. ACI 318-14
4. ACI 332-08

Calculate interior girder size

2nd floor and above wall loading taken from the reaction of the 2nd floor panel design. Openings in the interior walls are neglected for this calculation and WP2-2, for simplification purposes, is assumed to be aligned with WP2-1.

$\sigma_{self\_410} = 9.84\ psf$ $\sigma_{self\_690} = 16.4\ psf$ $\sigma_{self\_970} = 23.06\ psf$ $\sigma_{self\_4.125} := 11.1\ psf$

$\sigma_{collateral\_f} = 5\ psf$ $\sigma_{live} := 40\ psf$

$\omega_{d\_2nd} := 973\ plf$ $\omega_{l\_2nd} := 1438\ plf$

$$\omega_d := \omega_{d\_2nd} + \sigma_{self\_4.125} \cdot H + \sigma_{self\_690} \cdot \left(\frac{16.08}{2}\ ft + \frac{13.92}{2}\ ft\right) = 1307.8\ plf$$

$$\omega_l := \omega_{l\_2nd} + \sigma_{live} \cdot \left(\frac{16.08}{2}\ ft + \frac{13.92}{2}\ ft\right) = 2038\ plf$$

Size beam in Tedds (See Appendix C)

W8x18 is adequate for strength; however, the column bearing must be evaluated per Ref. 2 specification section J10

Single concentrated (compressive) load

$P_D := 13.2\ kip$ $P_L := 20.2\ kip$ $P_u := 1.2\ P_D + 1.6 \cdot P_L = 48.16\ kip$

Member Properties (W8x18)

$F_{yw} := 50\ ksi$ $t_w := 0.230\ in$ $t_f := 0.330\ in$ $E_s := 29000\ ksi$ $k := 0.630\ in$ $d := 8.14\ in$

Web Local Yielding

$\phi_{WLY} := 1.0 \qquad l_b := 4\ in$

$\phi R_{n\_WLY} := \phi_{WLY} \cdot F_{yw} \cdot t_w \cdot (5 \cdot k + l_b) = 82.23\ kip \quad \geq P_u = 48.16\ kip$ => OK

Web Crippling

$\phi_{WC} := 0.75$

$\phi R_{n\_WC} := \phi_{WC} \cdot 0.80 \cdot t_w^{\ 2} \left(1 + 3 \cdot \left(\frac{l_b}{d}\right) \cdot \left(\frac{t_w}{t_f}\right)^{1.5}\right) \cdot \sqrt{\frac{E_s \cdot F_{yw} \cdot t_f}{t_w}} = 85.05\ kip \geq P_u = 48.16\ kip$ => OK

Web Sidesway Buckling

$\phi_{WSB} := 0.85 \qquad h := d - 2 \cdot k = 6.88\ in \quad b_f := 5.25\ in \qquad S_{xx} := 15.2\ in^3 \qquad C_r := 960000\ ksi$

$L_b := 8.75\ ft$ Assume interior bearing points restrained against rotation. Add bracing.

$F_1 := \frac{\left(\frac{h}{t_w}\right)}{\left(\frac{L_b}{b_f}\right)} = 1.5$ $\leq$ 2.3=> J10-6 Applies

$\phi R_{n\_WSB} := \phi_{WSB} \cdot \left(\frac{C_r \cdot t_w^{\ 3} \cdot t_f}{h^2}\right) \cdot \left(1 + 0.4 \cdot F_1^{\ 3}\right) = 161.85\ kip \quad \geq P_u = 48.16\ kip$ => OK

Web Compression Buckling

$\phi_{WCB} := 0.90$

$\phi R_{n\_WCB} := \frac{24 \cdot t_w^{\ 3} \cdot \sqrt{E_s \cdot F_{yw}}}{h} = 51.11\ kip \quad \geq P_u = 48.16\ kip$ => OK

W8x18 passes all applicable concentrated force checks. Specify adjustable interior column. See Figure 1.

$P_a := P_D + P_L = 33400\ lbf \qquad \leq P_{allow} := 33800\ lbf$ => OK

AC3580216	8'0" – 8'4"	34,700
AC3583216	8'3" – 8'7"	33,800
AC3586216	8'6" – 8'10"	32,800

Figure 1. Marshall Stamping Co. capacities for 3.5" diameter 0.216" wall thickness.

Use a Marshall adjustable 3.5" column (wall thickness = 0.216"). Note this column is at the limit of it's capacity. Also, note that diagonal bracing must be installed at each interior column location to ensure compliance with the web sidesway buckling check. See Figure 2 for generalized bracing detail.

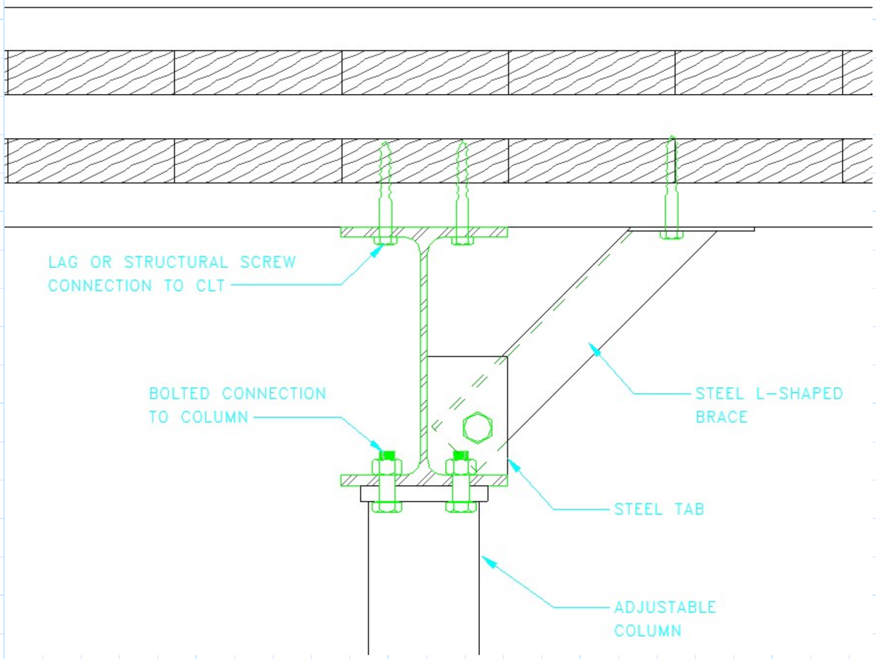


Figure 2. Girder bearing/connection bracing detail

Check CLT Bearing Capacity at Girder Bearing

$P_D := 1307.8\ lbf$ $P_L := 2038\ lbf$ $F_{c\_perp} := 425\ psi$ $C_b := 1.0$

$A := b_f \cdot 12\ in = 63\ in^2$ $F'_{c\_perp} := F_{c\_perp} \cdot C_b = 425\ psi$

$P_D + P_L = 3345.8\ lbf$ < $F'_{c\_perp} \cdot A = 26775\ lbf$; therefore, bearing OK.

Pad footings sized in Tedd's see Appendix C.

4.5'x4.5'x1.0' required for bearings B and D, 4.0'x4.0'x1.0' required for bearing C.

Size wall footings. Design based on Line 1 loading. Loads from CLT floor and roof panel design reactions. See Appendix C.

$\omega_{d\_r} := 500\ plf$ $\omega_{s\_r} := 400\ plf$ $\omega_{d\_a} := 42\ plf$ $\omega_{l\_a} := 152\ plf$

$\omega_{d\_2} := 236\ plf$ $\omega_{l\_2} := 406\ plf$ $\omega_{d\_1} := 136\ plf$ $\omega_{l\_1} := 289\ plf$

$\sigma_{self\_4.125} := 11.1\ psf$ $\gamma_{con} := 150\ pcf$ $H := 8\ ft$

$\omega_d := \omega_{d\_r} + \omega_{d\_a} + \omega_{d\_2} + \omega_{d\_1} + 2 \cdot H \cdot \sigma_{self\_4.125} + \gamma_{con} \cdot 9\ ft \cdot 8\ in = 1991.6\ plf$

$\omega_l := \omega_{l\_a} + \omega_{l\_2} + \omega_{l\_1} = 847\ plf$

$\omega_s := \omega_{s\_r} = 400\ plf$

Design a plain concrete strip footing (Ref 3. Chapter 22)

Bearing pressure and preliminary footing size calculated in Tedds (See Appendix C)

Check the 24 inch wide by 10" deep footing computed by Tedds; however, reduce footing thickness to 8" to align with Ref. 1 and conventional residential footing depths.

Neglect one-way shear check. Projections roughly equivalent to footing depth, therefore shear is not a concern.

Check moment capacity

$h := 10\ in$ $\quad d := h - 2\ in = 8\ in$ $\quad b := 24\ in$ $\quad S := \dfrac{b \cdot d^2}{6} = 256\ in^3$ $\quad t_{wall} := 8\ in$

$Proj := \dfrac{b - t_{wall}}{2} = 8\ in$ $\quad \phi := 0.9$ $\quad f'c := 3000$ PSI

$q_u := \dfrac{1.2 \cdot \omega_d + 1.6 \cdot \omega_l + 0.5 \cdot \omega_s}{b} = 1972.56\ psf$ $\quad \phi M_n := \phi \cdot 7.5 \cdot \sqrt{f'c}\ psi \cdot S = 7.89\ kip \cdot ft$ Ref. 4

$M_u := q_u \cdot Proj \cdot 1\ ft \cdot \left(\dfrac{Proj}{2}\right) = 0.44\ kip \cdot ft$

$M_u = 0.44\ kip \cdot ft << \phi M_n = 7.89\ kip \cdot ft$; therefore OK.

24"x8" Plain footing OK. Add #4 longitudinal bars to span soil discontinuities and aid with resistance to temperature and shrinkage cracking. Add #3 transverse bars at 24" O.C. for ties to support longitudinal bars

8" Concrete walls OK. Same specification as Ref. 1

Appendix C – Supplementary Design Calculations

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N2	0	0	0	
2	N3	6	0	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot [k-ft/rad]	Y Rot [k-ft/rad]	Z Rot [k-ft/rad]
1	ALL	Reaction	Reaction	Reaction			
2	N2	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction
3	N3	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.163	-0.163	0	3.167
2	M1	Y	-0.977	-0.843	3.167	%100

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.241	-0.241	0	3.167
2	M1	Y	-0.583	-0.523	3.167	%100

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.56	-0.462	3.167	%100

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.152	-0.125	3.167	%100

Load Combinations

	Description	Solve	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	ASD_1	Yes	DL	1	LL	1				
2	ASD_2	Yes	DL	1	SL	1				
3	ASD_3	Yes	DL	1	LL	0.75	SL	0.75	WL	0.45

Load Combination Design

	Description	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
1	ASD_1		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
2	ASD_2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
3	ASD_3		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

Basic Load Cases

	BLC Description	Category	Y Gravity	Distributed
1	Self	DL	-1	
2	DSI	DL		2
3	Floor Live	LL		2
4	Snow	SL		1
5	Wind_Positive	WL		1

Member Section Forces

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
1	1	M1	1	0	1.765	0	0	0	2.131
2			2	0	1.149	0	0	0	-0.054
3			3	0	0.533	0	0	0	-1.315
4			4	0	-1.563	0	0	0	-0.653
5			5	0	-3.699	0	0	0	3.313
6	2	M1	1	0	1.142	0	0	0	1.577
7			2	0	0.888	0	0	0	0.055
8			3	0	0.633	0	0	0	-1.086
9			4	0	-1.38	0	0	0	-0.656
10			5	0	-3.439	0	0	0	2.981
11	3	M1	1	0	1.769	0	0	0	2.263

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
12			2	0	1.244	0	0	0	0.003
13			3	0	0.718	0	0	0	-1.468
14			4	0	-1.802	0	0	0	-0.803
15			5	0	-4.374	0	0	0	3.855

Maximum Member Section Forces

	LC	Member Label		Axial[k]	Loc[ft]	y Shear[k]	Loc[ft]	z Shear[k]	Loc[ft]	Torque[k-ft]	Loc[ft]	y-y Moment[k-ft]	Loc[ft]	z-z Moment[k-ft]	Loc[ft]
1	1	M1	max	0	6	1.765	0	0	6	0	6	0	6	3.313	6
2			min	0	0	-3.699	6	0	0	0	0	0	0	-1.467	3.438
3	2	M1	max	0	6	1.142	0	0	6	0	6	0	6	2.981	6
4			min	0	0	-3.439	6	0	0	0	0	0	0	-1.308	3.563
5	3	M1	max	0	6	1.769	0	0	6	0	6	0	6	3.855	6
6			min	0	0	-4.374	6	0	0	0	0	0	0	-1.698	3.5

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N2	0	0	0	
2	N3	6	0	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot [k-ft/rad]	Y Rot [k-ft/rad]
1	ALL	Reaction	Reaction	Reaction		
2	N2	Reaction	Reaction	Reaction		
3	N3		Reaction		Reaction	Reaction

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.163	-0.163	0	3.167
2	M1	Y	-0.977	-0.843	3.167	%100

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.241	-0.241	0	3.167
2	M1	Y	-0.583	-0.523	3.167	%100

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.56	-0.462	3.167	%100

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.152	-0.125	3.167	%100

Load Combinations

	Description	Solve	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	ASD_1	Yes	DL	1	LL	1				
2	ASD_2	Yes	DL	1	SL	1				
3	ASD_3	Yes	DL	1	LL	0.75	SL	0.75	WL	0.45

Load Combination Design

	Description	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
1	ASD_1		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
2	ASD_2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
3	ASD_3		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

Basic Load Cases

	BLC Description	Category	Y Gravity	Distributed
1	Self	DL	-1	
2	DSI	DL		2
3	Floor Live	LL		2
4	Snow	SL		1
5	Wind_Positive	WL		1

Member Section Forces

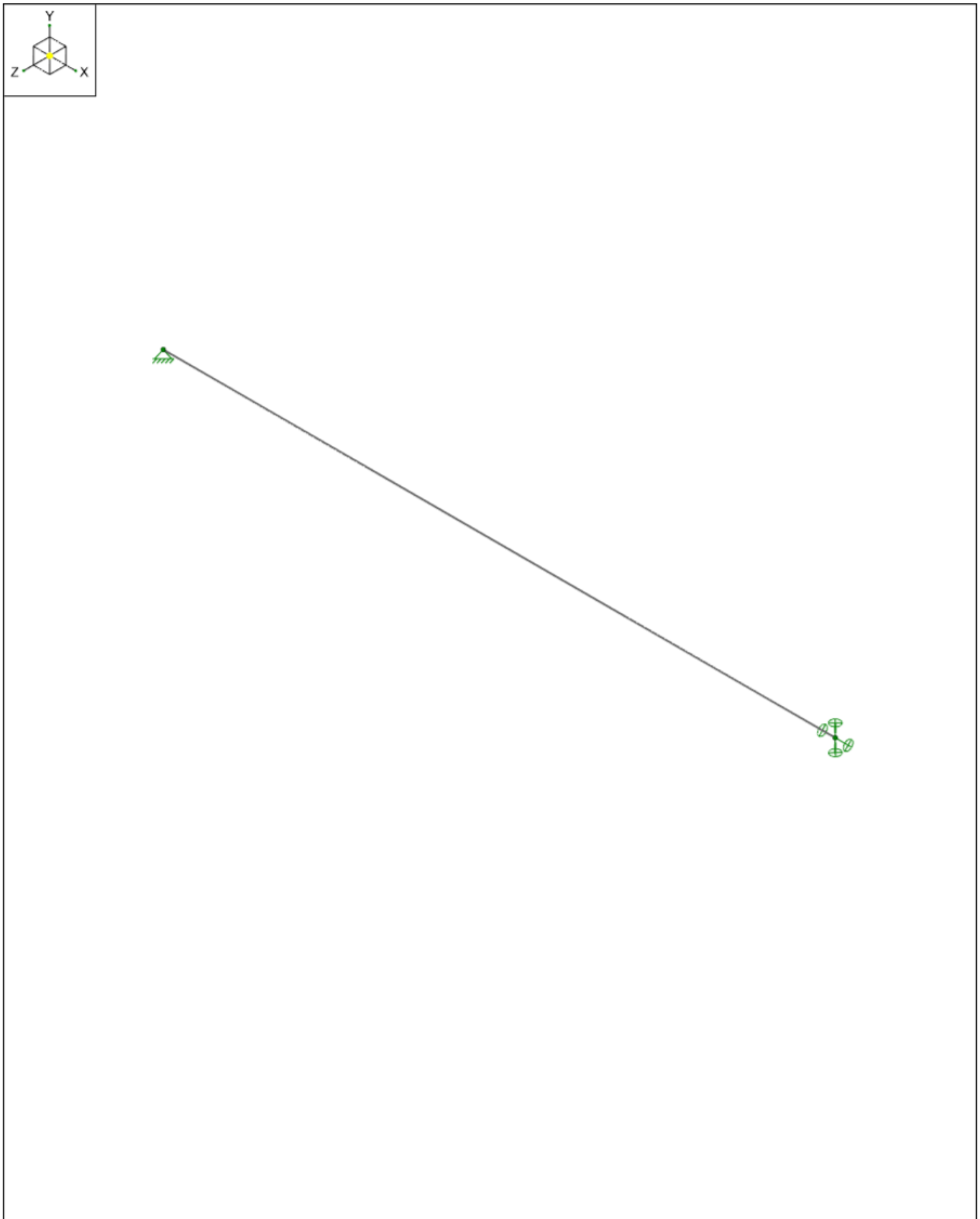
	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
1	1	M1	1	0	1.962	0	0	0	0
2			2	0	1.346	0	0	0	-2.481
3			3	0	0.73	0	0	0	-4.037
4			4	0	-1.366	0	0	0	-3.67
5			5	0	-3.502	0	0	0	0
6	2	M1	1	0	1.376	0	0	0	0
7			2	0	1.122	0	0	0	-1.873
8			3	0	0.867	0	0	0	-3.365
9			4	0	-1.146	0	0	0	-3.286
10			5	0	-3.205	0	0	0	0
11	3	M1	1	0	2.035	0	0	0	0

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
12			2	0	1.509	0	0	0	-2.658
13			3	0	0.984	0	0	0	-4.527
14			4	0	-1.537	0	0	0	-4.261
15			5	0	-4.109	0	0	0	0

Maximum Member Section Forces

	LC	Member Label		Axial[k]	Loc[ft]	y Shear[k]	Loc[ft]	z Shear[k]	Loc[ft]	Torque[k-ft]	Loc[ft]	y-y Moment[k-ft]	Loc[ft]	z-z Moment[k-ft]	Loc[ft]
1	1	M1	max	0	6	1.962	0	0	6	0	6	0	6	0	6
2			min	0	0	-3.502	6	0	0	0	0	0	0	-4.293	3.563
3	2	M1	max	0	6	1.376	0	0	6	0	6	0	6	0	6
4			min	0	0	-3.205	6	0	0	0	0	0	0	-3.737	3.688
5	3	M1	max	0	6	2.035	0	0	6	0	6	0	6	0	6
6			min	0	0	-4.109	6	0	0	0	0	0	0	-4.912	3.688



JES		SK-1
ACJ		Aug 05, 2021
		B-3_Simple.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N2	0	0	0	
2	N3	3	0	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot [k-ft/rad]	Y Rot [k-ft/rad]	Z Rot [k-ft/rad]
1	ALL	Reaction	Reaction	Reaction			
2	N2	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction
3	N3	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.977	-0.843	0	%100

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.583	-0.523	0	%100

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.56	-0.462	0	%100

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.152	-0.125	0	%100

Load Combinations

	Description	Solve	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	ASD_1	Yes	DL	1	LL	1				
2	ASD_2	Yes	DL	1	SL	1				
3	ASD_3	Yes	DL	1	LL	0.75	SL	0.75	WL	0.45
4	SERVICE_1	Yes	LL	1						
5	SERVICE_2	Yes	SL	1						

Load Combination Design

	Description	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
1	ASD_1		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
2	ASD_2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
3	ASD_3		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
4	SERVICE_1		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
5	SERVICE_2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

Basic Load Cases

	BLC Description	Category	Y Gravity	Distributed
1	Self	DL	-1	
2	DSI	DL		1
3	Floor Live	LL		1
4	Snow	SL		1
5	Wind_Positive	WL		1

Member Section Forces

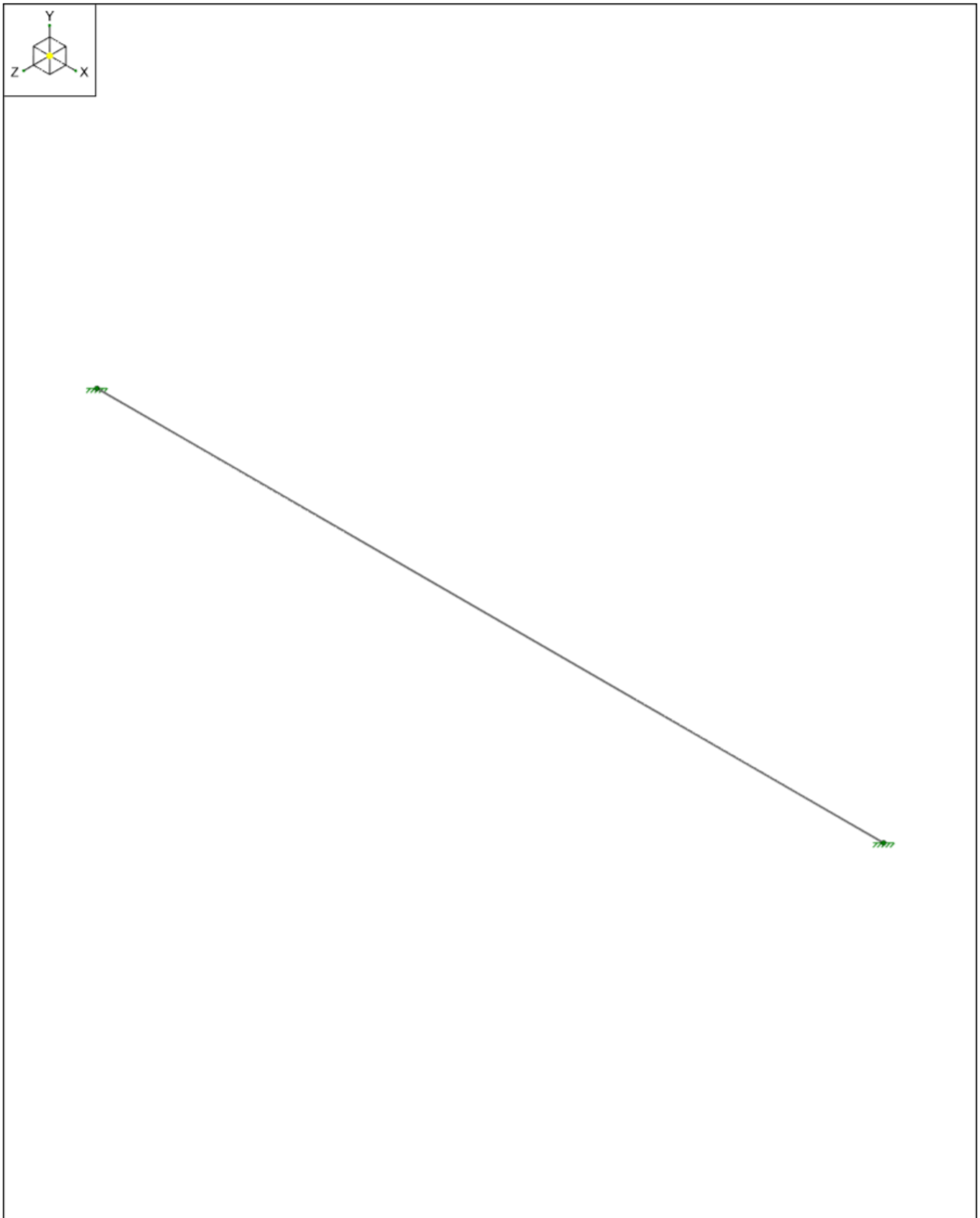
	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
1	1	M1	1	0	2.266	0	0	0	1.119
2			2	0	1.108	0	0	0	-0.144
3			3	0	-0.015	0	0	0	-0.552
4			4	0	-1.1	0	0	0	-0.132
5			5	0	-2.15	0	0	0	1.089
6	2	M1	1	0	2.215	0	0	0	1.09
7			2	0	1.077	0	0	0	-0.142
8			3	0	-0.017	0	0	0	-0.536
9			4	0	-1.068	0	0	0	-0.126

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
10			5	0	-2.075	0	0	0	1.055
11	3	M1	1	0	2.748	0	0	0	1.354
12			2	0	1.339	0	0	0	-0.175
13			3	0	-0.02	0	0	0	-0.667
14			4	0	-1.33	0	0	0	-0.158
15			5	0	-2.59	0	0	0	1.315
16	4	M1	1	0	0.847	0	0	0	0.419
17			2	0	0.416	0	0	0	-0.054
18			3	0	-0.005	0	0	0	-0.207
19			4	0	-0.414	0	0	0	-0.05
20			5	0	-0.811	0	0	0	0.41
21	5	M1	1	0	0.796	0	0	0	0.391
22			2	0	0.385	0	0	0	-0.051
23			3	0	-0.007	0	0	0	-0.192
24			4	0	-0.381	0	0	0	-0.045
25			5	0	-0.737	0	0	0	0.376

Maximum Member Section Forces

	LC	Member Label		Axial[k]	Loc[ft]	y Shear[k]	Loc[ft]	z Shear[k]	Loc[ft]	Torque[k-ft]	Loc[ft]	y-y Moment[k-ft]	Loc[ft]	z-z Moment[k-ft]	Loc[ft]
1	1	M1	max	0	3	2.266	0	0	3	0	3	0	3	1.119	0
2			min	0	0	-2.15	3	0	0	0	0	0	0	-0.552	1.5
3	2	M1	max	0	3	2.215	0	0	3	0	3	0	3	1.09	0
4			min	0	0	-2.075	3	0	0	0	0	0	0	-0.536	1.5
5	3	M1	max	0	3	2.748	0	0	3	0	3	0	3	1.354	0
6			min	0	0	-2.59	3	0	0	0	0	0	0	-0.667	1.5
7	4	M1	max	0	3	0.847	0	0	3	0	3	0	3	0.419	0
8			min	0	0	-0.811	3	0	0	0	0	0	0	-0.207	1.5
9	5	M1	max	0	3	0.796	0	0	3	0	3	0	3	0.391	0
10			min	0	0	-0.737	3	0	0	0	0	0	0	-0.192	1.5



JES		SK-2
ACJ		Aug 05, 2021
		B-6.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N2	0	0	0	
2	N3	16.1	0	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot [k-ft/rad]
1	ALL	Reaction	Reaction	Reaction	
2	N2	Reaction	Reaction	Reaction	
3	N3		Reaction	Reaction	Reaction

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.316	-0.316	0	%100

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.325	-0.325	0	%100

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.28	-0.28	0	%100

Load Combinations

	Description	Solve	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	ASD_1	Yes	DL	1	LL	1				
2	ASD_2	Yes	DL	1	SL	1				
3	ASD_3	Yes	DL	1	LL	0.75	SL	0.75	WL	0.45
4	SERVICE_1	Yes	LL	1						
5	SERVICE_2	Yes	SL	1						

Load Combination Design

	Description	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
1	ASD_1		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
2	ASD_2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
3	ASD_3		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
4	SERVICE_1		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
5	SERVICE_2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

Basic Load Cases

	BLC Description	Category	Y Gravity	Distributed
1	Self	DL	-1	
2	DSI	DL		1
3	Floor Live	LL		
4	Snow	SL		1
5	Wind_Positive	WL		1

