NORTHERN ARIZONA UNIVERSITY

Memorandum

Letter of Transmittal

To: Mark Lamer

CC: Robin Tuchscherer

From: Ponderosa TimberJacks

Date: 5/7/2024

Re: Letter of Transmittal

Dear Mark Lamer,

In the following document, you will find the final report for the Ponderosa TimberJacks. The project began on August 28th, 2023, and concluded on May 7th, 2024.

The report includes the project understanding, design, implementation, schedule, project staffing, cost of engineering services, and project impacts. The schedule of the project includes the completion of the ASCE competition and of all CENE 486 deliverables. The total project cost comes to $68,358 which includes funds for personnel, travel, lab use, and materials. If you have any questions or concerns, please feel free to contact our project manager, Jenna Hays.

Thank you,

Ponderosa TimberJacks

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2024 ASCE TIMBERSTRONG

PONDEROSA TIMBERJACKS

Mariah Boler, Mourtice Clitso, Jenna Hays

May 7, 2024, Draft #5

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List of Abbreviations

APA	American Plywood Association
ASCE	American Society of Civil Engineers
ASD	Allowable Stress Design
Avg	Average
AWC	American Wood Council
BIM	Building Information Model
CECMEE	Civil Engineering, Construction Management, and Environmental Engineering
FS	Factor of Safety
FTAO	Force Transfer Around Openings
ISWS	Intermountain Southwest Student Symposium
LB	Pounds
NAU	Northern Arizona University
NDS	National Design Specifications
OSB	Oriented Strand Board
PLF	Pounds per Linear Foot
PSF	Pounds per Square Foot
QA/QC	Quality Assurance/Quality Control
SDPWS	Special Design Provisions for Wind and Seismic
SST	Simpson Strong Tie
USU	Utah State University
Wtd.	Weighted

Acknowledgements

We would like to thank our sponsors (Sirius Structures, POWER Engineers, Simpson Strong Tie, the American Society of Civil Engineers, the American Wood Council, the American Plywood Association) for their support.

We would also like to thank our Technical Advisor, Dr. Robin Tuchscherer, and our client, Mark Lamer, for their guidance throughout the project.

Lastly, we would like to thank our mentees for their help with design, modeling, and construction.

1.0 Project Introduction

1.1 Project Purpose

The purpose of the American Society of Civil Engineers (ASCE) TimberStrong Design Build (TSDB) Student Competition is to provide civil engineering students with real-world experience in structural timber design and construction. The American Wood Council (AWC), Simpson Strong Tie (SST), American Plywood Association (APA), and ASCE sought ASCE Student Teams to act as a design-build construction firm and create a two-story light-framed lumber building that is structurally durable, aesthetically pleasing, and sustainable. This competition exposes students to various aspects of the structural engineering and construction industries, including design and analysis calculations, design code navigation, structural drawings, Building Information Modeling (BIM), and construction planning and execution.

1.2 Project Overview

Northern Arizona University (NAU) ASCE students have been participating in TimberStrong since the inaugural competition in 2018. The project scope has evolved from a scaled 'doghouse,' to a full two-story, twelve-foot-tall timber house over the competition lifetime. The stages of the project are shown in Figure 1-1.

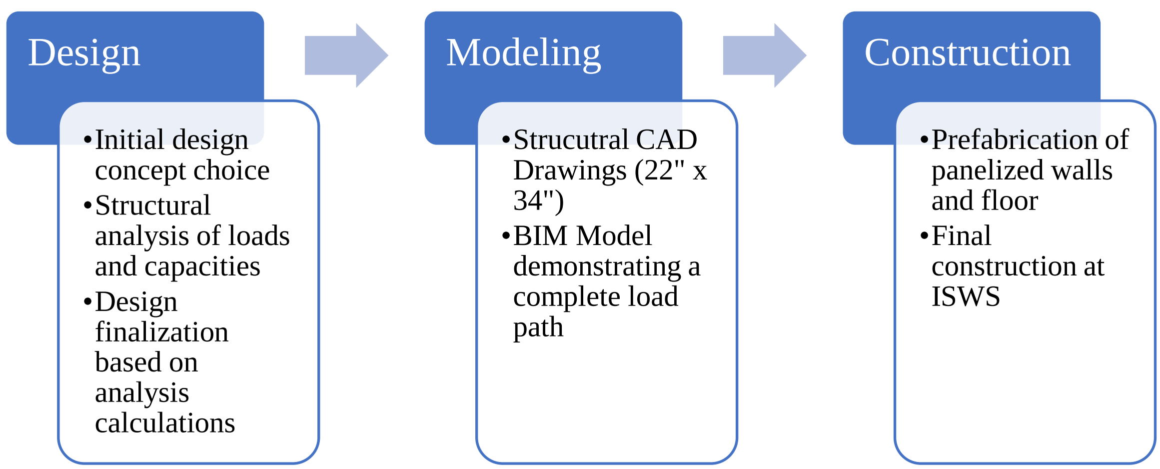

Figure 1-1: Project Stages

Prefabrication construction will take place at the NAU 'Farm,' officially titled the Civil Engineering, Construction Management, and Environmental Engineering (CECMEE) Field Station. The location of the project within Arizona as well as the location of NAU within Flagstaff is shown below in Figure 1-2.

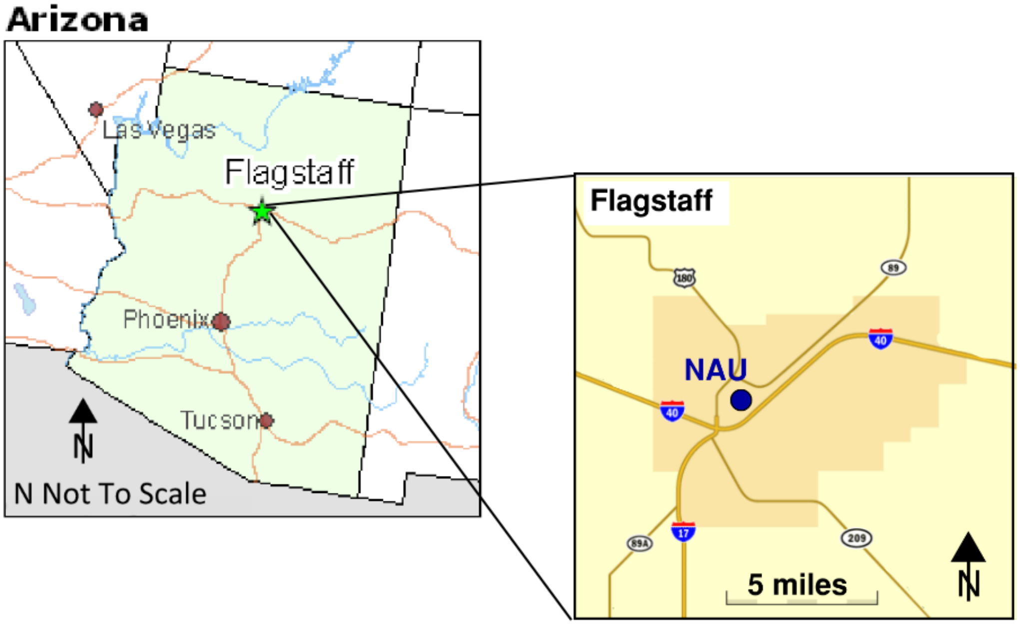

Figure 1-2: Location Maps of Flagstaff, Arizona [1] [2]

Final construction will occur during the ASCE Intermountain Southwest Student Symposium (ISWS) at Utah State University (USU) in Logan, Utah in April of 2024. The location map of Logan within Utah is shown below in Figure 1-3.

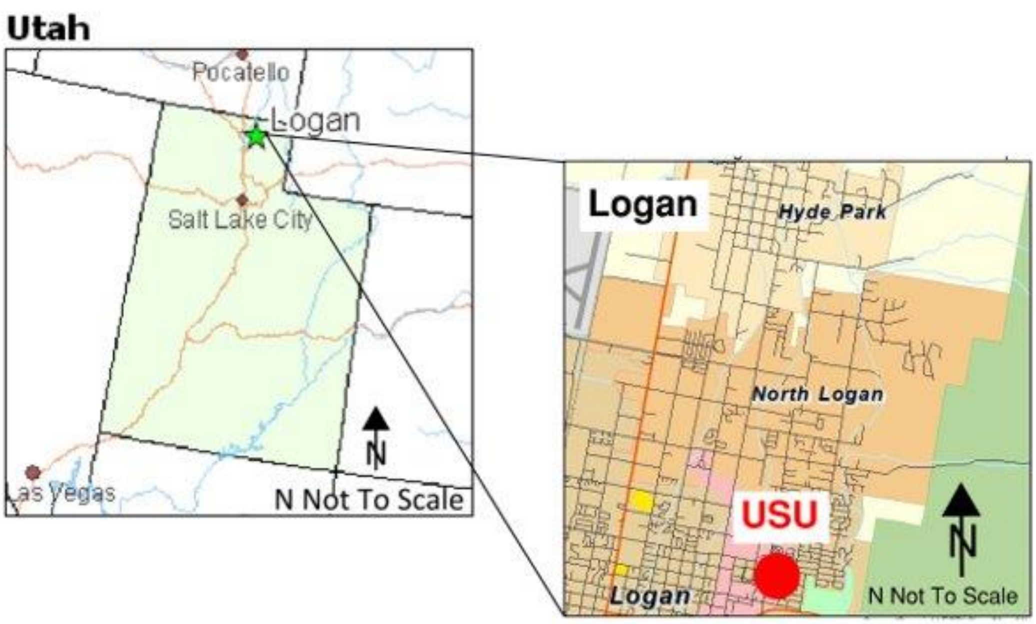

Figure 1-3: Location maps of Logan, Utah [1]

2.0 Project Background

2.1 Allowable Stress Design

The structural design is required to use the Allowable Stress Design (ASD) method to ensure that stresses caused by applied loads do not exceed design capacities. Structural design shall be completed in accordance with the AWC Special Design Provisions for Wind and Seismic (SDPWS) [3] and the AWC National Design Specifications (NDS) [4]

2.2 Timber Grade Species

The softwood grades one through five are the constraining timber grades for the design. The grades are specified based on strength, quality, and appearance [5]; details are shown below in Table 2-1.

Table 2-1: Timber Grades

Timber Grades	
Grade	Condition Description
1- Construction	Moderate number of tight knots
2- Standard	Higher number of knots
3- Utility	Splits and knotholes
4- Economy	Numerous splits and defects
5- Economy	Large number of defects

The timber species designated as design options by the TimberStrong rules are shown below in Table 2-2.

Table 2-2: Timber Species

Timber Species
Douglas Fir (DF)
Southern Pine (SP)
Douglas-Fir-Larch (DFL)
Hem Fir (HF)
Spruce-Pine-Fir (SPF)

2.3 Design Loads and Dimensions

The design must demonstrate a complete and continuous load path for both gravity and lateral loads through the structure and into the foundation. The dimensional constraints for the structure are shown in Appendix A. Demonstration of load path can be seen below in Figure 2-1.

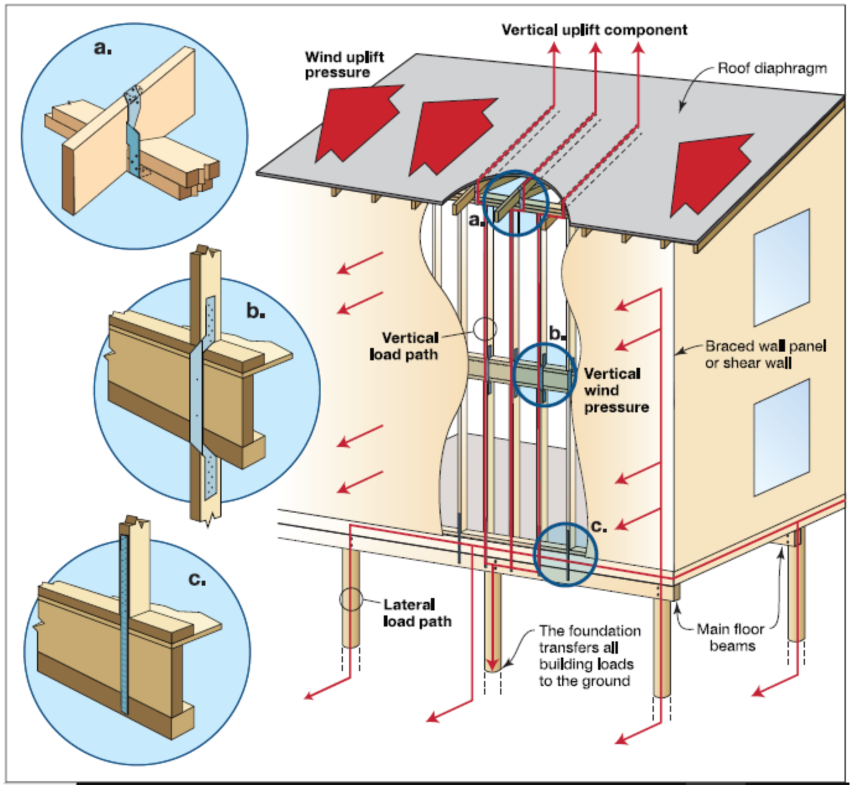

Figure 2-1: Load Path [6]

The structural design of the project requires that the proposed timber structure can withstand all self-weight dead loads and the loads established by the TimberStrong Competition Rules in pounds per square foot (psf), pounds per foot (plf), and pounds (lb), found in Table 2-3 below.

Table 2-3: Structure Loads

Load Type	Load Value
Live, Roof	20 psf
Live, Second Floor	50 psf
Point Load, Cantilever	150 lb
Wind Uplift, Roof	30 psf
Seismic, Roof Diaphragm	275 plf
Seismic, Second Floor Diaphragm	225 plf

The cantilever load's location will be determined on the day of competition by a dice roll. The cantilever is a beam in the floor diaphragm unsupported at one end and supported by the structure walls at the other. The beam protrudes four feet and one inch from the back wall of the structure. The measured deflection will be compared to the predicted deflection value at the specified location. The possible test load locations are shown below in Table 2-4.

Table 2-4: Cantilever Point Load Placements

Load Placement from Exterior Wall
4’-0”
3’-9”
3’-6”

The cantilever deflection must be between 0.5 in. to 1 in. when the load is placed 4 ft from the exterior wall. Dead loads were calculated from the self-weight of the structure elements. The live, wind, and seismic loads are shown below in Figure 2-2.

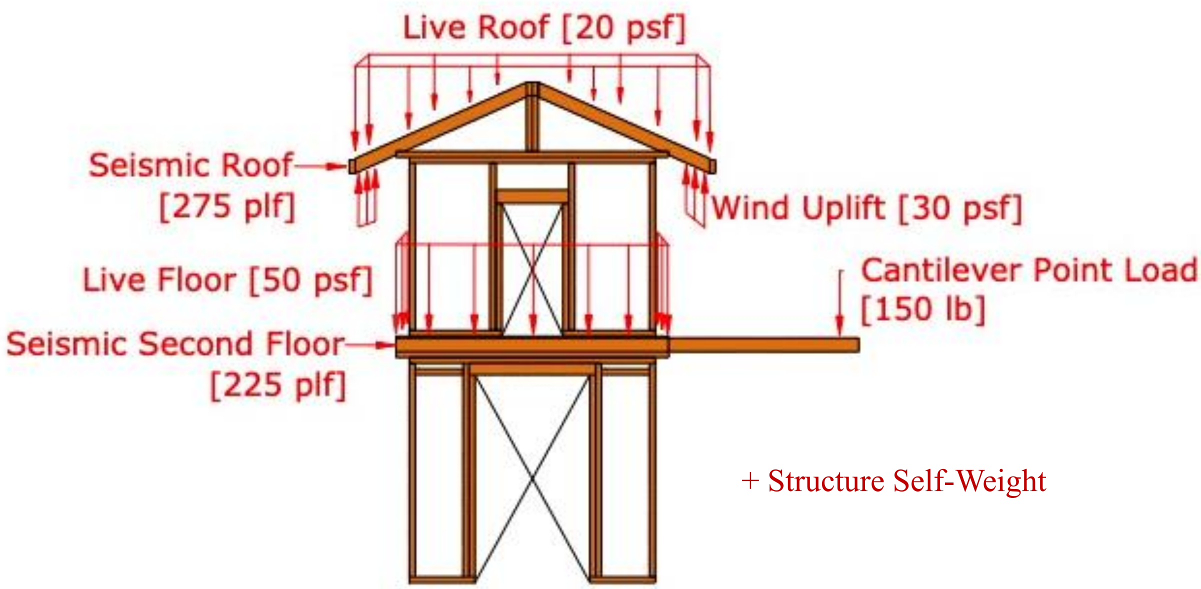

Figure 2-2:Structure Load Placements

2.4 Construction Rules

The three phases of the construction process and criteria are shown below in Figure 2-3.

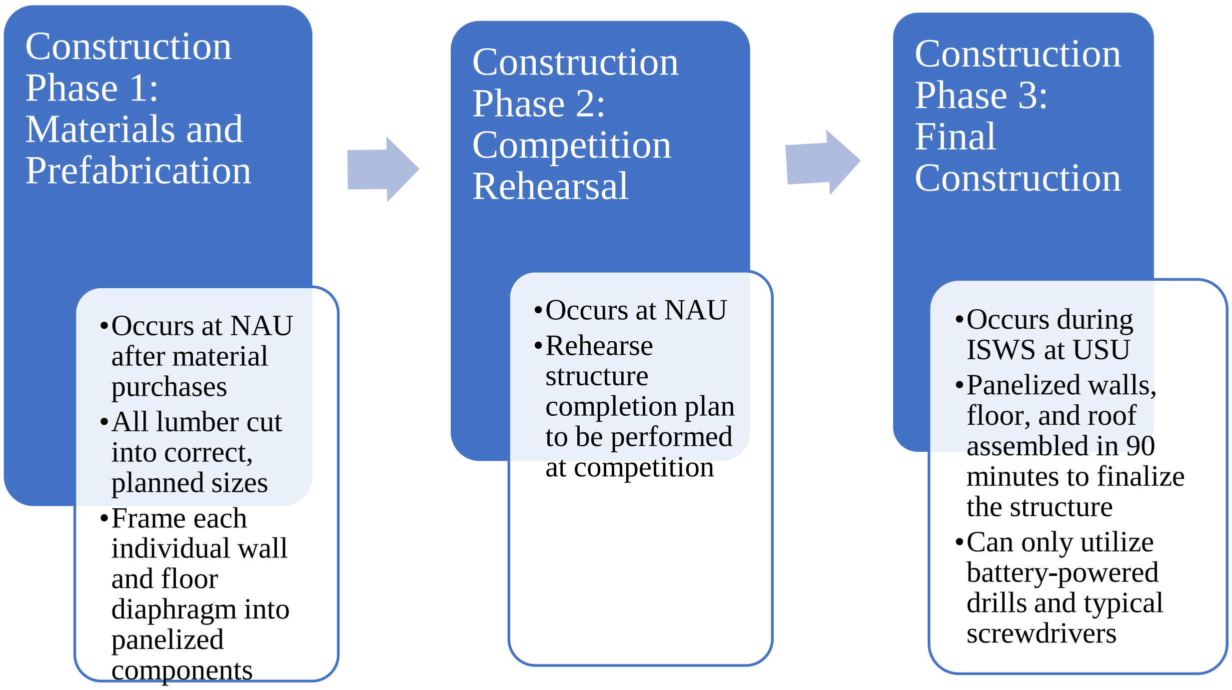

Figure 2-3: Construction Phases

2.5 Scoring

Report scoring for the design and modeling phases provides a maximum of 290 points and is based on the following sections as seen in Table 2-5 below.

Table 2-5: Report Scoring

Report Scoring	Maximum Points
Design Strength and Durability Analysis	82
Sustainability	18
Costs	20
Creativity and Aesthetics	20
Presentation	11
Visual Aid	9
Report Requirements	10
BIM	70
Construction Drawings	50
Design Points Possible	**290 (+5 bonus)**

Sustainability is determined from the design's potential carbon benefit and calculated carbon sequestration. Budget costs are scored relatively between teams; the team with the lowest budget is awarded the most points. Creativity and aesthetics are a subjective score given by the judges.

During the construction phase at ISWS, the first floor is tested for structural stability before building the top floor. After all construction is completed at ISWS, the structure is tested by applying the cantilever point load and measuring the deflection. The cantilever deflection is included in the Design Strength and Durability Analysis category and is scored on the ratio of predicted to actual deflection within the allowable range. The subsections of the Design Strength and Durability Analysis category scoring is shown below in Table 2-6.

Table 2-6: Design and Durability Scoring

Design and Durability Scoring	Maximum Points
Average Diaphragm Factor of Safety	6
Average Shear Wall Factor of Safety	6
Completeness and Accuracy of Calculations	55
Deflection	15
Design and Durability Points Possible	**82**

The details for the scoring of the average diaphragm and shear wall factors of safety are shown below in Table 2-7.

Table 2-7: Factor of Safety Scoring

Factor of Safety Results	Points Awarded
$1.50 \leq FS \leq 1.65$	Maximum
$1.65 \leq FS \leq 1.80$	Partial
$FS < 1.50$ or $FS > 1.80$	None

The subsections of the BIM model category scoring are shown below in Table 2-8.

Table 2-8: BIM Scoring

Construction Scoring	Maximum Points
Accuracy of Model	30
Load Path	20
Complete Structure	20

The maximum construction points awarded, 130 points, is distributed among the categories of consistency/accuracy of the completed structure to the structural drawings submitted, the continuous load path that is demonstrated in the structure, and the completion of the structure on competition day. There are also bonus points awarded to the first team to finish the construction of their structure. The scoring of the construction portion of the competition will be based on these sections as seen in Table 2-9 below.

Table 2-9: Construction Scoring

Construction Scoring	Maximum Points
Consistency/Accuracy	130
Continuous Load Path	
Completion of Structure	
Build Time (Bonus)	5
Construction Points Possible	**130 (+5 bonus)**

3.0 Preliminary Design and Analysis

3.1 Timber Decision Matrices

The lumber grade chosen for the design was determined with a decision matrix. The criteria of cost, appropriate strength, and availability were chosen to maximize competition points and aid the construction process.