Member Section Forces

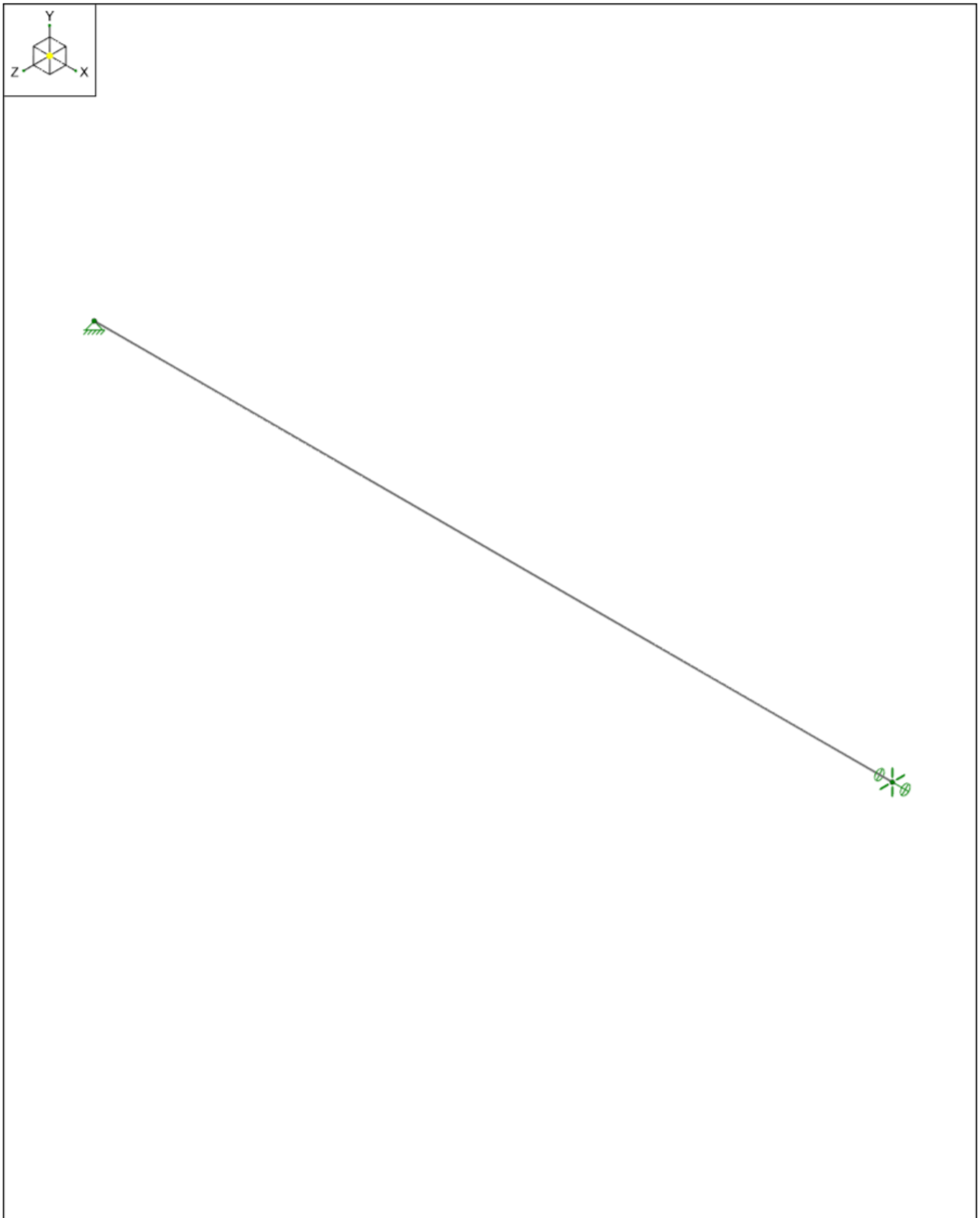
	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
1	1	M1	1	0	2.699	0	0	0	0
2			2	0	1.349	0	0	0	-8.147
3			3	0	0	0	0	0	-10.863
4			4	0	-1.349	0	0	0	-8.147
5			5	0	-2.699	0	0	0	0
6	2	M1	1	0	5.315	0	0	0	0
7			2	0	2.658	0	0	0	-16.045
8			3	0	0	0	0	0	-21.393
9			4	0	-2.658	0	0	0	-16.045
10			5	0	-5.315	0	0	0	0
11	3	M1	1	0	5.675	0	0	0	0
12			2	0	2.838	0	0	0	-17.132
13			3	0	0	0	0	0	-22.843
14			4	0	-2.838	0	0	0	-17.132

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
15			5	0	-5.675	0	0	0	0
16	4	M1	1	0	0	0	0	0	0
17			2	0	0	0	0	0	0
18			3	0	0	0	0	0	0
19			4	0	0	0	0	0	0
20			5	0	0	0	0	0	0
21	5	M1	1	0	2.616	0	0	0	0
22			2	0	1.308	0	0	0	-7.898
23			3	0	0	0	0	0	-10.53
24			4	0	-1.308	0	0	0	-7.898
25			5	0	-2.616	0	0	0	0

Maximum Member Section Forces

	LC	Member Label		Axial[k]	Loc[ft]	y Shear[k]	Loc[ft]	z Shear[k]	Loc[ft]	Torque[k-ft]	Loc[ft]	y-y Moment[k-ft]	Loc[ft]	z-z Moment[k-ft]	Loc[ft]
1	1	M1	max	0	16.1	2.699	0	0	16.1	0	16.1	0	16.1	0	16.1
2			min	0	0	-2.699	16.1	0	0	0	0	0	0	-10.863	8.05
3	2	M1	max	0	16.1	5.315	0	0	16.1	0	16.1	0	16.1	0	16.1
4			min	0	0	-5.315	16.1	0	0	0	0	0	0	-21.393	8.05
5	3	M1	max	0	16.1	5.675	0	0	16.1	0	16.1	0	16.1	0	16.1
6			min	0	0	-5.675	16.1	0	0	0	0	0	0	-22.843	8.05
7	4	M1	max	0	16.1	0	16.1	0	16.1	0	16.1	0	16.1	0	16.1
8			min	0	0	0	0	0	0	0	0	0	0	0	0
9	5	M1	max	0	16.1	2.616	0	0	16.1	0	16.1	0	16.1	0	16.1
10			min	0	0	-2.616	16.1	0	0	0	0	0	0	-10.53	8.05



JES		SK-3
ACJ		Aug 05, 2021
		B-2.r3d

CLT Home Design
1st_Floor
Apr. 1, 2021 09:21

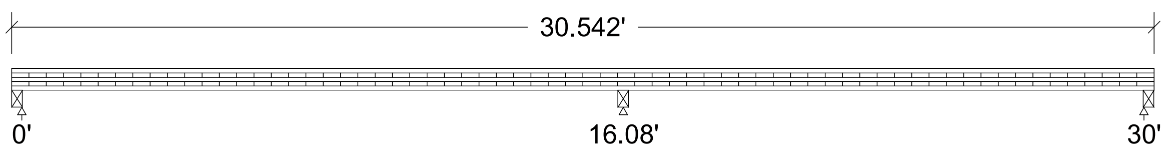
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 1)

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	Location [ft] End	Magnitude Start	Magnitude End	Unit
wd1	Dead	Full Area	No			21.40(1.00')		psf
wl1	Live	Full Area	Yes			40.00(1.00')		psf
wd2	Dead	Partial UDL	No	26.08	30.27	34.1	34.1	plf
wl2	Live	Partial UDL	Yes	26.08	30.27	63.8	63.8	plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:					
Dead	136		433		227
Live	289		810		475
Factored:					
Total	425		1243		703
Bearing:					
Capacity					
Beam	17850		19762		17850
Support	17850		17850		17850
Des ratio					
Beam	0.02		0.06		0.04
Support	0.02		0.07		0.04
Load comb	#3		#2		#4
Length	3.50		3.50		3.50
Min req'd	0.50*		0.50*		0.50*
Cb	1.00		1.11		1.00
Cb min	1.00		1.75		1.00
Cb support	1.00		1.00		1.00
Fcp sup	425		425		425

*Minimum bearing length setting used: 1/2" for end supports

1st Floor Panel

CLT Floor Panel, S-P-F, V2, 5 Layers 6-7/8" (12" width)

Supports: All - Lumber-soft Beam, No.3

Total length: 30.54'; Clear span: 15.913', 13.753'; Volume = 17.5 cu.ft. / ft.; Panel orientation: Longitudinal axis

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	V = 592	Vs' = 2475	lbs	V/Vs' = 0.24
Bending(+)	M = 1478	M' = 4675	lbs-ft	M/M' = 0.32
Bending(-)	M = 1944	M' = 4675	lbs-ft	M/M' = 0.42
Live Defl'n	0.13 = < L/999	0.54 = L/360	in	0.24
Total Defl'n	0.21 = L/913	0.80 = L/240	in	0.26
Vibration	Lmax = 16.063	Lv = 16.938	ft	Lmax/Lv = 0.95

Additional Data:

FACTORS:	F(psi)	CD	CM	Ct	CL	CF	Cfu	Cr	Cfrt	Ci	CLT	LC#
Fs	45	-	1.00	1.00	-	-	-	-	-	-	-	4
Fb+	875	1.00	1.00	1.00	1.000	-	-	-	-	-	0.85	4
Fb-	875	1.00	1.00	1.00	1.000	-	-	-	-	-	0.85	2
Fcp'	425	-	1.00	1.00	-	-	-	-	-	-	-	-
EIapp	311.8 million		1.00	1.00	-	-	-	-	-	-	-	3

CRITICAL LOAD COMBINATIONS:

```
 Shear     : LC #4  = D+L (pattern: _L)
 Bending(+): LC #4  = D+L (pattern: _L)
 Bending(-): LC #2  = D+L
 Deflection: LC #3  =   (live)
             LC #3  =   (total)
 Bearing   : Support 1 - LC #3 = D+L (pattern: L_)
             Support 2 - LC #2 = D+L
             Support 3 - LC #4 = D+L (pattern: _L)
 D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
 All LC's are listed in the Analysis output
 Load Patterns: s=S/2,  X=L+S or L+Lr,  _=no pattern load in this span
 Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.2
```

CALCULATIONS:

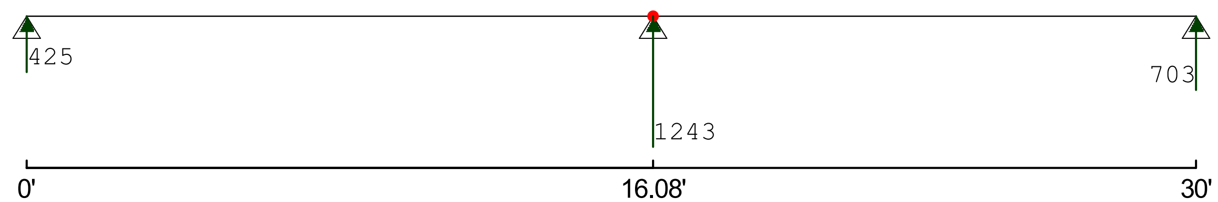
```
 V max = 686, V design = 592 (NDS 3.4.3.1(a)), Vs = 2475 lbs
 Seff,0 = 75.43 in^3; (FbS)eff = 4675 lbs-ft; (GA)eff,0 = 0.91e06 lb
 (EI)eff,0 = 363.00e06; (EI)app' = 311.76e06 lb-in^2
 E = 1400000 psi; G = 87500 psi; E⊥ = 40000 psi; G⊥ = 7500 psi
 "Live" deflection is due to all non-dead loads (live, wind, snow…)
 Total deflection = 2.0 dead + "live"
 (EI)app' for shear deflection is based on Ks = 11.5 for uniform loading on a single
 span and is approximate for other loading conditions.
```

Design Notes:

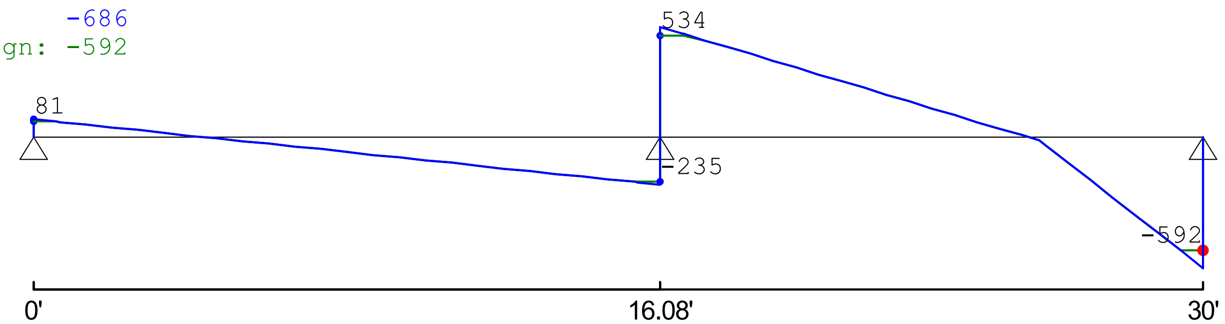
1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2018), the National Design Specification (NDS 2018), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. FIRE RATING: Joists, wall studs, and multi-ply members are not rated for fire endurance.
4. CLT design is according to NDS Ch. 10 and APA PRG 320-19. Where needed for customized lay-ups or fire-reduced sections, 2013 FPInnovations CLT Handbook Chs. 3 and 8, and 2014 CSA O86 Ch. 8 are used. Floor vibration from CSA O86 A.8.5.3.

ANALYSIS DIAGRAMS (known section)

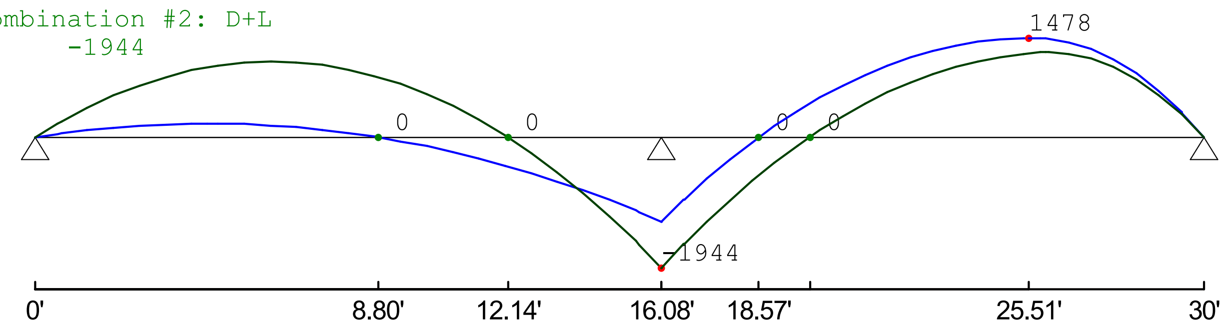
REACTION [lbs]
Maximum...
Uplift: 0
Bearing: 1243 (LC #4)



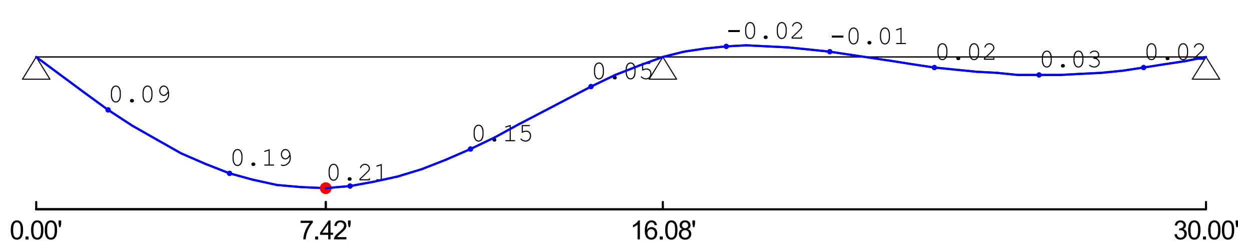
SHEAR [lbs]
Load Combination #4: D+L (pattern: _L)
+V max: 579
-V max: -686
V design: -592



BENDING [lbs-ft]
Load Combination #4: D+L (pattern: _L)
+M max: 1478
Load Combination #2: D+L
-M max: -1944



TOTAL DEFLECTION [in]
Load Combination #3:
Total = 2.00 x Dead + Live (all others)
Critical Live: 0.13
Critical Total: 0.21



CLT Home Design
2nd_Floor
Apr. 1, 2021 10:53

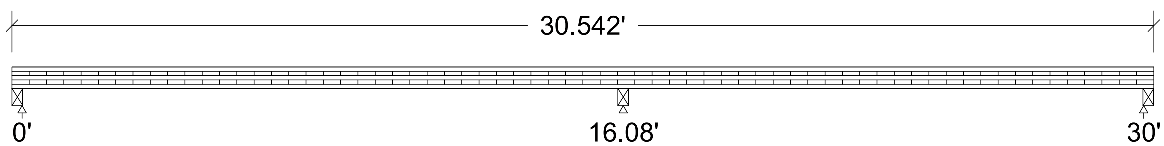
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 1)

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	Location [ft] End	Magnitude Start	Magnitude End	Unit
wd1	Dead	Full Area	No			21.40(1.00')		psf
wl1	Live	Full Area	Yes			40.00(1.00')		psf
wd2	Dead	Partial UDL	No	23.71	30.27	34.1	34.1	plf
wl2	Live	Partial UDL	Yes	23.71	30.27	63.8	63.8	plf
PD1	Dead	Point UDL	No	11.70		544		plf
PL1	Live	Point UDL	Yes	11.70		600		plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:					
Dead	236		973		212
Live	406		1438		557
Factored:					
Total	643		2411		768
Bearing:					
Capacity					
Beam	17850		19762		17850
Support	17850		17850		17850
Des ratio					
Beam	0.04		0.12		0.04
Support	0.04		0.14		0.04
Load comb	#3		#2		#4
Length	3.50		3.50		3.50
Min req'd	0.50*		0.50*		0.50*
Cb	1.00		1.11		1.00
Cb min	1.00		1.75		1.00
Cb support	1.00		1.00		1.00
Fcp sup	425		425		425

*Minimum bearing length setting used: 1/2" for end supports

2nd Floor Panel

CLT Floor Panel, S-P-F, V2, 5 Layers 6-7/8" (12" width)

Supports: All - Lumber-soft Beam, No.3

Total length: 30.54'; Clear span: 15.913', 13.753'; Volume = 17.5 cu.ft. / ft.; Panel orientation: Longitudinal axis

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	V = 1507	Vs' = 2475	lbs	V/Vs' = 0.61
Bending(+)	M = 3193	M' = 4675	lbs-ft	M/M' = 0.68
Bending(-)	M = 3922	M' = 4675	lbs-ft	M/M' = 0.84
Live Defl'n	0.26 = L/736	0.54 = L/360	in	0.49
Total Defl'n	0.58 = L/335	0.80 = L/240	in	0.72
Vibration	Lmax = 16.063	Lv = 16.938	ft	Lmax/Lv = 0.95

Additional Data:

FACTORS:	F(psi)	CD	CM	Ct	CL	CF	Cfu	Cr	Cfrt	Ci	CLT	LC#
Fs	45	-	1.00	1.00	-	-	-	-	-	-	-	2
Fb+	875	1.00	1.00	1.00	1.000	-	-	-	-	-	0.85	3
Fb-	875	1.00	1.00	1.00	1.000	-	-	-	-	-	0.85	2
Fcp'	425	-	1.00	1.00	-	-	-	-	-	-	-	-
EIapp	311.8 million		1.00	1.00	-	-	-	-	-	-	-	3

CRITICAL LOAD COMBINATIONS:

```
Shear     : LC #2  = D+L
Bending(+): LC #3  = D+L (pattern: L_)
Bending(-): LC #2  = D+L
Deflection: LC #3  =   (live)
            LC #3  =   (total)
Bearing   : Support 1 - LC #3 = D+L (pattern: L_)
            Support 2 - LC #2 = D+L
            Support 3 - LC #4 = D+L (pattern: _L)
D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
All LC's are listed in the Analysis output
Load Patterns: s=S/2,  X=L+S or L+Lr,  _=no pattern load in this span
Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.2
```

CALCULATIONS:

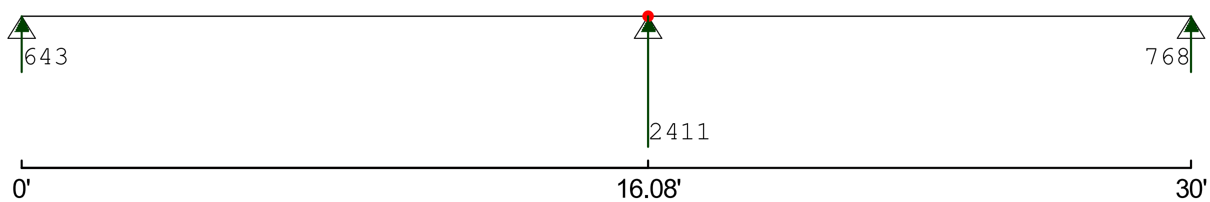
```
V max = 1551, V design = 1507 (NDS 3.4.3.1(a)), Vs = 2475 lbs
Seff,0 = 75.43 in^3; (FbS)eff = 4675 lbs-ft; (GA)eff,0 = 0.91e06 lb
(EI)eff,0 = 363.00e06; (EI)app' = 311.76e06 lb-in^2
E = 1400000 psi; G = 87500 psi; E⊥ = 40000 psi; G⊥ = 7500 psi
"Live" deflection is due to all non-dead loads (live, wind, snow…)
Total deflection = 2.0 dead + "live"
(EI)app' for shear deflection is based on Ks = 11.5 for uniform loading on a single
span and is approximate for other loading conditions.
```

Design Notes:

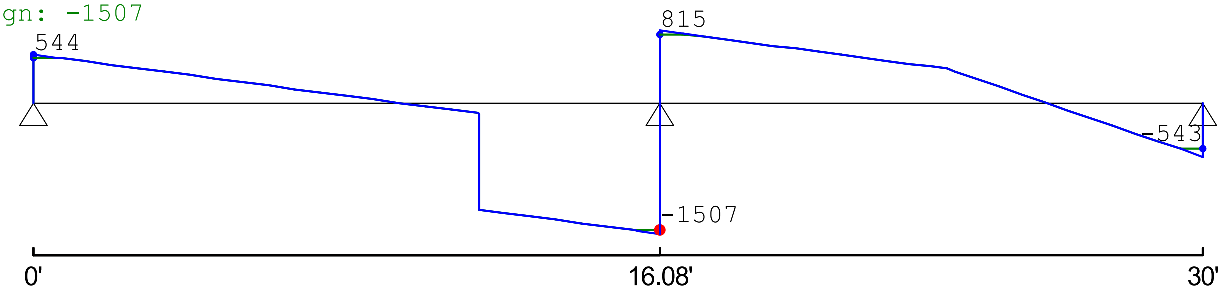
1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2018), the National Design Specification (NDS 2018), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. FIRE RATING: Joists, wall studs, and multi-ply members are not rated for fire endurance.
4. CLT design is according to NDS Ch. 10 and APA PRG 320-19. Where needed for customized lay-ups or fire-reduced sections, 2013 FPInnovations CLT Handbook Chs. 3 and 8, and 2014 CSA O86 Ch. 8 are used. Floor vibration from CSA O86 A.8.5.3.

ANALYSIS DIAGRAMS (known section)

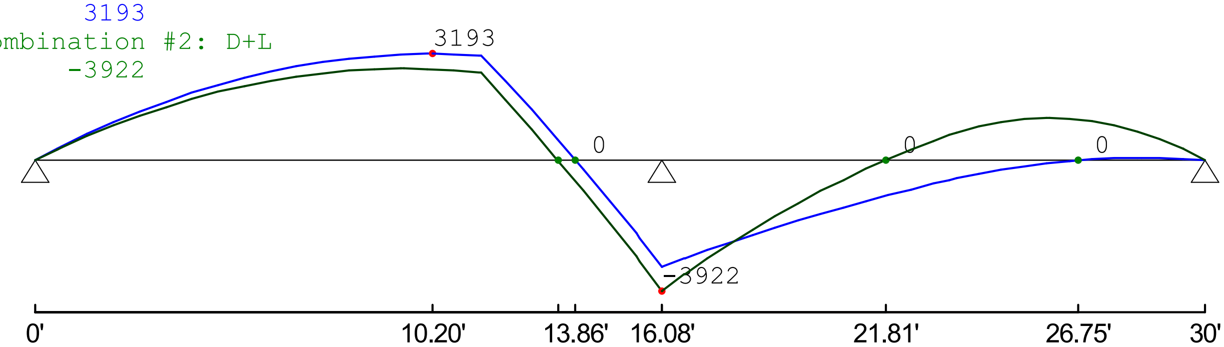
REACTION [lbs]
Maximum...
Uplift: 0
Bearing: 2411 (LC #4)



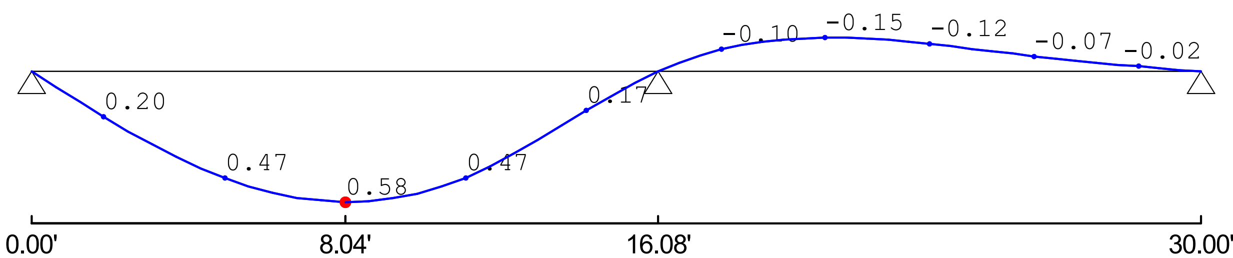
SHEAR [lbs]
Load Combination #2: D+L
+V max: 860
-V max: -1551
V design: -1507



BENDING [lbs-ft]
Load Combination #3: D+L (pattern: L_)
+M max: 3193
Load Combination #2: D+L
-M max: -3922



TOTAL DEFLECTION [in]
Load Combination #3:
Total = 2.00 x Dead + Live (all others)
Critical Live: 0.26
Critical Total: 0.58



CLT Home Design
Attic_Floor
Mar. 22, 2021 15:55

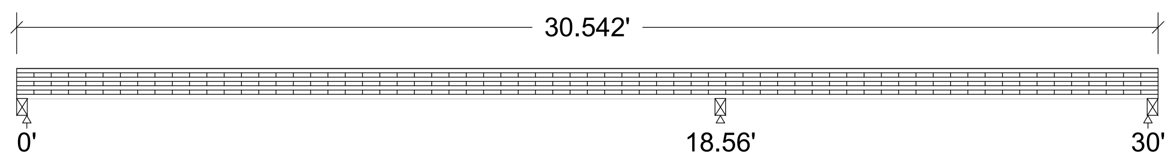
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 1)

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
wd1	Dead	Full Area	No			28.10(1.00')		psf
wl1	Live	Full Area	Yes			40.00(1.00')		psf
wd2	Dead	Partial UDL	No	0.00	6.63	44.7	44.7	plf
wl2	Live	Partial UDL	No	0.00	6.63	63.8	63.8	plf
wd2_2	Dead	Partial UDL	No	13.77	18.56	44.7	44.7	plf
wl2_2	Live	Partial UDL	No	13.77	18.56	63.8	63.8	plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