The cost criterion was weighted at 30% due to the scoring of competition budgets based on economy. The alternatives were scored on a scale of 1 to 5, with 1 being the most expensive option and 5 being the least expensive option.

The appropriate strength criterion was weighted at 20%. This criterion relates to creating a design that is appropriate for the loads that a small residential structure must withstand; overdesigning would cause issues in the budget and in material weights for construction processes. The strength appropriateness was weighted the lowest because it is not a detail that is directly scored in the competition, but it is still a relevant consideration in economic and sensible design. The alternatives were scored on a scale of 1 to 5, with 1 being the least appropriate level of strength and 5 being the most appropriate level of strength for residential construction.

The local availability criterion was weighted at 50%. This criterion was weighted the highest because lumber is the main material necessary for constructing the building. The alternatives were scored on a scale of 1 to 5, with 1 being the lowest availability and 5 being the highest availability in local lumber stores.

Based on these criteria, Grade 2 scored the highest in the decision matrix shown below in Table 3-1 and was chosen for the final design.

Table 3-1: Timber Grade Decision Matrix

Grade Decision Matrix		Grade 1		Grade 2		Grade 3		Grade 4		Grade 5	
Criteria	**Weight (%)**	**Score**	**Wtd. Score**	**Score**	**Wtd. Score**	**Score**	**Wtd. Score**	**Score**	**Wtd. Score**	**Score**	**Wtd. Score**
Cost	30	1	0.3	2	0.6	3	0.9	4	1.2	5	1.5
Efficient Strength	20	4	0.8	5	1	4	0.8	2	0.4	1	0.2
Availability	50	4	2	5	2.5	2	1	2	1	1	0.5
Total	**100**		**3.1**		**4.1**		**2.7**		**1.7**		**2.2**

The softwood species chosen for the design was also determined with a decision matrix. The criteria of cost, appropriate strength, and availability were maintained to maximize competition points and aid the construction process. Wood species considered are douglas fir (DF), southern pine (SP), douglas fir-larch (DFL), hem fir (HF) and southern pine-fir (SPF). The criteria weights and score scales were also maintained from the Grade Decision Matrix.

Table 3-2:Timber Species Decision Matrix

Species Decision Matrix		Douglas Fir (DF)		Spruce Pine (SP)		DF Larch		Hem Fir		SP Fir	
Criteria	**Weight (%)**	**Score**	**Wtd. Score**	**Score**	**Wtd. Score**	**Score**	**Wtd. Score**	**Score**	**Wtd. Score**	**Score**	**Wtd. Score**
Cost	30	2	0.6	1	0.3	3	0.9	4	1.2	5	1.5
Efficient Strength	20	5	1	2	0.4	2	0.4	4	0.8	3	0.6
Availability	50	4	2	2	1	3	1.5	5	2.5	2	1
Total	**100**		**3.6**		**1.7**		**2.8**		**4.5**		**3.1**

Based on the criteria, Hem Fir scored the highest in the decision matrix shown below in Table 3-2 and was chosen for the final design.

3.2 Design Decision Matrix

Three design alternatives were considered for the structure. These alternatives included general framing plans, aesthetics, and roof type. They did not consider any exact dimensions or design calculations; the qualitative design alternatives were instead formed relative to one another. These design aspects and alternatives are illustrated in Figure 3-1 and compared in in Table 3-3.

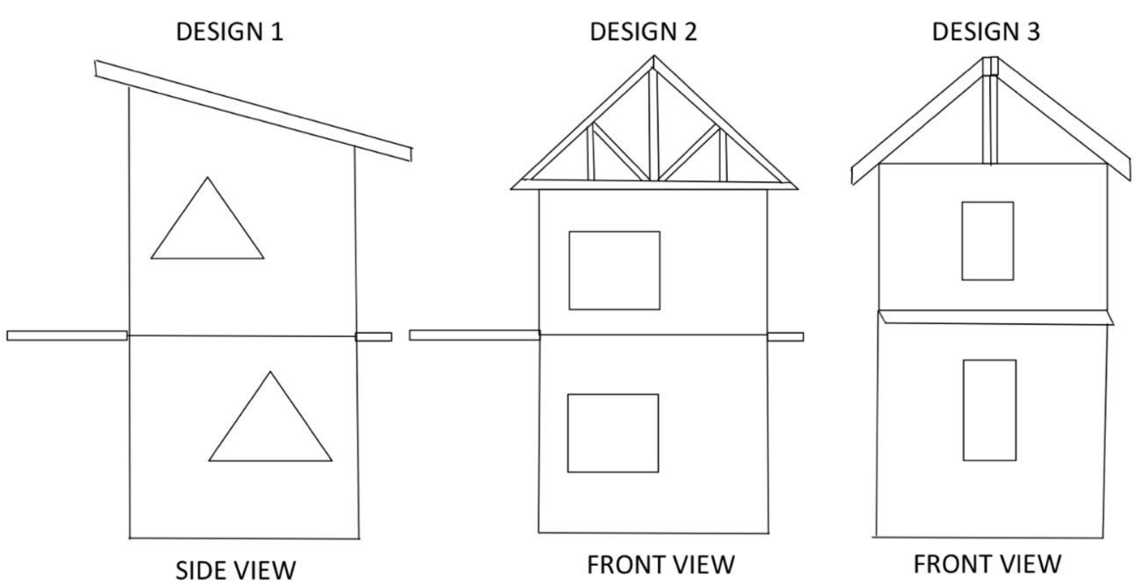

Figure 3 - 1: Design Alternatives

Table 3-3: Design Alternatives

Design Descriptions	Design 1	Design 2	Design 3
Roof	Mono-pitched	Trusses	Gable
Window Sizes	About 2' wide. Triangles	About 1.5' wide. Squares	About 1' wide. Rectangles
Window Placement	Off-Center and Not Stacked	Off-Center and Stacked	Centered and Stacked
Cantilever Beam Placement	Front Wall	Side Wall	Back Wall
Floor Overhang Placement	Back Wall	Side Wall	Front Wall
Aesthetic Theme	Mountains	Pine Tree	Log Cabin

The three design alternatives were scored using a decision matrix. The criteria for this decision matrix were based on the goals of maximizing points for the competition design scores of budgets, aesthetics, and constructability.

The cost criterion was weighted at 20% due to the scoring of competition budgets based on economy. The alternatives were scored on a scale of 1 to 3, with 1 being the most expensive option and 3 being the least expensive option. The cost of each alternative was estimated based on amounts of lumber used in the roof design and or excluded due to window size. Design 2 scored the lowest of the three designs due to the high cost of prefabricated trusses.

The aesthetics and creativity criterion were weighted at 20% equal to the cost criterion, as these aspects are weighted the same in the competition scoring. This criterion is scored subjectively by the judges at competition. Scores for this matrix were awarded based on creativity of the roof, window shape, window placement, cantilever/overhang placement, and theme. The alternatives were scored on a scale of 1 to 3, with 1 being the most creative and aesthetic and 3 being the least creative and aesthetic.

The prefabrication constructability criterion was weighted higher at 25% because the team must be able to practically construct the wall panels, floor panels, and roof pieces prior to competition. This criterion considered the window geometries within the walls for repeatability and ease of dimensional cutting. The alternatives were scored on a scale of 1 to 3, with 1 being the most difficult and 3 being easiest to construct in the prefabrication stage.

The roof constructability criterion was weighted the highest at 35% because a large portion of competition points is dependent on the completion of the structure at competition, with a small number of additional points related to a faster construction completion time within the 90-minute period. The roof is the only structural component that cannot be completely prefabricated prior to the competition, so it requires the most time to construct on the Build Day. The alternatives were scored on a scale of 1 to 3, with 1 being the most difficult and 3 being easiest to construct in the Build Day construction. Design 1 scored the lowest of the three designs due to the difficulty of constructing a mono-pitched roof on stepladders.

Based on the criteria, Design 3 scored the highest in the decision matrix shown below in Table 3-4 and was chosen for the final design.

Table 3-4: Design Decision Matrix

Design Decision Matrix		**Design 1**		**Design 2**		**Design 3**	
Criteria	**Weight (%)**	**Score**	**Weighted Score**	**Score**	**Weighted Score**	**Score**	**Weighted Score**
Cost	20	2	0.4	1	0.2	3	0.6
Aesthetics and Creativity	20	3	0.6	2	0.4	2	0.4
Prefabrication Constructability	25	1	0.25	1	0.25	3	0.75
Roof Constructability	35	1	0.35	3	1.05	2	0.7
Total	**100**		**1.6**		**1.9**		**2.45**

4.0 Final Design and Analysis

Design analyses considered the Bernoulli Beam theory in which all sections of a beam are assumed to remain plane without perpendicular deformations. Lumber was considered ideal with no imperfections. All members sizes were assumed to be 2x4's, as this is the smallest member size allowed by the TimberStrong Rules.

Live, seismic, and wind uplift loads described in Section 2.3 were applied to the appropriate elements of the structure along with assumed self-weight dead loads of framing and sheathing members. These self-weights were confirmed through the design of each member and would have been adjusted if the design resulted in a different outcome than what was assumed.

Deflections were assumed negligible due to the small size of the structure and were disregarded in the design process, excluding the required deflection prediction of the cantilever beam from the applied point load described in Section 2.3.

All designs followed the ASD method as specified by the TimberStrong Rules.

The complete list of structural elements designed for can be seen in Table 4-1 below.

Table 4- 1: Construction Details Required

Structural Elements Designed
Framing Member Sizes
Sheathing Sizes
Nail Size
Nail Spacing
Connections (Straps and Anchor Bolts)

Complete design calculations can be found in Appendix B.

4.1 Roof Design

The framing roof members were designed to resist self-weight dead loads and live roof loads. The ridge beam was modeled as a simply supported beam, and the rafters were modeled with pin and roller boundary conditions accurate to the length that hangs over the wall top plate acting as an eave for the roof. The studs supporting the ridge beam were modeled as columns.

The lateral roof design assumed the two rectangular sections of the sheathed roof to act as one diaphragm that resisted the seismic load. Sheathing size, nail size, and nail spacing were iterated to ensure a diaphragm design closest to a Factor of Safety of 1.5 to maximize competition points and a design that complemented constructability with the lateral design of the walls and floor.

Rafter tie downs were designed to satisfy a continuous load path for the roof loads transferring to the second story walls. Roof design results, including the average (avg) roof diaphragm Factor of Safety, are shown below in Table 4-2.

Table 4-2: Roof Design Results

Roof Design Results	
Design Aspect	**Design Result**
2x4 Member Size	All Framing Members
3/8” Sheathing	All Sheathing Pieces
6” Nail Spacing	All Diaphragm Edges
6D Nail Size	All Diaphragm Nailing
Rafter Tie Downs	SST H3 on each Rafter
Avg Roof Diaphragm FS	**1.52**

The final framing design of the roof can be seen in Figure 4-1 below.

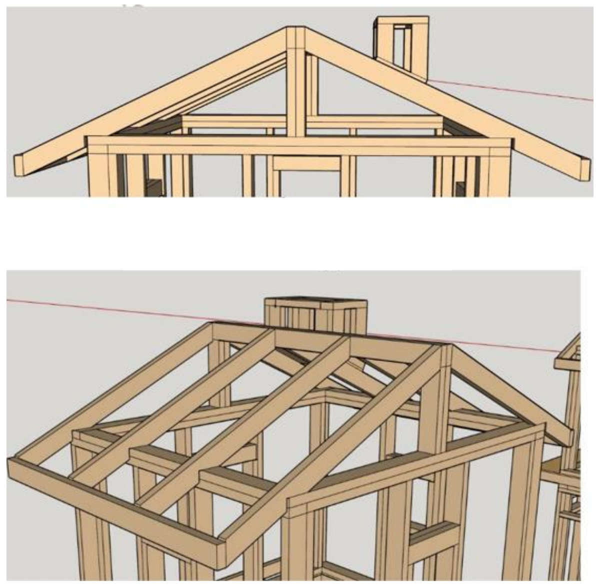

Figure 4- 1:Final Roof Design

4.2 Wall Design

The wall stud members were modeled as columns to resist the loads transferred through the walls for a continuous structure load path. The window and door headers were modeled as simply supported beams.

Wall lateral design utilized the segmented method for the first story door wall, as this wall met the aspect ratio requirements due to the horizontal blocking placement within the framing plan. This method assumes each full-height wall segment resists lateral loads individually.

The Force Transfer Around Opening (FTAO) shear wall method was used for all other walls. This method assumes wall segments above and below openings can also contribute to the lateral wall resistance. FTAO allows for a more economic use of straps and anchor bolts used to secure and connect shear walls to the surrounding structure elements.

Sheathing size, nail size, and nail spacing were iterated to ensure a shear wall design closest to a Factor of Safety of 1.5 to maximize competition points and a design that complemented constructability with the lateral design of the roof and floor. A tighter nail spacing was required on four of the walls to accommodate higher loads and lower wall capacities.

Wall design results are shown below in Table 4-3.

Table 4-3: Wall Design Results

<table>
<tr><th colspan="3">Wall Design Results</th></tr>
<tr><th colspan="2">Design Aspect</th><th>Design Result</th></tr>
<tr><td colspan="2">2x4 Member Size</td><td>All Framing Members</td></tr>
<tr><td colspan="2">3/8” Sheathing</td><td>All Sheathing Pieces</td></tr>
<tr><td rowspan="2">Nail Spacing</td><td>6”</td><td>1st Story Front Wall
2nd Story Front Wall
2nd Story Sidewalls</td></tr>
<tr><td>4”</td><td>1st Story Back Wall
2nd Story Back Wall
1st Story Sidewalls</td></tr>
<tr><td colspan="2">6D Nail Size</td><td>All Shear Walls</td></tr>
<tr><td colspan="2">Opening and Shear Wall Straps</td><td>SST LSTA24</td></tr>
<tr><td colspan="2">Anchor Bolts</td><td>STB2-50234R25</td></tr>
<tr><td colspan="2">Avg Shear Wall FS</td><td>1.57</td></tr>
</table>

The final framing design of one of the walls can be seen in Figure 4-2 below.

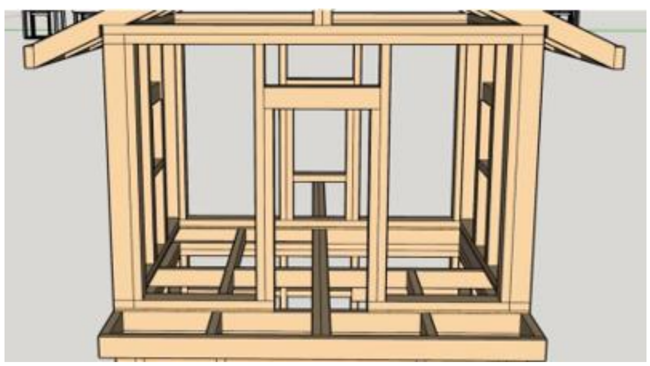

Figure 4- 2: Second Floor Wall

4.3 Floor Design

The framing floor members were designed with pin and roller boundary conditions placed at locations accurate to the first story wall supports, the floor overhang, and the floor cantilever beam. All members were designed as 2x4’s, and the cantilever beam design yielded a result that required two 2x4 members acting as one to resist the applied loads.

Sheathing size, nail size, and nail spacing were iterated to ensure a floor diaphragm design closest to a Factor of Safety (FS) of 1.5 to maximize competition points and a design that complemented constructability with the lateral design of the roof and shear walls. The opening in the floor diaphragm design required additional nailing and an additional strap for the members running along the opening to transfer additional load incurred by the opening.

Floor design results are shown below in Table 4-4.

Table 4-4: Floor Design Results

Floor Design Results	
Design Aspect	**Design Result**
2x4 Member Size	All Framing Members
Double 2x4 Member	Cantilever Beam
3/8” Sheathing	All Sheathing Pieces
6” Nail Spacing	All Diaphragm Edges and Beam along Opening
6D Nail Size	All Diaphragm Nailing
Strap on Beam along Opening	SST LSTA24
Avg Floor Diaphragm FS	**1.57**

The final design of the floor framing can be seen in Figure 4-3 below.

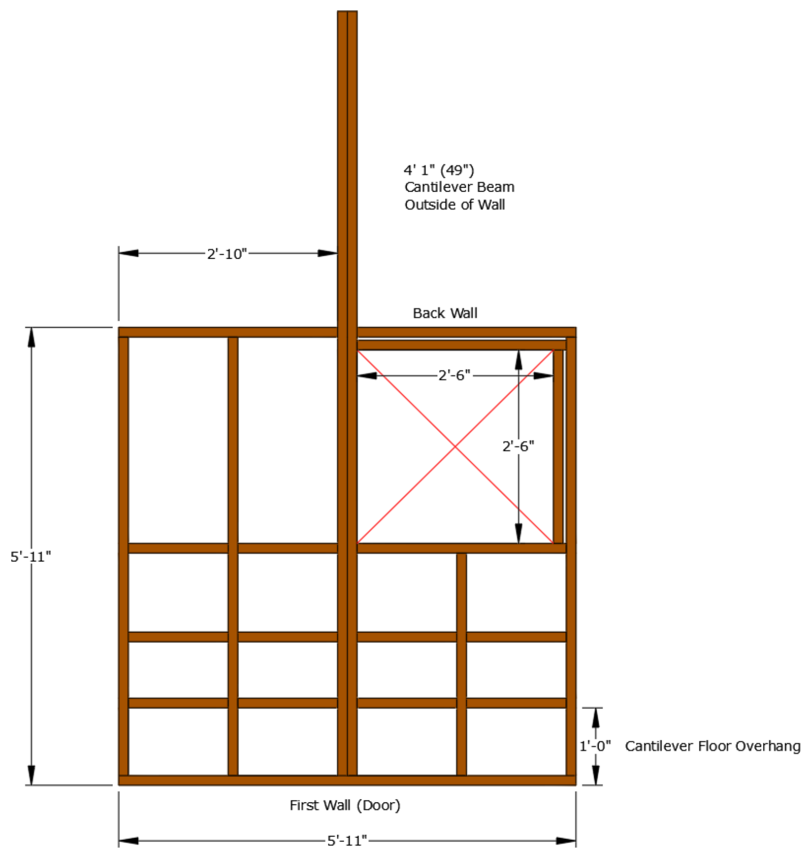

Figure 4- 3:Final Floor Design

4.4 Cantilever Deflection

The cantilever deflection was predicted using the Method of Virtual Work. This method assumes the system responds linearly to the applied loads and that the boundary condition constraints act how they are modeled as a pin and roller. As shown below in Table 4-5, the deflections meet the required range within 0.5 in. to 1.0 in.

Table 4-5: Cantilever Deflection Results

Cantilever Deflection		
Load Placement from Exterior Wall	Deflection, Δ (in.)	Meets 0.5in.<Δ<1.0in.
4'-0"	0.78	Yes
3'-9"	0.59	Yes
3'-6"	0.52	Yes

Complete cantilever design and deflection calculations can be found in Appendix C.

4.5 Diaphragms and Shear Wall Factor of Safety

The average diaphragm and shear wall factors of safety fell within the range of maximum competition points as outlined in Table 2-7 of Section 2.5. These results are shown below in Table 4-6.

Table 4-6: Average Factor of Safety Results

Average Lateral Factor of Safety Results	
Lateral Design Group	Average Factor of Safety
Diaphragms	1.54
Shear Walls	1.57

The worst-case factors of safety for the roof, floor, and wall framing gravity systems are shown below in Table 4-7. All factors of safety were greater than 1.0, meaning the capacity was greater than the demand.

Table 4- 7: Gravity Factor of Safety Results

Gravity Factor of Safety Results	
Gravity Design Group	Worst-Case Factor of Safety
Roof	4.17
Floor	1.53
Wall Framing	17.0

5.0 Modeling and Competition Presentation

5.1 2D Modeling

Two-dimensional modeling was completed using AutoCAD to convey wood framing details to competition judges. Framing details include dimensioning of wood members and Simpson Strong-Tie product placement on the structure. The model demonstrated a continuous load path with plan, elevation, and cross-sectional views. The requirements for the 22 in. x 34 in. structural drawings are listed below in Table 5-1.

Table 5-1:Drawing Requirements

Structural Drawing Requirements
Framing Plans
Shear Wall Connection Details
Panelized diaphragm and shear wall sheathing type and fastening schedule
Connectors, blocking, and fasteners for continuous load path
Plan views, elevations, and cross-sectional details demonstrating continuous load path
Anchorage to the foundation

The complete structural drawings are included in Appendix D. An example of the AutoCAD 2D modeling can be seen in Figure 5-1 below.

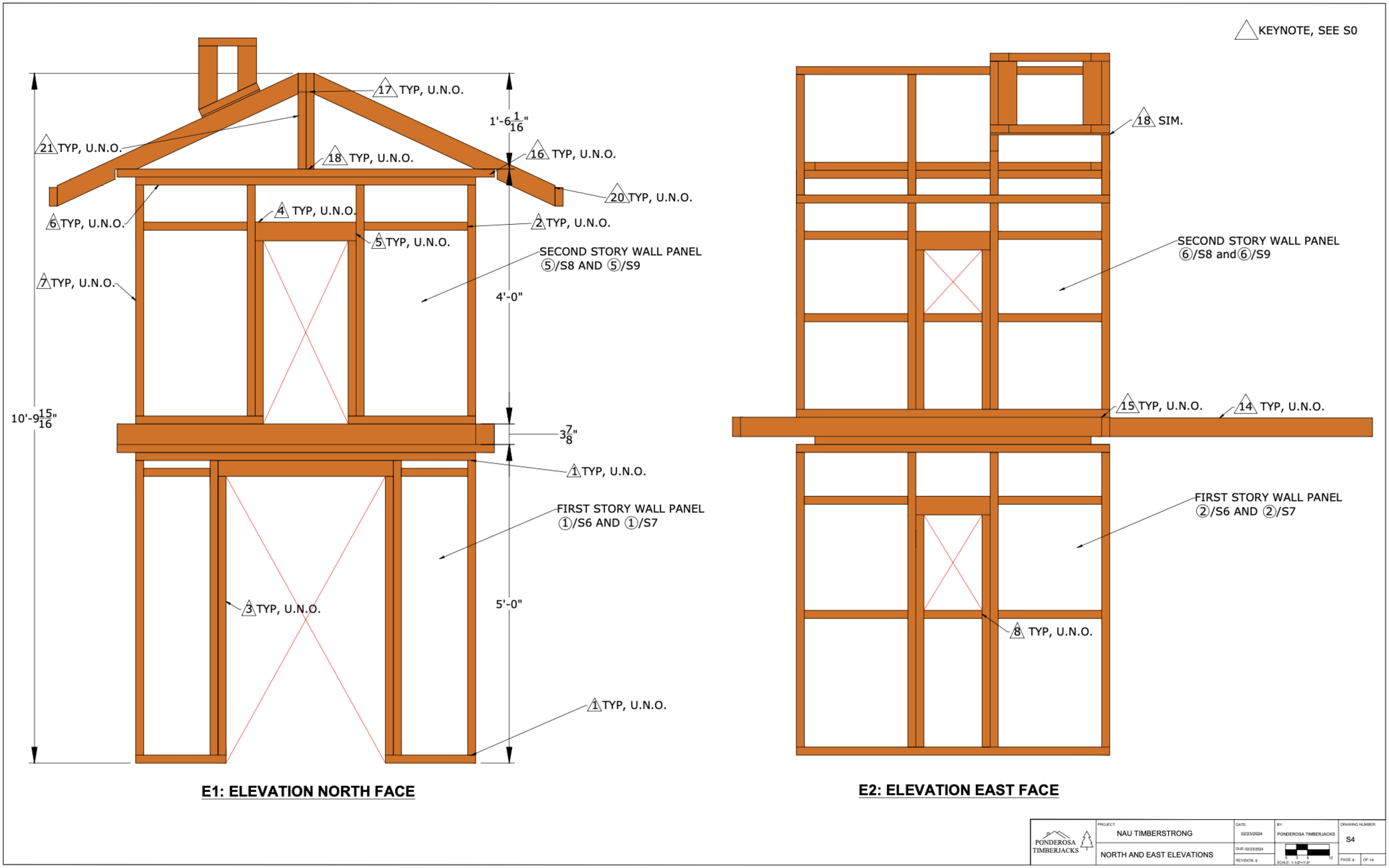

Figure 5-1: Elevation View

5.2 3D Modeling

Three-dimensional modeling was completed using Revit to demonstrate to the judges that the load path for gravity, wind, and seismic loads is continuous between the point application and the foundation, including all necessary connectors and fasteners. All structural members were required to be modeled in three dimensions.

The scoring criteria for the BIM model is shown in Table 2-8 in Section 2.5. The 3D model seen from the front and side view can be seen in Figures 5-2 and 5-3 respectively.

Figure 5-2: Revit BIM Front View

Figure 5-3: Revit BIM Side View

5.3 ASCE Presentation

The Phase 3 presentation was recorded and submitted with all the required components shown below in Table 5-2.

Table 5-2: Presentation Requirements

Presentation Requirements
Student chapter and team member names
Graphics and snapshots of the structure
Factor of Safety for the diaphragm and the shear walls
A table indicating the calculated cantilever beam deflections and bearing force per linear foot of the sill plate of the wall opposite the cantilever beam for each of the three possible point load locations
Design features
Total calculated carbon stored in structure and the total potential carbon benefit
Total material cost of the structure
Total calculated weight of the structure
Logos of all the host and sponsors (ASCE, AWC, APA & SST)

6.0 Design Implementation

6.1 Construction

6.1.1 Material Acquirement

Materials were purchased in March and stored at the field station. The lumber was acquired from HomCo, the connectors and fasteners were donated from Simpson Strong-Tie, and the building tools such as a toolbox and battery powered drills were acquired from Home Depot.

6.1.2 Prefabrication

Partial prefabrication of the structure was completed to ensure the full structure could be completed within the 90-minute time frame at competition. Prefabrication included the framing and sheathing of separate walls and the floor to create separate panels that could be combined to form a full structure at competition. Per Section 1.2, prefabrication was done at NAU's CECMEE Field Station, "the Farm" shown below in Figures 6-1 and 6-2.

Figure 6- 1: Farm Construction

Figure 6- 2: Floor Diaphragm

6.1.3 Construction Practice

Construction practice was conducted at the Farm after prefabrication, shown below in Figures 6-3 and 6-4. This ensured the team was familiar with the prefabricated panels of structure and their roles during competition to help increase efficiency during the 90-minute competition.

Figure 6- 3: Construction Practice

Figure 6- 4: Roof Construction

6.1.4 Competition Build Day

Competition Build Day occurred on April 11th, 2024. The team was given a 20-ft by 20-ft space to build within during the 90-minute competition, shown below in Figure 6-5. All materials, tools, and builders had to be placed within the space before the competition timer started. Anything or anyone outside the space before the competition timer began was not able to be used.

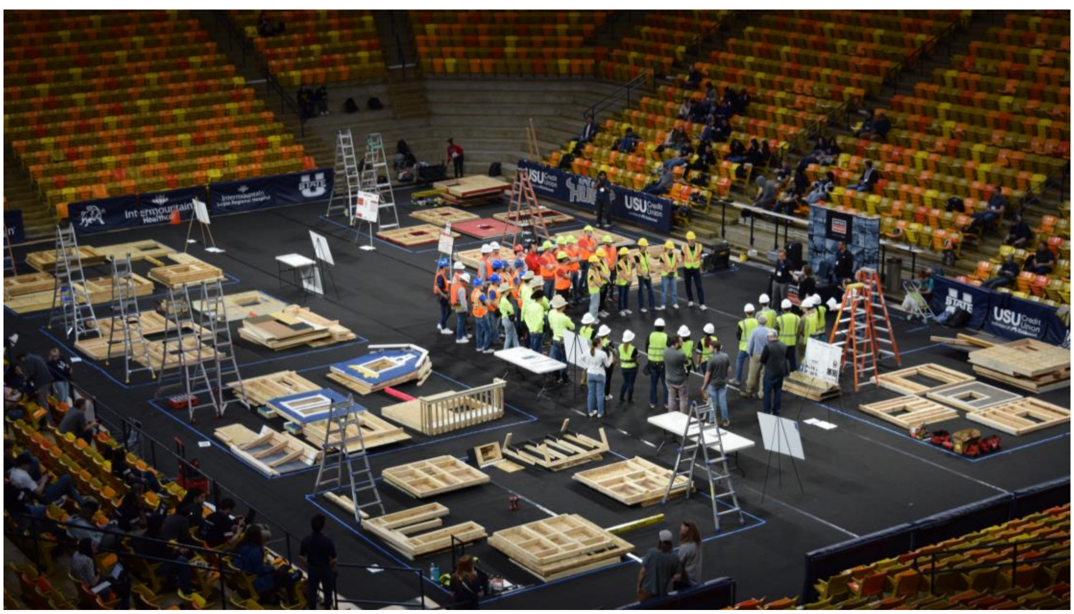

Figure 6- 5: Competition

During the competition, judges watched for safety concerns and stopped the timer if any safety concerns were seen. The time was also stopped after completing the first story to be assessed for sturdiness before builders were allowed to climb up and assemble the second story and roof.

After 90 minutes, all builders had to cease construction. The final structure is shown below in Figure 6-6.

Figure 6- 6: Completed Structure

The judges then scored each structure based on criteria in Section 2.5. After scoring, the judges met with each team to discuss their design and complete the cantilever deflection test, shown below in Figure 6-7.

Figure 6- 7: Cantilever Deflection Test

6.2 Competition Results

The team placed second overall out of the seven teams competing in the TimberStrong design build at ISWS. The measured deflection at 3'-9" from the exterior wall was 0.58 in., which was 0.01 in. off from the predicted deflection of 0.59 in. The judges commented that more blocking was needed within the roof framing and diaphragm connectors were needed between the floor and walls to help the structure withstand lateral loads in full-scale residential construction.

7.0 Summary of Engineering Work

7.1 Schedule Overview

The project started on August 28th, 2023, and ended on May 7th, 2024, for a total duration of 182 days which is consistent with the proposed timeframe. School breaks were accounted for in the project working days. The project schedule is displayed as a Gantt chart in Appendix E. The major tasks of the project included design, modeling, construction, competition, and capstone deliverables. The major CENE 486C deliverables included 30%, 60%, and 90% submittals, a final presentation, a website, and a final report. Major competition deliverables included a design report, structure modeling, a presentation, and a visual aid.

7.2 Schedule Changes

All project deliverables and milestones were met on time, so there were no changes to the proposed schedule or scope.

8.0 Summary of Engineering Costs

8.1 Staffing Matrix

A detailed breakdown of each task and the hours worked can be found in Appendix F. The staffing matrix presented in Table 8-1 summarizes the total number of hours worked by each position across all tasks and compares to the proposed effort. No major scope or task changes were made from the proposed estimate; the main differences between the proposed and actual hours worked were slight increases on most tasks, due to ambitious estimates of the time required for the tasks.

Table 8 - 1:Staffing Matrix

Table 8-1:Position	Task 1	Task 2	Task 3	Task 4	Task 5	Task 6	Task 7	Task 8	Task 9	TOTAL	PROPOSED
Senior Engineer	3	2	15	2	0	0	2	15	36	75	71
Engineer	9	6	59	24	3	0	0	49	57	207	185
Field Technician	2	0	0	0	31	19	2	2	7	63	56
Safety Officer	1	0	0	0	22	4	1	0	20	48	26
Intern	3	0	18	13	26	0	0	22	1	83	117
TOTAL	18	8	92	39	82	23	5	88	121	476	-
PROPOSED	20	10	90	30	73	32	7	81	112	-	455

8.2 Cost Overview

The cost of engineering services for this project included personnel, transportation, lab use, and materials.

Personnel hours were tracked throughout the project and incurred costs were calculated using each staffing position's hourly rate. The hourly rates of all staffing positions were estimated based on the US Department of Labor Employment and Earnings by Occupation [7], and team members' experience with service rates in the industry.

The team rented one nine-passenger van to fit all members through NAU's Fleet Services, resulting in a van charge of $215 for three days. There was an additional $0.42 milage rate for fuel [8]. Transportation costs accounted for the mileage accumulated during the round-trip to Logan, Utah from Flagstaff, Arizona. The price for one hotel room per night was $160. Three rooms were needed to separate five women and three men for three nights. Per diem of $60 per day was provided to each team member, based on the current per diem rate of Logan, Utah [9].

Lab usage costs included the time spent prefabricating at the Farm for $100 per day. Seven days were estimated for prefabrication, resulting in a total cost of $700 for lab usage.

Construction materials include lumber, OSB, fasteners and hardware, connectors, and paint. The material costs were determined based on quotes from local companies and the initial structure design.

The total cost of the entire project was originally estimated as $62,744. The specifications of price and quantity of each material are listed below in Table 8-2.

Table 8 - 2: Estimation of Project Cost

Description	Quantity	Unit of Measure	Rate ($)	Cost ($)
Personnel				
Senior Engineer	71	Hr.	250.00	17,750
Engineer	185	Hr.	160.00	29,600
Field Technician	56	Hr.	60.00	3,360
Intern	113	Hr.	40.00	4,520
Safety Officer	26	Hr.	85.00	2,210
			Subtotal Personnel	$57,440
Travel For Competition				
Transportation	600	Miles	0.42	252
Van Rental	3	Day	71.40	214
Hotel Rooms	3	Nights (3 Rooms)	480.00	1,440
Per Diem	8	People ($60 per day for 3 days)	180.00	1,440
			Subtotal Travel	$3,346
Lab Use				
Field Station "Farm"	7	Days	100.00	700
			Subtotal Lab Use	$700
Materials				
2x4x8 Hem Fir	80	Unit	5.69	455
2x4x10 Hem Fir	2	Unit	10.67	21
OSB	15	Sheets	29.98	450
Fasteners	2	Unit	40.53	81
Connectors/Hardware	1	Unit	130.30	130
Paint	3	GAL	40.00	120
			Subtotal Materials	$1,258
			Project Total	$62,744

8.3 Cost Changes

The increased personnel hours described in Section 8.1 added $5,610 to the project cost. The project material costs varied slightly from the proposed budget. The number of 2x4x8 Hem Fir studs was decreased from 80 to 65 due to more efficient planning in the use of lumber. The number of OSB sheets was decreased from 15 to 14 for this same reason. Extra aesthetic items of wallpaper and staples were added, which also decreased the gallons of paint needed. These developments resulted in a net increase of $4 to the material costs. The final project cost was $5,614 more than the original prediction. The breakdown of the final cost is listed below in Table 8-3.

Table 8 - 3: Final Project Cost

Description	Quantity	Unit of Measure	Rate ($)	Cost ($)
Personnel				
Senior Engineer	75	Hr.	250.00	18,750
Engineer	207	Hr.	160.00	33,120
Field Technician	63	Hr.	60.00	3,780
Intern	83	Hr.	40.00	3,320
Safety Officer	48	Hr.	85.00	4,080
			Subtotal Personnel	$63,050
Travel For Competition				
Transportation	600	Miles	0.42	252
Van Rental	3	Day	71.40	214
Hotel Rooms	3	Nights (3 Rooms)	480.00	1,440
Per Diem	8	People ($60/day for 3 days)	180.00	1,440
			Subtotal Travel	$3,346
Lab Use				
Field Station	7	Days	100.00	700
			Subtotal Lab Use	$700
Materials				
2x4x8 Hem Fir	65	Unit	5.69	370
2x4x10 Hem Fir	2	Unit	10.67	21
OSB	14	Sheets	29.98	420
Fasteners	5	Unit	40.53	203
Connectors/Hardware	1	Unit	0.00	-
Paint	1	Gal	40.00	40
Wallpaper	4	Box	49.50	198
Staples	1	Pack	9.99	10
			Subtotal Materials	$1,262
			Project Total	$68,358

9.0 Impacts

The social, economic, and environmental impacts for the project were assessed for the use of timber compared to concrete masonry units (CMU) in residential construction. The pros and cons of both alternatives on the three impacts are listed in Table 9-1.

Table 9- 1:Impact Analysis

Impact Analysis				
Alternative	**Type**	**Social**	**Economic**	**Environmental**
Timber	**Pros (+)**	• Timber provides design versatility and aesthetic appeal [10] • Framing cavities provide room for insulation [11]	• Quick construction time with framing crews [11] • Lower labor and material costs [11]	• Timber is a renewable resource [12] • Lighter for transportation - less fuel used
	Cons (-)	• Moisture vulnerability increases mold, damage, and insects • Susceptible to fire, wind, and earthquake damage [11]	• High maintenance costs from damage or moisture [13] • High insurance rates for homeowners [11]	• Energy and fuel consumed in material production • Timber demolition waste is put in landfills [12]
CMU	**Pros (+)**	• Not susceptible to damage [10] • CMU provides some soundproofing for the user [10]	• Very durable/lasting - less maintenance [10] • Lower insurance rates (durability/termite resistance) [11]	• Very durable/lasting - less replacement • Concrete batching produces little waste [13]
	Cons (-)	• Blocks are thick and take up space [11] • Less architectural variation	• High labor and material costs [11] • High remodel costs due to low versatility [11]	• Cement production - carbon emissions • Heavy for transportation - more fuel used

These alternatives were scored out of 100 for each impact. The timber alternative resulted in a higher Sustainability Index (SI) as shown in Table 9-2.

Table 9- 2: Impact Scoring

Scoring						
Alternative	**Social**	**Economic**	**Environmental**	**Total**	**Max - Min**	**SI**
Timber	80	65	85	230	20	210
CMU	75	70	35	180	40	140

After the project, the structure was donated to a local family to be used a children's playhouse. This donation had a positive impact on the community and repurposed the wood sustainably. Scrap wood leftover from the project was donated to the local Habitat for Humanity.

10.0 Conclusion

The TimberStrong competition tasked ASCE student teams to act as a design-build construction firm, aiming to create a two-story light-framed timber building that met criteria for structural durability, aesthetic appeal, and sustainability. The Ponderosa TimberJacks successfully designed and constructed a building within all competition guidelines and constraints. The building was structurally sound in line with wood design codes and featured a continuous load from the roof to the foundation. Its symmetric layout, chimney, and wallpaper contributed to its visual appeal within the theme of a summer camp log cabin. 2D drawings and a BIM model demonstrated structural details, aiding in an accurate construction process. The construction of a timber house in comparison to a CMU structure was more sustainable within the social, economic, and environmental impacts of the alternatives. The Ponderosa TimberJacks secured 2nd place for the overall competition.

11.0 References

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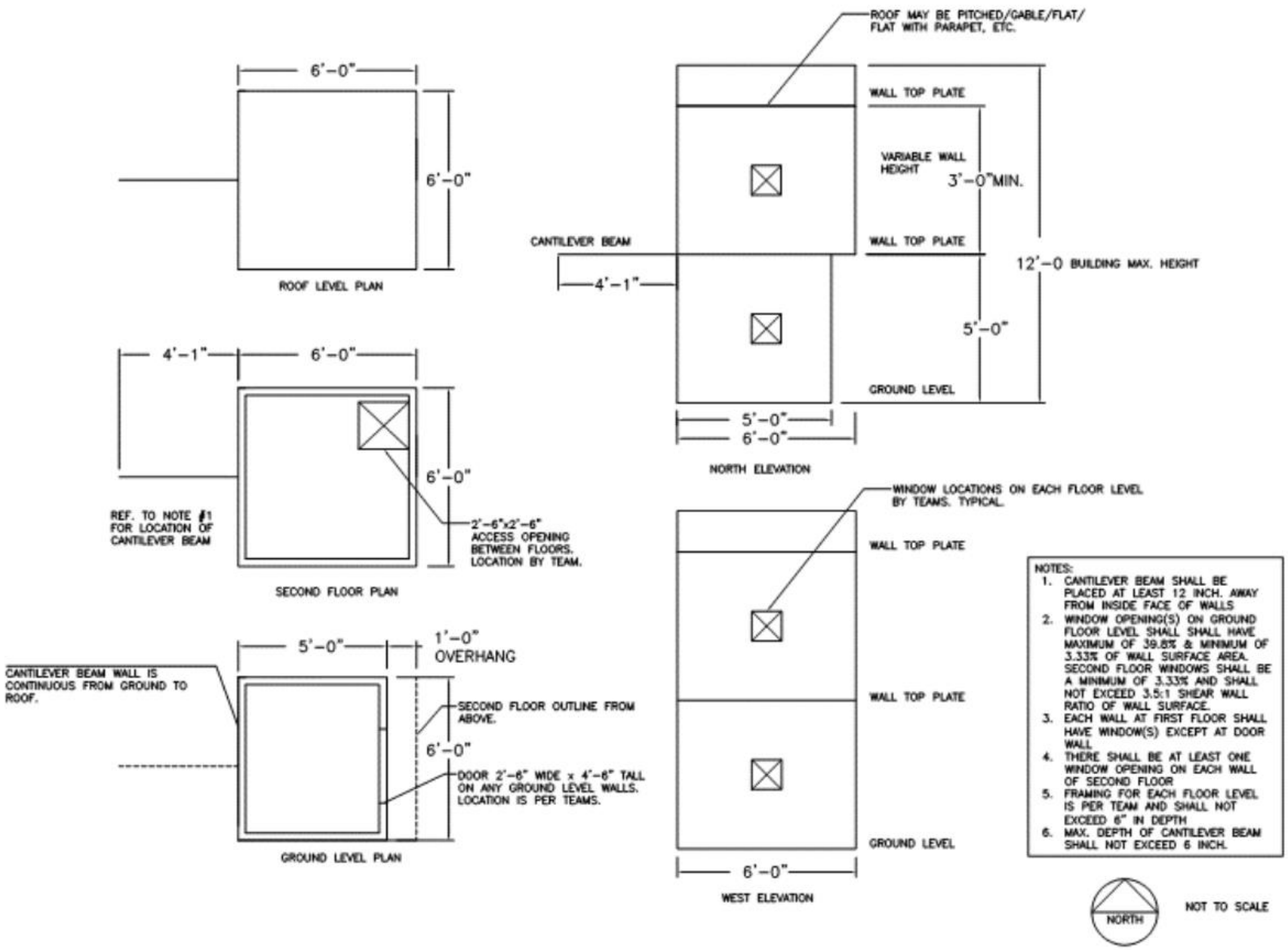
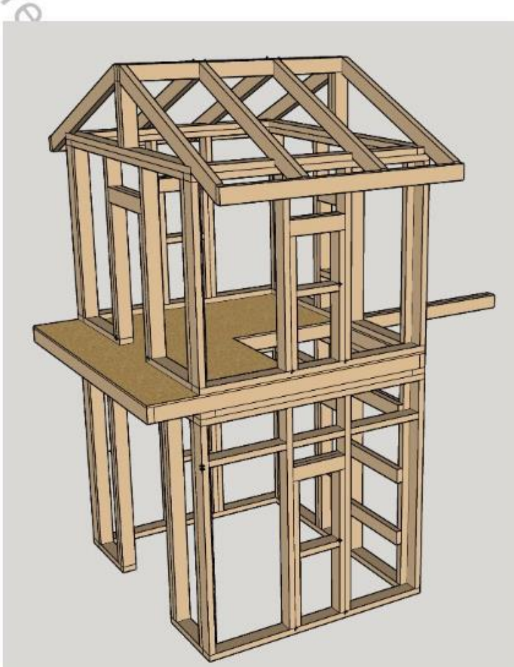
APPENDICIES

APPENDIX A: OFFICIAL DESIGN CONSTRAINTS



APPENDIX B: DESIGN CALCULATIONS

DESIGN



The roof will be composed of two panels (not sheathed before competition), so there will be two "ridge beams" that sit on two stud columns.