	0'	18.56'	30'
Unfactored:			
Dead	469	858	42
Live	682	1223	152
Factored:			
Uplift			-106
Total	1150	2082	194
Bearing:			
Capacity			
Beam	17850	19762	17850
Support	17850	17850	17850
Des ratio			
Beam	0.06	0.11	0.01
Support	0.06	0.12	0.01
Load comb	#3	#2	#4
Length	3.50	3.50	3.50
Min req'd	0.50*	0.50*	0.50*
Cb	1.00	1.11	1.00
Cb min	1.00	1.75	1.00
Cb support	1.00	1.00	1.00
Fcp sup	425	425	425

*Minimum bearing length setting used: 1/2" for end supports

Attic Floor Panel

CLT Floor Panel, S-P-F, V2, 7 Layers 9-5/8" (12" width)

Supports: All - Lumber-soft Beam, No.3

Total length: 30.54'; Clear span: 18.393', 11.273'; Volume = 24.5 cu.ft. / ft.; Panel orientation: Longitudinal axis

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	V = 1248	Vs' = 3465	lbs	V/Vs' = 0.36
Bending(+)	M = 3441	M' = 8264	lbs-ft	M/M' = 0.42
Bending(-)	M = 3517	M' = 8264	lbs-ft	M/M' = 0.43
Live Defl'n	0.15 = < L/999	0.62 = L/360	in	0.24
Total Defl'n	0.33 = L/667	0.93 = L/240	in	0.36
Vibration	Lmax = 18.563	Lv = 21.125	ft	Lmax/Lv = 0.88

Additional Data:

FACTORS:	F(psi)	CD	CM	Ct	CL	CF	Cfu	Cr	Cfrt	Ci	CLT	LC#
Fs	45	-	1.00	1.00	-	-	-	-	-	-	-	2
Fb+	875	1.00	1.00	1.00	1.000	-	-	-	-	-	0.85	3
Fb-	875	1.00	1.00	1.00	1.000	-	-	-	-	-	0.85	2
Fcp'	425	-	1.00	1.00	-	-	-	-	-	-	-	-
EIapp	641.0 million		1.00	1.00	-	-	-	-	-	-	-	3

CRITICAL LOAD COMBINATIONS:

```
 Shear     : LC #2  = D+L
 Bending(+): LC #3  = D+L (pattern: L_)
 Bending(-): LC #2  = D+L
 Deflection: LC #3  =   (live)
             LC #3  =   (total)
 Bearing   : Support 1 - LC #3 = D+L (pattern: L_)
             Support 2 - LC #2 = D+L
             Support 3 - LC #4 = D+L (pattern: _L)
             Support 3 - LC #3 = D+L (pattern: L_)
 D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
 All LC's are listed in the Analysis output
 Load Patterns: s=S/2,  X=L+S or L+Lr,  _=no pattern load in this span
 Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.2
```

CALCULATIONS:

```
 V max = 1385, V design = 1248 (NDS 3.4.3.1(a)), Vs = 3465 lbs
 Seff,0 = 133.33 in^3; (FbS)eff = 8264 lbs-ft; (GA)eff,0 = 1.37e06 lb
 (EI)eff,0 = 898.31e06; (EI)app' = 640.97e06 lb-in^2
 E = 1400000 psi; G = 87500 psi; E⊥ = 40000 psi; G⊥ = 7500 psi
 "Live" deflection is due to all non-dead loads (live, wind, snow…)
 Total deflection = 2.0 dead + "live"
 (EI)app' for shear deflection is based on Ks = 11.5 for uniform loading on a single
 span and is approximate for other loading conditions.
```

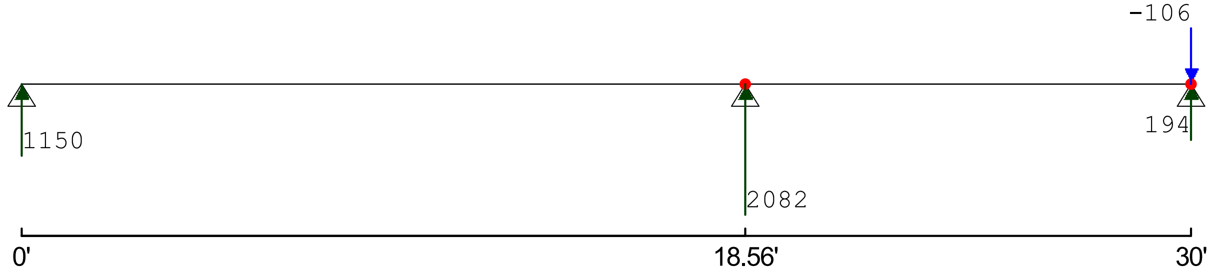
Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2018), the National Design Specification (NDS 2018), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. FIRE RATING: Joists, wall studs, and multi-ply members are not rated for fire endurance.
4. CLT design is according to NDS Ch. 10 and APA PRG 320-19. Where needed for customized lay-ups or fire-reduced sections, 2013 FPInnovations CLT Handbook Chs. 3 and 8, and 2014 CSA O86 Ch. 8 are used. Floor vibration from CSA O86 A.8.5.3.

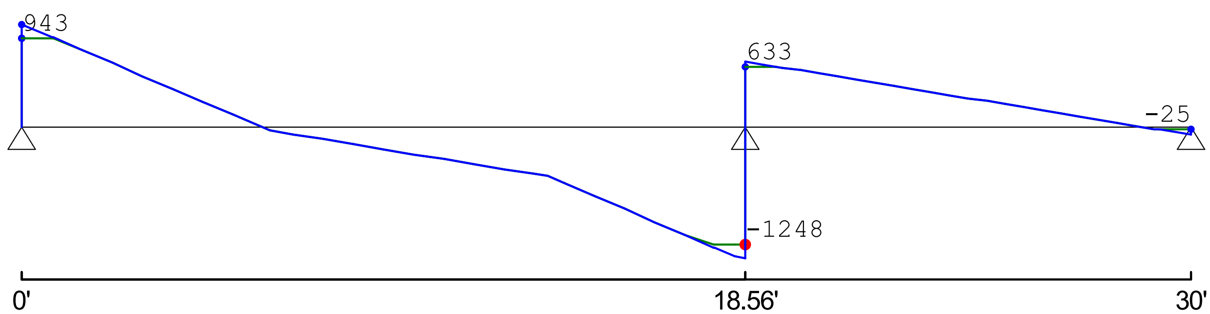
Attic_Floor **WoodWorks® Sizer 2019 (Update 1)** **Mar. 22, 2021 15:54:41**
Critical Results

ANALYSIS DIAGRAMS (known section)

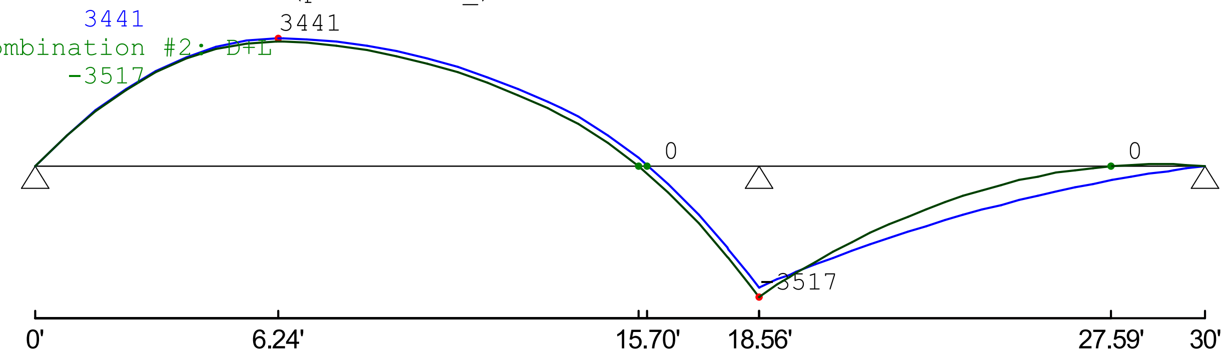
REACTION [lbs]
Maximum...
Uplift: -106 (LC #3)
Bearing: 2082 (LC #4)



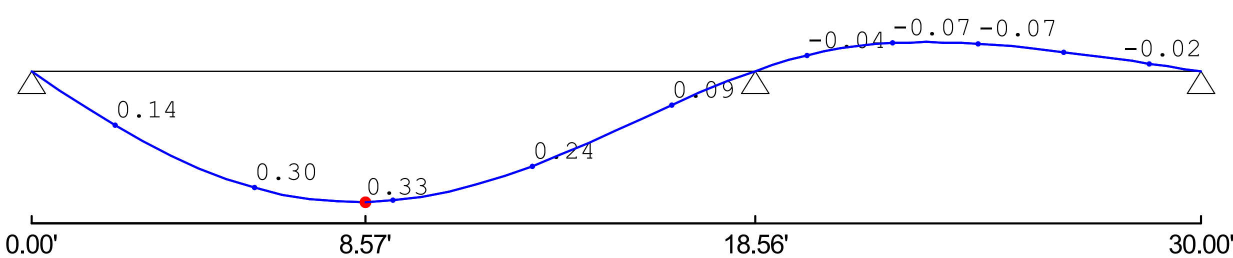
SHEAR [lbs]
Load Combination #2: D+L
+V max: 1089



BENDING [lbs-ft]
Load Combination #3: D+L (pattern: L_)
+M max: 3441
Load Combination #2: D+L
-M max: -3517



TOTAL DEFLECTION [in]
Load Combination #3:
Total = 2.00 x Dead + Live (all others)
Critical Live: 0.15
Critical Total: 0.33



CLT Home Design
Garage
Apr. 1, 2021 12:37

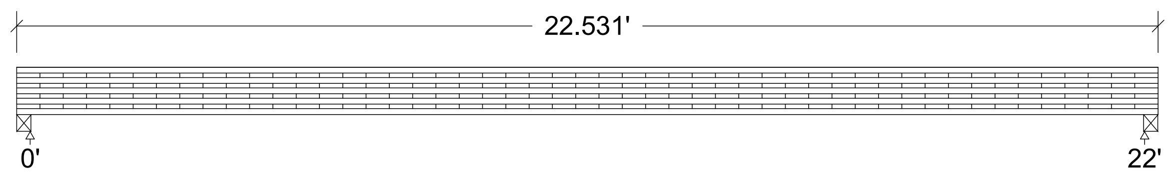
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 1)

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start End	Magnitude Start End	Unit
w1d	Dead	Full Area			31.60(1.00')	psf
w1l	Live	Full Area			40.00(1.00')	psf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:			
Dead	356		356
Live	451		451
Factored:			
Total	807		807
Bearing:			
Capacity			
Beam	17531		17544
Support	-		-
Des ratio			
Beam	0.05		0.05
Support	-		-
Load comb	#2		#2
Length	3.44		3.44
Min req'd	0.50*		0.50*
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	-		-
Fcp sup	425		425

*Minimum bearing length setting used: 1/2" for end supports

Garage Floor Panel

CLT Floor Panel, S-P-F, V2, 9 Layers 11-1/4" (12" width)

Supports: All - CLT Wall panel, V2

Total length: 22.53'; Clear span: 21.958'; Volume = 21.1 cu.ft. / ft.; Panel orientation: Longitudinal axis

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	V = 719	Vs' = 4039	lbs	V/Vs' = 0.18
Bending(+)	M = 4332	M' = 11342	lbs-ft	M/M' = 0.38
Live Defl'n	0.17 = < L/999	0.73 = L/360	in	0.23
Total Defl'n	0.43 = L/616	1.10 = L/240	in	0.39
Vibration	Lmax = 22.000	Lv = 23.750	ft	Lmax/Lv = 0.93

Additional Data:

FACTORS:	F(psi)	CD	CM	Ct	CL	CF	Cfu	Cr	Cfrt	Ci	CLT	LC#
Fs	45	-	1.00	1.00	-	-	-	-	-	-	-	2
Fb+	875	1.00	1.00	1.00	1.000	-	-	-	-	-	0.85	2
Fcp'	425	-	1.00	1.00	-	-	-	-	-	-	-	-
EIapp	1271.2 million		1.00	1.00	-	-	-	-	-	-	-	2

CRITICAL LOAD COMBINATIONS:

```
 Shear     : LC #2  = D+L
 Bending(+): LC #2  = D+L
 Deflection: LC #2  = D+L  (live)
             LC #2  = D+L  (total)
 Bearing   : Support 1 - LC #2 = D+L
             Support 2 - LC #2 = D+L
 D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
 All LC's are listed in the Analysis output
 Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.2
```

CALCULATIONS:

```
 V max = 788, V design = 719 (NDS 3.4.3.1(a)), Vs = 4039 lbs
 Seff,0 = 183.00 in^3; (FbS)eff = 11342 lbs-ft; (GA)eff,0 = 1.82e06 lb
 (EI)eff,0 = 1437.26e06; (EI)app' = 1271.16e06 lb-in^2
 E = 1400000 psi; G = 87500 psi; E⊥ = 40000 psi; G⊥ = 7500 psi
 "Live" deflection is due to all non-dead loads (live, wind, snow…)
 Total deflection = 2.0 dead + "live"
 (EI)app' for shear deflection is based on Ks = 11.5 for uniform loading on a single
 span and is approximate for other loading conditions.
```

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2018), the National Design Specification (NDS 2018), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. FIRE RATING: Joists, wall studs, and multi-ply members are not rated for fire endurance.
4. CLT design is according to NDS Ch. 10 and APA PRG 320-19. Where needed for customized lay-ups or fire-reduced sections, 2013 FPInnovations CLT Handbook Chs. 3 and 8, and 2014 CSA O86 Ch. 8 are used. Floor vibration from CSA O86 A.8.5.3.

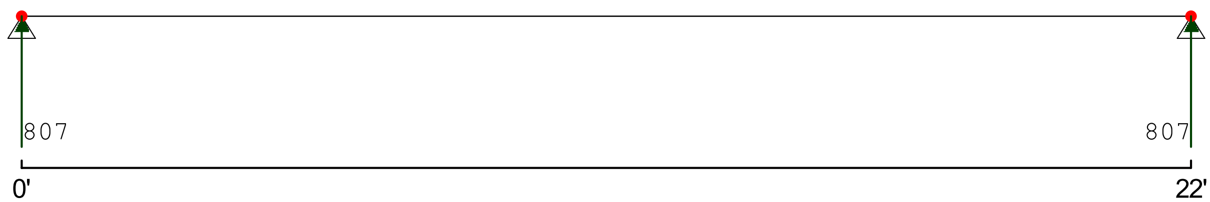
Garage
Critical Results

WoodWorks® Sizer 2019 (Update 1)

Apr. 1, 2021 12:36:21

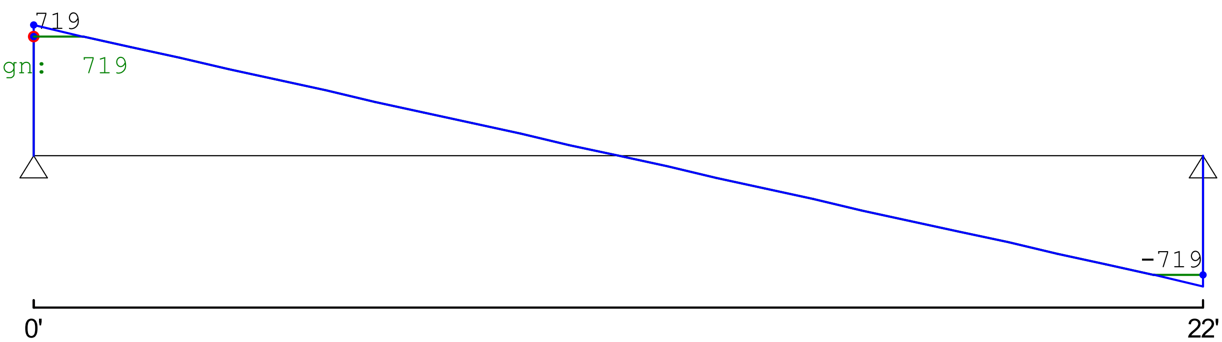
ANALYSIS DIAGRAMS (known section)

REACTION [lbs]
Maximum...
Uplift: 0
Bearing: 807 (LC #2)

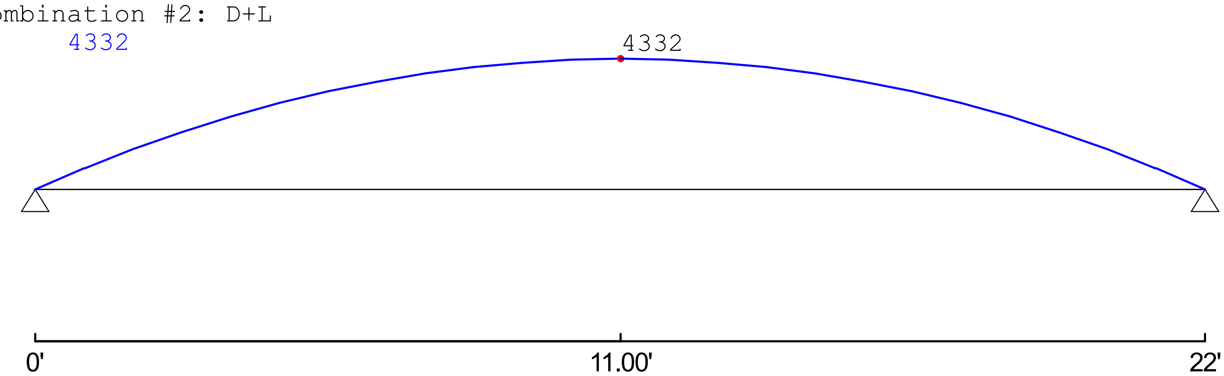


SHEAR [lbs]
Load Combination #2: D+L

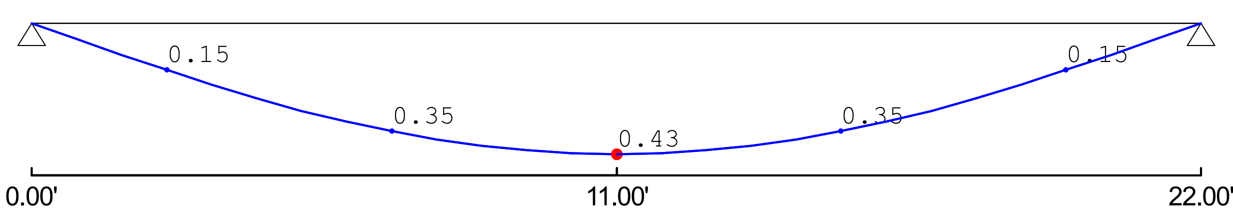
V design: 719



BENDING [lbs-ft]
Load Combination #2: D+L
+M max: 4332



TOTAL DEFLECTION [in]
Load Combination #2: D+L
Total = 2.00 x Dead + Live (all others)
Critical Live: 0.17
Critical Total: 0.43



JES		SK-5
ACJ		Aug 05, 2021
		1st_Floor.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	16.083333	0	0	
3	N3	30	0	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot [k-ft/rad]
1	N1	Reaction	Reaction	Reaction	Reaction
2	N3	Reaction	Reaction	Reaction	Reaction
3	ALL		Reaction		
4	N2		Reaction		

Member Primary Data

	Label	I Node	J Node	Rotate(deg)	Section/Shape	Type	Design List	Material	Design Rule
1	M1	N1	N2	90	6.16X12FS	Beam	None	SPF_CLT	Typical
2	M2	N2	N3	90	6.16X12FS	Beam	None	SPF_CLT	Typical

Wood Material Properties

	Label	Type	Database	Species	Grade	Cm	Emod	Nu	Therm. Coeff. $[1e^{5}{}^\circ F^{-1}]$	Density $[k/ft^3]$
1	DF	Solid Sawn	Visually Graded	Douglas Fir-Larch	No.1		1	0.3	0.3	0.035
2	SP	Solid Sawn	Visually Graded	Southern Pine	No.1		1	0.3	0.3	0.035
3	HF	Solid Sawn	Visually Graded	Hem-Fir	No.1		1	0.3	0.3	0.035
4	SPF	Solid Sawn	Visually Graded	Spruce-Pine-fir	No.1		1	0.3	0.3	0.035
5	24F-1.8E DF Balanced	Glulam	NDS Table 5A	24F-1.8E_DF_BAL	na		1	0.3	0.3	0.035
6	24F-1.8E DF Unbalanced	Glulam	NDS Table 5A	24F-1.8E_DF_UNBAL	na		1	0.3	0.3	0.035
7	24F-1.8E SP Balanced	Glulam	NDS Table 5A	24F-1.8E_SP_BAL	na		1	0.3	0.3	0.035
8	24F-1.8E SP Unbalanced	Glulam	NDS Table 5A	24F-1.8E_SP_UNBAL	na		1	0.3	0.3	0.035
9	1.3E-1600F_VERSALAM	SCL	Boise Cascade	1.3E-1600F_VERSALAM	na		1	0.3	0.3	0.035
10	1.35E LSL_SolidStart	SCL	Louisiana Pacific	1.35E LSL_SolidStart	na		1	0.3	0.3	0.035
11	1.3E_RIGIDLAM LVL	SCL	Roseburg Forest Products	1.3E_RIGIDLAM LVL	na		1	0.3	0.3	0.035
12	2.0E_DF Parallam PSL	SCL	TrusJoist	2.0E_DF Parallam PSL	na		1	0.3	0.3	0.035
13	SPF_CLT	Custom	N/A	CLT_SPF_1_2	na		1	0.3	0.3	0.035
14	LVL_Microlam_1.9E_2600F	Custom	N/A	LVL_Microllam_1.9E_2600F	na		1	0.3	0.3	0.035
15	PSL_Parallam_2.0E_2900F	Custom	N/A	PSL_Parallam_2.0E_2900F	na		1	0.3	0.3	0.035
16	LSL_TimberStrand_1.55E_2325F	Custom	N/A	LSL_TimberStrand_1.55E_2325F	na		1	0.3	0.3	0.035

Wood Design Parameters

	Label	Shape	Length [ft]	le-bend top [ft]	Cr	y sway	z sway
1	M1	6.16X12FS	16.083	Lbyy			
2	M2	6.16X12FS	13.917	Lbyy			

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.021	-0.021	0	%100
2	M2	Y	-0.021	-0.021	0	%100
3	M2	Y	-0.034	-0.034	9.73	%100

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.04	-0.04	0	%100
2	M2	Y	-0.04	-0.04	0	%100
3	M2	Y	-0.064	-0.064	9.73	%100

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.021	-0.021	0	%100
2	M2	Y	-0.021	-0.021	0	%100
3	M2	Y	-0.064	-0.064	9.73	%100

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.04	-0.04	0	%100

Load Combinations

	Description	Solve	BLC	Factor	BLC	Factor
1	ASD_1	Yes	DL	1	LL	1
2	Service_Dead	Yes	DL	1		
3	Service_Live	Yes	LL	1		
4	Serv_Dead_Left	Yes	3	1		
5	Serv_Live_Left	Yes	4	1		
6	ASD_2	Yes	4	1	3	1

Load Combination Design

	Description	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
1	ASD_1		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
2	Service_Dead	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
3	Service_Live	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
4	Serv_Dead_Left	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
5	Serv_Live_Left	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6	ASD_2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

Basic Load Cases

	BLC Description	Category	Distributed
1	Dead	DL	3
2	Live	LL	3
3	Dead_Left	None	3
4	Live_Left	None	1

Member Section Forces

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
1	1	M1	1	0	0	-0.373	0	0	0
2			2	0	0	-0.126	0	-1.003	0
3			3	0	0	0.121	0	-1.014	0
4			4	0	0	0.368	0	-0.032	0
5			5	0	0	0.615	0	1.942	0
6	1	M2	1	0	0	-0.629	0	1.942	0
7			2	0	0	-0.415	0	0.127	0
8			3	0	0	-0.201	0	-0.945	0
9			4	0	0	0.082	0	-1.25	0
10			5	0	0	0.637	0	0	0
11	2	M1	1	0	0	-0.13	0	0	0
12			2	0	0	-0.044	0	-0.35	0
13			3	0	0	0.042	0	-0.354	0
14			4	0	0	0.128	0	-0.011	0
15			5	0	0	0.214	0	0.677	0
16	2	M2	1	0	0	-0.219	0	0.677	0
17			2	0	0	-0.145	0	0.044	0
18			3	0	0	-0.07	0	-0.329	0
19			4	0	0	0.028	0	-0.435	0
20			5	0	0	0.222	0	0	0
21	3	M1	1	0	0	-0.243	0	0	0
22			2	0	0	-0.082	0	-0.654	0
23			3	0	0	0.079	0	-0.661	0
24			4	0	0	0.24	0	-0.021	0
25			5	0	0	0.4	0	1.266	0
26	3	M2	1	0	0	-0.41	0	1.266	0
27			2	0	0	-0.27	0	0.083	0
28			3	0	0	-0.131	0	-0.616	0
29			4	0	0	0.053	0	-0.815	0
30			5	0	0	0.415	0	0	0
31	4	M1	1	0	0	-0.126	0	0	0
32			2	0	0	-0.04	0	-0.335	0
33			3	0	0	0.046	0	-0.325	0
34			4	0	0	0.132	0	0.032	0

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
35			5	0	0	0.218	0	0.735	0
36	4	M2	1	0	0	-0.242	0	0.735	0
37			2	0	0	-0.168	0	0.022	0
38			3	0	0	-0.093	0	-0.431	0
39			4	0	0	0.027	0	-0.61	0
40			5	0	0	0.324	0	0	0
41	5	M1	1	0	0	-0.279	0	0	0
42			2	0	0	-0.118	0	-0.797	0
43			3	0	0	0.043	0	-0.947	0
44			4	0	0	0.204	0	-0.45	0
45			5	0	0	0.365	0	0.693	0
46	5	M2	1	0	0	-0.05	0	0.693	0
47			2	0	0	-0.05	0	0.52	0
48			3	0	0	-0.05	0	0.346	0
49			4	0	0	-0.05	0	0.173	0
50			5	0	0	-0.05	0	0	0
51	6	M1	1	0	0	-0.405	0	0	0
52			2	0	0	-0.158	0	-1.132	0
53			3	0	0	0.089	0	-1.272	0
54			4	0	0	0.336	0	-0.418	0
55			5	0	0	0.583	0	1.427	0
56	6	M2	1	0	0	-0.292	0	1.427	0
57			2	0	0	-0.217	0	0.542	0
58			3	0	0	-0.143	0	-0.085	0
59			4	0	0	-0.023	0	-0.436	0
60			5	0	0	0.274	0	0	0