Material Properties

$L_{RB} := 59\ in$ Length of Ridge Beam
$L_{Rafter} := 37.22\ in$ Measured from Ridge Beam to Top Plate (bearing area spot)
$L_{OH} := 12\ in$ Length of Rafter Overhang
$SG_{hem} := 0.43$ [NDS SUPP. T4.A pg. 35]
$DENS_w := 62.3\ pcf$
$DENS_{hem} := SG_{hem} \cdot DENS_w = 26.789\ pcf$
$b := 1.5\ in$ Assumed 2x due to small structure size
$XS_{assumed} := b \cdot 3.5\ in = 0.036\ ft^2$

Loads

$L_r := 20\ psf$
$L_f := 50\ psf$ [TimberStrong Rules]
$R_{WU} := 30\ psf$
$w_{stud} := DENS_{hem} \cdot XS_{assumed} = 0.977\ plf$ Assumed self-weight stud dead load (one 2x4 Hem Fir)
$SH := 40\ lbf \div (4\ ft \cdot 8\ ft) = 1.25\ psf$ Dead load due to OSB and stud in one square foot of area
$DL_{OSB} := 1.25\ psf$ Assumed $DL_{2x4} := 1.65\ psf$ Assumed $DL := DL_{2x4} + DL_{OSB} = 2.9\ psf$

Beam Adjustment Factors

[NDS Supplement Table 4A Adjustment Factors]

$C_R := 1.15$ B/C Rafters are spaced less than 24in on center

$C_M := 1.0$ Assumed moisture content less than 19%

$C_{FU} := 1.0$ Load is applied edge-wise not flat-wise

$C_F := 1.5$ Grade No. 2

$d := 3.5\ in$

$K_e := 0.5$

$L := 5\ ft$

$L_o := L \cdot K_e = 30\ in$

$E_{min} := 470000\ psi$

$$F_{cE} := \frac{(0.822 \cdot E_{min})}{\left(\frac{L_o}{d}\right)^2} \qquad C_F := 1.5$$

$c := 0.8$ For Sawn Lumber NDS 3.7.1

$f_{cprime} := 1300\ psi \cdot C_M \cdot C_F = 1950\ psi$ Compression Parallel to the Grain multiplied by all applicable adjustment factors except for Cp

$$C_P := \frac{1 + (F_{cE} \div f_{cprime})}{2\ c} - \sqrt{\left(\frac{1 + (F_{cE} \div f_{cprime})}{2\ c}\right)^2 - \frac{(F_{cE} \div f_{cprime})}{c}} = 0.908$$ NDS 3.7-1

Bucking Length Coefficient NDS Table G1

Adjusted Capacity Values

$f_s := 850\ psi \cdot C_R \cdot C_M \cdot C_{FU} \cdot C_F = 1466.25\ psi$ [NDS Supplement Table 4A Page 35]

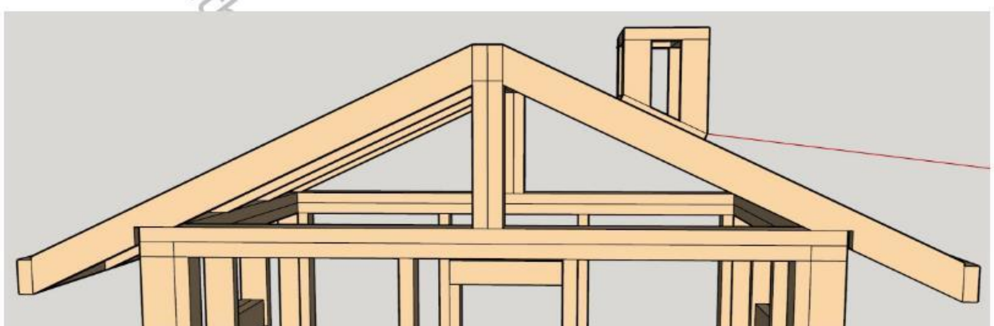
$f_v := 150\ psi \cdot C_R \cdot C_M \cdot C_{FU} = 172.5\ psi$

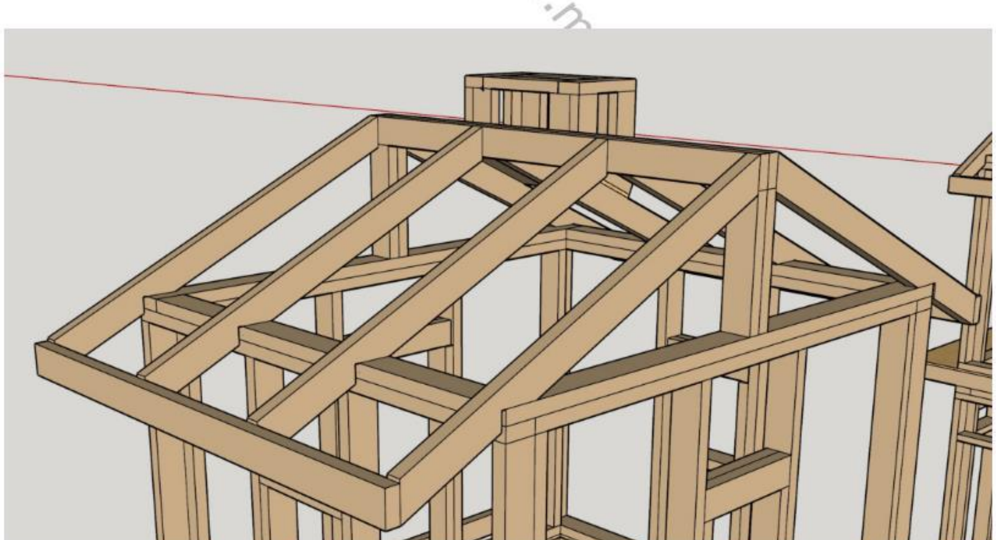
$f_{compParallel} := 1300\ psi \cdot C_M \cdot C_F \cdot C_P = 1770.309\ psi$

$f_{cperp} := 405\ psi \cdot C_M = 405\ psi$

$f_t := 525\ psi \cdot C_F = 787.5\ psi$

ROOF DESIGN





Rafter Design

Design for one interior rafter on the roof panel that includes the chimney. This design will be conservative (largest tributary and largest load), so the design of the interior rafters and the other half of the roof can be the same design.

Loads

$PL_{CH} := 17\ lbf$ Dead load self weight of the chimney 2x4's and sheathing.

$PL_{CH} \div 2 = 8.5\ lbf$ Half goes to interior rafter

$Trib_{int} := 19.167\ in$ On-center spacing of the interior rafters

$w_{LR} := L_r \cdot Trib_{int} = 31.945\ plf$

$w_D := SH \cdot Trib_{int} + w_{stud} = 2.973\ plf$

Using ASD Load Combination 1.0D+1.0L

$w_{DandL} := 1.0\ w_D + 1.0 \cdot w_{LR} = 34.918\ plf$ Distributed Load on the Exterior Rafter

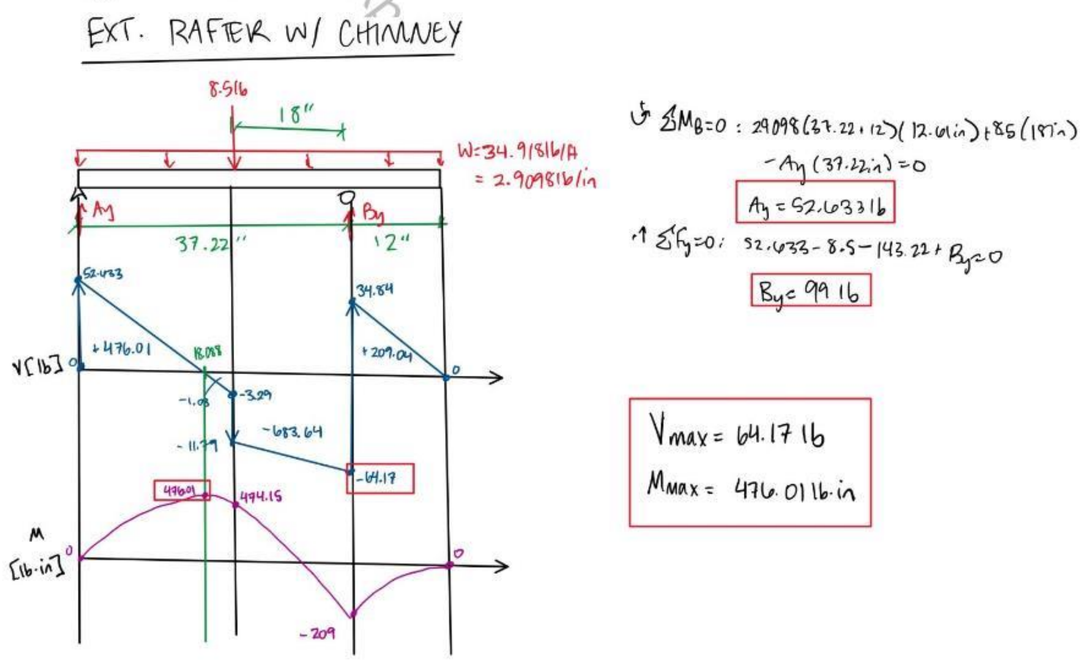
Solve For

$Rea_{1Raft} := 52.633\ lbf$ Reaction of the rafter at the ridge beam

$Rea_{2Raft} := 99\ lbf$ Reaction of the rafter at the top plate

$M_{max} := 476\ lbf \cdot in$ Maximum moment

$V_{max} := 64.17\ lbf$ Maximum shear



Flexure Design

$d := \sqrt{(6 \cdot M_{max}) \div (f_s \cdot b)} = 1.14\ in$ [NDS 3.3-2] $\text{if}(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}) = \text{“Good”}$

Shear Design

$d := (3\ V_{max}) \div (2\ b \cdot f_v) = 0.372\ in$ [NDS 3.4.2] $\text{if}(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}) = \text{“Good”}$

Ridge Beam Design

The ridge beam takes the point loads from the rafters and it's own self-weight

$PL_{RB} := Rea_{1Raft} = 52.633\ lbf$ Point load from the rafters on the ridge beam

$w_D := w_{stud} + SH \cdot 1.5\ in = 1.133\ plf$ Dead load from stud and the sheathing on the ridge beam

Solve For

$Rea_{RB} := 108.05\ lbf$ Reaction of the ridge beam on the stud column

$M_{max} := 1076.39\ lbf \cdot in$ $V_{max} := 55.42\ lbf$

Flexure Design

$d := \sqrt{(6 \cdot M_{max}) \div (f_s \cdot b)} = 1.714\ in$

[NDS 3.3-2]

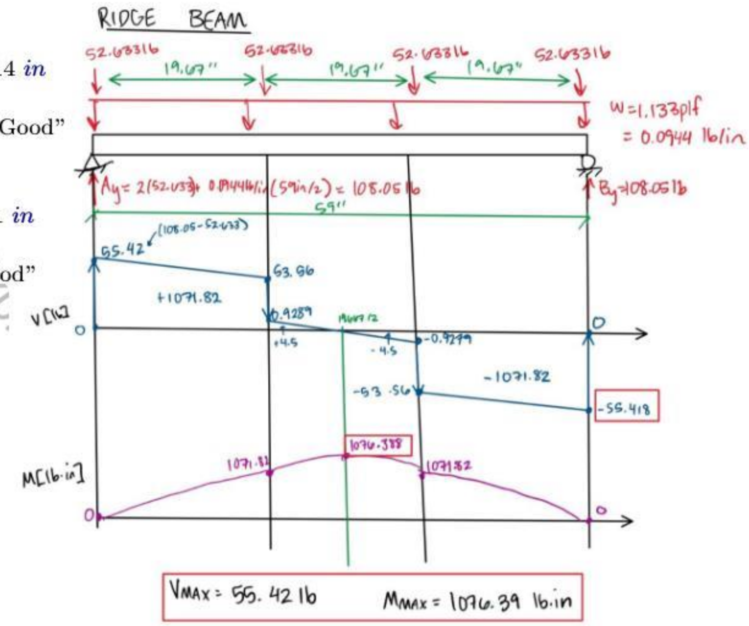
$\text{if}\left(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}\right) = \text{“Good”}$

Shear Design

$d := (3\ V_{max}) \div (2\ b \cdot f_v) = 0.321\ in$

[NDS 3.4.2]

$\text{if}\left(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}\right) = \text{“Good”}$



Front and Back Wall Center Stud Column

Compression stress parallel to the grain shall not exceed the adjusted compression design value [NDS 3.6.3]

Compression Design stress equals force over area (area is bd) -> solved for d

$d := Rea_{RB} \div (b \cdot f_{compParallel}) = 0.041\ in$ $\text{if}\left(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}\right) = \text{“Good”}$

Wind Uplift

Dimensions

$h_{secstory} := 4\ ft$ Height of the second story

$Trib_{roof} := h_{secstory} \div 2 = 2\ ft$ Vertical Tributary

$Trib_{rafter} := 19.967\ in$ For interior rafter - conservative

Loads

$R_{wu} := 30\ psf \cdot 0.6 = 18\ psf$ 0.6 for ASD

[TimberStrong Rules, wind load at roof overhang]

$R_w := R_{wu} \cdot Trib_{roof} = 36\ plf$ Distributed Load on Roof

$U := R_w \cdot Trib_{rafter} = 59.901\ lbf$ Uplift force on one rafter

Capacity

$U_{capacity} := 225\ lbf$ H3.SS Tie - Capacity $\text{if}\left(U \le U_{capacity}, \text{“Good”}, \text{“Bad”}\right) = \text{“Good”}$

LATERAL DESIGN

"Lateral seismic load of E = 275 plf at the roof diaphragm in both directions (not simultaneously)."

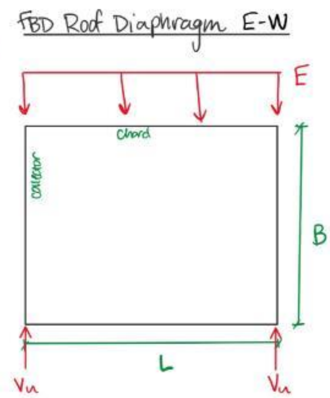
East to West Roof Diaphragm (Direction One)

Capacity

$v_n := 520\ plf$ Table 4.2A SDPWS (Nail Size 6d, Sheathing 3/8, 6 in spa.)

$v_{na} := v_n \cdot (1 - 0.5 \cdot SG_{hem}) \div 2.8 = 145.786\ plf$

Adjusted Nominal Unit Shear Capacity



Loads

$E_s := 275\ plf \cdot 0.7 = 192.5\ plf$ 0.7 for ASD

$L := 59\ in$ $B := 59\ in$

$V_{rEW} := E_s \cdot L \div 2 = 473.229\ lbf$

Shear. Modeled as a simply supported beam, reaction equals load/2

$v_u := V_{rEW} \div B = 96.25\ plf$ Unit Shear

if $(v_u \le v_{na}, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

Factor of Safety: $FS_1 := v_{na} \div v_u = 1.515$

Chord Design

$M_{max} := (E_s \cdot L^2) \div 8 = 0.582\ kip \cdot ft$ $T := M_{max} \div B = 0.118\ kip$ Tension Force In Chord

$d_{assu} := 3.5\ in$ $\sigma := T \div (b \cdot d_{assu}) = 22.535\ psi$

if $(\sigma \le f_{compParallel}, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$ if $(\sigma \le f_t, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

Collector Design

$\sigma := V_{rEW} \div (b \cdot d_{assu}) = 90.139\ psi$ if $(\sigma \le f_{compParallel}, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

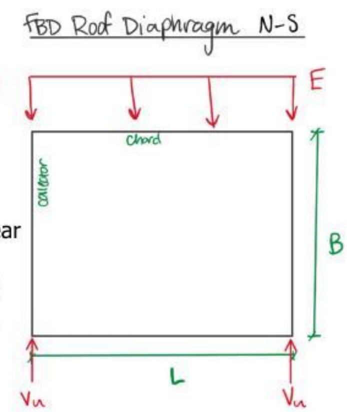
North to South Roof Diaphragm (Direction Two)

Capacity

$v_n := 520\ plf$ Table 4.2A SDPWS (Nail Size 6d, Sheathing 3/8, 6 in sp

$v_{na} := v_n \cdot (1 - 0.5 \cdot SG_{hem}) \div 2.8 = 145.786\ plf$

Adjusted Nominal Unit Shear Capacity



Loads

$E_s := 275\ plf \cdot 0.7 = 192.5\ plf$ $L := 59\ in$ $B := 59\ in$

$V_{rNS} := E_s \cdot L \div 2 = 473.229\ lbf$ $v_u := V_{rNS} \div B = 96.25\ plf$ Unit Shear

if $(v_u \le v_{na}, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

Factor of Safety: $FS_2 := v_{na} \div v_u = 1.515$

Chord Design

$M_{max} := (E_s \cdot L^2) \div 8 = 0.582\ kip \cdot ft$

$T := M_{max} \div L = 0.118\ kip$ Tension Force In Chord

$d_{assu} := 3.5\ in$

$\sigma := T \div (b \cdot d_{assu}) = 22.535\ psi$ if $(\sigma \le f_{compParallel}, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

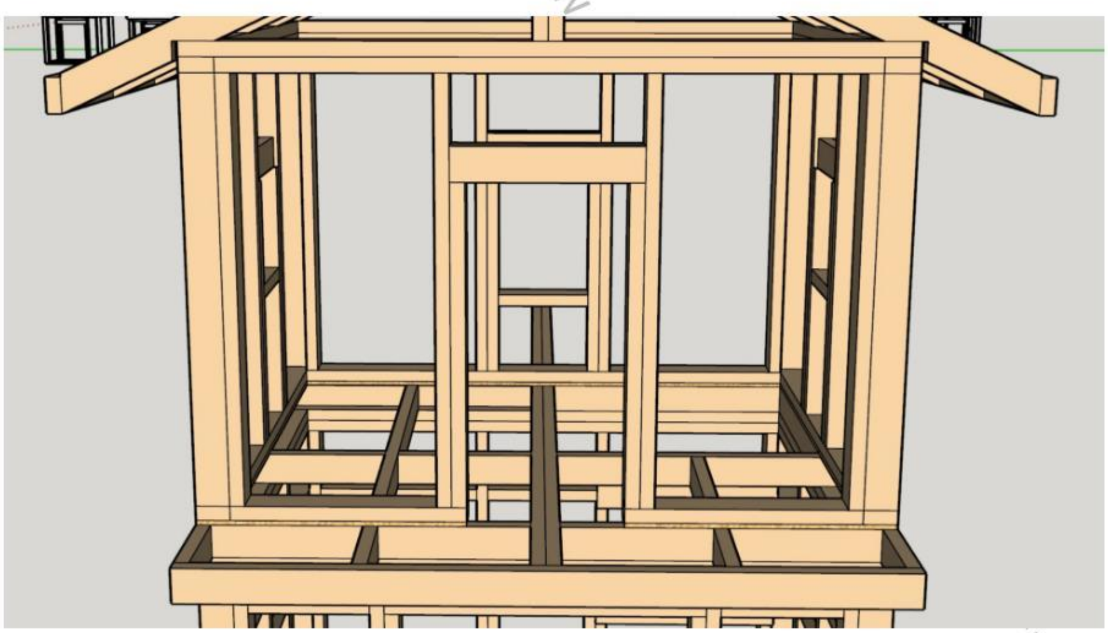
if $(\sigma \le f_t, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

Collector Design

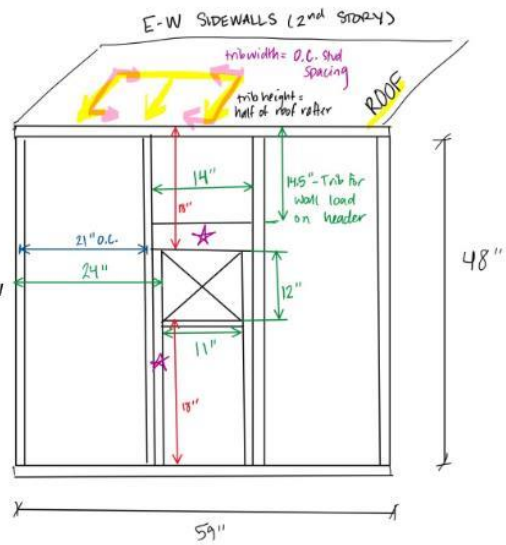
$\sigma := V_{rNS} \div (b \cdot d_{assu}) = 90.139\ psi$ if $(\sigma \le f_{compParallel}, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

Roof Design Summary
-All members are 2x4's
-H3.SS Roof Tie Downs on all rafters
-Sheathing is 3/8"
-Nails are 6d
-Nails are spaced at 6" along the diaphragm edges
-Roof Diaphragm FS=1.515

2nd STORY WALLS GRAVITY DESIGN



Second Story East-West Sidewalls



$LF_{2x4} := 59\ in \cdot 2 + 48\ in \cdot 4 + 40\ in \cdot 2 + 15\ in + 12\ in = 417\ in$
$Area_{SH} := (51\ in \cdot 59\ in) - (12\ in \cdot 12\ in) = 19.896\ ft^2$
$SW_{EW2Wall} := w_{stud} \cdot LF_{2x4} + SH \cdot Area_{SH} = 0.0588\ kip$

HEADER:

Dimensions

$L_{Raft} := (L_{Rafter} + L_{OH})$ Length of rafter
$L_{wall} := 59\ in$ Length of wall
$L_{header} := 14\ in$ Length of header
$Trib_{WALL} := 14.5\ in$ Height of wall above window

Loads

$w_{header} := DL \cdot Trib_{WALL} + w_{stud} = 4.481\ plf$
Distributed Load on the header

Solve For

$Reaction_{Header} := (w_{header} \cdot L_{header}) \div 2 = 2.614\ lbf$
$V_{Max} := Reaction_{Header} = 2.614\ lbf$
$M_{Max} := (w_{header} \cdot L_{header}{}^2) \div 8 = 9.148\ lbf \cdot in$

Flexure Design

$d := \sqrt{(6 \cdot M_{Max}) \div (f_s \cdot b)} = 0.158\ in$ [NDS 3.3-2] $\text{if}(d \le 3.5\ in, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

Shear Design

$d := (3\ V_{Max}) \div (2\ b \cdot f_v) = 0.015\ in$ [NDS 3.4.2] $\text{if}(d \le 3.5\ in, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

TRIMMER STUDS:

Compression Design

$d := \dfrac{Reaction_{Header}}{b \cdot f_{compParallel}} = 0.001\ in$ $\text{if}(d \le 3.5\ in, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

KING STUD:

Checking one "king stud" because it has the largest tributary and is only braced in one direction

Dimensions

$Trib_{height} := L_{Raft} \div 2 = 24.61\ in$ $Trib_{width} := 18.75\ in$ On-center stud spacing

$Trib_{area} := Trib_{width} \cdot Trib_{height} = 461.438\ in^2$

Loads

$PL_{CH} := 17.6\ lbf$ Point load due to the chimney on the roof (applied to load-bearing sidewalls)

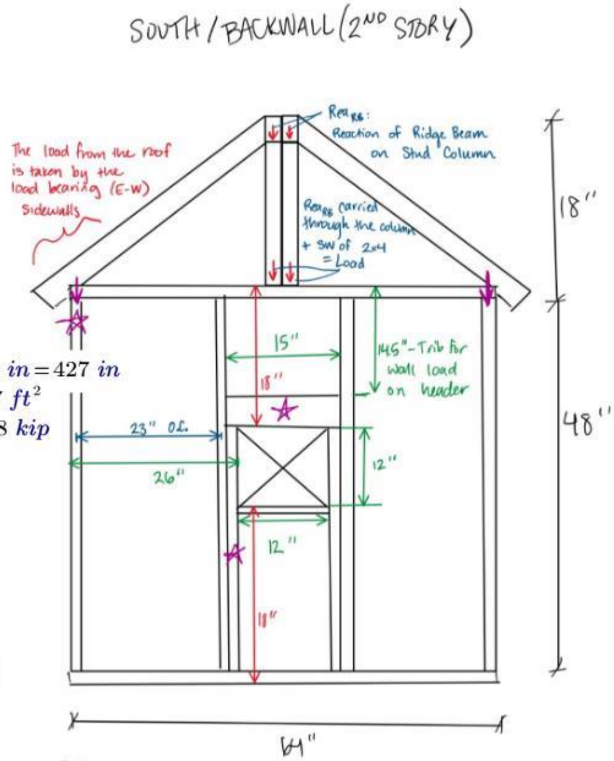
$Load := PL_{CH} + (DL + L_r) \cdot Trib_{area} = 90.981\ lbf$ Load applied in compression to the king stud

$w_{rEW} := Load \div L_{wall} = 18.5047\ plf$ Roof Load that comes down onto the first story walls
$w_{EW} := w_{rEW} + SW_{EW2Wall} \div L_{wall} = 30.466\ plf$ Distributed Load applied to the floor due to roof dead load (except for chimney), roof live load, and second story wall dead load

Compression Design

$d := \dfrac{Load}{b \cdot f_{compParallel}} = 0.034\ in$ $\text{if}(d \le 3.5\ in, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

Second Story, South/Back Wall



$L_{wall} := 64\ in$

$LF_{2x4} := 64\ in \cdot 2 + 48\ in \cdot 4 + 40\ in \cdot 2 + 12\ in + 15\ in = 427\ in$

$Area_{SH} := (51\ in \cdot 64\ in) - (12\ in \cdot 12\ in) = 21.667\ ft^2$

$SW_{S2Wall} := w_{stud} \cdot LF_{2x4} + DL_{OSB} \cdot Area_{SH} = 0.0618\ kip$

$w_S := SW_{S2Wall} \div L_{wall} = 11.594\ plf$

HEADER:

Dimensions

$L_{header} := 15\ in$ $\quad Trib_{WALL} := 14.5\ in$ Tributary is the height of the wall above the window header

Loads

$w_{header} := DL \cdot Trib_{WALL} + w_{stud} = 4.481\ plf$ Distributed Load on the header

Solve For

$Reaction_{Header} := (w_{header} \cdot L_{header}) \div 2 = 2.801\ lbf$ $\quad V_{Max} := Reaction_{Header} = 2.801\ lbf$

$M_{Max} := (w_{header} \cdot L_{header}{}^2) \div 8 = 10.502\ lbf \cdot in$

Flexure Design

$d := \sqrt{(6 \cdot M_{Max}) \div (f_s \cdot b)} = 0.169\ in$ [NDS 3.3-2] $\quad \text{if}(d \le 3.5\ in, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

Shear Design

$d := (3\ V_{Max}) \div (2\ b \cdot f_v) = 0.016\ in$ [NDS 3.4.2] $\quad \text{if}(d \le 3.5\ in, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

TRIMMER STUDS:

Compression Design

$d := Reaction_{Header} \div (b \cdot f_{compParallel}) = 0.0011\ in$ $\quad \text{if}(d \le 3.5\ in, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

EXTERIOR STUD:

Conservative: apply one of the ridge beam reactions that comes through the center column to the exterior stud (smaller trib). The top plate would have four supports (wall studs), but simply supported is conservative.

Loads

$Load_{ext} := Rea_{RB} + w_{stud} \cdot 18\ in = 109.515\ lbf$ Load from reaction of top plate, which was from the reaction of the ridge beam on the supporting stud

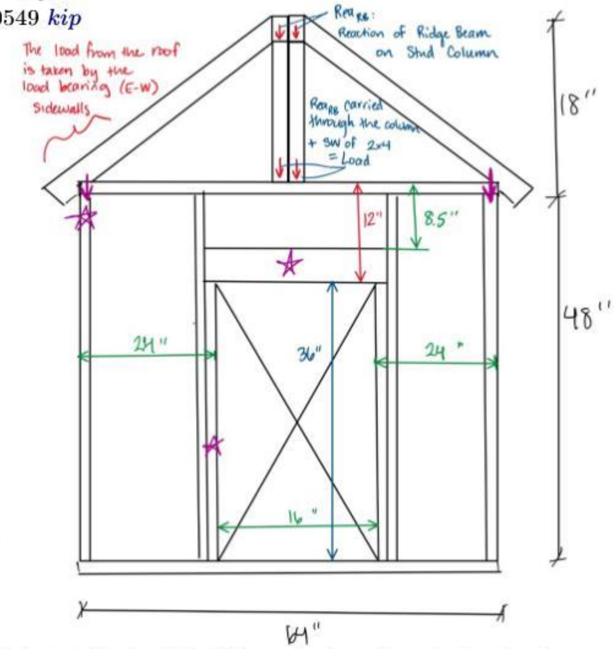
Compression Design

$d := Rea_{RB} \div (b \cdot f_{compParallel}) = 0.041\ in$ $\quad \text{if}(d \le 3.5\ in, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

Second Story, North/Front Wall

NORTH / FRONT WALL (2ND STORY)

$L_{wall} := 64\ in$
$LF_{2x4} := 64\ in \cdot 2 + 48\ in \cdot 4 + 36\ in \cdot 2 + 27\ in = 419\ in$
$Area_{SH} := (51\ in \cdot 64\ in) - (24\ in \cdot 36\ in) = 16.667\ ft^2$
$SW_{N2Wall} := w_{stud} \cdot LF_{2x4} + DL_{OSB} \cdot Area_{SH} = 0.0549\ kip$
$w_N := SW_{N2Wall} \div L_{wall} = 10.3\ plf$



HEADER:

Dimensions

$L_{header} := 27\ in$ $\quad Trib_{WALL} := 8.5\ in$ Tributary is the height of the wall above the window header

Loads

$w_{header} := DL \cdot Trib_{WALL} + w_{header} = 6.535\ plf$ Distributed Load on the header

Solve For

$Reaction_{Header} := (w_{header} \cdot L_{header}) \div 2 = 7.352\ lbf$ $\quad V_{Max} := Reaction_{Header} = 7.352\ lbf$
$M_{Max} := (w_{header} \cdot L_{header}^2) \div 8 = 49.625\ lbf \cdot in$

Flexure Design

$d := \sqrt{(6 \cdot M_{Max}) \div (f_s \cdot b)} = 0.368\ in$ [NDS 3.3-2] $\quad$ if$(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}) = \text{“Good”}$

Shear Design

$d := (3\ V_{Max}) \div (2\ b \cdot f_v) = 0.043\ in$ [NDS 3.4.2] $\quad$ if$(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}) = \text{“Good”}$

TRIMMER STUDS:

Compression Design

$d := Reaction_{Header} \div (b \cdot f_{compParallel}) = 0.0028\ in$ $\quad$ if$(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}) = \text{“Good”}$

EXTERIOR STUD:

Conservative: apply one of the ridge beam reactions that comes through the center column to the exterior stud (smaller trib). The top plate would have four supports (wall studs), but simply supported is conservative.

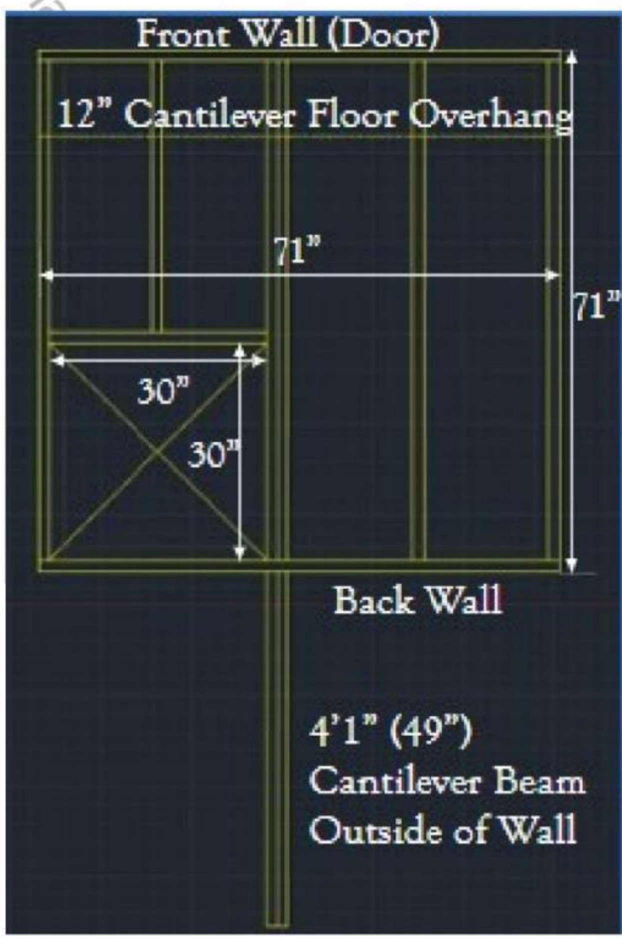
Loads

$Load_{ext} := Rea_{RB} + w_{stud} \cdot 18\ in = 109.515\ lbf$ Load from reaction of top plate, which was from the reaction of the ridge beam on the supporting stud

Compression Design

$d := Rea_{RB} \div (b \cdot f_{compParallel}) = 0.041\ in$ $\quad$ if$(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}) = \text{“Good”}$

FLOOR DESIGN



Floor Design

$Trib_1 := (17\ in) \div 2 = 8.5\ in$ $L_1 := 68\ in$

$Trib_2 := 17\ in$ $L_2 := 34.5\ in$

$Trib_{3.1} := (19.5 \div 2)\ in$ $L_3 := 118.5\ in$

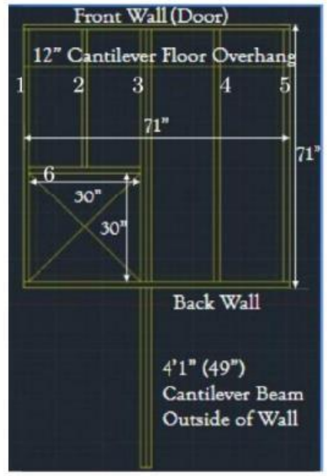
$Trib_{3.2} := 19.5\ in$

$Trib_4 := 17\ in$ $L_4 := 68\ in$

$Trib_5 := (17\ in) \div 2 = 8.5\ in$ $L_5 := 68\ in$

$Trib_6 := 1.5\ in$ $L_6 := 30\ in$

$a := 12\ in$ $x := 30.75\ in$



$L_f = 50\ psf$

$w_{stud} = 0.977\ plf$

$SH = 1.25\ psf$

Shear and Moment calculations for Beams 1-5 were solved using Canned Equations

Beam 2 Design:

Loads

$w_2 := Trib_2 \cdot (SH + L_f) + w_{stud} = 73.581\ plf$

Solve For

$R_{end2} := (w_2 \div (2 \cdot L_2)) \cdot (L_2{}^2 - a^2) = 0.093\ kip$ $R_{int} := (w_2 \div (2 \cdot L_2)) \cdot (L_2 + a)^2 = 0.192\ kip$

$V_2 := w_2 \cdot a = 0.074\ kip$ $V_3 := (w_2 \div (2 \cdot L_2)) \cdot (L_2{}^2 + a^2) = 0.119\ kip$

$V_{max} := \max(R_{end2}, V_2, V_3) = 118.569\ lbf$

$M_1 := (w_2 \div (8 \cdot L_2{}^2)) \cdot (L_2 + a)^2 \cdot (L_2 - a)^2 = 0.059\ kip \cdot ft$ $M_2 := (w_2 \cdot a^2) \div 2 = 0.037\ kip \cdot ft$

$M_{max} := \max(M_1, M_2) = 0.059\ kip \cdot ft$

Flexure Design

$d := \sqrt{(6 \cdot M_{max}) \div (f_s \cdot b)} = 1.387\ in$ [NDS 3.3-2] $\mathbf{if}(d \le 3.5\ in, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

Shear Design

$d := (3\ V_{max}) \div (2\ b \cdot f_v) = 0.687\ in$ [NDS 3.4.2] $\mathbf{if}(d \le 3.5\ in, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

Beam 6 Design:

Loads

$w_6 := Trib_6 \cdot (SH) + w_{stud} = 1.133\ plf$

Solve For

$PL_6 := R_{end2} = 92.976\ lbf$ Reaction of Beam 2, is a point load on the center of Beam 6

$R_6 := (w_6 \cdot L_6 + PL_6) \div 2 = 47.904\ lbf$ Reaction of Beam 6, on the exterior beam and the cantilever beam

$V_{max} := (PL_6 \div 2) + (w_6 \cdot L_6) \div 2 = 47.904\ lbf$ $M_{max} := (PL_6 \cdot L_6) \div 4 + (w_6 \cdot L_6^2) \div 8 = 0.059\ kip \cdot ft$

Flexure Design

$d := \sqrt{(6 \cdot M_{max}) \div (f_s \cdot b)} = 1.39\ in$ [NDS 3.3-2] if$(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}) = \text{“Good”}$

Shear Design

$d := (3\ V_{max}) \div (2\ b \cdot f_v) = 0.278\ in$ [NDS 3.4.2] if$(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}) = \text{“Good”}$

Beam 1 Design:

Loads

$PL_1 := R_6 = 47.904\ lbf$ Point Load on Beam 1 from Beam 6 $PL_{CH} := 17.598\ lbf$

$w_1 := Trib_1 \cdot (SH + L_f) + w_{stud} + w_{EW} = 67.745\ plf$

Solve For

$V_{max} := 144.14\ lbf$

$M_{max} := 0.119\ kip \cdot ft$

Flexure Design

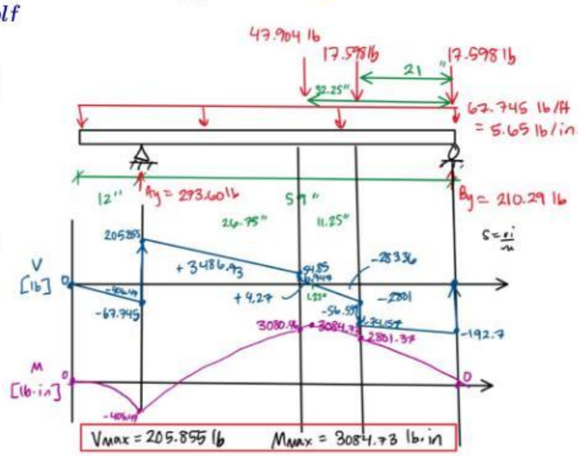
$d := \sqrt{(6 \cdot M_{max}) \div (f_s \cdot b)} = 1.974\ in$

if$(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}) = \text{“Good”}$

Shear Design

$d := (3\ V_{max}) \div (2\ b \cdot f_v) = 0.836\ in$

if$(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}) = \text{“Good”}$



Beam 4 Design:

Loads

$w_4 := Trib_4 \cdot (SH + L_f) + w_{stud} = 73.581\ plf$

Solve For

$R_{end} := (w_4 \div (2 \cdot L_4)) \cdot (L_4^2 - a^2) = 201.987\ lbf$ $R_{int} := (w_4 \div (2 \cdot L_4)) \cdot (L_4 + a)^2 = 288.552\ lbf$

$V_2 := w_4 \cdot a = 73.581\ lbf$ $V_3 := (w_4 \div (2 \cdot L_4))\ (L_4^2 + a^2) = 214.971\ lbf$

$V_{max} := \max(R_{end}, V_2, V_3) = 214.971\ lbf$

$M_1 := (w_4 \div (8 \cdot L_4^2)) \cdot (L_4 + a)^2 \cdot (L_4 - a)^2 = 0.277\ kip \cdot ft$ $M_2 := (w_4 \cdot a^2) \div 2 = 0.037\ kip \cdot ft$

$M_{max} := \max(M_1, M_2) = 0.277\ kip \cdot ft$

Flexure Design

$d := \sqrt{(6 \cdot M_{max}) \div (f_s \cdot b)} = 3.013\ in$ [NDS 3.3-2] if$(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}) = \text{“Good”}$

Shear Design

$d := (3\ V_{max}) \div (2\ b \cdot f_v) = 1.246\ in$ if$(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}) = \text{“Good”}$

Beam 5 Design:

Loads

$w_5 := Trib_5 \cdot (SH + L_f) + w_{stud} + w_{EW} = 67.745\ plf$

Solve For

$R_{end} := (w_5 \div (2 \cdot L_5)) \cdot (L_5^2 - a^2) = 0.186\ kip$ $R_{int} := (w_5 \div (2 \cdot L_5)) \cdot (L_5 + a)^2 = 0.266\ kip$

$V_2 := w_5 \cdot a = 67.745\ lbf$ $V_3 := (w_5 \div (2 \cdot L_5))\ (L_5^2 + a^2) = 197.921\ lbf$

$V_{max} := \max(R_{end}, V_2, V_3) = 197.921\ lbf$

$M_1 := (w_5 \div (8 \cdot L_5^2)) \cdot (L_5 + a)^2 \cdot (L_5 - a)^2 = 0.255\ kip \cdot ft$ $M_2 := (w_5 \cdot a^2) \div 2 = 0.034\ kip \cdot ft$

$M_{max} := \max(M_1, M_2) = 0.255\ kip \cdot ft$

Flexure Design

$d := \sqrt{(6 \cdot M_{max}) \div (f_s \cdot b)} = 2.891\ in$ [NDS 3.3-2] $\text{if}(d \le 3.5\ in, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

Shear Design

$d := (3\ V_{max}) \div (2\ b \cdot f_v) = 1.147\ in$ $\text{if}(d \le 3.5\ in, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

Beam 3 Design (Cantilever - Two 2x4's):

Loads

$w_{3.1} := Trib_{3.1} \cdot (SH + L_f) + w_{beam} = 0.04359\ kip \div ft$ dist load next to opening

$w_{3.2} := Trib_{3.2} \cdot (SH + L_f) + w_{beam} = 0.08523\ kip \div ft$ dist load non-opening length

$w_{3.3} := w_{stud} = 0.00098\ kip \div ft$ dist load cantilever

$w_{3.4L} := Trib_{3.2} \cdot (SH + L_f) + w_{beam} = 0.08523\ kip \div ft$ dist load overhang

$PL_1 := w_S \cdot Trib_{3.1} = 0.00942\ kip$ Point Load from the weight of 2nd Story South Wall

$PL_2 := R_6 = 0.0479\ kip$ Point Load on Beam 1 from Beam 6

$PL_3 := w_N \cdot Trib_{3.2} = 0.01674\ kip$ Point Load from the weight of 2nd Story North Wall

$PL_4 := 0.150\ kip$ Point Load at the end of the Cantilever Beam

Solve For

$M_{max} := 608\ lbf \cdot ft$

$V_{max} := 256.6\ lbf$

$R_{C.S} := 0.429\ kip$ Reaction of the cantilever on the first story south wall

$R_{C.N} := 0.199\ kip$ Reaction of the cantilever on the first story north wall

The Shear and Moment Diagram and Deflection Calculations for the cantilever beam are shown in Appendix C. The team chose two 2x4's instead of one 2x6 because the deflection of the 2x6 would be too low (0.35") due to a higher MOI.