Maximum Member Section Forces

	LC	Member Label		Axial[k]	Loc[ft]	y Shear[k]	Loc[ft]	z Shear[k]	Loc[ft]	Torque[k-ft]	Loc[ft]	y-y Moment[k-ft]	Loc[ft]	z-z Moment[k-ft]	Loc[ft]
1	1	M1	max	0	16.083	0	16.083	0.615	16.083	0	16.083	1.942	16.083	0	16.083
2			min	0	0	0	0	-0.373	0	0	0	-1.133	6.031	0	0
3	1	M2	max	0	13.917	0	13.917	0.637	13.917	0	13.917	1.942	0	0	13.917
4			min	0	0	0	0	-0.629	0	0	0	-1.27	9.858	0	0
5	2	M1	max	0	16.083	0	16.083	0.214	16.083	0	16.083	0.677	16.083	0	16.083
6			min	0	0	0	0	-0.13	0	0	0	-0.395	6.031	0	0
7	2	M2	max	0	13.917	0	13.917	0.222	13.917	0	13.917	0.677	0	0	13.917
8			min	0	0	0	0	-0.219	0	0	0	-0.442	9.858	0	0
9	3	M1	max	0	16.083	0	16.083	0.4	16.083	0	16.083	1.266	16.083	0	16.083
10			min	0	0	0	0	-0.243	0	0	0	-0.738	6.031	0	0
11	3	M2	max	0	13.917	0	13.917	0.415	13.917	0	13.917	1.266	0	0	13.917
12			min	0	0	0	0	-0.41	0	0	0	-0.828	9.858	0	0
13	4	M1	max	0	16.083	0	16.083	0.218	16.083	0	16.083	0.735	16.083	0	16.083
14			min	0	0	0	0	-0.126	0	0	0	-0.373	5.864	0	0
15	4	M2	max	0	13.917	0	13.917	0.324	13.917	0	13.917	0.735	0	0	13.917
16			min	0	0	0	0	-0.242	0	0	0	-0.614	10.148	0	0
17	5	M1	max	0	16.083	0	16.083	0.365	16.083	0	16.083	0.693	16.083	0	16.083
18			min	0	0	0	0	-0.279	0	0	0	-0.97	7.036	0	0
19	5	M2	max	0	13.917	0	13.917	-0.05	13.917	0	13.917	0.693	0	0	13.917
20			min	0	0	0	0	-0.05	0	0	0	0	13.917	0	0
21	6	M1	max	0	16.083	0	16.083	0.583	16.083	0	16.083	1.427	16.083	0	16.083
22			min	0	0	0	0	-0.405	0	0	0	-1.336	6.534	0	0
23	6	M2	max	0	13.917	0	13.917	0.274	13.917	0	13.917	1.427	0	0	13.917
24			min	0	0	0	0	-0.292	0	0	0	-0.439	10.727	0	0

Member Section Deflections Strength

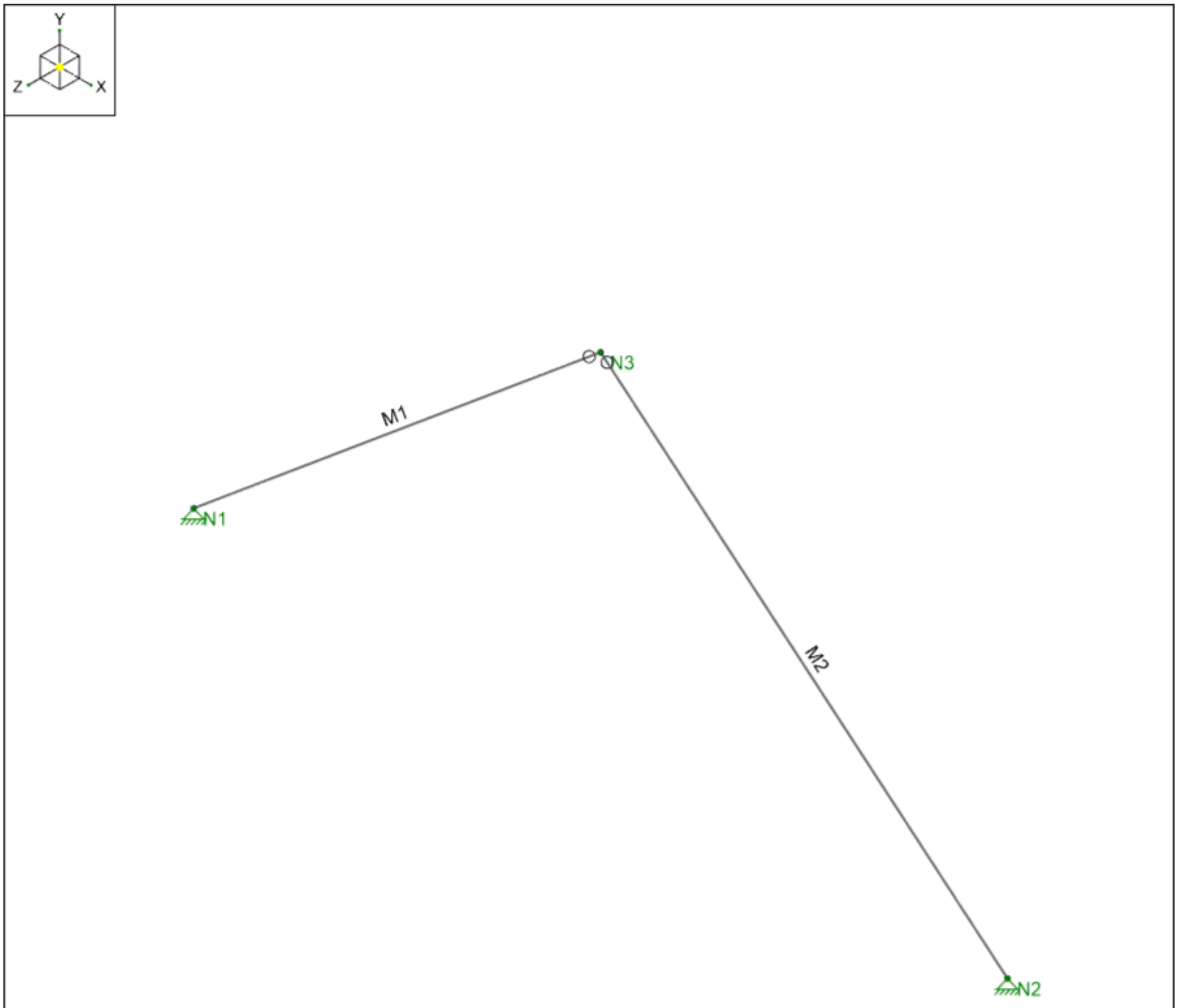
	LC	Member Label	Sec	x [in]	y [in]	z [in]	x Rotate[rad]	(n) L/y' Ratio	(n) L/z' Ratio
1	1	M1	1	0	0	0	0	NC	NC
2			2	0	0	0.098	0	NC	1976
3			3	0	0	0.117	0	NC	1654
4			4	0	0	0.056	0	NC	3435
5			5	0	0	0	0	NC	NC
6	1	M2	1	0	0	0	0	NC	NC
7			2	0	0	0.037	0	NC	4528
8			3	0	0	0.086	0	NC	1945

Member Section Deflections Strength (Continued)

	LC	Member Label	Sec	x [in]	y [in]	z [in]	x Rotate[rad]	(n) L/y' Ratio	(n) L/z' Ratio
9			4	0	0	0.078	0	NC	2131
10			5	0	0	0	0	NC	NC
11	6	M1	1	0	0	0	0	NC	NC
12			2	0	0	0.125	0	NC	1542
13			3	0	0	0.161	0	NC	1201
14			4	0	0	0.095	0	NC	2039
15			5	0	0	0	0	NC	NC
16	6	M2	1	0	0	0	0	NC	NC
17			2	0	0	-0.019	0	NC	8703
18			3	0	0	-0.002	0	NC	NC
19			4	0	0	0.01	0	NC	NC
20			5	0	0	0	0	NC	NC

Member Section Deflections Service

	LC	Member Label	Sec	x [in]	y [in]	z [in]	x Rotate[rad]	(n) L/y' Ratio	(n) L/z' Ratio
1	2	M1	1	0	0	0	0	NC	NC
2			2	0	0	0.034	0	NC	5670
3			3	0	0	0.041	0	NC	4744
4			4	0	0	0.02	0	NC	9850
5			5	0	0	0	0	NC	NC
6	2	M2	1	0	0	0	0	NC	NC
7			2	0	0	0.013	0	NC	NC
8			3	0	0	0.03	0	NC	5588
9			4	0	0	0.027	0	NC	6123
10			5	0	0	0	0	NC	NC
11	3	M1	1	0	0	0	0	NC	NC
12			2	0	0	0.064	0	NC	3035
13			3	0	0	0.076	0	NC	2540
14			4	0	0	0.037	0	NC	5275
15			5	0	0	0	0	NC	NC
16	3	M2	1	0	0	0	0	NC	NC
17			2	0	0	0.024	0	NC	6944
18			3	0	0	0.056	0	NC	2984
19			4	0	0	0.051	0	NC	3269
20			5	0	0	0	0	NC	NC
21	4	M1	1	0	0	0	0	NC	NC
22			2	0	0	0.031	0	NC	6236
23			3	0	0	0.036	0	NC	5401
24			4	0	0	0.015	0	NC	NC
25			5	0	0	0	0	NC	NC
26	4	M2	1	0	0	0	0	NC	NC
27			2	0	0	0.02	0	NC	8538
28			3	0	0	0.042	0	NC	3984
29			4	0	0	0.038	0	NC	4383
30			5	0	0	0	0	NC	NC
31	5	M1	1	0	0	0	0	NC	NC
32			2	0	0	0.094	0	NC	2049
33			3	0	0	0.125	0	NC	1545
34			4	0	0	0.079	0	NC	2431
35			5	0	0	0	0	NC	NC
36	5	M2	1	0	0	0	0	NC	NC
37			2	0	0	-0.039	0	NC	4310
38			3	0	0	-0.044	0	NC	3771
39			4	0	0	-0.028	0	NC	6034
40			5	0	0	0	0	NC	NC



JES		SK-7
ACJ		Aug 05, 2021
		Garage_Roof.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	25	0	0	
3	N3	12.5	10.4	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]
1	ALL			Fixed
2	N1	Reaction	Reaction	Reaction
3	N2	Reaction	Reaction	Reaction

Member Primary Data

	Label	I Node	J Node	Rotate(deg)	Section/Shape	Type	Design List	Material	Design Rule
1	M1	N1	N3	90	4.01X12FS	Beam	None	SPF_CLT	Typical
2	M2	N3	N2	90	4.01X12FS	Beam	None	SPF_CLT	Typical

Wood Material Properties

	Label	Type	Database	Species	Grade	Cm	Emod	Nu	Therm. Coeff. [$1e^{5}$°F^{-1}]	Density [k/ft^3]
1	DF	Solid Sawn	Visually Graded	Douglas Fir-Larch	No.1		1	0.3	0.3	0.035
2	SP	Solid Sawn	Visually Graded	Southern Pine	No.1		1	0.3	0.3	0.035
3	HF	Solid Sawn	Visually Graded	Hem-Fir	No.1		1	0.3	0.3	0.035
4	SPF	Solid Sawn	Visually Graded	Spruce-Pine-fir	No.1		1	0.3	0.3	0.035
5	24F-1.8E DF Balanced	Glulam	NDS Table 5A	24F-1.8E_DF_BAL	na		1	0.3	0.3	0.035
6	24F-1.8E DF Unbalanced	Glulam	NDS Table 5A	24F-1.8E_DF_UNBAL	na		1	0.3	0.3	0.035
7	24F-1.8E SP Balanced	Glulam	NDS Table 5A	24F-1.8E_SP_BAL	na		1	0.3	0.3	0.035
8	24F-1.8E SP Unbalanced	Glulam	NDS Table 5A	24F-1.8E_SP_UNBAL	na		1	0.3	0.3	0.035
9	1.3E-1600F_VERSALAM	SCL	Boise Cascade	1.3E-1600F_VERSALAM	na		1	0.3	0.3	0.035
10	1.35E LSL_SolidStart	SCL	Louisiana Pacific	1.35E LSL_SolidStart	na		1	0.3	0.3	0.035
11	1.3E_RIGIDLAM LVL	SCL	Roseburg Forest Products	1.3E_RIGIDLAM LVL	na		1	0.3	0.3	0.035
12	2.0E_DF Parallam PSL	SCL	TrusJoist	2.0E_DF Parallam PSL	na		1	0.3	0.3	0.035
13	SPF_CLT	Custom	N/A	CLT_SPF_1_2	na		1	0.3	0.3	0.035
14	LVL_Microlam_1.9E_2600F	Custom	N/A	LVL_Microllam_1.9E_2600F	na		1	0.3	0.3	0.035
15	PSL_Parallam_2.0E_2900F	Custom	N/A	PSL_Parallam_2.0E_2900F	na		1	0.3	0.3	0.035
16	LSL_TimberStrand_1.55E_2325F	Custom	N/A	LSL_TimberStrand_1.55E_2325F	na		1	0.3	0.3	0.035

Wood Design Parameters

	Label	Shape	Length [ft]	le-bend top [ft]	Cr	y sway	z sway
1	M1	4.01X12FS	16.261	Lbyy			
2	M2	4.01X12FS	16.261	Lbyy			

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.019	-0.02	0	%100
2	M2	Y	-0.02	-0.02	0	%100

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.048	-0.048	0	%100
2	M2	Y	-0.048	-0.048	0	%100

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	z	-0.024	-0.024	0	%100
2	M2	z	-0.024	-0.024	0	%100

Load Combinations

	Description	Solve	BLC	Factor	BLC	Factor	BLC	Factor
1	ASD_1	Yes	DL	1	SL	1		
2	ASD_2	Yes	DL	1	SL	0.75	WL	0.45
3	ASD_3	Yes	DL	0.6	WL	0.6		
4	Service_Dead	Yes	DL	1				
5	Service_Snow	Yes	SL	1				

Load Combinations (Continued)

	Description	Solve	BLC	Factor	BLC	Factor	BLC	Factor
6	Service_Wind	Yes	WL	1				

Load Combination Design

	Description	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
1	ASD_1		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
2	ASD_2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
3	ASD_3		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
4	Service_Dead	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
5	Service_Snow	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6	Service_Wind	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

Basic Load Cases

	BLC Description	Category	Distributed
1	Dead	DL	2
2	Snow	SL	2
3	Wind	WL	2

Member Section Forces

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
1	1	M1	1	1.201	0	-0.417	0	0	0
2			2	1.028	0	-0.209	0	-1.274	0
3			3	0.854	0	0	0	-1.7	0
4			4	0.68	0	0.209	0	-1.276	0
5			5	0.505	0	0.419	0	0	0
6	1	M2	1	0.504	0	-0.421	0	0	0
7			2	0.679	0	-0.21	0	-1.282	0
8			3	0.854	0	0	0	-1.71	0
9			4	1.029	0	0.21	0	-1.282	0
10			5	1.204	0	0.421	0	0	0
11	2	M1	1	0.884	0	-0.256	0	0	0
12			2	0.741	0	-0.129	0	-0.782	0
13			3	0.599	0	0	0	-1.044	0
14			4	0.455	0	0.128	0	-0.784	0
15			5	0.311	0	0.258	0	0	0
16	2	M2	1	0.31	0	-0.259	0	0	0
17			2	0.454	0	-0.13	0	-0.791	0
18			3	0.598	0	0	0	-1.054	0
19			4	0.742	0	0.13	0	-0.791	0
20			5	0.886	0	0.259	0	0	0
21	3	M1	1	0.071	0	0.044	0	0	0
22			2	0.041	0	0.022	0	0.133	0
23			3	0.011	0	0	0	0.176	0
24			4	-0.02	0	-0.022	0	0.131	0
25			5	-0.05	0	-0.043	0	0	0
26	3	M2	1	-0.051	0	0.042	0	0	0
27			2	-0.02	0	0.021	0	0.128	0
28			3	0.01	0	0	0	0.17	0
29			4	0.041	0	-0.021	0	0.128	0
30			5	0.072	0	-0.042	0	0	0
31	4	M1	1	0.35	0	-0.12	0	0	0
32			2	0.301	0	-0.061	0	-0.368	0
33			3	0.25	0	0	0	-0.493	0
34			4	0.2	0	0.061	0	-0.371	0
35			5	0.148	0	0.122	0	0	0
36	4	M2	1	0.147	0	-0.124	0	0	0
37			2	0.199	0	-0.062	0	-0.377	0
38			3	0.25	0	0	0	-0.503	0
39			4	0.301	0	0.062	0	-0.377	0
40			5	0.353	0	0.124	0	0	0
41	5	M1	1	0.851	0	-0.297	0	0	0
42			2	0.727	0	-0.148	0	-0.905	0
43			3	0.604	0	0	0	-1.207	0
44			4	0.48	0	0.148	0	-0.905	0

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
45			5	0.357	0	0.297	0	0	0
46	5	M2	1	0.357	0	-0.297	0	0	0
47			2	0.48	0	-0.148	0	-0.905	0
48			3	0.604	0	0	0	-1.207	0
49			4	0.727	0	0.148	0	-0.905	0
50			5	0.851	0	0.297	0	0	0
51	6	M1	1	-0.233	0	0.194	0	0	0
52			2	-0.233	0	0.097	0	0.59	0
53			3	-0.233	0	0	0	0.787	0
54			4	-0.233	0	-0.097	0	0.59	0
55			5	-0.233	0	-0.194	0	0	0
56	6	M2	1	-0.233	0	0.194	0	0	0
57			2	-0.233	0	0.097	0	0.59	0
58			3	-0.233	0	0	0	0.787	0
59			4	-0.233	0	-0.097	0	0.59	0
60			5	-0.233	0	-0.194	0	0	0

Maximum Member Section Forces

	LC	Member Label		Axial[k]	Loc[ft]	y Shear[k]	Loc[ft]	z Shear[k]	Loc[ft]	Torque[k-ft]	Loc[ft]	y-y Moment[k-ft]	Loc[ft]	z-z Moment[k-ft]	Loc[ft]
1	1	M1	max	1.201	0	0	16.261	0.419	16.261	0	16.261	0	16.261	0	16.261
2			min	0.505	16.261	0	0	-0.417	0	0	0	-1.7	8.13	0	0
3	1	M2	max	1.204	16.261	0	16.261	0.421	16.261	0	16.261	0	16.261	0	16.261
4			min	0.504	0	0	0	-0.421	0	0	0	-1.71	8.13	0	0
5	2	M1	max	0.884	0	0	16.261	0.258	16.261	0	16.261	0	16.261	0	16.261
6			min	0.311	16.261	0	0	-0.256	0	0	0	-1.044	8.13	0	0
7	2	M2	max	0.886	16.261	0	16.261	0.259	16.261	0	16.261	0	16.261	0	16.261
8			min	0.31	0	0	0	-0.259	0	0	0	-1.054	8.13	0	0
9	3	M1	max	0.071	0	0	16.261	0.044	0	0	16.261	0.176	8.13	0	16.261
10			min	-0.05	16.261	0	0	-0.043	16.261	0	0	0	0	0	0
11	3	M2	max	0.072	16.261	0	16.261	0.042	0	0	16.261	0.17	8.13	0	16.261
12			min	-0.051	0	0	0	-0.042	16.261	0	0	0	0	0	0
13	4	M1	max	0.35	0	0	16.261	0.122	16.261	0	16.261	0	16.261	0	16.261
14			min	0.148	16.261	0	0	-0.12	0	0	0	-0.493	8.13	0	0
15	4	M2	max	0.353	16.261	0	16.261	0.124	16.261	0	16.261	0	16.261	0	16.261
16			min	0.147	0	0	0	-0.124	0	0	0	-0.503	8.13	0	0
17	5	M1	max	0.851	0	0	16.261	0.297	16.261	0	16.261	0	16.261	0	16.261
18			min	0.357	16.261	0	0	-0.297	0	0	0	-1.207	8.13	0	0
19	5	M2	max	0.851	16.261	0	16.261	0.297	16.261	0	16.261	0	16.261	0	16.261
20			min	0.357	0	0	0	-0.297	0	0	0	-1.207	8.13	0	0
21	6	M1	max	-0.233	16.261	0	16.261	0.194	0	0	16.261	0.787	8.13	0	16.261
22			min	-0.233	0	0	0	-0.194	16.261	0	0	0	0	0	0
23	6	M2	max	-0.233	16.261	0	16.261	0.194	0	0	16.261	0.787	8.13	0	16.261
24			min	-0.233	0	0	0	-0.194	16.261	0	0	0	0	0	0

Member Section Deflections Strength

	LC	Member Label	Sec	x [in]	y [in]	z [in]	x Rotate[rad]	(n) L/y' Ratio	(n) L/z' Ratio
1	1	M1	1	0	0	0	0	NC	NC
2			2	0	0	0.639	0	NC	305
3			3	-0.001	0	0.898	0	NC	217
4			4	-0.002	0	0.641	0	NC	304
5			5	-0.002	0	0.003	0	NC	NC
6	1	M2	1	0.002	0	0.003	0	NC	NC
7			2	0.002	0	0.645	0	NC	302
8			3	0.001	0	0.903	0	NC	216
9			4	0	0	0.643	0	NC	303
10			5	0	0	0	0	NC	NC
11	2	M1	1	0	0	0	0	NC	NC
12			2	0	0	0.393	0	NC	497
13			3	0	0	0.551	0	NC	353
14			4	-0.001	0	0.394	0	NC	495
15			5	-0.002	0	0.002	0	NC	NC
16	2	M2	1	0.002	0	0.002	0	NC	NC
17			2	0.001	0	0.398	0	NC	490
18			3	0	0	0.557	0	NC	350

Member Section Deflections Strength (Continued)

	LC	Member Label	Sec	x [in]	y [in]	z [in]	x Rotate[rad]	(n) L/y' Ratio	(n) L/z' Ratio
19			4	0	0	0.397	0	NC	492
20			5	0	0	0	0	NC	NC
21	3	M1	1	0	0	0	0	NC	NC
22			2	0	0	-0.066	0	NC	2943
23			3	0	0	-0.093	0	NC	2100
24			4	0	0	-0.066	0	NC	2953
25			5	0	0	0	0	NC	NC
26	3	M2	1	0	0	0	0	NC	NC
27			2	0	0	-0.064	0	NC	3054
28			3	0	0	-0.09	0	NC	2175
29			4	0	0	-0.064	0	NC	3053
30			5	0	0	0	0	NC	NC

Member Section Deflections Service

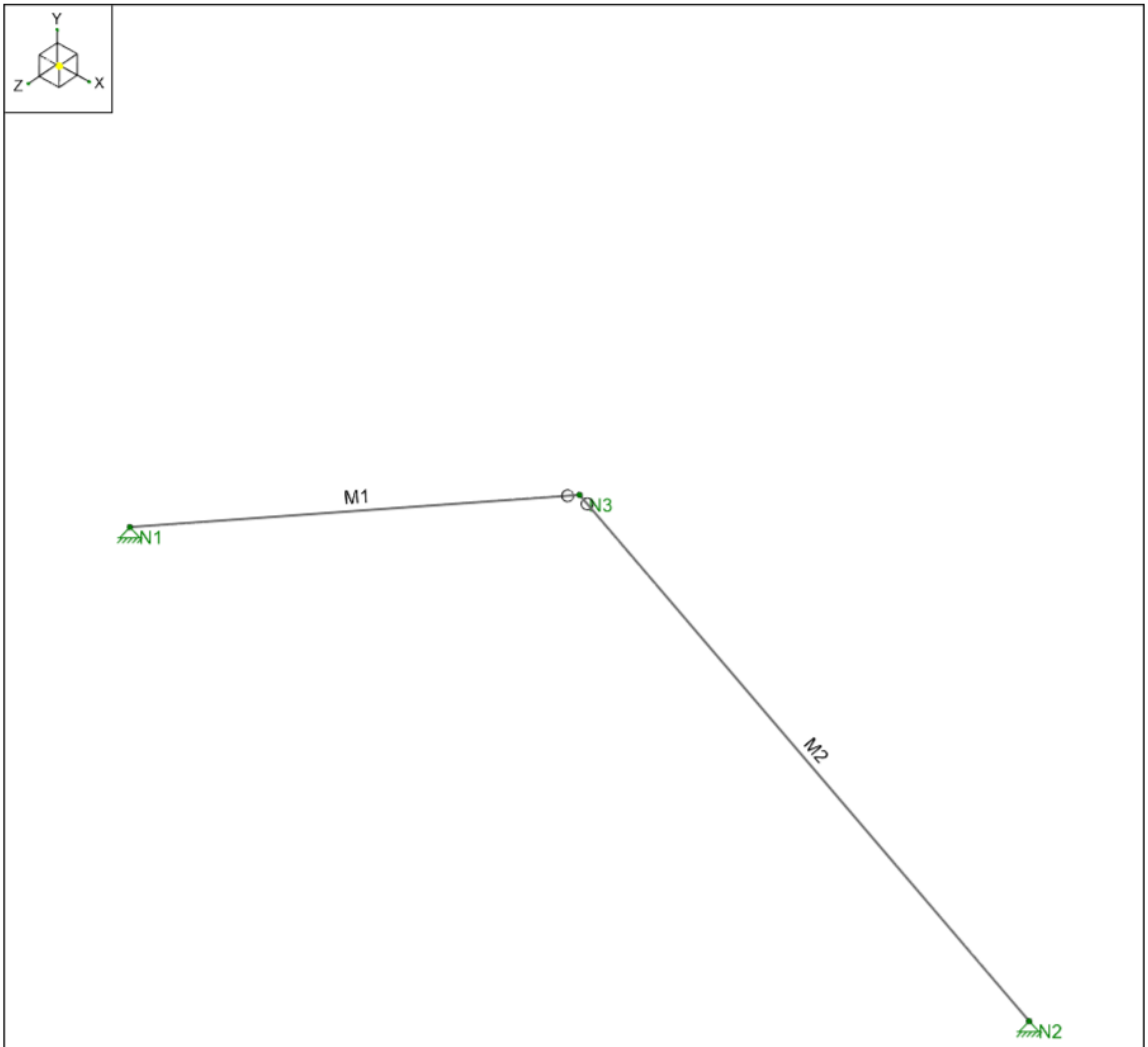
	LC	Member Label	Sec	x [in]	y [in]	z [in]	x Rotate[rad]	(n) L/y' Ratio	(n) L/z' Ratio
1	4	M1	1	0	0	0	0	NC	NC
2			2	0	0	0.185	0	NC	1053
3			3	0	0	0.26	0	NC	749
4			4	0	0	0.186	0	NC	1049
5			5	0	0	0	0	NC	NC
6	4	M2	1	0	0	0	0	NC	NC
7			2	0	0	0.19	0	NC	1029
8			3	0	0	0.266	0	NC	734
9			4	0	0	0.189	0	NC	1031
10			5	0	0	0	0	NC	NC
11	5	M1	1	0	0	0	0	NC	NC
12			2	0	0	0.454	0	NC	430
13			3	0	0	0.637	0	NC	306
14			4	-0.001	0	0.455	0	NC	429
15			5	-0.002	0	0.002	0	NC	NC
16	5	M2	1	0.002	0	0.002	0	NC	NC
17			2	0.001	0	0.455	0	NC	429
18			3	0	0	0.637	0	NC	306
19			4	0	0	0.454	0	NC	430
20			5	0	0	0	0	NC	NC
21	6	M1	1	0	0	0	0	NC	NC
22			2	0	0	-0.296	0	NC	660
23			3	0	0	-0.415	0	NC	470
24			4	0	0	-0.296	0	NC	659
25			5	0	0	0	0	NC	NC
26	6	M2	1	0	0	0	0	NC	NC
27			2	0	0	-0.296	0	NC	659
28			3	0	0	-0.415	0	NC	470
29			4	0	0	-0.296	0	NC	660
30			5	0	0	0	0	NC	NC

Node Reactions

	LC	Node Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
1	1	N1	0.656	1.089	0	0	0	0
2	1	N2	-0.656	1.093	0	0	0	0
3	1	N3	NC	NC	NC	LOCKED	LOCKED	NC
4	1	Totals:	0	2.182	0			
5	1	COG (ft):	X: 12.525	Y: 5.205	Z: 0			
6	2	N1	0.516	0.762	0	0	0	0
7	2	N2	-0.516	0.766	0	0	0	0
8	2	N3	NC	NC	NC	LOCKED	LOCKED	NC
9	2	Totals:	0	1.528	0			
10	2	COG (ft):	X: 12.535	Y: 5.207	Z: 0			
11	3	N1	0.082	0.011	0	0	0	0
12	3	N2	-0.082	0.014	0	0	0	0
13	3	N3	NC	NC	NC	LOCKED	LOCKED	NC
14	3	Totals:	0	0.025	0			
15	3	COG (ft):	X: 13.778	Y: 5.466	Z: 0			
16	4	N1	0.192	0.317	0	0	0	0