Flexure Design

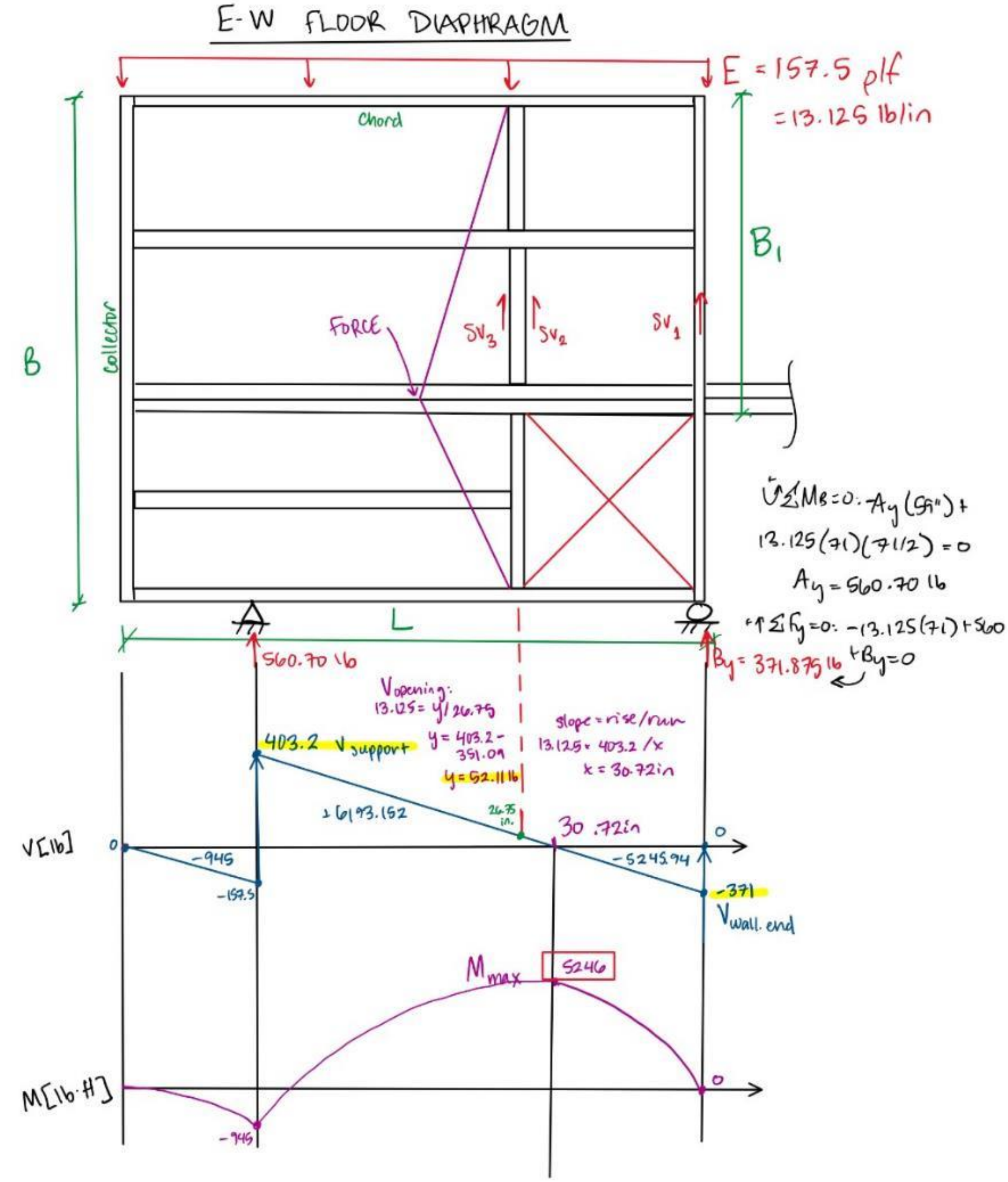
$d := \sqrt{(6 \cdot M_{max}) \div (f_s \cdot b)} = 4.461\ in$ [NDS 3.3-2] $\text{if}(d \le 7\ in, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

Shear Design

$d := (3\ V_{max}) \div (2\ b \cdot f_v) = 1.488\ in$ $\text{if}(d \le 7\ in, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

FLOOR LATERAL DESIGN

East to West Floor Diaphragm (Direction One)



East to West Floor Diaphragm (Direction One)

Capacity

$v_n := 520\ plf$ Table 4.2A SDPWS (Nail Size 6d, Sheathing 3/8, 6 in spa.)

$v_{na} := (v_n \div 2.8) \cdot (1 - 0.5 \cdot SG_{hem}) = 145.786\ plf$ Adjusted Nominal Unit Shear Capacity

Loads

$E_s := 225\ plf \cdot 0.7 = 157.5\ plf$ Factored Load, 0.7Eh for ASD

$L := 71\ in$

$B := 71\ in$

$B_1 := 39.5\ in$ Length of non-opening parallel to the load

Solve For

$v_{wall.end} := 0.371\ kip$

$v_{support} := 0.403\ kip$

$v_{opening} := 0.052\ kip$

$M_{max} := 0.44\ kip \cdot ft$

$V_{fEW} := \max(v_{wall.end}, v_{support}, v_{opening}) = 0.403\ kip$ Max shear, to apply to 1st story shear walls

$sv_1 := v_{wall.end} \div B_1 = 112.709\ plf$

$sv_2 := v_{opening} \div B_1 = 15.797\ plf$

$sv_3 := v_{opening} \div B = 8.789\ plf$

$v_u := sv_1 = 112.709\ plf$ Max unit shear, to design diaphragm

if $(v_u \le v_{na}$, "Good", "Bad"$)$ = "Good" **Factor of Safety:** $FS_3 := v_{na} \div v_u = 1.293$

Extra Force Due to Opening

$Force := (sv_2 - sv_3) \cdot B_1 = 23.07\ lbf$ Force at the opening corner

$F_a := 1235\ lbf$ Allowable force on a SST LSTA24 Light Strap Tie

if $(F_a > Force$, "Strap Design Good", "Bad"$)$ = "Strap Design Good"

Chord Design

$T := M_{max} \div L = 0.074\ kip$ Tension Force In Chord $d_{assu} := 3.5\ in$

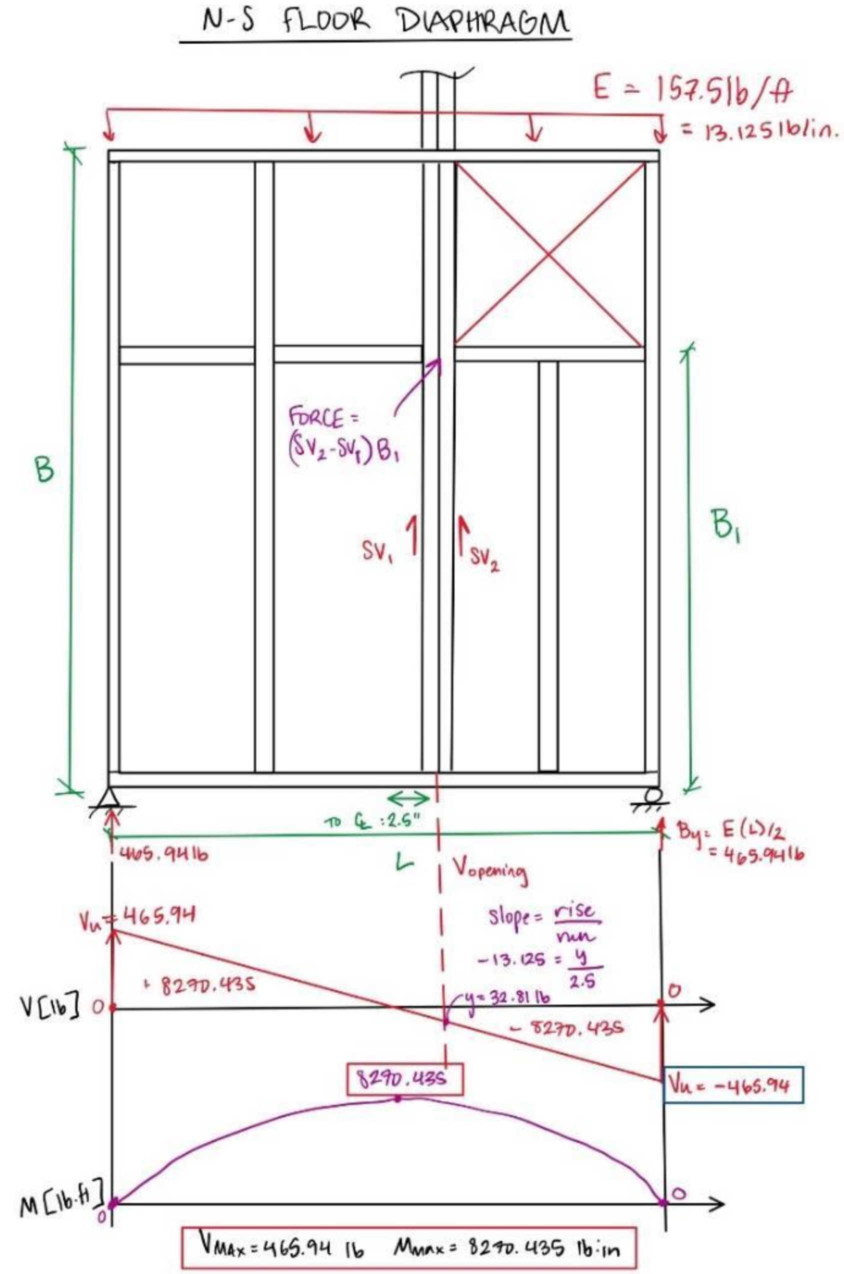
$\sigma := T \div (b \cdot d_{assu}) = 14.165\ psi$ if $(\sigma \le f_{compParallel}$, "Good", "Bad"$)$ = "Good"

if $(\sigma \le f_t$, "Good", "Bad"$)$ = "Good"

Collector Design

$\sigma := v_{wall.end} \div (b \cdot d_{assu}) = 70.667\ psi$ if $(\sigma \le f_{compParallel}$, "Good", "Bad"$)$ = "Good"

North to South Floor Diaphragm (Direction Two)



North to South Floor Diaphragm (Direction Two)

Capacity

$v_n := 520\ plf$ Table 4.2A SDPWS (Nail Size 6d, Sheathing 3/8, 6 in spa.)

$v_{na} := (v_n \div 2.8) \cdot (1 - 0.5 \cdot SG_{hem}) = 145.786\ plf$ Adjusted Nominal Unit Shear Capacity

Loads

$E_s := 225\ plf \cdot 0.7 = 157.5\ plf$ Factored Load, 0.7Eh for ASD

$L := 71\ in$ $B := 71\ in$ $B_1 := 39.5\ in$ Length of non-opening parallel to the load

$v_{opening} := 32.81\ lbf$ $v_{max} := 465.94\ lbf$ $M_{max} := 8270.435\ lbf \cdot in$

$V_{fNS} := v_{max} = 0.466\ kip$ Max shear, to apply to 1st story shear walls

$v_u := v_{max} \div B = 78.75\ plf$

if $(v_u \le v_{na}$, "Good", "Bad"$)$ = "Good" **Factor of Safety:** $FS_4 := v_{na} \div v_u = 1.851$

Check Beam Next to Opening

$sv_1 := v_{opening} \div B = 5.545\ plf$ $sv_2 := v_{opening} \div B_1 = 9.968\ plf$

$Force := (sv_2 - sv_1) \cdot B_1 = 14.557\ lbf$ Apply diaphragm nail spacing along beam 3

$v_u := Force \div B = 2.46\ plf$ if $(v_u \le v_{na}$, "Good", "Bad"$)$ = "Good"

Chord Design

$T := M_{max} \div L = 0.116\ kip$ Tension Force In Chord $d_{assu} := 3.5\ in$

$\sigma := T \div (b \cdot d_{assu}) = 22.188\ psi$ if $(\sigma \le f_{compParallel}$, "Good", "Bad"$)$ = "Good"

if $(\sigma \le f_t$, "Good", "Bad"$)$ = "Good"

Collector Design

$\sigma := v_{max} \div (b \cdot d_{assu}) = 88.75\ psi$ if $(\sigma \le f_{compParallel}$, "Good", "Bad"$)$ = "Good"

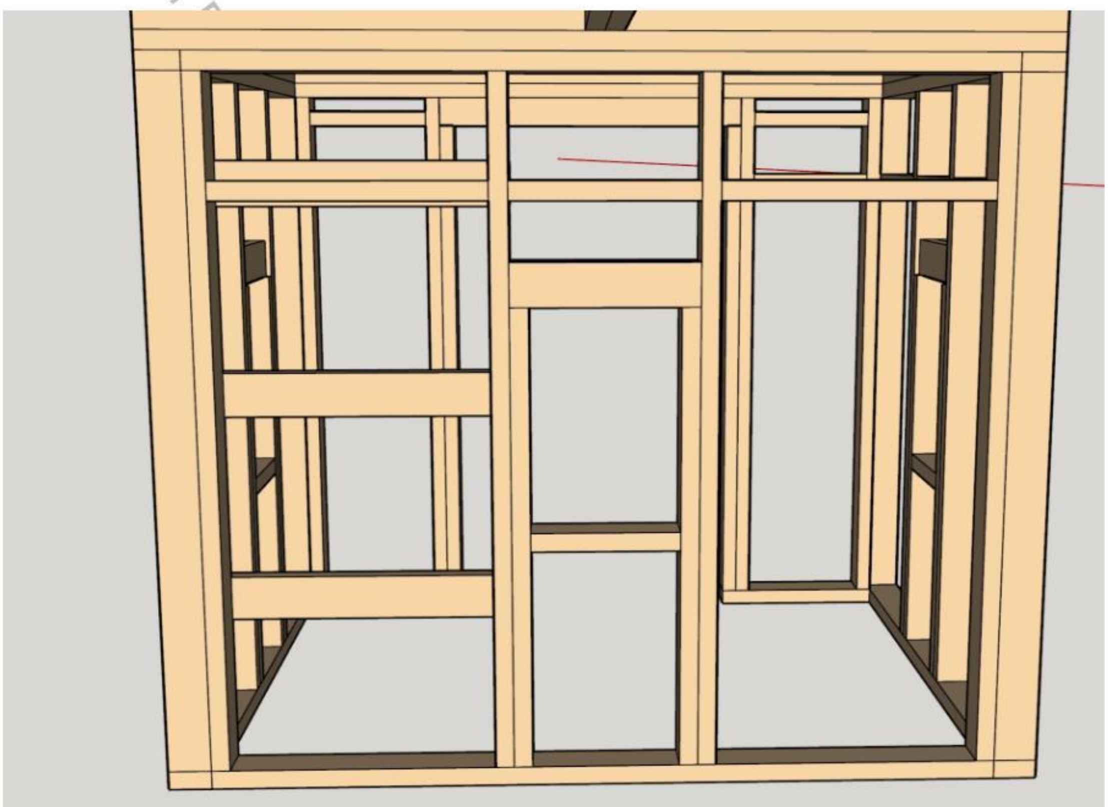
$Average := (FS_1 + FS_2 + FS_3 + FS_4) \div 4 = 1.544$

1.5<FS<1.65 = BEST
1.65<FS<1.8 = GOOD
FS<1.5 or FS>1.8 = BAD

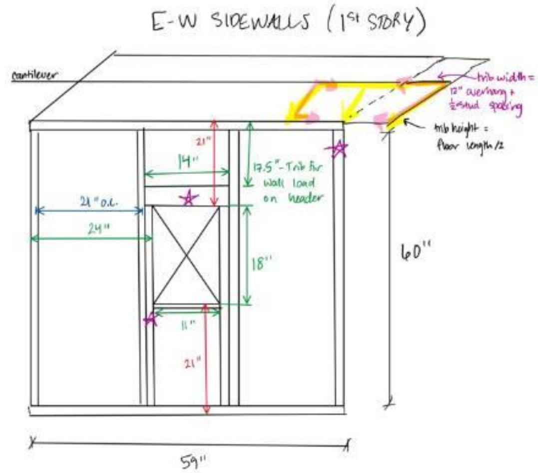
Floor Design Summary

-All members are 2x4's
-Cantilever Beam is double 2x4
-Sheathing is 3/8"
-Nails are 6d
-Nails are spaced at 6" along the diaphragm edges and the cantilever beam inside the diaphragm
-SST LSTA24 used along Beam 6 & subsequent blocking
-Total Diaphragm FS=1.544

1st STORY WALLS GRAVITY DESIGN



First Story East-West Sidewalls



HEADER:

Dimensions

$L_{Floor} := 71\ in$ $\quad L_{Wall} := 59\ in$ $\quad L_{header} := 15\ in$

$Trib_{WALL} := 17.5\ in$

Tributary ht of the wall above the window header

$Trib_{FLOOR} := L_{Floor} \div 2 = 35.5\ in$

Tributary length is the half of the floor

Loads

$w_{header} := DL \cdot Trib_{WALL} + w_{header} = 10.764\ plf$

Distributed Load on the header

Solve For

$Reaction_{Header} := \left(w_{header} \cdot L_{header}\right) \div 2 = 6.728\ lbf$

$V_{Max} := Reaction_{Header} = 6.728\ lbf$

$M_{Max} := \left(w_{header} \cdot {L_{header}}^{2}\right) \div 8 = 25.229\ lbf \cdot in$

Flexure Design

$d := \sqrt{\left(6 \cdot M_{Max}\right) \div \left(f_s \cdot b\right)} = 0.262\ in$ [NDS 3.3-2] $\quad \text{if}\left(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}\right) = \text{“Good”}$

Shear Design

$d := \left(3\ V_{Max}\right) \div \left(2\ b \cdot f_v\right) = 0.039\ in$ [NDS 3.4.2] $\quad \text{if}\left(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}\right) = \text{“Good”}$

TRIMMER STUDS:

Compression Design

$d := \dfrac{Reaction_{Header}}{b \cdot f_{compParallel}} = 0.0025\ in$ $\quad \text{if}\left(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}\right) = \text{“Good”}$

EXTERIOR STUD:

Checking the exterior stud because it has the largest tributary (due to the overhang)

Dimensions

$Trib_{height} := L_{Floor} \div 2 = 35.5\ in$ $\quad Trib_{width} := 22.5\ in$ 12" overhang + 0.5 of stud spacing

$Trib_{area} := Trib_{width} \cdot Trib_{height} = 798.75\ in^2$

Loads

$PL_{CH} := 17.6\ lbf$ Point load due to the chimney on the roof (applied to load-bearing sidewalls)

$w_{EW2Wall} := SW_{EW2Wall} \div L_{Wall} = 11.961\ plf$ Distributed Load from selfweight of the second story wall that sits on the first story wall

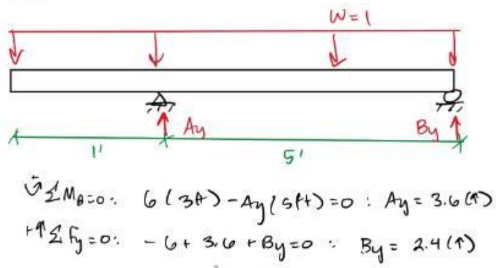
$Load := PL_{CH} + \left(DL + L_f\right) \cdot Trib_{area} + w_{EW} \cdot Trib_{width} = 368.153\ lbf$

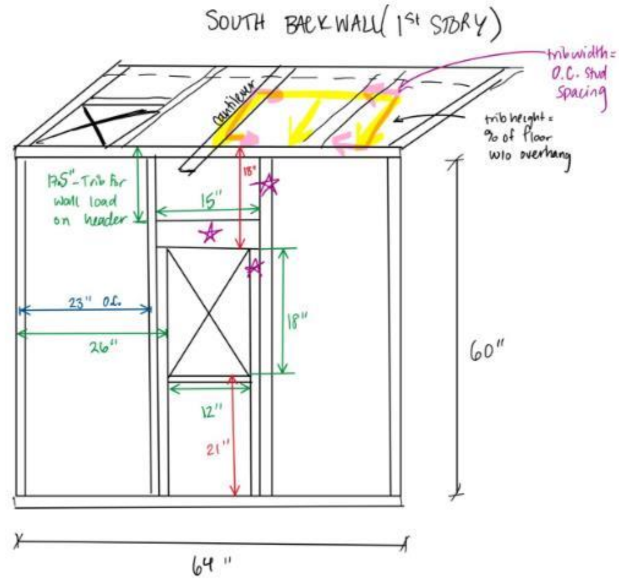
Compression Design

$d := \dfrac{Load}{b \cdot \left(f_{compParallel} \div C_P\right)} = 0.126\ in$ $\quad \text{if}\left(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}\right) = \text{“Good”}$

Braced in two directions (corner stud), so C_P=1.0 (divide out the old C_P)

First Story South/Back Wall





$3.6 \div (3.6 + 2.4) = 0.6$

60% of the non-cantilevered floor goes to the North (front) Wall tributary of the floor load

40% goes to the South (back) Wall

HEADER:

Dimensions

$L_{Floor} := 71\ in \quad L_{header} := 15\ in \quad Trib_{WALL} := 14.5\ in$ Tributary ht of the wall above the window header

Loads

$w_{header} := DL \cdot Trib_{WALL} + w_{stud} = 4.481\ plf$ Distributed Load on the header

Solve For

$Reaction_{Header} := (w_{header} \cdot L_{header}) \div 2 = 2.801\ lbf \quad V_{Max} := Reaction_{Header} = 2.801\ lbf$

$M_{Max} := (w_{header} \cdot L_{header}^{2}) \div 8 = 10.502\ lbf \cdot in$

Flexure Design

$d := \sqrt{(6 \cdot M_{Max}) \div (f_s \cdot b)} = 0.169\ in$ [NDS 3.3-2] $\text{if}(d \le 3.5\ in, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

Shear Design

$d := (3\ V_{Max}) \div (2\ b \cdot f_v) = 0.016\ in$ [NDS 3.4.2] $\text{if}(d \le 3.5\ in, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

TRIMMER STUDS:

Compression Design

$d := Reaction_{Header} \div (b \cdot f_{compParallel}) = 0.0011\ in \quad \text{if}(d \le 3.5\ in, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

KING STUD:

Checking one "king stud" because it has the largest tributary and is only braced in one direction

Dimensions

$Trib_{height} := (L_{Floor} - 12\ in) \cdot 0.4 = 23.6\ in \quad Trib_{width} := 19.75\ in$ On-center stud spacing

$Trib_{area} := Trib_{width} \cdot Trib_{height} = 466.1\ in^2 \quad L_{Wall} := 64\ in$

Loads

$PL_{Cant} := R_{C.S} = 429\ lbf$

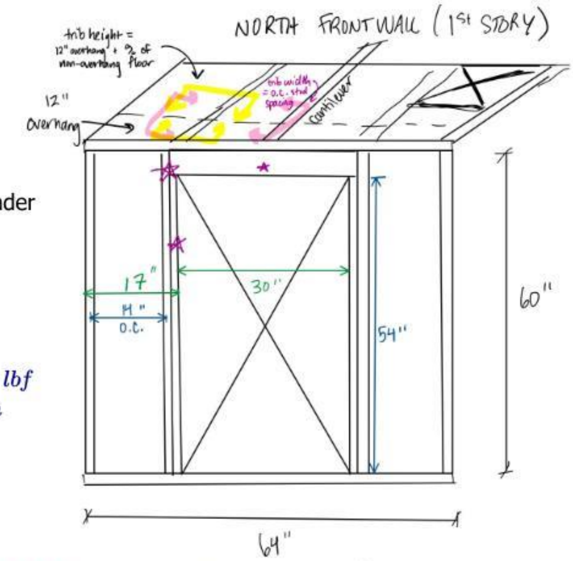
$Load := (DL + L_f) \cdot Trib_{area} + Trib_{width} \cdot w_S + (PL_{Cant} \div 2) = 404.81\ lbf$