Node Reactions (Continued)

	LC	Node Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
17	4	N2	-0.192	0.321	0	0	0	0
18	4	N3	NC	NC	NC	LOCKED	LOCKED	NC
19	4	Totals:	0	0.637	0			
20	4	COG (ft):	X: 12.585	Y: 5.218	Z: 0			
21	5	N1	0.464	0.772	0	0	0	0
22	5	N2	-0.464	0.772	0	0	0	0
23	5	N3	NC	NC	NC	LOCKED	LOCKED	NC
24	5	Totals:	0	1.545	0			
25	5	COG (ft):	X: 12.5	Y: 5.2	Z: 0			
26	6	N1	-0.055	-0.298	0	0	0	0
27	6	N2	0.055	-0.298	0	0	0	0
28	6	N3	NC	NC	NC	LOCKED	LOCKED	NC
29	6	Totals:	0	-0.595	0			
30	6	COG (ft):	X: 12.5	Y: 5.2	Z: 0			



JES		SK-8
ACJ		Aug 05, 2021
		Main_Roof.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	31	0	0	
3	N3	15.5	9.06	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]
1	ALL			Fixed
2	N1	Reaction	Reaction	Reaction
3	N2	Reaction	Reaction	Reaction

Member Primary Data

	Label	I Node	J Node	Rotate(deg)	Section/Shape	Type	Design List	Material	Design Rule
1	M1	N1	N3	90	4.02X12FS	Beam	None	SPF_CLT	Typical
2	M2	N3	N2	90	4.02X12FS	Beam	None	SPF_CLT	Typical

Wood Material Properties

	Label	Type	Database	Species	Grade	Cm	Emod	Nu	Therm. Coeff. [1e⁵°F⁻¹]	Density [k/ft³]
1	DF	Solid Sawn	Visually Graded	Douglas Fir-Larch	No.1		1	0.3	0.3	0.035
2	SP	Solid Sawn	Visually Graded	Southern Pine	No.1		1	0.3	0.3	0.035
3	HF	Solid Sawn	Visually Graded	Hem-Fir	No.1		1	0.3	0.3	0.035
4	SPF	Solid Sawn	Visually Graded	Spruce-Pine-fir	No.1		1	0.3	0.3	0.035
5	24F-1.8E DF Balanced	Glulam	NDS Table 5A	24F-1.8E_DF_BAL	na		1	0.3	0.3	0.035
6	24F-1.8E DF Unbalanced	Glulam	NDS Table 5A	24F-1.8E_DF_UNBAL	na		1	0.3	0.3	0.035
7	24F-1.8E SP Balanced	Glulam	NDS Table 5A	24F-1.8E_SP_BAL	na		1	0.3	0.3	0.035
8	24F-1.8E SP Unbalanced	Glulam	NDS Table 5A	24F-1.8E_SP_UNBAL	na		1	0.3	0.3	0.035
9	1.3E-1600F_VERSALAM	SCL	Boise Cascade	1.3E-1600F_VERSALAM	na		1	0.3	0.3	0.035
10	1.35E LSL_SolidStart	SCL	Louisiana Pacific	1.35E LSL_SolidStart	na		1	0.3	0.3	0.035
11	1.3E_RIGIDLAM LVL	SCL	Roseburg Forest Products	1.3E_RIGIDLAM LVL	na		1	0.3	0.3	0.035
12	2.0E_DF Parallam PSL	SCL	TrusJoist	2.0E_DF Parallam PSL	na		1	0.3	0.3	0.035
13	SPF_CLT	Custom	N/A	CLT_SPF_1_2	na		1	0.3	0.3	0.035
14	LVL_Microlam_1.9E_2600F	Custom	N/A	LVL_Microllam_1.9E_2600F	na		1	0.3	0.3	0.035
15	PSL_Parallam_2.0E_2900F	Custom	N/A	PSL_Parallam_2.0E_2900F	na		1	0.3	0.3	0.035
16	LSL_TimberStrand_1.55E_2325F	Custom	N/A	LSL_TimberStrand_1.55E_2325F	na		1	0.3	0.3	0.035

Wood Design Parameters

	Label	Shape	Length [ft]	le-bend top [ft]	Cr	y sway	z sway
1	M1	4.02X12FS	17.954	Lbyy			
2	M2	4.02X12FS	17.954	Lbyy			

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.02	-0.02	0	%100
2	M2	Y	-0.02	-0.02	0	%100

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.025	-0.025	0	%100
2	M2	Y	-0.025	-0.025	0	%100

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	z	-0.024	-0.024	0	%100
2	M2	z	-0.024	-0.024	0	%100

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.01	-0.01	0	%100
2	M2	Y	-0.01	-0.01	0	%100

Load Combinations

	Description	Solve	BLC	Factor	BLC	Factor	BLC	Factor
1	ASD_1	Yes	1	1	SL	1		
2	ASD_2	Yes	1	1	SL	0.75	WL	0.45
3	ASD_3	Yes	1	0.6	WL	0.6		
4	ASD_4	Yes	4	0.6	WL	0.6		
5	Service_Dead	Yes	DL	1				
6	Service_Snow	Yes	SL	1				
7	Service_Wind	Yes	WL	1				

Load Combination Design

	Description	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
1	ASD_1		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
2	ASD_2		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
3	ASD_3		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
4	ASD_4		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
5	Service_Dead	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6	Service_Snow	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
7	Service_Wind	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

Basic Load Cases

	BLC Description	Category	Distributed
1	Dead	DL	2
2	Snow	SL	2
3	Wind	WL	2
4	Dead_Min	DL	2

Member Section Forces

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
1	1	M1	1	1	0	-0.347	0	0	0
2			2	0.898	0	-0.174	0	-1.169	0
3			3	0.797	0	0	0	-1.558	0
4			4	0.695	0	0.174	0	-1.169	0
5			5	0.594	0	0.347	0	0	0
6	1	M2	1	0.594	0	-0.347	0	0	0
7			2	0.695	0	-0.174	0	-1.169	0
8			3	0.797	0	0	0	-1.558	0
9			4	0.898	0	0.174	0	-1.169	0
10			5	1	0	0.347	0	0	0
11	2	M1	1	0.696	0	-0.203	0	0	0
12			2	0.609	0	-0.101	0	-0.682	0
13			3	0.521	0	0	0	-0.909	0
14			4	0.434	0	0.101	0	-0.682	0
15			5	0.347	0	0.203	0	0	0
16	2	M2	1	0.347	0	-0.203	0	0	0
17			2	0.434	0	-0.101	0	-0.682	0
18			3	0.521	0	0	0	-0.909	0
19			4	0.609	0	0.101	0	-0.682	0
20			5	0.696	0	0.203	0	0	0
21	3	M1	1	0.046	0	0.036	0	0	0
22			2	0.019	0	0.018	0	0.122	0
23			3	-0.008	0	0	0	0.162	0
24			4	-0.035	0	-0.018	0	0.122	0
25			5	-0.062	0	-0.036	0	0	0
26	3	M2	1	-0.062	0	0.036	0	0	0
27			2	-0.035	0	0.018	0	0.122	0
28			3	-0.008	0	0	0	0.162	0
29			4	0.019	0	-0.018	0	0.122	0
30			5	0.046	0	-0.036	0	0	0
31	4	M1	1	-0.085	0	0.082	0	0	0
32			2	-0.099	0	0.041	0	0.275	0
33			3	-0.113	0	0	0	0.367	0
34			4	-0.126	0	-0.041	0	0.275	0
35			5	-0.14	0	-0.082	0	0	0
36	4	M2	1	-0.14	0	0.082	0	0	0

Member Section Forces (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
37			2	-0.126	0	0.041	0	0.275	0
38			3	-0.113	0	0	0	0.367	0
39			4	-0.099	0	-0.041	0	0.275	0
40			5	-0.085	0	-0.082	0	0	0
41	5	M1	1	0.665	0	-0.231	0	0	0
42			2	0.598	0	-0.115	0	-0.777	0
43			3	0.53	0	0	0	-1.037	0
44			4	0.463	0	0.115	0	-0.777	0
45			5	0.395	0	0.231	0	0	0
46	5	M2	1	0.395	0	-0.231	0	0	0
47			2	0.463	0	-0.115	0	-0.777	0
48			3	0.53	0	0	0	-1.037	0
49			4	0.598	0	0.115	0	-0.777	0
50			5	0.665	0	0.231	0	0	0
51	6	M1	1	0.558	0	-0.194	0	0	0
52			2	0.501	0	-0.097	0	-0.652	0
53			3	0.445	0	0	0	-0.87	0
54			4	0.388	0	0.097	0	-0.652	0
55			5	0.331	0	0.194	0	0	0
56	6	M2	1	0.331	0	-0.194	0	0	0
57			2	0.388	0	-0.097	0	-0.652	0
58			3	0.445	0	0	0	-0.87	0
59			4	0.501	0	0.097	0	-0.652	0
60			5	0.558	0	0.194	0	0	0
61	7	M1	1	-0.366	0	0.214	0	0	0
62			2	-0.366	0	0.107	0	0.719	0
63			3	-0.366	0	0	0	0.959	0
64			4	-0.366	0	-0.107	0	0.719	0
65			5	-0.366	0	-0.214	0	0	0
66	7	M2	1	-0.366	0	0.214	0	0	0
67			2	-0.366	0	0.107	0	0.719	0
68			3	-0.366	0	0	0	0.959	0
69			4	-0.366	0	-0.107	0	0.719	0
70			5	-0.366	0	-0.214	0	0	0

Maximum Member Section Forces

	LC	Member Label		Axial[k]	Loc[ft]	y Shear[k]	Loc[ft]	z Shear[k]	Loc[ft]	Torque[k-ft]	Loc[ft]	y-y Moment[k-ft]	Loc[ft]	z-z Moment[k-ft]	Loc[ft]
1	1	M1	max	1	0	0	17.954	0.347	17.954	0	17.954	0	17.954	0	17.954
2			min	0.594	17.954	0	0	-0.347	0	0	0	-1.558	8.977	0	0
3	1	M2	max	1	17.954	0	17.954	0.347	17.954	0	17.954	0	17.954	0	17.954
4			min	0.594	0	0	0	-0.347	0	0	0	-1.558	8.977	0	0
5	2	M1	max	0.696	0	0	17.954	0.203	17.954	0	17.954	0	17.954	0	17.954
6			min	0.347	17.954	0	0	-0.203	0	0	0	-0.909	8.977	0	0
7	2	M2	max	0.696	17.954	0	17.954	0.203	17.954	0	17.954	0	17.954	0	17.954
8			min	0.347	0	0	0	-0.203	0	0	0	-0.909	8.977	0	0
9	3	M1	max	0.046	0	0	17.954	0.036	0	0	17.954	0.162	8.977	0	17.954
10			min	-0.062	17.954	0	0	-0.036	17.954	0	0	0	0	0	0
11	3	M2	max	0.046	17.954	0	17.954	0.036	0	0	17.954	0.162	8.977	0	17.954
12			min	-0.062	0	0	0	-0.036	17.954	0	0	0	0	0	0
13	4	M1	max	-0.085	0	0	17.954	0.082	0	0	17.954	0.367	8.977	0	17.954
14			min	-0.14	17.954	0	0	-0.082	17.954	0	0	0	0	0	0
15	4	M2	max	-0.085	17.954	0	17.954	0.082	0	0	17.954	0.367	8.977	0	17.954
16			min	-0.14	0	0	0	-0.082	17.954	0	0	0	0	0	0
17	5	M1	max	0.665	0	0	17.954	0.231	17.954	0	17.954	0	17.954	0	17.954
18			min	0.395	17.954	0	0	-0.231	0	0	0	-1.037	8.977	0	0
19	5	M2	max	0.665	17.954	0	17.954	0.231	17.954	0	17.954	0	17.954	0	17.954
20			min	0.395	0	0	0	-0.231	0	0	0	-1.037	8.977	0	0
21	6	M1	max	0.558	0	0	17.954	0.194	17.954	0	17.954	0	17.954	0	17.954
22			min	0.331	17.954	0	0	-0.194	0	0	0	-0.87	8.977	0	0
23	6	M2	max	0.558	17.954	0	17.954	0.194	17.954	0	17.954	0	17.954	0	17.954
24			min	0.331	0	0	0	-0.194	0	0	0	-0.87	8.977	0	0
25	7	M1	max	-0.366	17.954	0	17.954	0.214	0	0	17.954	0.959	8.977	0	17.954
26			min	-0.366	0	0	0	-0.214	17.954	0	0	0	0	0	0
27	7	M2	max	-0.366	17.954	0	17.954	0.214	0	0	17.954	0.959	8.977	0	17.954

Maximum Member Section Forces (Continued)

	LC	Member Label		Axial[k]	Loc[ft]	y Shear[k]	Loc[ft]	z Shear[k]	Loc[ft]	Torque[k-ft]	Loc[ft]	y-y Moment[k-ft]	Loc[ft]	z-z Moment[k-ft]	Loc[ft]
28			min	-0.366	0	0	0	-0.214	17.954	0	0	0	0	0	0

Member Section Deflections Strength

	LC	Member Label	Sec	x [in]	y [in]	z [in]	x Rotate[rad]	(n) L/y' Ratio	(n) L/z' Ratio
1	1	M1	1	0	0	0	0	NC	NC
2			2	0	0	0.709	0	NC	303
3			3	-0.001	0	0.996	0	NC	216
4			4	-0.002	0	0.712	0	NC	302
5			5	-0.003	0	0.004	0	NC	NC
6	1	M2	1	0.003	0	0.004	0	NC	NC
7			2	0.002	0	0.712	0	NC	302
8			3	0.001	0	0.996	0	NC	216
9			4	0	0	0.709	0	NC	303
10			5	0	0	0	0	NC	NC
11	2	M1	1	0	0	0	0	NC	NC
12			2	0	0	0.414	0	NC	520
13			3	0	0	0.582	0	NC	370
14			4	-0.001	0	0.415	0	NC	518
15			5	-0.002	0	0.003	0	NC	NC
16	2	M2	1	0.002	0	0.003	0	NC	NC
17			2	0.001	0	0.415	0	NC	518
18			3	0	0	0.582	0	NC	370
19			4	0	0	0.414	0	NC	520
20			5	0	0	0	0	NC	NC
21	3	M1	1	0	0	0	0	NC	NC
22			2	0	0	-0.074	0	NC	2923
23			3	0	0	-0.103	0	NC	2082
24			4	0	0	-0.074	0	NC	2922
25			5	0	0	0	0	NC	NC
26	3	M2	1	0	0	0	0	NC	NC
27			2	0	0	-0.074	0	NC	2922
28			3	0	0	-0.103	0	NC	2082
29			4	0	0	-0.074	0	NC	2923
30			5	0	0	0	0	NC	NC
31	4	M1	1	0	0	0	0	NC	NC
32			2	0	0	-0.167	0	NC	1291
33			3	0	0	-0.234	0	NC	919
34			4	0	0	-0.167	0	NC	1289
35			5	0	0	0	0	NC	NC
36	4	M2	1	0	0	0	0	NC	NC
37			2	0	0	-0.167	0	NC	1289
38			3	0	0	-0.234	0	NC	919
39			4	0	0	-0.167	0	NC	1291
40			5	0	0	0	0	NC	NC

Member Section Deflections Service

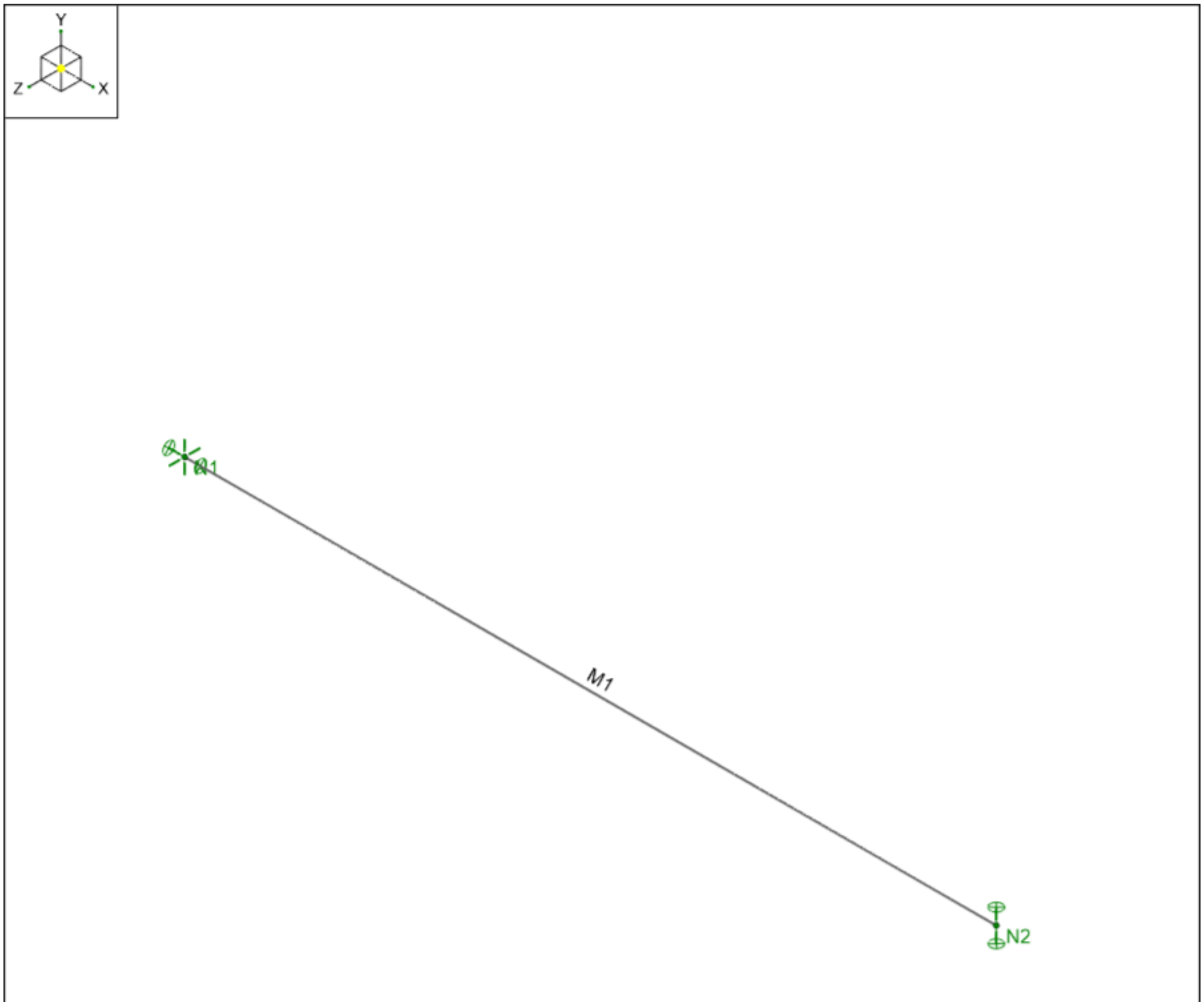
	LC	Member Label	Sec	x [in]	y [in]	z [in]	x Rotate[rad]	(n) L/y' Ratio	(n) L/z' Ratio
1	5	M1	1	0	0	0	0	NC	NC
2			2	0	0	0.472	0	NC	456
3			3	0	0	0.663	0	NC	325
4			4	-0.001	0	0.473	0	NC	455
5			5	-0.002	0	0.003	0	NC	NC
6	5	M2	1	0.002	0	0.003	0	NC	NC
7			2	0.001	0	0.473	0	NC	455
8			3	0	0	0.663	0	NC	325
9			4	0	0	0.472	0	NC	456
10			5	0	0	0	0	NC	NC
11	6	M1	1	0	0	0	0	NC	NC
12			2	0	0	0.396	0	NC	544
13			3	0	0	0.556	0	NC	387
14			4	-0.001	0	0.397	0	NC	542
15			5	-0.001	0	0.002	0	NC	NC
16	6	M2	1	0.001	0	0.002	0	NC	NC
17			2	0.001	0	0.397	0	NC	542

Member Section Deflections Service (Continued)

	LC	Member Label	Sec	x [in]	y [in]	z [in]	x Rotate[rad]	(n) L/y' Ratio	(n) L/z' Ratio
18			3	0	0	0.556	0	NC	387
19			4	0	0	0.396	0	NC	544
20			5	0	0	0	0	NC	NC
21	7	M1	1	0	0	0	0	NC	NC
22			2	0	0	-0.436	0	NC	493
23			3	0	0	-0.613	0	NC	351
24			4	0	0	-0.437	0	NC	492
25			5	0.001	0	-0.002	0	NC	NC
26	7	M2	1	-0.001	0	-0.002	0	NC	NC
27			2	0	0	-0.437	0	NC	492
28			3	0	0	-0.613	0	NC	351
29			4	0	0	-0.436	0	NC	493
30			5	0	0	0	0	NC	NC

Node Reactions

	LC	Node Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
1	1	N1	0.688	0.804	0	0	0	0
2	1	N2	-0.688	0.804	0	0	0	0
3	1	N3	NC	NC	NC	LOCKED	LOCKED	NC
4	1	Totals:	0	1.609	0			
5	1	COG (ft):	X: 15.5	Y: 4.53	Z: 0			
6	2	N1	0.499	0.526	0	0	0	0
7	2	N2	-0.499	0.526	0	0	0	0
8	2	N3	NC	NC	NC	LOCKED	LOCKED	NC
9	2	Totals:	0	1.052	0			
10	2	COG (ft):	X: 15.5	Y: 4.53	Z: 0			
11	3	N1	0.058	-0.008	0	0	0	0
12	3	N2	-0.058	-0.008	0	0	0	0
13	3	N3	NC	NC	NC	LOCKED	LOCKED	NC
14	3	Totals:	0	-0.016	0			
15	3	COG (ft):	X: 15.5	Y: 4.53	Z: 0			
16	4	N1	-0.033	-0.114	0	0	0	0
17	4	N2	0.033	-0.114	0	0	0	0
18	4	N3	NC	NC	NC	LOCKED	LOCKED	NC
19	4	Totals:	0	-0.227	0			
20	4	COG (ft):	X: 15.5	Y: 4.53	Z: 0			
21	5	N1	0.458	0.535	0	0	0	0
22	5	N2	-0.458	0.535	0	0	0	0
23	5	N3	NC	NC	NC	LOCKED	LOCKED	NC
24	5	Totals:	0	1.07	0			
25	5	COG (ft):	X: 15.5	Y: 4.53	Z: 0			
26	6	N1	0.384	0.449	0	0	0	0
27	6	N2	-0.384	0.449	0	0	0	0
28	6	N3	NC	NC	NC	LOCKED	LOCKED	NC
29	6	Totals:	0	0.898	0			
30	6	COG (ft):	X: 15.5	Y: 4.53	Z: 0			
31	7	N1	-0.208	-0.369	0	0	0	0
32	7	N2	0.208	-0.369	0	0	0	0
33	7	N3	NC	NC	NC	LOCKED	LOCKED	NC
34	7	Totals:	0	-0.738	0			
35	7	COG (ft):	X: 15.5	Y: 4.53	Z: 0			



JES		SK-9
ACJ		Aug 05, 2021
		Roof.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	30	0	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot [k-ft/rad]	Y Rot [k-ft/rad]
1	N1	Reaction	Reaction	Reaction	Reaction	
2	N2		Reaction			Reaction

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.092	0	15	30
2	M1	Y	0	-0.092	0	15

Load Combinations

	Description	Solve	BLC	Factor
1	ASD	Yes	1	0.6
2		Yes	1	0.6

Load Combination Design

	Description	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
1	ASD		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
2			Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

Basic Load Cases

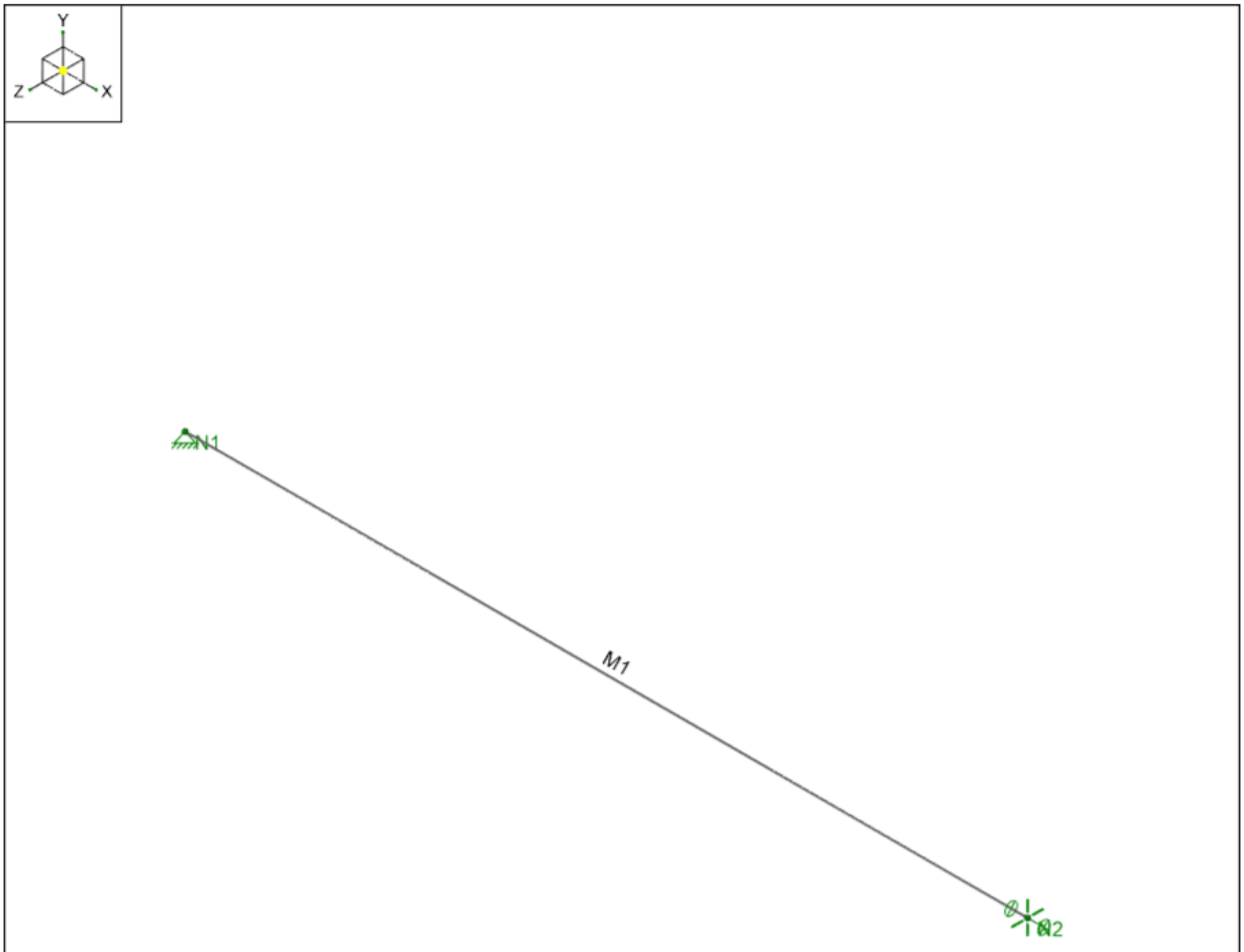
	BLC Description	Category	Distributed
1	Wind	None	2

Member Section Forces

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
1	1	M1	1	0	0.416	0	0	0	0
2			2	0	0.312	0	0	0	-2.859
3			3	0	0	0	0	0	-4.158
4			4	0	-0.312	0	0	0	-2.859
5			5	0	-0.416	0	0	0	0

Maximum Member Section Forces

	LC	Member Label		Axial[k]	Loc[ft]	y Shear[k]	Loc[ft]	z Shear[k]	Loc[ft]	Torque[k-ft]	Loc[ft]	y-y Moment[k-ft]	Loc[ft]	z-z Moment[k-ft]	Loc[ft]
1	1	M1	max	0	30	0.416	0	0	30	0	30	0	30	0	30
2			min	0	0	-0.416	30	0	0	0	0	0	0	-4.158	15



JES		SK-10
ACJ		Aug 05, 2021
		Attic.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	30	0	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot [k-ft/rad]
1	N1	Reaction	Reaction	Reaction	
2	N2		Reaction	Reaction	Reaction

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.132	-0.21	0	%100

Load Combinations

	Description	Solve	BLC	Factor
1	LC1	Yes	1	1

Load Combination Design

	Description	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
1	LC1		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

Basic Load Cases

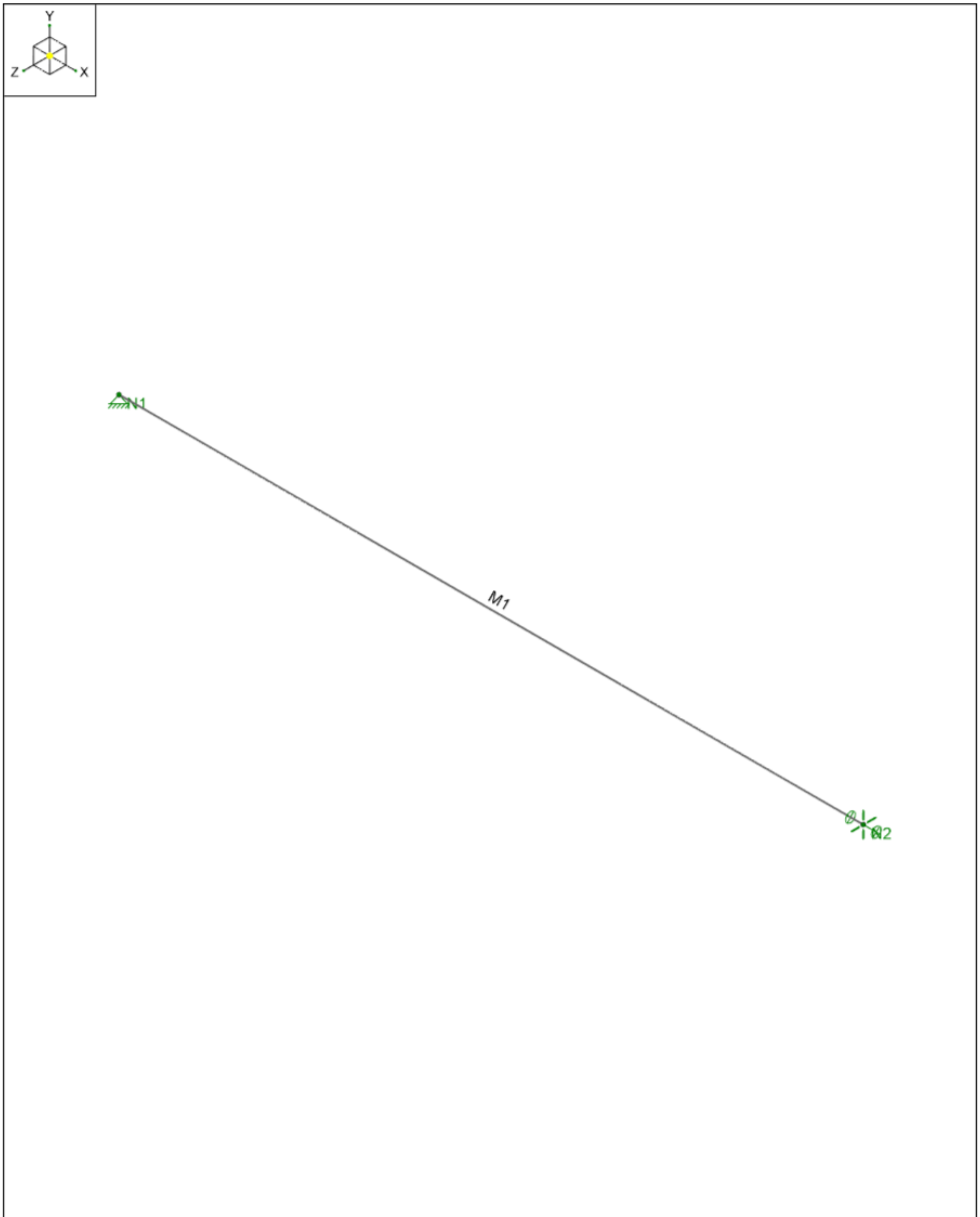
	BLC Description	Category	Distributed
1	Wind	None	1

Member Section Forces

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
1	1	M1	1	0	2.365	0	0	0	0
2			2	0	1.305	0	0	0	-13.856
3			3	0	0.098	0	0	0	-19.209
4			4	0	-1.256	0	0	0	-14.958
5			5	0	-2.757	0	0	0	0

Maximum Member Section Forces

	LC	Member Label		Axial[k]	Loc[ft]	y Shear[k]	Loc[ft]	z Shear[k]	Loc[ft]	Torque[k-ft]	Loc[ft]	y-y Moment[k-ft]	Loc[ft]	z-z Moment[k-ft]	Loc[ft]
1	1	M1	max	0	30	2.365	0	0	30	0	30	0	30	0	30
2			min	0	0	-2.757	30	0	0	0	0	0	0	-19.237	15.625



JES		SK-11
ACJ		Aug 05, 2021
		2nd.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	36	0	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot [k-ft/rad]
1	N1	Reaction	Reaction	Reaction	
2	N2		Reaction	Reaction	Reaction

Member Distributed Loads

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.165	-0.165	0	%100

Load Combinations

	Description	Solve	BLC	Factor
1	LC1	Yes	1	1

Load Combination Design

	Description	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
1	LC1		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

Basic Load Cases

	BLC Description	Category	Distributed
1	Wind	None	1

Member Section Forces

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
1	1	M1	1	0	2.964	0	0	0	0
2			2	0	1.482	0	0	0	-20.007
3			3	0	0	0	0	0	-26.677
4			4	0	-1.482	0	0	0	-20.007
5			5	0	-2.964	0	0	0	0

Maximum Member Section Forces

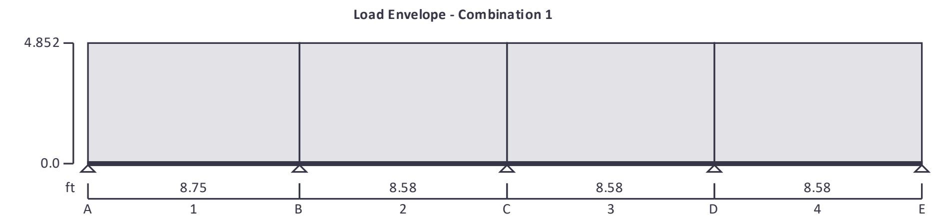
	LC	Member Label		Axial[k]	Loc[ft]	y Shear[k]	Loc[ft]	z Shear[k]	Loc[ft]	Torque[k-ft]	Loc[ft]	y-y Moment[k-ft]	Loc[ft]	z-z Moment[k-ft]	Loc[ft]
1	1	M1	max	0	36	2.964	0	0	36	0	36	0	36	0	36
2			min	0	0	-2.964	36	0	0	0	0	0	0	-26.677	18

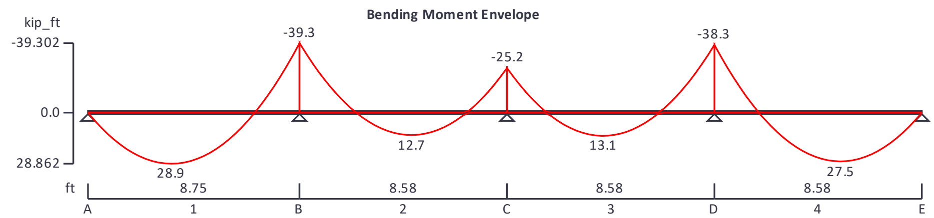
Tekla Tedds	Project CLT Home Design				Job Ref.	
	Section Interior Basement Girder				Sheet no./rev. 1	
	Calc. by ACJ	Date 7/22/2021	Chk'd by	Date	App'd by	Date

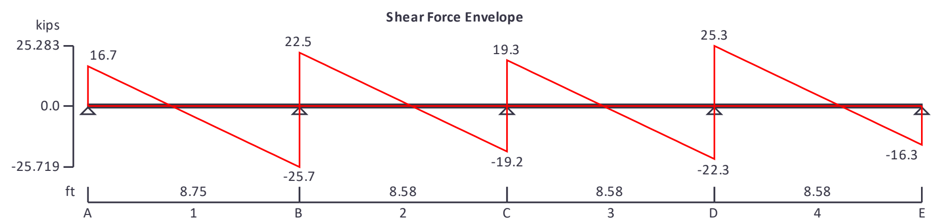
STEEL BEAM ANALYSIS & DESIGN (AISC360-10)

In accordance with AISC360-10 using the LRFD method

Tedds calculation version 3.0.15







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free
Support C	Vertically restrained
	Rotationally free
Support D	Vertically restrained
	Rotationally free
Support E	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam × 1
	wd - Dead full UDL 1.308 kips/ft
	wl - Live full UDL 2.038 kips/ft

Load combinations

Load combination 1	Support A	Dead × 1.20
		Live × 1.60
		Dead × 1.20
		Live × 1.60
	Support B	Dead × 1.20
		Live × 1.60
		Dead × 1.20
		Live × 1.60
	Support C	Dead × 1.20
		Live × 1.60
		Dead × 1.20
		Live × 1.60
	Support D	Dead × 1.20
		Live × 1.60
		Dead × 1.20
		Live × 1.60
	Support E	Dead × 1.20
		Live × 1.60

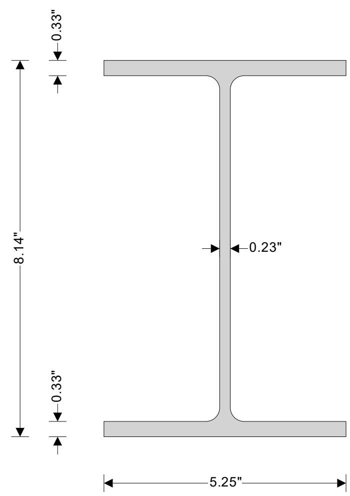
Analysis results

Maximum moment	M_{max} = **28.9** kips_ft	M_{min} = **-39.3** kips_ft
Maximum moment span 1	M_{s1_max} = **28.9** kips_ft	M_{s1_min} = **-39.3** kips_ft
Maximum moment span 2	M_{s2_max} = **12.7** kips_ft	M_{s2_min} = **-39.3** kips_ft
Maximum moment span 3	M_{s3_max} = **13.1** kips_ft	M_{s3_min} = **-38.3** kips_ft
Maximum moment span 4	M_{s4_max} = **27.5** kips_ft	M_{s4_min} = **-38.3** kips_ft
Maximum shear	V_{max} = **25.3** kips	V_{min} = **-25.7** kips
Maximum shear span 1	V_{s1_max} = **16.7** kips	V_{s1_min} = **-25.7** kips
Maximum shear span 2	V_{s2_max} = **22.5** kips	V_{s2_min} = **-19.2** kips
Maximum shear span 3	V_{s3_max} = **19.3** kips	V_{s3_min} = **-22.3** kips
Maximum shear span 4	V_{s4_max} = **25.3** kips	V_{s4_min} = **-16.3** kips
Deflection	δ_{max} = **0.1** in	δ_{min} = **0** in
Deflection span 1	δ_{s1_max} = **0.1** in	δ_{s1_min} = **0** in
Deflection span 2	δ_{s2_max} = **0** in	δ_{s2_min} = **0** in
Deflection span 3	δ_{s3_max} = **0** in	δ_{s3_min} = **0** in
Deflection span 4	δ_{s4_max} = **0.1** in	δ_{s4_min} = **0** in
Maximum reaction at support A	R_{A_max} = **16.7** kips	R_{A_min} = **16.7** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **4.6** kips	
Unfactored live load reaction at support A	R_{A_Live} = **7** kips	
Maximum reaction at support B	R_{B_max} = **48.2** kips	R_{B_min} = **48.2** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **13.2** kips	
Unfactored live load reaction at support B	R_{B_Live} = **20.2** kips	
Maximum reaction at support C	R_{C_max} = **38.5** kips	R_{C_min} = **38.5** kips
Unfactored dead load reaction at support C	R_{C_Dead} = **10.5** kips	

Unfactored live load reaction at support C	R_{C_Live} = **16.2** kips	
Maximum reaction at support D	R_{D_max} = **47.6** kips	R_{D_min} = **47.6** kips
Unfactored dead load reaction at support D	R_{D_Dead} = **13** kips	
Unfactored live load reaction at support D	R_{D_Live} = **20** kips	
Maximum reaction at support E	R_{E_max} = **16.3** kips	R_{E_min} = **16.3** kips
Unfactored dead load reaction at support E	R_{E_Dead} = **4.5** kips	
Unfactored live load reaction at support E	R_{E_Live} = **6.9** kips	

Section details

Section type	**W 8x18 (AISC 15th Edn (v15.0))**
ASTM steel designation	**A992**
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi



Resistance factors

Resistance factor for tensile yielding	ϕ_{ty} = **0.90**
Resistance factor for tensile rupture	ϕ_{tr} = **0.75**
Resistance factor for compression	ϕ_c = **0.90**
Resistance factor for flexure	ϕ_b = **0.90**

Lateral bracing

Span 1 has lateral bracing at supports only
Span 2 has lateral bracing at supports only
Span 3 has lateral bracing at supports only
Span 4 has lateral bracing at supports only

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio	$b_f / (2 \times t_f)$ = **7.95**	
Limiting ratio for compact section	$\lambda_{pff} = 0.38 \times \sqrt{[E / F_y]}$ = **9.15**	
Limiting ratio for non-compact section	$\lambda_{rff} = 1.0 \times \sqrt{[E / F_y]}$ = **24.08**	Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio $(d - 2 \times k) / t_w = \mathbf{29.91}$

Limiting ratio for compact section $\lambda_{pwf} = 3.76 \times \sqrt{[E / F_y]} = \mathbf{90.55}$

Limiting ratio for non-compact section $\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]} = \mathbf{137.27}$ Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{25.719}$ kips

Web area $A_w = d \times t_w = \mathbf{1.872}$ in^2

Web plate buckling coefficient $k_v = \mathbf{5}$

Web shear coefficient - eq G2-3 $C_v = \mathbf{1}$

Nominal shear strength – eq G2-1 $V_n = 0.6 \times F_y \times A_w \times C_v = \mathbf{56.166}$ kips

Resistance factor for shear $\phi_v = \mathbf{1.00}$

Design shear strength $V_c = \phi_v \times V_n = \mathbf{56.166}$ kips

PASS - Design shear strength exceeds required shear strength

Design of members for flexure in the major axis at span 1 - Chapter F

Required flexural strength $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{39.302}$ kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y \times Z_x = \mathbf{70.833}$ kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length $L_b = L_{s1} = \mathbf{105}$ in

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 \times r_y \times \sqrt{[E / F_y]} = \mathbf{52.135}$ in

Distance between flange centroids $h_o = d - t_f = \mathbf{7.81}$ in

$c = \mathbf{1}$

$r_{ts} = \sqrt{[\sqrt{(I_y \times C_w)} / S_x]} = \mathbf{1.432}$ in

Limiting unbraced length for inelastic LTB - eq F2-6

$$L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{[(J \times c / (S_x \times h_o)) + \sqrt{((J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_y / E)^2)}]} = \mathbf{162.078} \text{ in}$$

Cross-section mono-symmetry parameter $R_m = \mathbf{1.000}$

Moment at quarter point of segment $M_A = \mathbf{25.000}$ kips_ft

Moment at center-line of segment $M_B = \mathbf{26.783}$ kips_ft

Moment at three quarter point of segment $M_C = \mathbf{5.349}$ kips_ft

Maximum moment in segment $M_{abs} = \mathbf{39.302}$ kips_ft

Lateral torsional buckling modification factor - eq F1-1 $C_b = 12.5 \times M_{abs} / [2.5 \times M_{abs} + 3 \times M_A + 4 \times M_B + 3 \times M_C] = \mathbf{1.657}$

Nominal flexural strength for lateral torsional buckling - eq F2-2 $M_{nltb} = C_b \times [M_p - (M_p - 0.7 \times F_y \times S_x) \times (L_b - L_p) / (L_r - L_p)] =$ $\mathbf{96.273}$ kips_ft

Nominal flexural strength $M_n = \min(M_{nyld}, M_{nltb}) = \mathbf{70.833}$ kips_ft

Design flexural strength $M_c = \phi_b \times M_n = \mathbf{63.750}$ kips_ft

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to live loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 = \mathbf{0.292}$ in

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{0.075}$ in

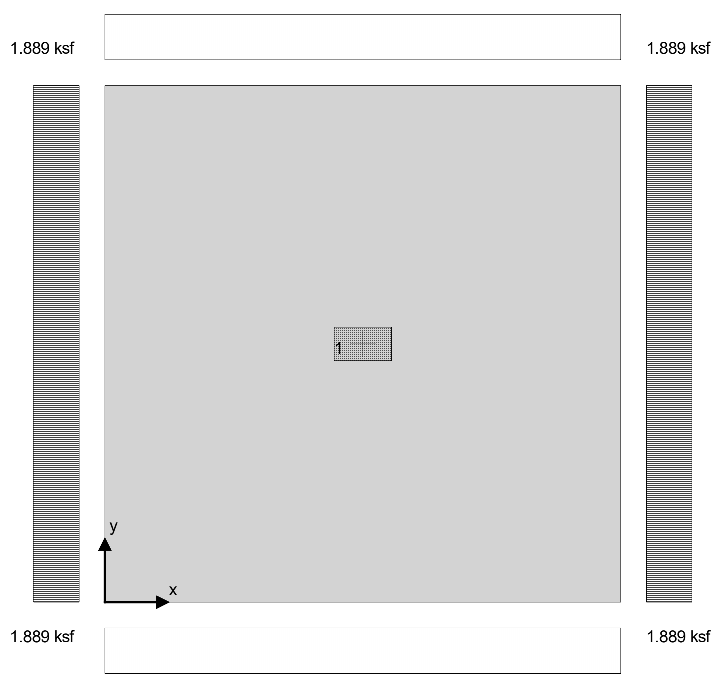
PASS - Maximum deflection does not exceed deflection limit

Foundation analysis & design (ACI318) in accordance with ACI318-14

Tedds calculation version 3.2.10

FOOTING ANALYSIS

Length of foundation	L_x = **4.5** ft
Width of foundation	L_y = **4.5** ft
Foundation area	$A = L_x \times L_y$ = **20.25** ft^2
Depth of foundation	h = **12** in
Depth of soil over foundation	h_{soil} = **0** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column	l_{x1} = **6.00** in
Width of column	l_{y1} = **3.50** in
position in x-axis	x_1 = **27.00** in
position in y-axis	y_1 = **27.00** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **2** ksf
Density of soil	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**

Foundation loads

Dead surcharge load F_{Dsur} = **50** psf

Live surcharge load F_{Lsur} = **40** psf

Self weight $F_{swt} = h \times \gamma_{conc}$ = **150** psf

Column no.1 loads

Dead load in z F_{Dz1} = **13.2** kips

Dead load in z F_{Dz1} = **13.2** kips

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Dead load in z F_{Dz1} = **13.2** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.426)

1.0D + 1.0L (0.945)

Combination 2 results: 1.0D + 1.0L

Forces on foundation

Force in z-axis $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{Dsur}) + \gamma_L \times A \times F_{Lsur} + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1}$ = **38.3** kips

Moments on foundation

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{Dsur}) \times L_x / 2) + \gamma_L \times A \times F_{Lsur} \times L_x / 2 + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **86.1** kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{Dsur}) \times L_y / 2) + \gamma_L \times A \times F_{Lsur} \times L_y / 2 + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **86.1** kip_ft

Uplift verification

Vertical force F_{dz} = **38.26** kips

PASS - Foundation is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.889** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **1.889** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **1.889** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **1.889** ksf

Minimum base pressure $q_{min} = \min(q_1,q_2,q_3,q_4) =$ **1.889** ksf

Maximum base pressure $q_{max} = \max(q_1,q_2,q_3,q_4) =$ **1.889** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross} =$ **2** ksf

$q_{max} / q_{allow} =$ **0.945**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete $f'_c =$ **3000** psi

Yield strength of reinforcement $f_y =$ **60000** psi

Compression-controlled strain limit (21.2.2) $\varepsilon_{ty} =$ **0.00200**

Cover to reinforcement $c_{nom} =$ **3** in

Concrete type Normal weight

Concrete modification factor $\lambda =$ **1.00**

Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.247)

1.2D + 1.6L + 0.5Lr (0.612)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on foundation

Ultimate force in z-axis $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{Dsur}) + \gamma_L \times A \times F_{Lsur} + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} =$ **54.3** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{Dsur}) \times L_x / 2) + \gamma_L \times A \times F_{Lsur} \times L_x / 2 + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) =$ **122.2** kip_ft

Ultimate moment in y-axis, about y is 0 $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{Dsur}) \times L_y / 2) + \gamma_L \times A \times F_{Lsur} \times L_y / 2 + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) =$ **122.2** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{ux} = M_{ux} / F_{uz} - L_x / 2 =$ **0** in

Eccentricity of base reaction in y-axis $e_{uy} = M_{uy} / F_{uz} - L_y / 2 =$ **0** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **2.682** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **2.682** ksf

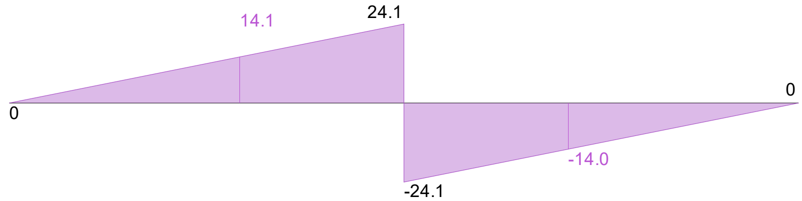
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **2.682** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **2.682** ksf

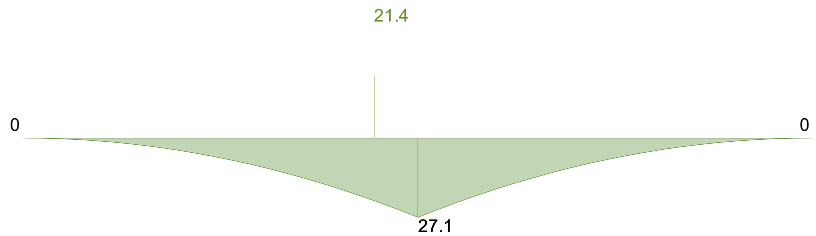
Minimum ultimate base pressure $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **2.682** ksf

Maximum ultimate base pressure $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **2.682** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max}$ = **21.418** kip_ft

Tension reinforcement provided 6 No.4 bottom bars (9.5 in c/c)

Area of tension reinforcement provided $A_{sx.bot.prov}$ = **1.2** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_y \times h$ = **1.166** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) $s_{max} = \min(2 \times h, 18\text{ in})$ = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement $d = h - c_{nom} - \phi_{y.bot} - \phi_{x.bot} / 2$ = **8.250** in

Depth of compression block $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.523** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **0.615** in

Strain in tensile reinforcement $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.03723**

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = 0.004$ = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **47.931** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9)$ = **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n$ = **43.138** kip_ft

$M_{u.x.max} / \phi M_n$ = **0.497**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force $V_{u.x}$ = **14.054** kips

Depth to reinforcement $d_v = h - c_{nom} - \phi_{x.bot} / 2$ = **8.75** in

Shear strength reduction factor ϕ_v = **0.75**

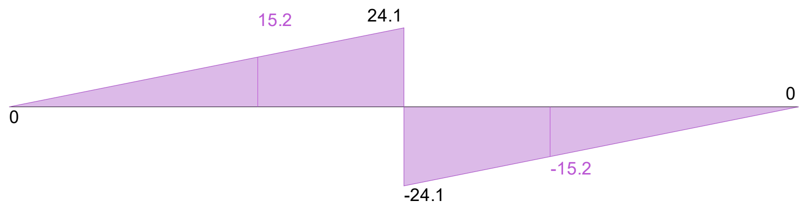
Nominal shear capacity (Eq. 22.5.5.1) $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_y \times d_v$ = **51.76** kips

Design shear capacity $\phi V_n = \phi_v \times V_n$ = **38.82** kips

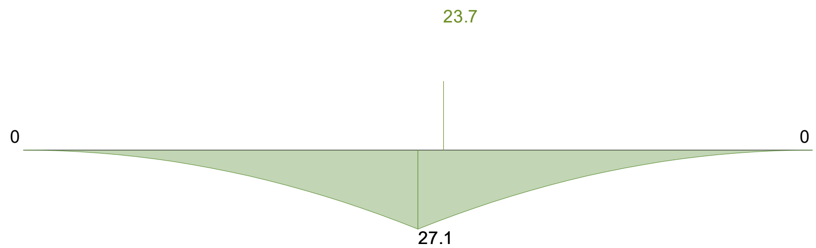
$V_{u.x} / \phi V_n$ = **0.362**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max}$ = **23.692** kip_ft

Tension reinforcement provided 6 No.4 bottom bars (9.5 in c/c)

Area of tension reinforcement provided $A_{sy.bot.prov}$ = **1.2** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_x \times h$ = **1.166** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) s_{max} = min(2 × h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement $d = h - c_{nom} - \phi_{y.bot} / 2$ = **8.750** in

Depth of compression block $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.523** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **0.615** in

Strain in tensile reinforcement $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.03967**

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = 0.004$ = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **50.931** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9)$ = **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n$ = **45.838** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.517**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force $V_{u.y}$ = **15.161** kips

Depth to reinforcement $d_v = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **8.25** in

Shear strength reduction factor — ϕ_v = **0.75**

Nominal shear capacity (Eq. 22.5.5.1) — $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_x \times d_v$ = **48.802** kips

Design shear capacity — $\phi V_n = \phi_v \times V_n$ = **36.602** kips

$V_{u.y} / \phi V_n$ = **0.414**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement — d_{v2} = **8.5** in

Shear perimeter length (22.6.4) — l_{xp} = **14.500** in

Shear perimeter width (22.6.4) — l_{yp} = **12.000** in

Shear perimeter (22.6.4) — $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2})$ = **53.000** in

Shear area — $A_p = l_{x,perim} \times l_{y,perim}$ = **174.000** in^2

Surcharge loaded area — $A_{sur} = A_p - l_{x1} \times l_{y1}$ = **153.000** in^2

Ultimate bearing pressure at center of shear area — $q_{up.avg}$ = **2.682** ksf

Ultimate shear load — $F_{up} = \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{Dsur} + \gamma_L \times A_{sur} \times F_{Lsur} - q_{up.avg} \times A_p$ = **45.268** kips

Ultimate shear stress from vertical load — $v_{ug} = \max(F_{up} / (b_o \times d_{v2}),0\text{ psi})$ = **100.484** psi

Column geometry factor (Table 22.6.5.2) — $\beta = l_{x1} / l_{y1}$ = **1.71**

Column location factor (22.6.5.3) — α_s =**40**

Concrete shear strength (22.6.5.2) — $v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **237.346** psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **460.914** psi