Compression Design

$d := Load \div (b \cdot f_{compParallel}) = 0.152\ in \quad \text{if}(d \le 3.5\ in, \text{"Good"}, \text{"Bad"}) = \text{"Good"}$

Ac
be
stu

First Story North/ Front Wall



HEADER:

Dimensions

$L_{Floor} := 71\ in$

$L_{header} := 33\ in$

$Trib_{WALL} := 6\ in$

Tributary ht of the wall above the window header

Loads

$w_{header} := DL \cdot Trib_{WALL} + w_{stud} = 2.427\ plf$

Distributed Load on the header

Solve For

$Reaction_{Header} := (w_{header} \cdot L_{header}) \div 2 = 3.337\ lbf$

$M_{Max} := (w_{header} \cdot L_{header}^2) \div 8 = 27.528\ lbf \cdot in$

$V_{Max} := Reaction_{Header} = 3.337\ lbf$

Flexure Design

$d := \sqrt{(6 \cdot M_{Max}) \div (f_s \cdot b)} = 0.274\ in$ [NDS 3.3-2] $\text{if}(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}) = \text{“Good”}$

Shear Design

$d := (3\ V_{Max}) \div (2\ b \cdot f_v) = 0.019\ in$ [NDS 3.4.2] $\text{if}(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}) = \text{“Good”}$

TRIMMER STUDS:

Compression Design

$d := \dfrac{Reaction_{Header}}{b \cdot f_{compParallel}} = 0.0013\ in$ $\text{if}(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}) = \text{“Good”}$

KING STUD:

Checking one "king stud" because it has the largest tributary and is only braced in one direction

Dimensions

$Trib_{height} := 12\ in + (L_{Floor} - 12\ in) \cdot 0.60 = 47.4\ in$ $Trib_{width} := 24.5\ in$ On-center stud spacing

$Trib_{area} := Trib_{width} \cdot Trib_{height} = (1.161 \cdot 10^3)\ in^2$ $L_{Wall} := 64\ in$

Loads

$PL_{Cant} := R_{C.N} = 199\ lbf$

$Load := (DL + L_f) \cdot Trib_{area} + Trib_{width} \cdot w_N + (PL_{Cant} \div 2) = 547.147\ lbf$

Added half of the reaction from the cantilever point load on the king stud. This is conservative because it is not accounting for the exterior studs (just modeling the top plate between the king studs as a simply supported beam)

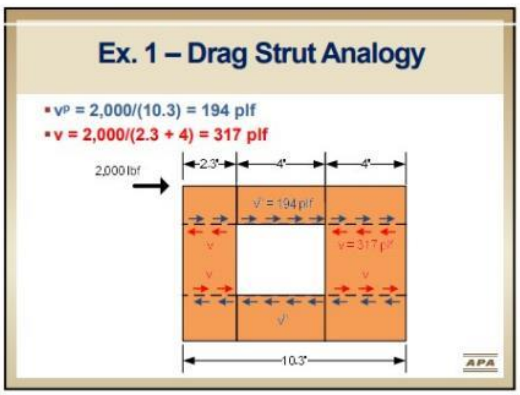
Compression Design

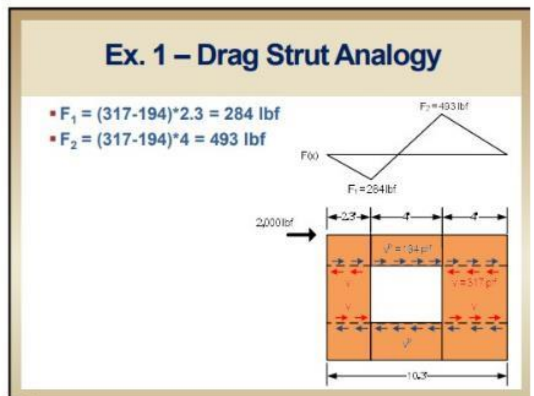
$d := Load \div (b \cdot f_{compParallel}) = 0.206\ in$ $\text{if}(d \le 3.5\ in, \text{“Good”}, \text{“Bad”}) = \text{“Good”}$

Gravity Wall Design Summary

-All members are 2x4's

LATERAL WALL DESIGN





Using Force Transfer Around Openings - Better Hardware Straps. 24" length minimum
Drag Strut Method (same result as APA Worksheet)
FTAO requires vertical strap at each end of the entire wall (not each shear wall end), and horizontal straps above and below the opening.

Lateral Wall Design

2nd Story South/Back Wall

Dimensions

$L := 64\ in$ $H := 48\ in$ $L_o := 12\ in$ $L_1 := 26\ in$ $L_2 := L_1 = 26\ in$

$SG := 0.43$

Loads

$V_{rEW} = 473.229\ lbf$ Shear from East-West Roof Diaphragm

$v_p := V_{rEW} \div L = 88.73\ plf$ $v := V_{rEW} \div (L_1 + L_2) = 109.207\ plf$

Capacity

$v_n := 560\ plf$ Nominal Unit Shear Capacity [SDPWS Table 4.3A] - sheathing, 3/8 thick, 6d nail, 6" spa

$v_{na} := v_n \cdot (1 - 0.5 \cdot SG) \div 2.8 = 157\ plf$ Adjusted Nominal Unit Shear Capacity

Factor of Safety: $FS_a := v_{na} \div v = 1.438$

Hardware

$T := V_{rEW} \cdot H \div L = 354.922\ lbf$ Overturning at ends of total wall - use for vertical straps

$F_1 := (v - v_p) \cdot L_1 = 44.365\ lbf$

$F_s := F_1 + (v_p \cdot L_o) = 133.096\ lbf$ Force carried by the horizontal strap between the side and middle "panels"

$F_a := 1235\ lbf$ Allowable force on a SST LSTA24 Light Strap Tie

if $(F_a > F_s$, "Horizontal Strap Design Good", "Bad"$)$ = "Horizontal Strap Design Good"

if $(F_a > T$, "Vertical Strap Design Good", "Bad"$)$ = "Vertical Strap Design Good"

2nd Story Sidewalls

Dimensions

$L := 59\ in$ $H := 48\ in$ $L_o := 11\ in$ $L_1 := 24\ in$ $L_2 := L_1 = 24\ in$

Loads

$V_{rNS} = 0.473\ kip$ Shear from North-South Roof Diaphragm

$v_p := V_{rNS} \div L = 96.25\ plf$ $v := V_{rNS} \div (L_1 + L_2) = 118.307\ plf$

Capacity

$v_n := 560\ plf$ Nominal Unit Shear Capacity [SDPWS Table 4.3A] - sheathing, 3/8 thick, 6d nail, 6" spa

$v_{na} := v_n \cdot (1 - 0.5 \cdot SG) \div 2.8 = 157\ plf$ Adjusted Nominal Unit Shear Capacity

Factor of Safety: $FS_b := v_{na} \div v = 1.327$

Hardware

$T := V_{rNS} \cdot H \div L = 385\ lbf$ Overturning at ends of total wall - use for vertical straps

$F_1 := (v - v_p) \cdot L_1 = 44.115\ lbf$

$F_s := F_1 + (v_p \cdot L_o) = 132.344\ lbf$ Force carried by the horizontal strap between the side and middle "panels"

$F_a := 1235\ lbf$ Allowable force on a SST LSTA24 Light Strap Tie

if $(F_a > F_s$, "Horizontal Strap Design Good", "Bad"$)$ = "Horizontal Strap Design Good"

if $(F_a > T$, "Vertical Strap Design Good", "Bad"$)$ = "Vertical Strap Design Good"

2nd Story North/Front Wall

Dimensions

$L := 64\ in$ $H := 48\ in$ $L_o := 16\ in$ $L_1 := 24\ in$ $L_2 := L_1 = 24\ in$

Loads

$V_{rEW} = 0.473\ kip$ Shear from East-West Roof Diaphragm

$v_p := V_{rEW} \div L = 88.73\ plf$ $v := V_{rEW} \div (L_1 + L_2) = 118.307\ plf$

Capacity

$v_n := 840\ plf$ Nominal Unit Shear Capacity [SDPWS Table 4.3A] - sheathing, 3/8 thick, 6d nail, 4" spa

$v_{na} := v_n \cdot (1 - 0.5 \cdot SG) \div 2.8 = 235.5\ plf$ Adjusted Nominal Unit Shear Capacity

Factor of Safety: $FS_c := v_{na} \div v = 1.991$

Hardware

$T := V_{rEW} \cdot H \div L = 354.922\ lbf$ Overturning at ends of total wall - use for vertical straps

$F_1 := (v - v_p) \cdot L_1 = 59.154\ lbf$

$F_s := F_1 + (v_p \cdot L_o) = 177.461\ lbf$ Force carried by the horizontal strap between the side and middle "panels"

$F_a := 1235\ lbf$ Allowable force on a SST LSTA24 Light Strap Tie

if $(F_a > F_s$, "Horizontal Strap Design Good", "Bad"$)$ = "Horizontal Strap Design Good"

if $(F_a > T$, "Vertical Strap Design Good", "Bad"$)$ = "Vertical Strap Design Good"

1st Story Sidewalls

Dimensions

$L := 59\ in$ $H := 60\ in$ $L_o := 11\ in$ $L_1 := 24\ in$ $L_2 := L_1 = 24\ in$

Loads

$V_{fNS} = 0.466\ kip$ Shear from North-South Floor Diaphragm

$v_p := V_{fNS} \div L = 94.767\ plf$ $v := V_{fNS} \div (L_1 + L_2) = 116.485\ plf$

Capacity

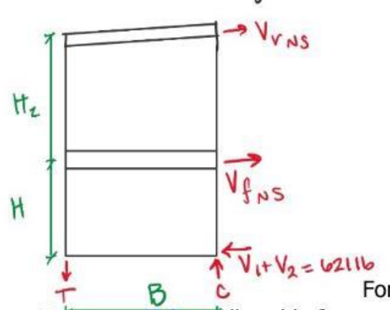
$v_n := 840\ plf$ Nominal Unit Shear Capacity [SDPWS Table 4.3A] - sheathing, 3/8 thick, 6d nail, 4" spa

$v_{na} := v_n \cdot (1 - 0.5 \cdot SG) \div 2.8 = 235.5\ plf$ Adjusted Nominal Unit Shear Capacity

Factor of Safety: $FS_d := v_{na} \div v = 2.022$

Hardware

Overturning:



$\circlearrowleft \Sigma M_c = 0$:

$T(B) - V_{fNS}(H) - V_{rNS}(H + H_2) = 0$

$T = \dfrac{V_{fNS}(H) + V_{rNS}(H + H_2)}{B}$

$H_2 := 48\ in$ $B := 5\ ft$

$T := ((V_{fNS} \cdot H) + (V_{rNS} \cdot (H + H_2))) \div B = 1.318\ kip$

Overturning at ends of total wall - use for anchor bolts

$F_1 := (v - v_p) \cdot L_1 = 43.435\ lbf$

$F_s := F_1 + (v_p \cdot L_o) = 130.305\ lbf$

Force carried by the horizontal strap between the side and middle "panels"

$F_a := 1235\ lbf$ Allowable force on a SST LSTA24 Light Strap Tie

$T_a := 3615\ lbf$ Allowable force on a SST STB2-50234R25 Concrete Anchor Bolt

if $(F_a > F_s$, "Horizontal Strap Design Good", "Bad"$)$ = "Horizontal Strap Design Good"

if $(T_a > T$, "Anchor Bolt Design Good", "Bad"$)$ = "Anchor Bolt Design Good"

1st Story South/Back Wall

Dimensions

$L := 71\ in$ $H := 60\ in$ $L_o := 12\ in$ $L_1 := 29.5\ in$ $L_2 := L_1 = 29.5\ in$

Loads

$V_{fEW} = 0.403\ kip$ Shear from East-West Floor Diaphragm

$v_p := V_{fEW} \div L = 68.113\ plf$ $v := V_{fEW} \div (L_1 + L_2) = 81.966\ plf$

Capacity

$v_n := 560\ plf$ Nominal Unit Shear Capacity [SDPWS Table 4.3A] - sheathing, 3/8 thick, 6d nail, 6" spa

$v_{na} := v_n \cdot (1 - 0.5 \cdot SG) \div 2.8 = 157\ plf$ Adjusted Nominal Unit Shear Capacity

Factor of Safety: $FS_e := v_{na} \div v = 1.915$

Hardware

$H_2 := 48\ in$ $B := 6\ ft$

$T := ((V_{fNS} \cdot H) + (V_{rNS} \cdot (H + H_2))) \div B = 1.098\ kip$ Overturning at ends of total wall - use for anchor bolts

$F_1 := (v - v_p) \cdot L_1 = 34.056\ lbf$

$F_s := F_1 + (v_p \cdot L_o) = 102.169\ lbf$ Force carried by the horizontal strap between the side and middle "panels"

$F_a := 1235\ lbf$ Allowable force on a SST LSTA24 Light Strap Tie

$T_a := 3615\ lbf$ Allowable force on a SST STB2-50234R25 Concrete Anchor Bolt

if $(F_a > F_s$, "Horizontal Strap Design Good", "Bad"$)$ = "Horizontal Strap Design Good"

if $(T_a > T$, "Anchor Bolt Design Good", "Bad"$)$ = "Anchor Bolt Design Good"

1st Story FrontWall

For Segmented Method: $h := 56.5\ in$ $b := 17\ in$ $h_1 := 3.5\ in$ $b_1 := 64\ in$

3.5 is Maximum Shear Wall Aspect Ratio for wood structual panels, blocked [SDPWS Table 4.3.3]

if $((h \div b) < 3.5$, "Can Use Segmented", "Use FTAO or Perforated"$)$ = "Can Use Segmented"

Loads

$V_{fNS} = 465.94\ lbf$ $M := V_{fNS} \cdot h_1 = 135.899\ lbf \cdot ft$

$T_1 := V_{fNS} \cdot h \div b = 1.549\ kip$ $T_2 := M \div b = 95.929\ lbf$

$T := T_1 + T_2 = 1.644\ kip$

$v := V_{fNS} \div b = 328.899\ plf$

Capacity

$v_n := 840\ plf$ Nominal Unit Shear Capacity [SDPWS Table 4.3A] - sheathing, 3/8 thick, 6d nail, 4" spa

$v_{na} := v_n \cdot (1 - 0.5 \cdot SG) \div 2.8 = 235.5\ plf$ Adjusted Nominal Unit Shear Capacity

Factor of Safety: $FS_f := v_{na} \div v = 0.716$

Hardware

$T = 1.644\ kip$ $T_a := 3615\ kip$ Allowable force on a SST STB2-50234R25 Concrete Anchor Bolt

if $(T_a > T$, "Anchor Bolt Design Good", "Bad"$)$ = "Anchor Bolt Design Good"

$sv_1 := V_{fNS} \div b_1 = 87.364\ plf$ $sv_2 := V_{fNS} \div (2 \cdot b) = 164.449\ plf$ $F_s := (sv_2 - sv_1) \cdot (2 \cdot b) = 218.409\ lbf$

$F_a := 1235\ lbf$ Allowable force on a SST LSTA24 Light Strap Tie

if $(F_a > F_s$, "Horizontal Strap Design Good", "Bad"$)$ = "Horizontal Strap Design Good"

$Average := (FS_a + FS_b + FS_c + FS_d + FS_e + FS_f) \div 6 = 1.568$

Lateral Wall Design Summary

-Sheathing is 3/8"
-Nails are 6d
-Nails are spaced at 6" along the shear wall edges for the 2nd story side walls and the 1st and 2nd story back (south) walls
-Nails are spaced at 4" along the shear wall edges for the 1st story side walls and the 1st and 2nd story front (north).
-SST LSTA24 used above and below all openings except for the 1st story door, and vertically at the end of each total wall between the 2nd and 1st stories
-STB2-50234R25 Anchor Bolt used at the end of each total wall on the 1st story, with one at the end of each individual shear wall on the front (north) wall.
-Total Shear Wall FS=1.568

APPENDIX C: CANTILEVER DEFLECTION AND DESIGN

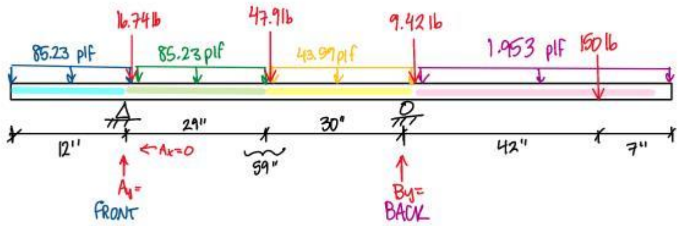
3'-6" Cantilever Deflection

OVERHANG (FRONT WALL)

CANTILEVER (BACK WALL)

OPENING

NON-OPENING



$\circlearrowleft^{+} \Sigma M_A = 0$: 85.23 plf (ft/12")(12")(6") − 85.23 plf (ft/12")(29")(14.5") − 47.9 lb (29")
− 43.59 plf (ft/12")(30")(40") − 9.42 lb (59") − 1.953 plf (ft/12")(49")(83.5")
− 150 lb (101") + By (59") = 0

By = 416.86 lb (↑)

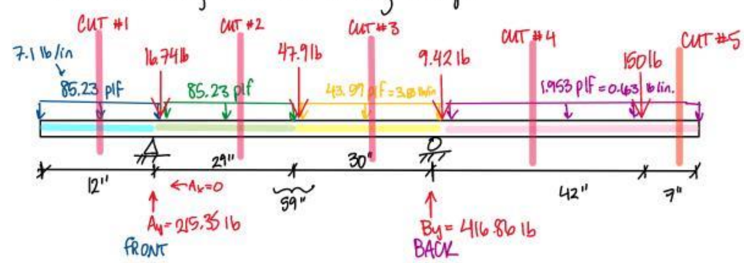
$+\uparrow \Sigma F_y = 0$: − 85.23 plf (12"/12") − 16.74 lb − 85.23 plf (29"/12") − 47.9 lb
− 43.59 plf (30"/12") − 9.42 lb − 1.953 plf (49"/12") − 150 lb + 416.86 lb + Ay = 0

Ay = 215.35 lb (↑)

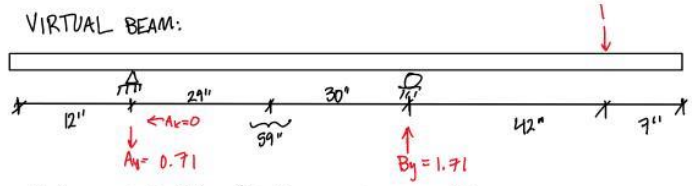
0.98 plf = 0.082 lb/in.

METHOD OF VIRTUAL WORK

• make cuts every time the loading changes:



VIRTUAL BEAM:

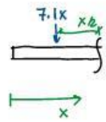


$\circlearrowleft \Sigma M_A = 0$: By (59") − 1 (101") = 0 : By = 1.71 (↑)

$+\uparrow \Sigma F_y = 0$: −Ay + 1.71 − 1 = 0 : Ay = 0.71 (↓)

CUT #1: $0 < x < 12"$

REAL

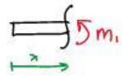


$\circlearrowleft \Sigma M_1 = 0:$

$M_1 + 7.1x(x/2) = 0$

$M_1 = -3.55x^2$

VIRTUAL

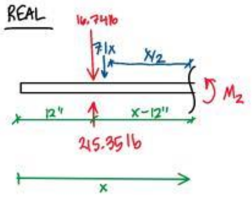


$\circlearrowleft \Sigma M_1 = 0:$

$m_1 = 0$

CUT #2: $12" < x < 41"$

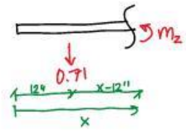
REAL



$\circlearrowleft \Sigma M_2 = 0: \ 7.1x\left(\frac{x}{2}\right) - 215.35(x-12) + 16.74(x-12) + M_2 = 0$

$M_2 = -3.55x^2 + 198.61x - 2383.32$

VIRTUAL



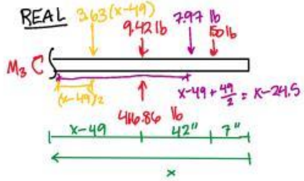
$\circlearrowleft \Sigma M_2 = 0:$

$m_2 + 0.71(x-12) = 0$

$m_2 = -0.71x + 8.52$

CUT #3: $49" < x < 79"$ (opposite direction)

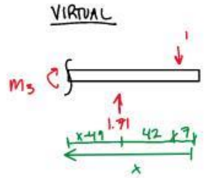
REAL



$\circlearrowleft \Sigma M_3 = 0: \ -M_3 - 3.63(x-49)\left(\frac{x-49}{2}\right) - 9.42(x-49) + 416.86(x-49) - 7.97(x-24.5) - 150(x-7) = 0$

$M_3 = -1.815x^2 + 427.34x - 23977.1$

VIRTUAL



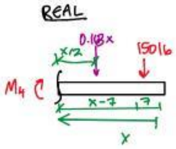
$\circlearrowleft \Sigma M_3 = 0:$

$-m_3 + 1.71(x-49) - 1(x-7) = 0$

$m_3 = 0.71x - 76.79$

CUT #4: $7" < x < 49"$ (opposite direction)

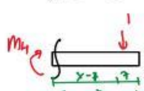
REAL



$\circlearrowleft \Sigma M_4 = 0: -0.163x(x/2) - 150(x-7) - M_4 = 0$

$M_4 = -0.0815x^2 - 150x + 1050$

VIRTUAL



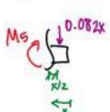
$\circlearrowleft \Sigma M_4 = 0:$

$-m_4 - 1(x-7) = 0$

$m_4 = -x + 7$

CUT #5: $0 < x < 7"$ (opp. direction)