$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **219.089** psi

$v_{cp} = \min(v_{cpa},v_{cpb},v_{cpc})$ = **219.089** psi

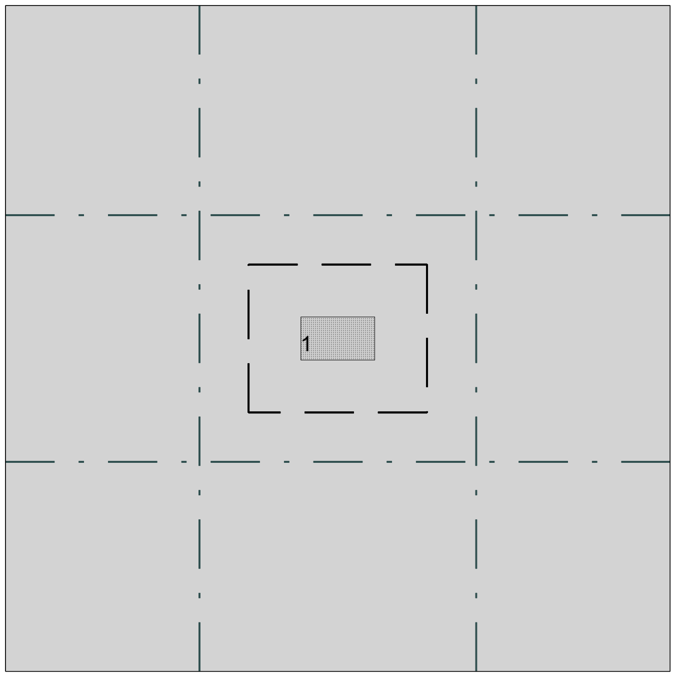
Shear strength reduction factor — ϕ_v = **0.75**

Nominal shear stress capacity (Eq. 22.6.1.2) — $v_n = v_{cp}$ = **219.089** psi

Design shear stress capacity (8.5.1.1(d)) — $\phi v_n = \phi_v \times v_n$ = **164.317** psi

$v_{ug} / \phi v_n$ = **0.612**

PASS - Design shear stress capacity exceeds ultimate shear stress load


1

Foundation analysis & design (ACI318) in accordance with ACI318-14

Tedds calculation version 3.2.10

FOOTING ANALYSIS

Length of foundation L_x = **4** ft

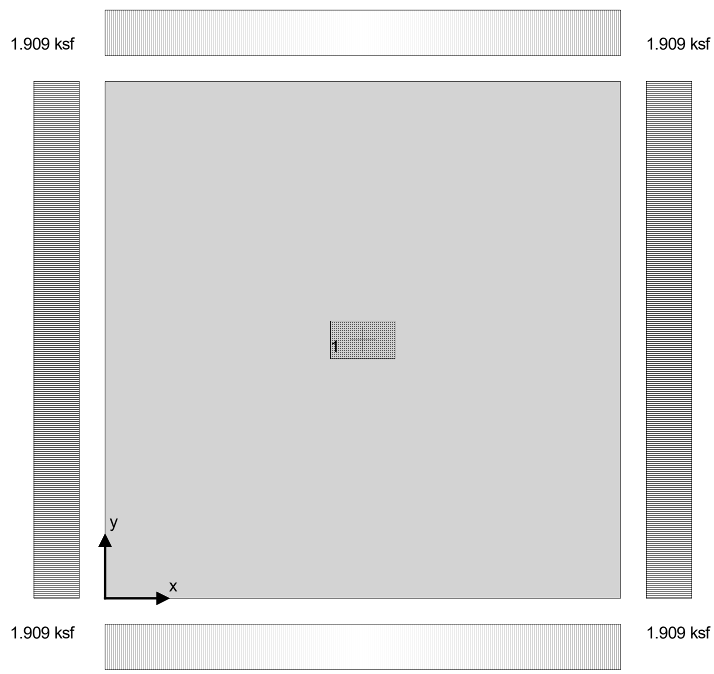
Width of foundation L_y = **4** ft

Foundation area $A = L_x \times L_y$ = **16** ft^2

Depth of foundation h = **12** in

Depth of soil over foundation h_{soil} = **0** in

Density of concrete γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column l_{x1} = **6.00** in

Width of column l_{y1} = **3.50** in

position in x-axis x_1 = **24.00** in

position in y-axis y_1 = **24.00** in

Soil properties

Gross allowable bearing pressure q_{allow_Gross} = **2** ksf

Density of soil γ_{soil} = **120.0** lb/ft^3

Angle of internal friction ϕ_b = **30.0** deg

Design base friction angle δ_{bb} = **30.0** deg

Coefficient of base friction $\tan(\delta_{bb})$ = **0.577**

Foundation loads

Dead surcharge load F_{Dsur} = **50** psf

Live surcharge load F_{Lsur} = **40** psf

Self weight $F_{swt} = h \times \gamma_{conc}$ = **150** psf

Column no.1 loads

Dead load in z F_{Dz1} = **10.5** kips

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Dead load in z F_{Dz1} = **10.5** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.428)

1.0D + 1.0L (0.954)

Combination 2 results: 1.0D + 1.0L

Forces on foundation

Force in z-axis $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{Dsur}) + \gamma_L \times A \times F_{Lsur} + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1}$ = **30.5** kips

Moments on foundation

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{Dsur}) \times L_x / 2) + \gamma_L \times A \times F_{Lsur} \times L_x / 2 + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **61.1** kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{Dsur}) \times L_y / 2) + \gamma_L \times A \times F_{Lsur} \times L_y / 2 + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **61.1** kip_ft

Uplift verification

Vertical force F_{dz} = **30.54** kips

PASS - Foundation is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.909** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **1.909** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **1.909** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **1.909** ksf

Minimum base pressure $q_{min} = \min(q_1,q_2,q_3,q_4) =$ **1.909** ksf

Maximum base pressure $q_{max} = \max(q_1,q_2,q_3,q_4) =$ **1.909** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross} =$ **2** ksf

$q_{max} / q_{allow} =$ **0.954**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete $f'_c =$ **3000** psi

Yield strength of reinforcement $f_y =$ **60000** psi

Compression-controlled strain limit (21.2.2) $\varepsilon_{ty} =$ **0.00200**

Cover to reinforcement $c_{nom} =$ **3** in

Concrete type Normal weight

Concrete modification factor $\lambda =$ **1.00**

Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.193)

1.2D + 1.6L + 0.5Lr (0.481)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on foundation

Ultimate force in z-axis $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{Dsur}) + \gamma_L \times A \times F_{Lsur} + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} =$ **43.4** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{Dsur}) \times L_x / 2) + \gamma_L \times A \times F_{Lsur} \times L_x / 2 + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) =$ **86.8** kip_ft

Ultimate moment in y-axis, about y is 0 $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{Dsur}) \times L_y / 2) + \gamma_L \times A \times F_{Lsur} \times L_y / 2 + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) =$ **86.8** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{ux} = M_{ux} / F_{uz} - L_x / 2 =$ **0** in

Eccentricity of base reaction in y-axis $e_{uy} = M_{uy} / F_{uz} - L_y / 2 =$ **0** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **2.711** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **2.711** ksf

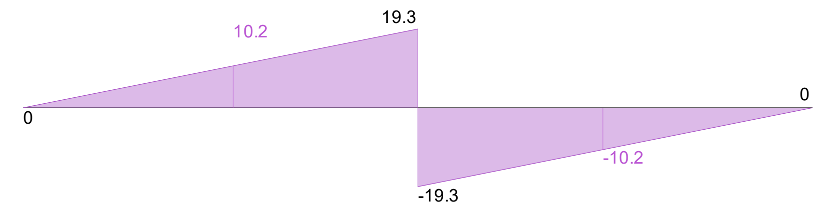
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **2.711** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **2.711** ksf

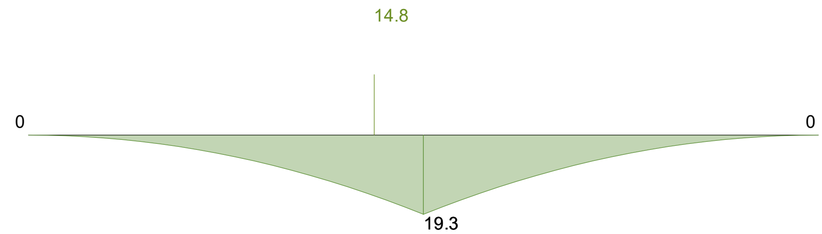
Minimum ultimate base pressure $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **2.711** ksf

Maximum ultimate base pressure $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{2.711}$ ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max} = \mathbf{14.757}$ kip_ft

Tension reinforcement provided 6 No.4 bottom bars (8.3 in c/c)

Area of tension reinforcement provided $A_{sx.bot.prov} = \mathbf{1.2}$ in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_y \times h = \mathbf{1.037}$ in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) $s_{max} = \min(2 \times h, 18\text{ in}) = \mathbf{18}$ in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement $d = h - c_{nom} - \phi_{y.bot} - \phi_{x.bot} / 2 = \mathbf{8.250}$ in

Depth of compression block $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y) = \mathbf{0.588}$ in

Neutral axis factor $\beta_1 = \mathbf{0.85}$

Depth to neutral axis $c = a / \beta_1 = \mathbf{0.692}$ in

Strain in tensile reinforcement $\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.03276}$

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = 0.004 = \mathbf{0.00400}$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) = \mathbf{47.735}$ kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity $\phi M_n = \phi_f \times M_n = \mathbf{42.962}$ kip_ft

$M_{u.x.max} / \phi M_n = \mathbf{0.343}$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force $V_{u.x} = \mathbf{10.238}$ kips

Depth to reinforcement $d_v = h - c_{nom} - \phi_{x.bot} / 2 = \mathbf{8.75}$ in

Shear strength reduction factor $\phi_v = \mathbf{0.75}$

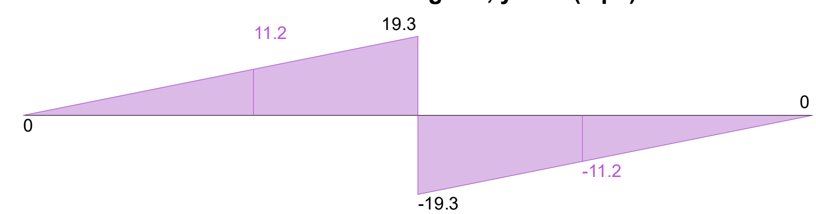
Nominal shear capacity (Eq. 22.5.5.1) $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_y \times d_v = \mathbf{46.009}$ kips

Design shear capacity $\phi V_n = \phi_v \times V_n =$ **34.507** kips

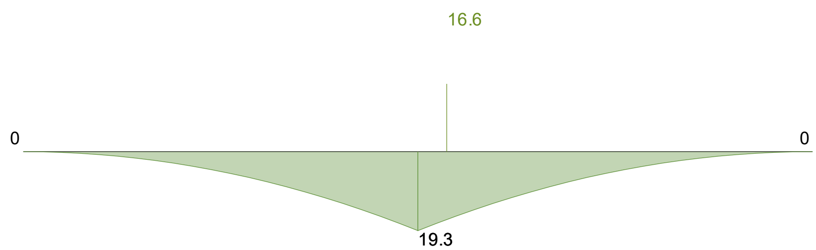
$V_{u.x} / \phi V_n =$ **0.297**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max} =$ **16.554** kip_ft

Tension reinforcement provided 6 No.4 bottom bars (8.3 in c/c)

Area of tension reinforcement provided $A_{sy.bot.prov} =$ **1.2** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_x \times h =$ **1.037** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) $s_{max} = \min(2 \times h, 18\text{ in}) =$ **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement $d = h - c_{nom} - \phi_{y.bot} / 2 =$ **8.750** in

Depth of compression block $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x) =$ **0.588** in

Neutral axis factor $\beta_1 =$ **0.85**

Depth to neutral axis $c = a / \beta_1 =$ **0.692** in

Strain in tensile reinforcement $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.03493**

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = 0.004 =$ **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) =$ **50.735** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9) =$ **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n =$ **45.662** kip_ft

$M_{u.y.max} / \phi M_n =$ **0.363**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force $V_{u.y} =$ **11.235** kips

Depth to reinforcement $d_v = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2 =$ **8.25** in

Shear strength reduction factor ϕ_v = **0.75**

Nominal shear capacity (Eq. 22.5.5.1) $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v$ = **43.38** kips

Design shear capacity $\phi V_n = \phi_v \times V_n$ = **32.535** kips

$V_{u.y} / \phi V_n$ = **0.345**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement d_{v2} = **8.5** in

Shear perimeter length (22.6.4) l_{xp} = **14.500** in

Shear perimeter width (22.6.4) l_{yp} = **12.000** in

Shear perimeter (22.6.4) $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2})$ = **53.000** in

Shear area $A_p = l_{x,perim} \times l_{y,perim}$ = **174.000** in^2

Surcharge loaded area $A_{sur} = A_p - l_{x1} \times l_{y1}$ = **153.000** in^2

Ultimate bearing pressure at center of shear area $q_{up.avg}$ = **2.711** ksf

Ultimate shear load $F_{up} = \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{Dsur} + \gamma_L \times A_{sur} \times F_{Lsur} - q_{up.avg} \times A_p$ = **35.593** kips

Ultimate shear stress from vertical load $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi})$ = **79.007** psi

Column geometry factor (Table 22.6.5.2) $\beta = l_{x1} / l_{y1}$ = **1.71**

Column location factor (22.6.5.3) α_s =**40**

Concrete shear strength (22.6.5.2) $v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})}$ = **237.346** psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})}$ = **460.914** psi

$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})}$ = **219.089** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc})$ = **219.089** psi

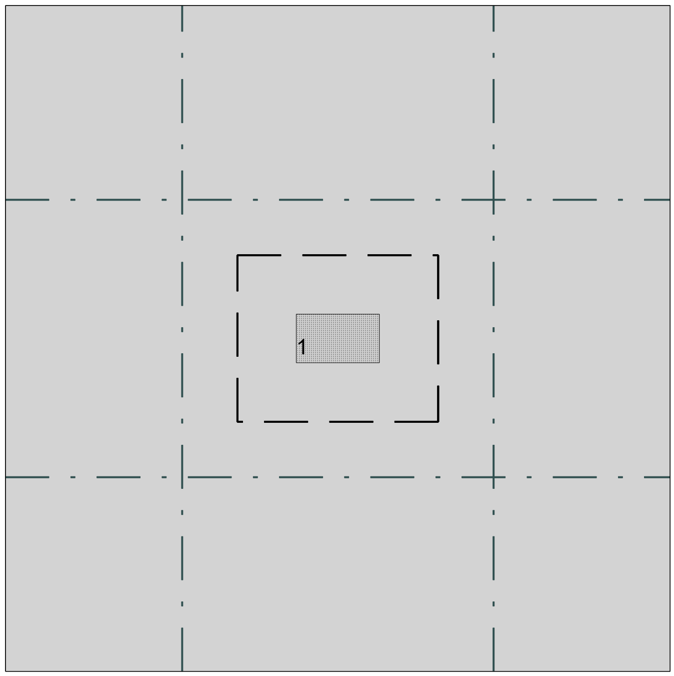
Shear strength reduction factor ϕ_v = **0.75**

Nominal shear stress capacity (Eq. 22.6.1.2) $v_n = v_{cp}$ = **219.089** psi

Design shear stress capacity (8.5.1.1(d)) $\phi v_n = \phi_v \times v_n$ = **164.317** psi

$v_{ug} / \phi v_n$ = **0.481**

PASS - Design shear stress capacity exceeds ultimate shear stress load



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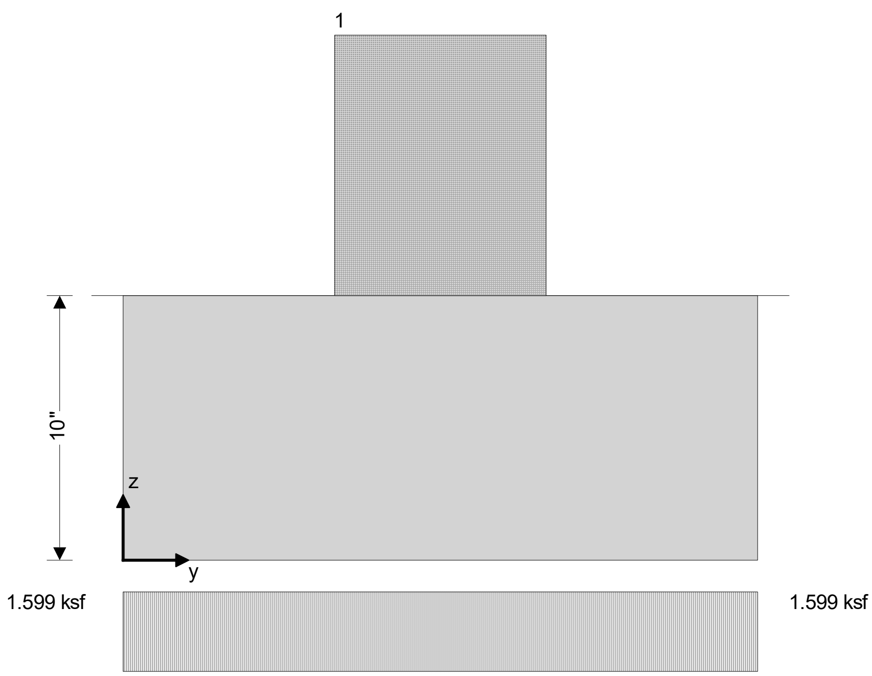
Foundation analysis & design (ACI318) in accordance with ACI318-14

Tedds calculation version 3.2.10

FOOTING ANALYSIS

Length of foundation	L_x = **1** ft
Width of foundation	L_y = **2** ft
Foundation area	$A = L_x \times L_y$ = **2** ft^2
Depth of foundation	h = **10** in
Depth of soil over foundation	h_{soil} = **0** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3



Wall no.1 details

Width of wall	l_{y1} = **8** in
position in y-axis	y_1 = **12** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **2** ksf
Density of soil	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**
Dead surcharge load	F_{Dsur} = **50** psf
Self weight	$F_{swt} = h \times \gamma_{conc}$ = **125** psf

Wall no.1 loads per linear foot

Dead load in z — F_{Dz1} = **2.0** kips

Dead load in z — F_{Dz1} = **2.0** kips

Dead load in z — F_{Dz1} = **2.0** kips

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Dead load in z — F_{Dz1} = **2.0** kips

Dead load in z — F_{Dz1} = **2.0** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.587)

1.0D + 1.0L (0.799)

1.0D + 1.0S (0.687)

Combination 2 results: 1.0D + 1.0L

Forces on foundation per linear foot

Force in z-axis — $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{Dsur}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1}$ = **3.2** kips

Moments on foundation per linear foot

Moment in y-axis, about y is 0 — $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{Dsur}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **3.2** kip_ft

Uplift verification

Vertical force — F_{dz} = **3.197** kips

PASS - Foundation is not subject to uplift

Stability against sliding

Resistance due to base friction — $F_{RFriction} = \max(F_{dz}, 0 \text{ kN}) \times \tan(\delta_{bb})$ = **1.846** kips

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in y-axis — $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0.000** in

Strip base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dy} / L_y) / (L_y \times 1 \text{ ft})$ = **1.598** ksf

$q_2 = F_{dz} \times (1 + 6 \times e_{dy} / L_y) / (L_y \times 1 \text{ ft})$ = **1.598** ksf

Minimum base pressure — $q_{min} = \min(q_1,q_2)$ = **1.598** ksf

Maximum base pressure — $q_{max} = \max(q_1,q_2)$ = **1.598** ksf

Allowable bearing capacity

Allowable bearing capacity — $q_{allow} = q_{allow\_Gross} = \mathbf{2}$ ksf

$q_{max} / q_{allow} = \mathbf{0.799}$

PASS - Allowable bearing capacity exceeds design base pressure

<u>FOOTING DESIGN (ACI318)</u>

In accordance with ACI318-14

Material details

Compressive strength of concrete — $f'_c = \mathbf{3000}$ psi

Yield strength of reinforcement — $f_y = \mathbf{60000}$ psi

Compression-controlled strain limit (21.2.2) — $\varepsilon_{ty} = \mathbf{0.00200}$

Cover to reinforcement — $c_{nom} = \mathbf{3}$ in

Concrete type — Normal weight

Concrete modification factor — $\lambda = \mathbf{1.00}$

Wall type — Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.044)

1.2D + 1.6L + 0.5Lr (0.059)

1.2D + 1.6L + 0.5S (0.063)

1.2D + 1.0L + 1.6S (0.061)

Combination 3 results: 1.2D + 1.6L + 0.5S

Forces on foundation per linear foot

Ultimate force in z-axis — $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{Dsur}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} + \gamma_S \times F_{Sz1} = \mathbf{4.4}$ kips

Moments on foundation per linear foot

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{Dsur}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_S \times (F_{Sz1} \times y_1) = \mathbf{4.4}$ kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in y-axis — $e_{uy} = M_{uy} / F_{uz} - L_y / 2 = \mathbf{0.000}$ in

Strip base pressures

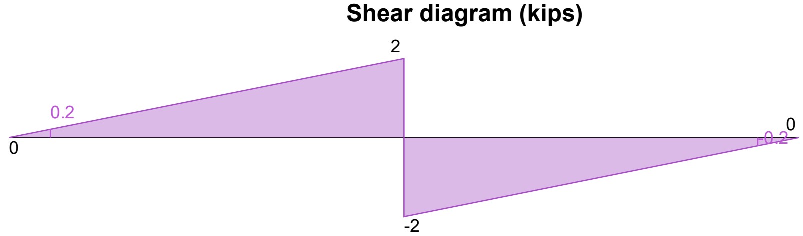
$q_{u1} = F_{uz} \times (1 - 6 \times e_{uy} / L_y) / (L_y \times 1 \text{ ft}) = \mathbf{2.188}$ ksf

$q_{u2} = F_{uz} \times (1 + 6 \times e_{uy} / L_y) / (L_y \times 1 \text{ ft}) = \mathbf{2.188}$ ksf

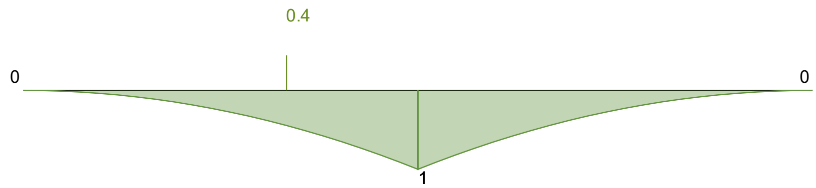
Minimum ultimate base pressure — $q_{umin} = \min(q_{u1},q_{u2}) = \mathbf{2.188}$ ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1},q_{u2}) = \mathbf{2.188}$ ksf

Shear diagram (kips)



Moment diagram (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment — $M_{u.y.max}$ = **0.44** kip_ft

Tension reinforcement provided — No.4 bars at 10.0 in c/c bottom

Area of tension reinforcement provided — $A_{sy.bot.prov}$ = **0.24** in^2

Minimum area of reinforcement (7.6.1.1) — $A_{s.min} = 0.0018 \times L_x \times h$ = **0.216** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (7.7.2.3) — s_{max} = min(3 × h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement — $d = h - c_{nom} - \phi_{y.bot} / 2$ = **6.750** in

Depth of compression block — $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.471** in

Neutral axis factor — β_1 = **0.85**

Depth to neutral axis — $c = a / \beta_1$ = **0.554** in

Strain in tensile reinforcement — $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.03358**

Minimum tensile strain(7.3.3.1) — $\varepsilon_{min} = 0.004$ = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity — $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **7.818** kip_ft

Flexural strength reduction factor — $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9)$ = **0.900**

Design moment capacity — $\phi M_n = \phi_f \times M_n$ = **7.036** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.063**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force — $V_{u.y}$ = **0.207** kips

Depth to reinforcement — $d_v = h - c_{nom} - \phi_{y.bot} / 2$ = **6.75** in

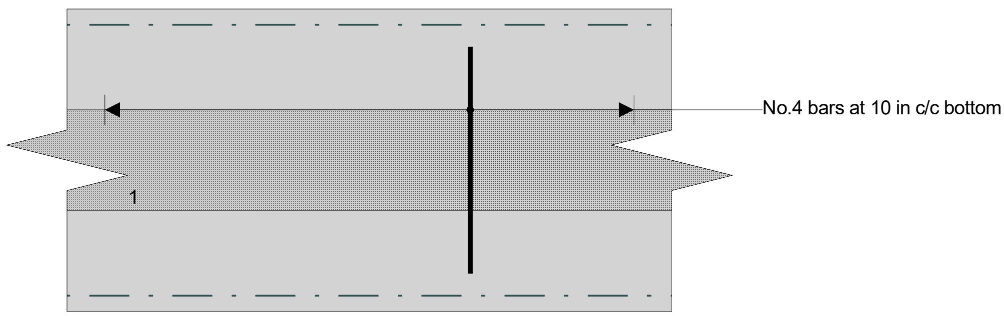
Shear strength reduction factor — ϕ_v = **0.75**

Nominal shear capacity (Eq. 22.5.5.1) — $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v$ = **8.873** kips

Design shear capacity — $\phi V_n = \phi_v \times V_n$ = **6.655** kips

$V_{u.y} / \phi V_n$ = **0.031**

PASS - Design shear capacity exceeds ultimate shear load



No.4 bars at 10 in c/c bottom
1

Appendix D – Reference Home Plans

Braced Wall Panel Method and Fastening Pattern

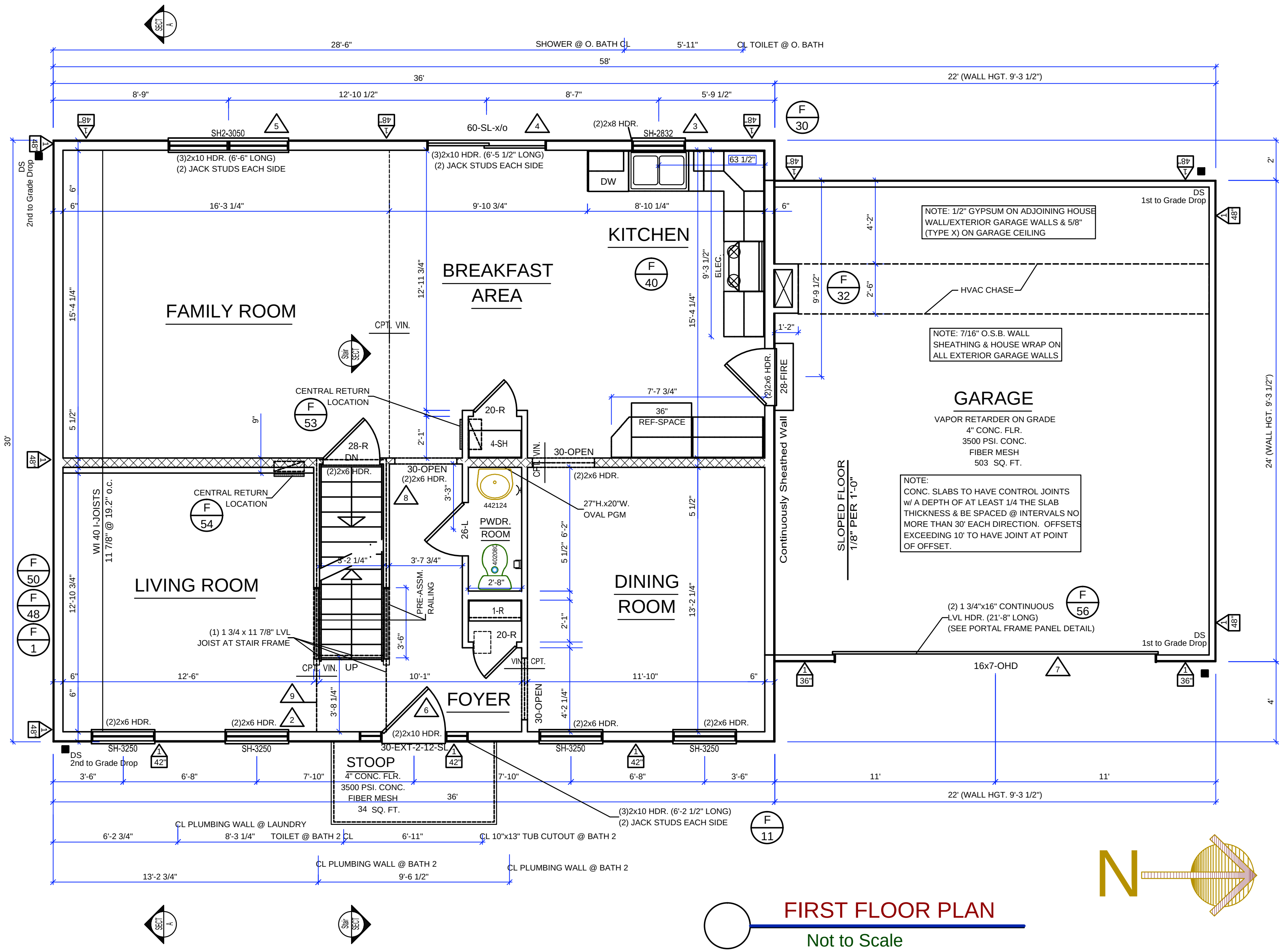
Mark	Method	Fastener	Fastening Schedule		CODE TABLE
			Edge	Field	
	WSP (Wood Structural Panel)	8D Common	6"	12"	2009 IRC R602.3(3)
2	GB (Gypsum Board)	1 1/4" Type "W" Screw	7"	16" Ceilings/24" Walls	2009 IRC R702.3.5
3	PFG (Intermittent Portal Frame at Garage)	Per Detail	Per Detail	Per Detail	2009 IRC R602.10.3.4
4	CS-WSP (Continuous Wood Structural Panel)	6D Common	6"	12"	2009 IRC R602.10.4.1
5	CS-G (Adjacent Garage Openings)	6D Common	6"	12"	2009 IRC R602.10.4.1
6	CS-G (Adjacent Garage Openings)	Per Detail	Per Detail	Per Detail	2009 IRC R602.10.4.1.1

1 — Method of Braced Wall Panel
48" — Length of Panel (inches)

Note: Table values are valid for the 2015 IRC as well.

FLOOR PLAN NOTES:

1) ALL INTERIOR PARTITIONS TO BE 2x4's UNLESS OTHERWISE NOTED.
2) ALL OPENINGS 48" AND LARGER REQUIRE DOUBLE JACK STUDS.
3) WRITTEN DIMENSIONS ON THESE DRAWINGS SHALL HAVE PRECEDENCE OVER SCALED DIMENSIONS.
4) ALL INTERIOR DIMENSIONS ARE FROM FACE OF STUD TO FACE OF STUD.
5) ALL STRUCTURAL MEMBERS (FLOOR AND CEILING JOIST, RAFTERS, HEADERS, BEAMS) MUST NOT BE CHANGED WITHOUT VERIFICATION & APPROVAL FROM DESIGN DEPARTMENT.
6) FIELD VERIFY ALL DIMENSIONS PRIOR TO CONSTRUCTION.
7) ALL HEADERS TO BE (2) 2x10 UNLESS NOTED OTHERWISE.
8) ALL LUMBER TO BE S.P.F. #2 OR BETTER UNLESS NOTED OTHERWISE.
9) ADD AN APPOVED AIR BARRIER TO ALL EXPOSED FIBERGLASS INSULATION (INCLUDING BEHIND TUBS ON EXTERIOR WALLS)
10)PSK REQUIRED ON ALL EXPOSED INSULATION IN UNFINISHED AREAS AND ENCLOSED CHASES.
11)INCREASE ENTRY DOOR ROUGH OPENING HEIGHT +3/4" WHEN THE FINISHED FLOOR IS 3/4" HARDWOOD



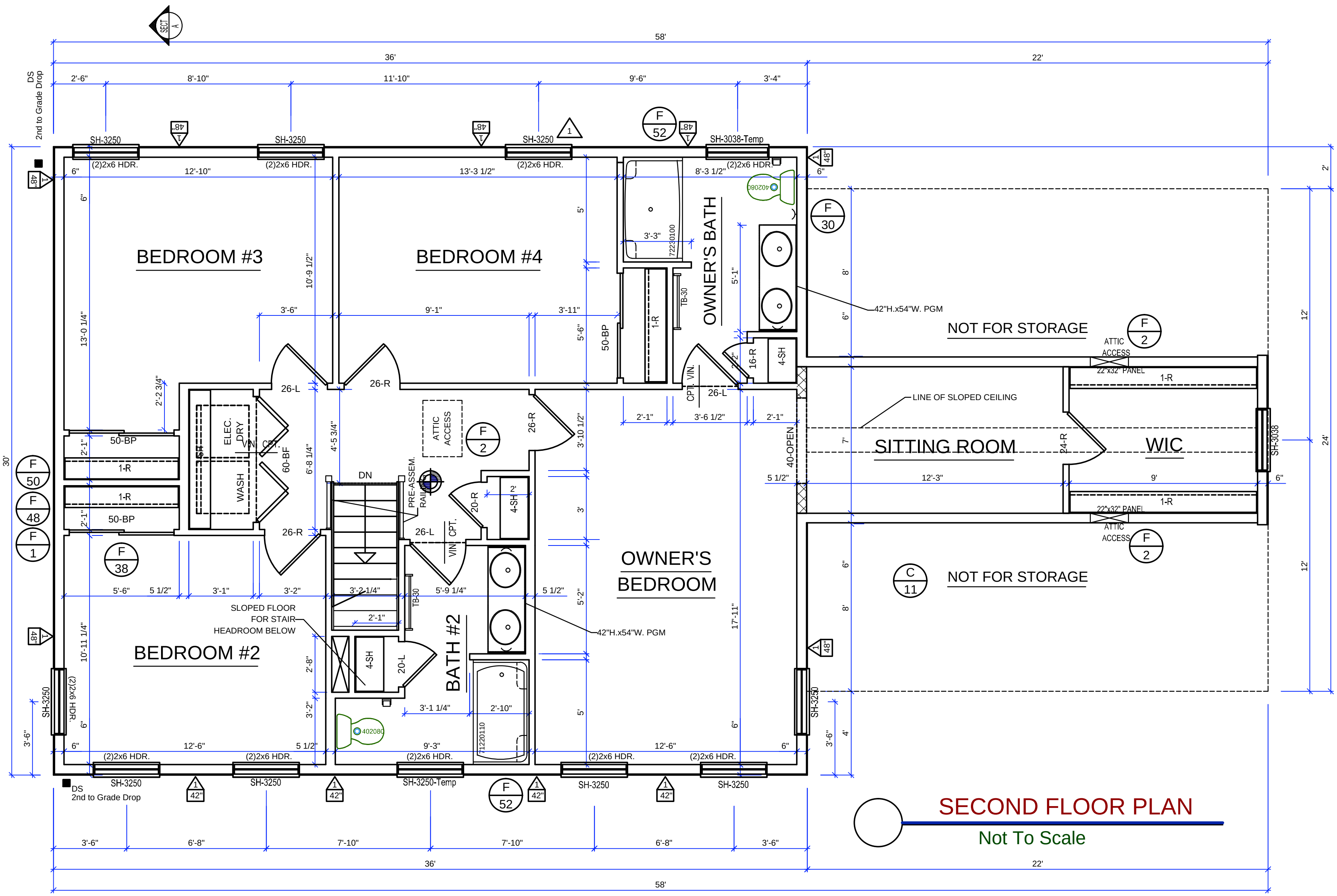
FIRST FLOOR PLAN
Not to Scale

Braced Wall Panel Method and Fastening Pattern

Mark	Method	Fastener	Fastening Schedule		CODE TABLE
			Edge	Field	
	WSP (Wood Structural Panel)	8D Common	6"	12"	2009 IRC R602.3(3)
2	GB (Gypsum Board)	1 1/4" Type "W" Screw	7"	16" Ceilings/24" Walls	2009 IRC R702.3.5
3	PFG (Intermittent Portal Frame at Garage)	Per Detail	Per Detail	Per Detail	2009 IRC R602.10.3.4
4	CS-WSP (Continuous Wood Structural Panel)	6D Common	6"	12"	2009 IRC R602.10.4.1
5	CS-G (Adjacent Garage Openings)	6D Common	6"	12"	2009 IRC R602.10.4.1
6	CS-G (Adjacent Garage Openings)	Per Detail	Per Detail	Per Detail	2009 IRC R602.10.4.1.1

1 — Method of Braced Wall Panel
48" — Length of Panel (inches)

Note: Table values are valid for the 2015 IRC as well.



SECOND FLOOR PLAN
Not To Scale

POURED WALL REBAR SIZE AND SPACING SPECIFICATIONS

WALL HEIGHT	UNBALANCED BACKFILL	WALL THICKNESS	REBAR SIZE AND SPACING		CODE TABLE
			HORIZONTAL	VERTICAL	
8'-0"	8'-0"	10"	2[a]	PLAIN CONCRETE	2009 IRC R404.1.2(8)
8'-0"	≤ 7'-0"	8"	3[b]	PLAIN CONCRETE	ACI 332-08 TABLE A.4
9'-0"	9'-0"	10"	3[a]	#5 REBAR @ 29" O.C.[c]	2009 IRC R404.1.2(8)
9'-0"	8'-0"	8"	4[b]	#5 REBAR @ 30" O.C.	ACI 332-08 TABLE A.4
9'-0"	<7'-0"	8"	4[b]	PLAIN CONCRETE	ACI 332-08 TABLE A.4

FOOTNOTES:

[a] - For wall heights < 8' - (1) 5 bar near mid-height of the wall story.
For wall heights > 8' - (1) 5 bar near third points in the wall story (2009 IRC R404.1.2(1))

[b] - For wall heights < 8' - (1) 5 bar near mid-height of the wall story.
For wall heights > 8' - (1) 5 bar near the third points in the wall story (ACI 332-08 R7.2.8)

[c] - Vertical reinforcement bars of different size than specified in R404.1.2(8) are permitted in accordance with Table R404.1.2(9)

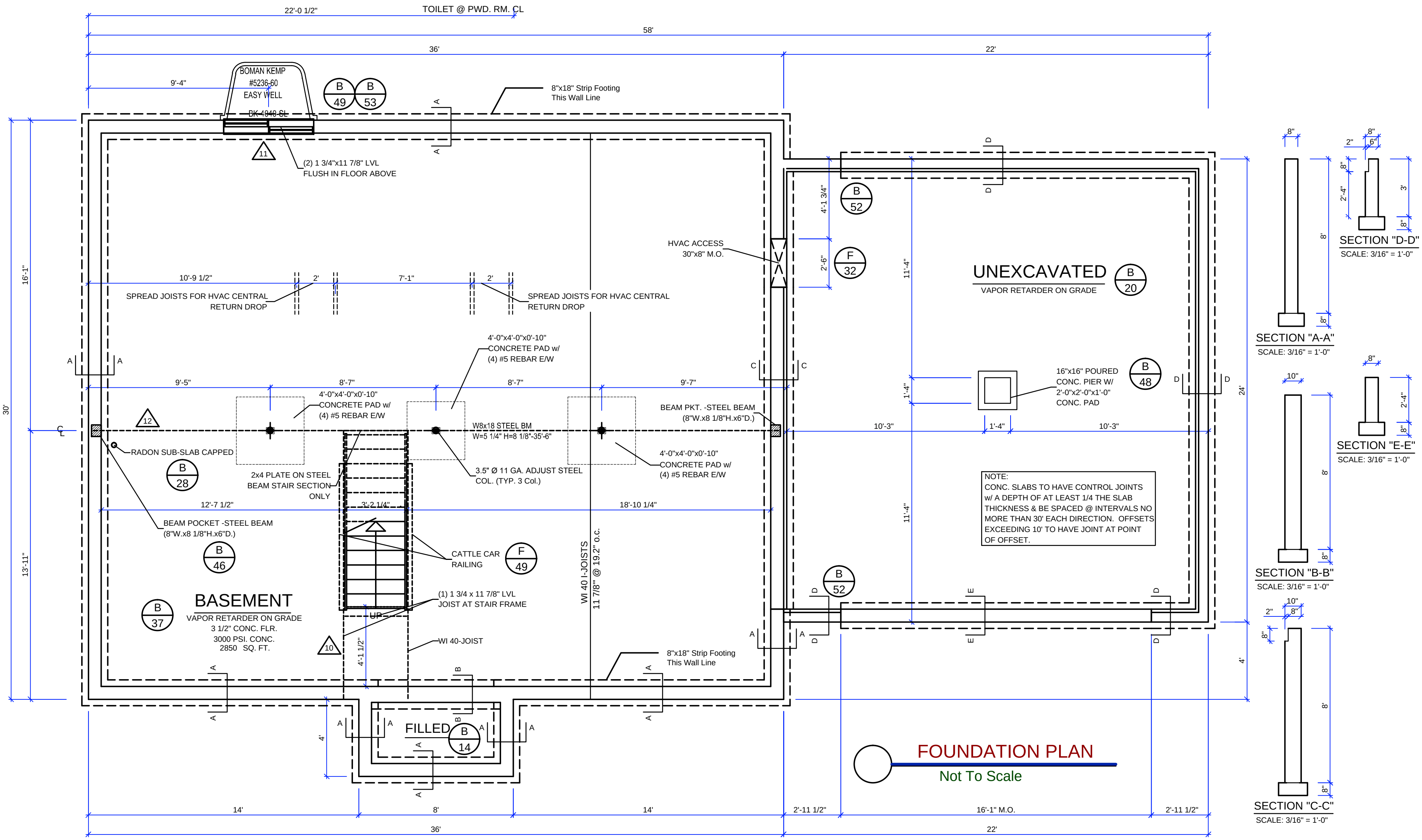
Note: Table values are valid for the 2015 IRC as well.

NOTES:

1.) MINIMUM CONCRETE CONPRESSIVE STRENGTH FOR WALLS: f'c=3,000 psi.
2.) MINIMUM REBAR YIELD STRENGTH: fy=60 ksi.
3.) MAXIMUM DESIGN LATERAL PRESSURE: 45psf/ft.
4.) LAP SPLICE LENGTHS TO BE 30" FOR #4 BARS AND 38" FOR #5 BARS (2009 IRC R611.5.1(1)

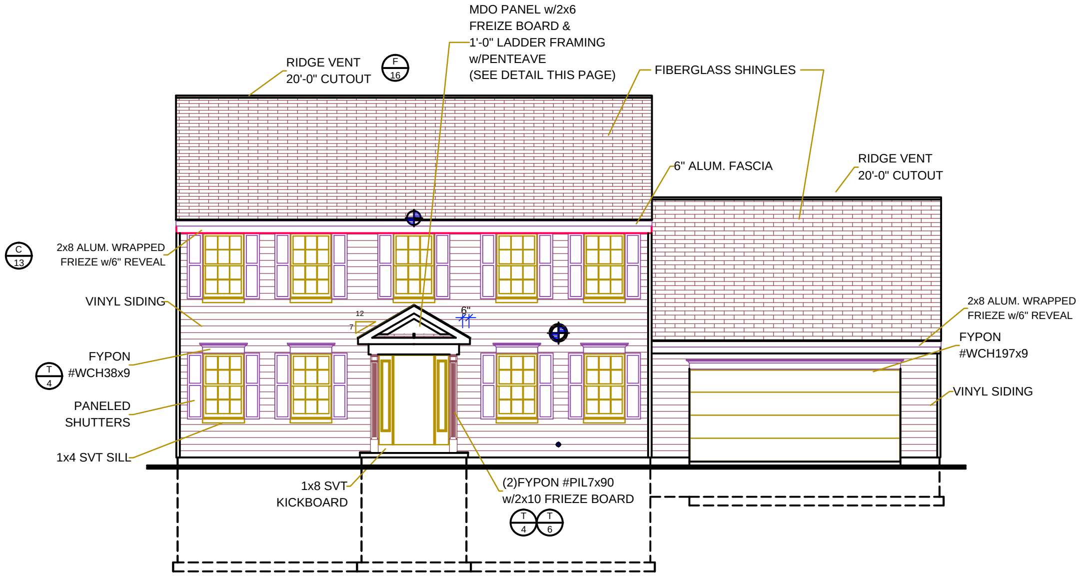
FOUNDATION NOTES:

1) MAIN BEAM(S) - STEEL BEAM AND/OR LVL BEAM, AS PER PLAN
2) STEEL BEAM - MECHANICALLY FASTEN 2x WOOD PLATE TO TOP FLANGE OF STEEL BEAM MIN. 24" O.C. STAGGERED BOTH SIDES AND 12" FROM EACH END
3) ADJUSTABLE STEEL COLUMNS, AS PER PLANS
4) FORMED CONCRETE PADS, AS PER PLANS
5) 1/2" DIA. x 18" ANCHOR BOLTS (IN BLOCK) OR 10" ANCHOR BOLTS (IN CONRETE) *AS PER CODE* w/SPACING @ 6'-0" O.C. & 1'-0" FROM END OF PLATE & FROM CORNERS,(EMBEDDED 15" -BLOCK & 7" CONCRETE).
6) HOLES TO BE DRILLED IN I-JOIST AS PER MANUFACTURER. REFER TO I-JOIST INSTALLATION DETAILS.
7) MIN. COMPRESSIVE STRENGTH FOR FOOTINGS TO BE 2500 PSI.
8) CONCRETE FOOTING SIZES:
(a) 8" WALL = 16" WIDE FORMED FOOTING
(b) 10" WALL = 18" WIDE FORMED FOOTING
(c) 12" WALL = 20" WIDE FORMED FOOTING
(d) TRENCH FOOTING TO BE 18" WIDE (MIN.)
9) VERTICAL GYPSUM FIRE PROTECTION METHOD (WEB ARMOR™) (ESR 1144,ESR 1336) TO MEET FIRE SAFETY RESISTANCE REQUIREMENT(PA ONLY)

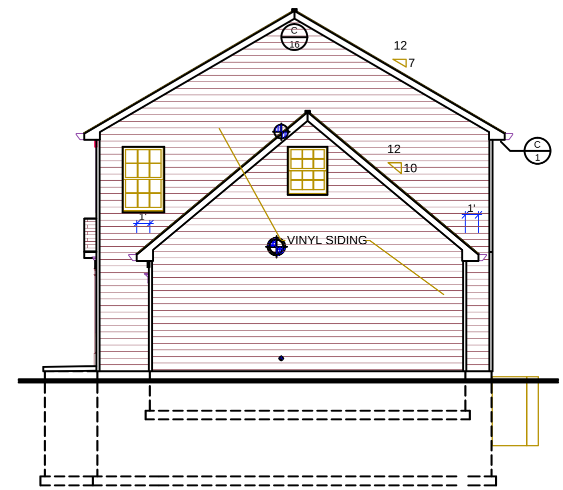


FOUNDATION PLAN

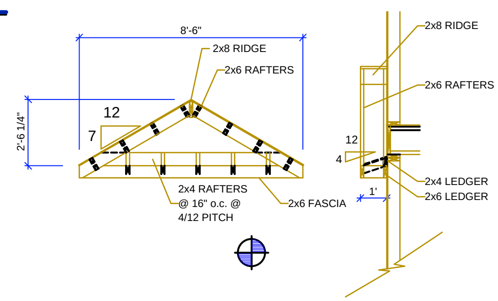
Not To Scale



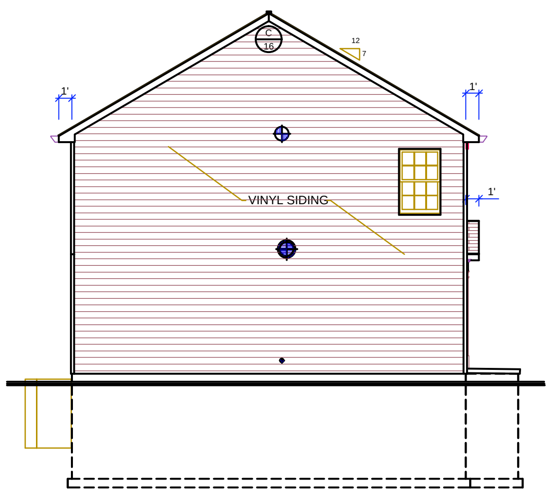
FRONT ELEVATION



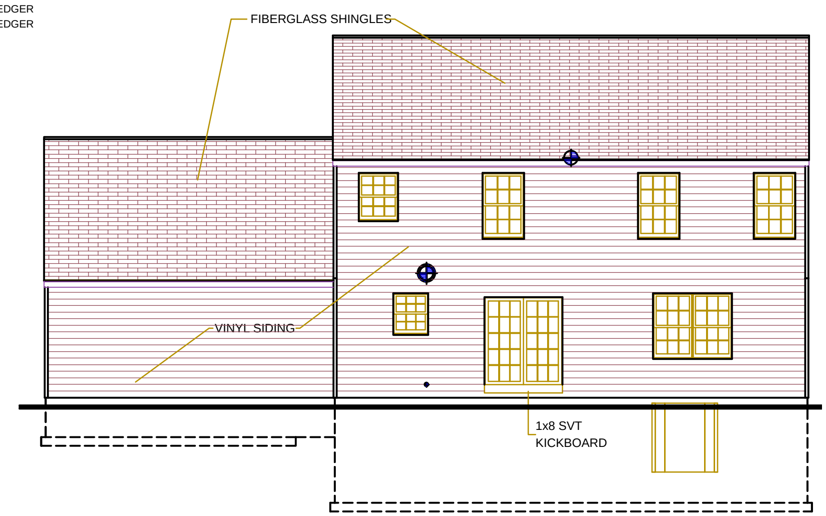
RIGHT SIDE ELEVATION



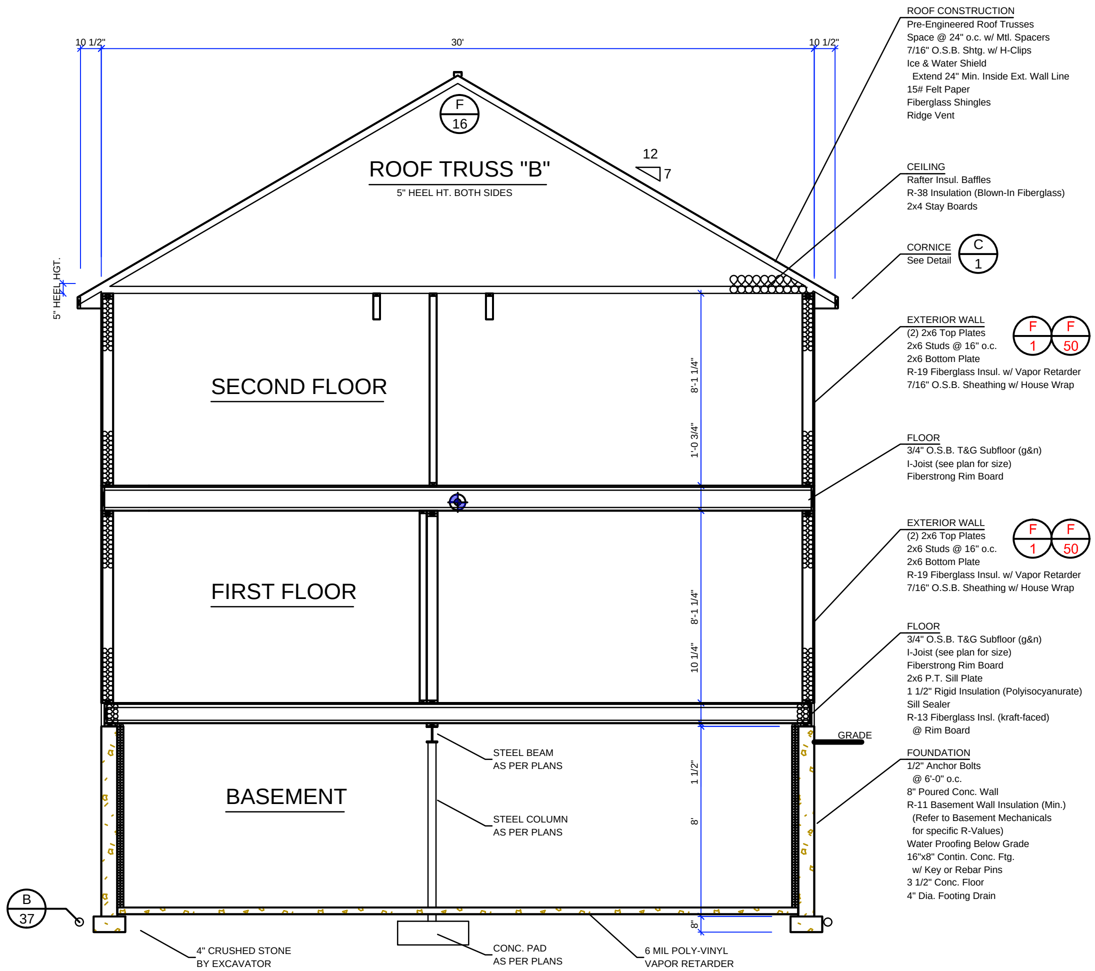
STOOP ROOF DETAIL



LEFT SIDE ELEVATION

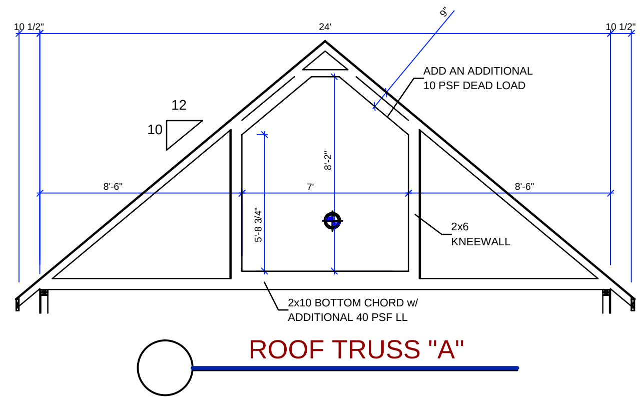


REAR ELEVATION



SECTION NOTES:

1) ALL BOTTOM PLATES SET UPON MASONRY OR CONCRETE TO BE PRESSURE TREATED.
2) ALL CONCRETE POURED AGAINST WOODEN MEMBERS TO HAVE FLASHING BETWEEN.
3) JOIST HANGERS REQ'D ON FLUSH CONNECTIONS WITH SPANS GREATER THAN 4'-0".
4) INT. WALL & CLG. FINISH TO BE 1/2" DRYWALL ALL AREAS UNLESS NOTED OTHERWISE.
5) HOUSEWRAP APPLIED OVER WOOD SHEATHING WHERE BRICK / MASONRY IS TO BE PLACED.
6) INTERMEDIATE GUARD REQUIRED AT OPEN SIDED STAIRS, WHICH WILL NOT PERMIT PASSAGE OF AN OBJECT 4 3/8" OR MORE IN DIAMETER.
7) BRICK TIES AT MASONRY VENEER; 16" OR 24" MAX SPACING -HORIZONTAL AND 16" MAX SPACING -VERTICAL.
8) TYVEK DRAINWRAP TO BE INSTALLED BEHIND ALL MANUFACTURER STONE VENEER THAT IS APPLIED OVER OSB WOOD SHEATHING.



ROOF TRUSS PLAN

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