REAL



$\circlearrowleft \Sigma M_5 = 0:$

$-M_5 - 0.082x\left(\frac{x}{2}\right) = 0$

$M_5 = -0.0815x^2$

VIRTUAL

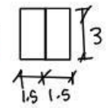
$m_5 = 0$

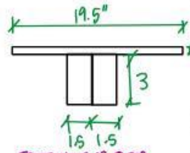
DEFLECTION

$E = 1{,}300{,}000$ psi

$$I_3 = \frac{bh^3}{12} = \frac{(1.5+1.5)(3.5)^3}{12} = 10.72 \text{ in}^4$$

↳ cantilever outside of floor





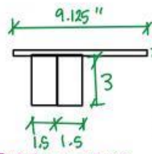
FRONT HALF OF FLOOR

$$\bar{y} = [(10.5 \text{ in}^2)(1.5 \text{ in}) + (6.7 \text{ in}^2)(3.17 \text{ in})] / [10.5 + 6.7]$$

$$\bar{y} = 2.15 \text{ in.}$$

$$I_1 = 10.72 \text{ in}^4 + (10.5 \text{ in}^2)(2.15-1.5)^2 + \frac{(19.5)(0.34375)^3}{12} + (6.9)(3.17-2.15)^2$$

$$I_1 = 22.19 \text{ in}^4$$



FLOOR NEXT TO OPENING

$$\bar{y} = [(10.5 \text{ in}^2)(1.5 \text{ in}) + (3.14 \text{ in}^2)(3.17 \text{ in})] / [10.5 + 3.14]$$

$$\bar{y} = 1.88 \text{ in.}$$

$$I_2 = 10.72 \text{ in}^4 + (10.5 \text{ in}^2)(1.88-1.5)^2 + \frac{(9.125)(0.34375)^3}{12} + (3.14)(3.17-1.88)^2$$

$$I_2 = 17.49 \text{ in}^4$$

$$I_4 = \frac{I_2(30) + I_3(49)}{30+49} = 13.29 \text{ in}^4$$

$$\Delta = \int \frac{m(x)M(x)}{EI}dx$$

$$\Delta = \int_0^{12} \frac{0}{EI}dx + \int_{12}^{41} \frac{(-3.95x^2 + 198.61x - 2363.32)(-0.71x + 8.52)}{EI_1}dx$$

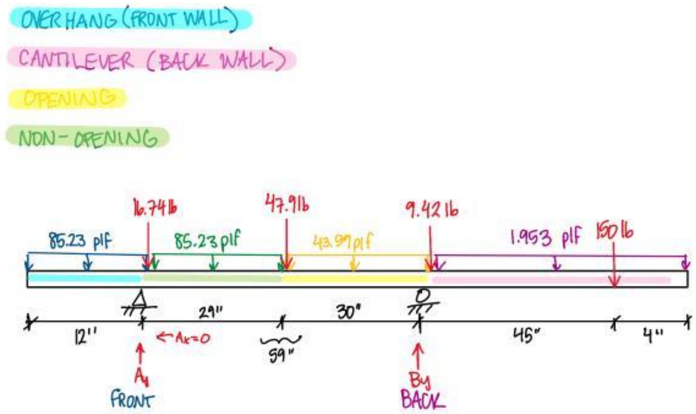
$$+ \int_{49}^{79} \frac{(-1.815x^2 + 427.34x - 23977.11)(0.71x - 76.79)}{EI_4}dx$$

$$+ \int_{7}^{49} \frac{(-0.0815x^2 - 150x + 650)(-x+7)}{EI_3}dx + \int_0^{7} \frac{0}{EI}dx$$

$\Delta = 0.51723$ in. $0.5 \text{ in} < \Delta < 1.0 \text{ in.}$ ✓

@3'-6" from exterior wall

3'-9" Cantilever Deflection



$\circlearrowleft^{+} \Sigma M_A = 0$: 85.23 plf (ft/12")(12")(6") − 85.23 plf (ft/12")(29")(14.5") − 47.9 lb (29")
− 43.59 plf (ft/12")(30")(40") − 9.42 lb (59") − 1.953 plf (ft/12")(49")(83.5")
− 150 lb (104") + B_y (59") = 0

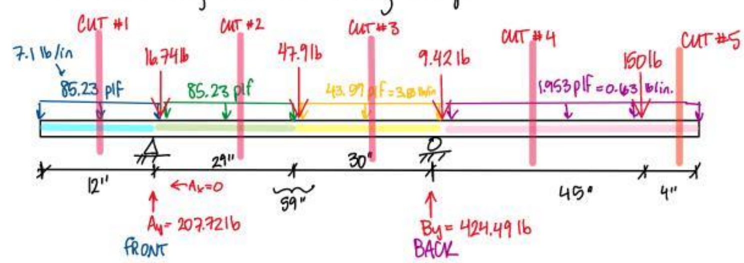
B_y = 424.49 lb (↑)

$+\uparrow \Sigma F_y = 0$: − 85.23 plf (12"/12") − 16.74 lb − 85.23 plf (29"/12") − 47.9 lb
− 43.59 plf (30"/12") − 9.42 lb − 1.953 plf (49"/12") − 150 lb + 424.49 lb + A_y = 0

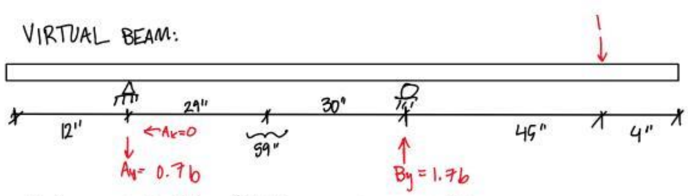
A_y = 207.72 lb (↑)

METHOD OF VIRTUAL WORK

• make cuts every time the loading changes:



VIRTUAL BEAM:

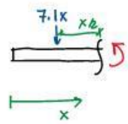


$\circlearrowleft^{+} \Sigma M_A = 0$: B_y (59") − 1 (104") = 0 : B_y = 1.76 (↑)

$+\uparrow \Sigma F_y = 0$: $-A_y$ + 1.76 − 1 = 0 : A_y = 0.76 (↓)

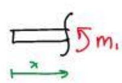
CUT #1: $0 < x < 12"$

REAL



$\circlearrowleft \sum M_1 = 0:$
$M_1 + 7.1x(x/2) = 0$
$M_1 = -3.55x^2$

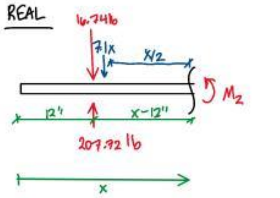
VIRTUAL



$\circlearrowleft \sum M_1 = 0:$
$m_1 = 0$

CUT #2: $12" < x < 41"$

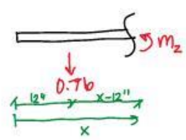
REAL



$\circlearrowleft \sum M_2 = 0: 7.1x\left(\frac{x}{2}\right) - 207.72(x-12) + 16.74(x-12) + M_2 = 0$

$M_2 = -3.55x^2 + 190.98x - 2291.76$

VIRTUAL

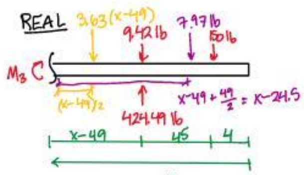


$\circlearrowleft \sum M_2 = 0:$
$m_2 + 0.76(x-12) = 0$

$m_2 = -0.76x + 9.12$

CUT #3: $49" < x < 79"$ (opposite direction)

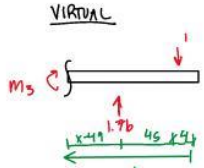
REAL



$\circlearrowleft \sum M_3 = 0: -M_3 - 3.63(x-49)\left(\frac{x-49}{2}\right) - 9.42(x-49) + 424.49(x-49) - 7.97(x-24.5) - 150(x-4) = 0$

$M_3 = -1.815x^2 + 434.97x - 24350.98$

VIRTUAL

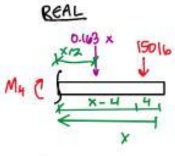


$\circlearrowleft \sum M_3 = 0:$
$-m_3 + 1.76(x-49) - 1(x-4) = 0$

$m_3 = 0.76x - 82.24$

CUT #4: $4" < x < 49"$ (opposite direction)

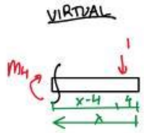
REAL



$\circlearrowleft \sum M_4 = 0: -0.163x(x/2) - 150(x-4) - M_4 = 0$

$M_4 = -0.0815x^2 - 150x + 600$

VIRTUAL



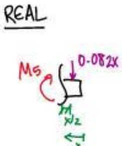
$\circlearrowleft \sum M_4 = 0:$

$-m_4 - 1(x-4) = 0$

$m_4 = -x + 4$

CUT #5: $0 < x < 4"$ (opp. direction)

REAL

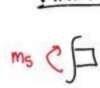


$\circlearrowleft \sum M_5 = 0:$

$-M_5 - 0.163x\left(\frac{x}{2}\right) = 0$

$M_5 = -0.082x^2$

VIRTUAL

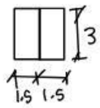


$m_5 = 0$

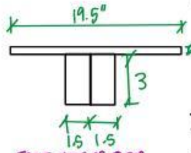
DEFLECTION

$E = 1{,}300{,}000$ psi

$I_3 = \frac{bh^3}{12} = \frac{(1.5+1.5)(3.5)^3}{12} = 10.72\ in^4$



↳ cantilever outside of floor



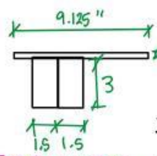
FRONT HALF OF FLOOR

$\bar{y} = [(10.5 in^2)(1.5 in) + (6.7 in^2)(3.17 in)] / [10.5 + 6.7]$

$\bar{y} = 2.15 in.$

$I_1 = 10.72\ in^4 + (10.5 in^2)(2.15-1.5)^2 + \frac{(19.5)(0.34375)^3}{12} + (6.7)(3.17-2.15)^2$

$I_1 = 22.19\ in^4$



FLOOR NEXT TO OPENING

$\bar{y} = [(10.5 in^2)(1.5 in) + (3.14 in^2)(3.17 in)] / [10.5 + 3.14]$

$\bar{y} = 1.88\ in.$

$I_2 = 10.72\ in^4 + (10.5 in^2)(1.88-1.5)^2 + \frac{(9.125)(0.34375)^3}{12} + (3.14)(3.17-1.88)^2$

$I_2 = 17.49\ in^4$

$$I_4 = \frac{I_2(30) + I_3(49)}{30+49} = 13.29\ in^4$$

$$\Delta = \int \frac{m(x)M(x)}{EI} dx$$

$$\Delta = \int_0^{12} \frac{0}{EI} dx + \int_{12}^{41} \frac{(-3.55x^2 + 190.98x - 2291.76)(-0.76x + 9.12)}{EI_1} dx$$

$$+ \int_{49}^{79} \frac{(-1.815x^2 + 434.97x - 24350.98)(0.76x - 82.24)}{EI_4} dx$$

$$+ \int_4^{49} \frac{(-0.0815x^2 - 150x + 600)(-x+4)}{EI_3} dx + \int_0^4 \frac{0}{EI} dx$$

$\Delta = 0.59274$ in. $\quad 0.5 in < \Delta < 1.0 in.$ ✓

@ 3'-9" from exterior wall

4’ Cantilever Deflection

CANTILEVER DESIGN

$w_{3.1} = Trib_{3.1}(SH + L_f) + w_{beam} = 0.04359$ kip/ft   dist load next to opening

$w_{3.2} = Trib_{3.2}(SH + L_f) + w_{beam} = 0.08523$ kip/ft   " non-opening length

$w_{3.3} = w_{beam}$ cantilever

$w_{3.4} = Trib_{3.2}(SH + L_f) + w_{beam} = 0.08523$ overhang floor

$PL_1 = w_s \cdot Trib_{3.1} = 0.00942$ PL from wt of 2nd story wall (south)

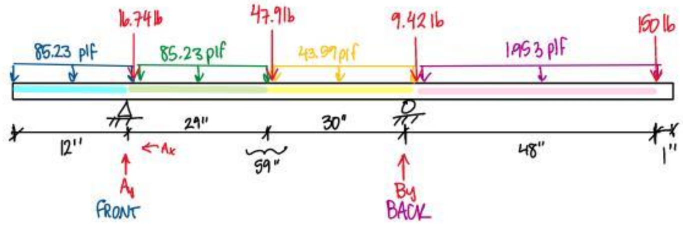
$PL_2 = R_6 = 0.0479$ PL on beam3 from beam6

$PL_3 = w_n \cdot Trib_{3.2} = 0.01674$ PL from wt of 2nd story wall (north)

$PL_4 = 0.15$ kip   PL applied to cant. beam

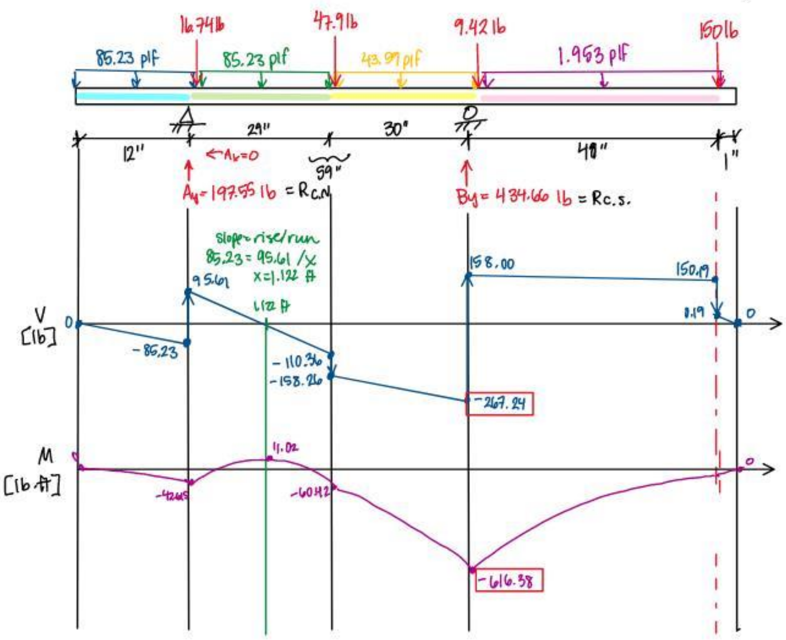
OVERHANG (FRONT WALL)   OPENING

CANTILEVER (BACK WALL)   NON-OPENING



$\circlearrowleft \sum M_A = 0$: 85.23 plf (ft/12")(12")(6") − 85.23 plf (ft/12")(29")(14.5") − 47.91b (29") − 43.59 plf (ft/12")(30")(40") − 9.42 lb (59") − 1.953 plf (ft/12")(49")(83.5") − 150 lb (108") + By (59") = 0   **By = 434.66 lb (↑)**

$+\uparrow \sum F_y = 0$: − 85.23 plf (12"/12") − 16.74 lb − 85.23 plf (29"/12") − 47.91b − 43.59 plf (30"/12") − 9.42 lb − 1.953 plf (49"/12") − 150 lb + 434.66 lb + Ay = 0   **Ay = 197.55 lb (↑)**



$V_{MAX} = 267.24$ lb $M_{MAX} = 616.38$ lb·ft

Flexure Design

$d = \sqrt{(6 \cdot M_{max}) \div f_b \cdot b} = 4.49$ in. ✓ ← less than 7" (two 2x4's)

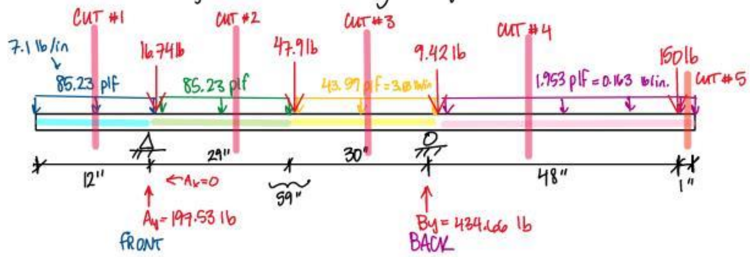
Shear Design

$d = (3 v_{max}) / (2 b \cdot f_v) = 1.48$ in. ✓

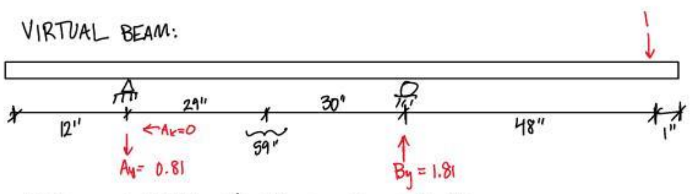
Design chose two 2x4's instead of one 2x6 because the deflection of the 2x6 would be too low (0.35") due to higher MOI.

METHOD OF VIRTUAL WORK

• make cuts every time the loading changes:



VIRTUAL BEAM:

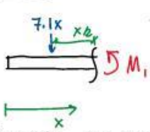


$\circlearrowleft \Sigma M_A = 0$: $B_y(59") - 1(107") = 0$: $B_y = 1.81$ (↑)

$+\uparrow \Sigma F_y = 0$: $-A_y + 1.81 - 1 = 0$: $A_y = 0.81$ (↓)

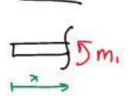
CUT #1: $0 < x < 12"$

REAL



$\circlearrowleft \Sigma M_1 = 0$:
$M_1 + 7.1x(x/2) = 0$
$M_1 = -3.55x^2$

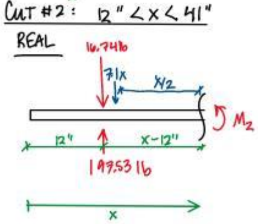
VIRTUAL



$\circlearrowleft \Sigma M_1 = 0$: $m_1 = 0$

CUT #2: $12" < x < 41"$

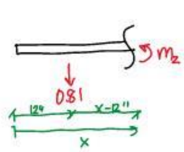
REAL



$\circlearrowleft \Sigma M_2 = 0$: $7.1x(\frac{x}{2}) - 199.53(x-12) + 16.74(x-12) + M_2 = 0$

$M_2 = -3.55x^2 + 180.79x - 2169.48$

VIRTUAL

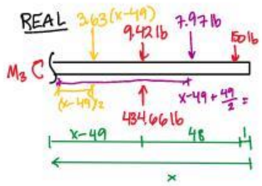


$\circlearrowleft \Sigma M_2 = 0$: $m_2 + 0.81(x-12) = 0$

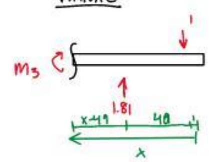
$m_2 = -0.81x + 9.72$

CUT #3: 49" < x < 79" (opposite direction)

REAL



VIRTUAL



$\circlearrowleft \Sigma M_3 = 0: -M_3 - 3.63(x-49)\left(\frac{x-49}{2}\right) - 9.42(x-49) + 434.66(x-49) - 7.97(x-24.5) - 150(\underbrace{x-49+48}_{x-1}) = 0$

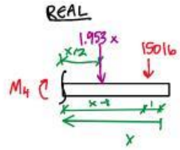
$M_3 = -1.815x^2 + 445.14x - 24849.31$

$\circlearrowleft \Sigma m_3 = 0: -m_3 + 1.81(x-49) - 1(x-1) = 0$

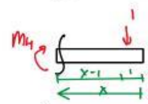
$m_3 = 0.81x - 87.69$

CUT #4: 1" < x < 49" (opposite direction)

REAL



VIRTUAL



$\circlearrowleft \Sigma M_4 = 0: -1.953x(x/2) - 150(x-1) - M_4 = 0$

$M_4 = -0.9765x^2 - 150x + 150$

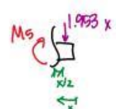
$\circlearrowleft \Sigma m_4 = 0:$

$-m_4 - 1(x-1) = 0$

$m_4 = -x + 1$

CUT #5: 0 < x < 1" (opp. direction)

REAL



$\circlearrowleft \Sigma M_5 = 0:$

$-M_5 - 1.953x\left(\frac{x}{2}\right) = 0$

$M_5 = -0.9765x^2$

VIRTUAL

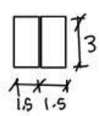
$m_5 = 0$

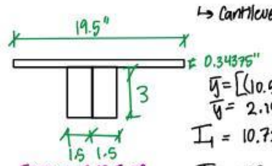
DEFLECTION

$E = 1{,}300{,}000$ psi

$I_3 = \frac{bh^3}{12} = \frac{(1.5+1.5)(3.5)^3}{12} = 10.72 \text{ in}^4$

↳ cantilever outside of floor





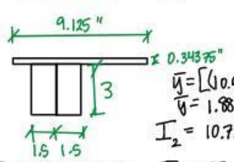
FRONT HALF OF FLOOR

$\bar{y} = [(10.5 \text{ in}^2)(1.5 \text{ in}) + (6.7 \text{ in}^2)(3.17 \text{ in})] / [10.5 + 6.7]$

$\bar{y} = 2.15 \text{ in.}$

$I_1 = 10.72 \text{ in}^4 + (10.5 \text{ in}^2)(2.15-1.5)^2 + \frac{(19.5)(0.34375)^3}{12} + (6.7)(3.17-2.15)^2$

$I_1 = 22.19 \text{ in}^4$



FLOOR NEXT TO OPENING

$\bar{y} = [(10.5 \text{ in}^2)(1.5 \text{ in}) + (3.14 \text{ in}^2)(3.17 \text{ in})] / [10.5 + 3.14]$

$\bar{y} = 1.88 \text{ in.}$

$I_2 = 10.72 \text{ in}^4 + (10.5 \text{ in}^2)(1.88-1.5)^2 + \frac{(9.125)(0.34375)^3}{12} + (3.14)(3.17-1.88)^2$

$I_2 = 17.49 \text{ in}^4$

$I_4 = \frac{I_2(30) + I_3(49)}{30+49} = 13.29 \text{ in}^4$

$$\Delta = \int \frac{m(x)M(x)}{EI} dx$$

$$\Delta = \int_0^{12} \frac{0}{EI} dx + \int_{12}^{41} \frac{(-3.55x^2 + 180.79x - 2169.48)(-0.81x + 9.72)}{EI_1} dx$$

$$+ \int_{49}^{79} \frac{(-1.815x^2 + 445.14x - 24849.31)(0.81x - 89.69)}{EI_4} dx$$

$$+ \int_1^{49} \frac{(-0.9765x^2 - 150x + 150)(-x+1)}{EI_3} dx + \int_0^1 \frac{0}{EI} dx$$

$\Delta = 0.77762$ in. 0.5 in. $< \Delta <$ 1.0 in. ✓

@ 4' from exterior wall

APPENDIX D: 2D STRUCTURAL DRAWINGS

NAU TIMBERJACKS

2024 TIMBERSTRONG DESIGN BUILD

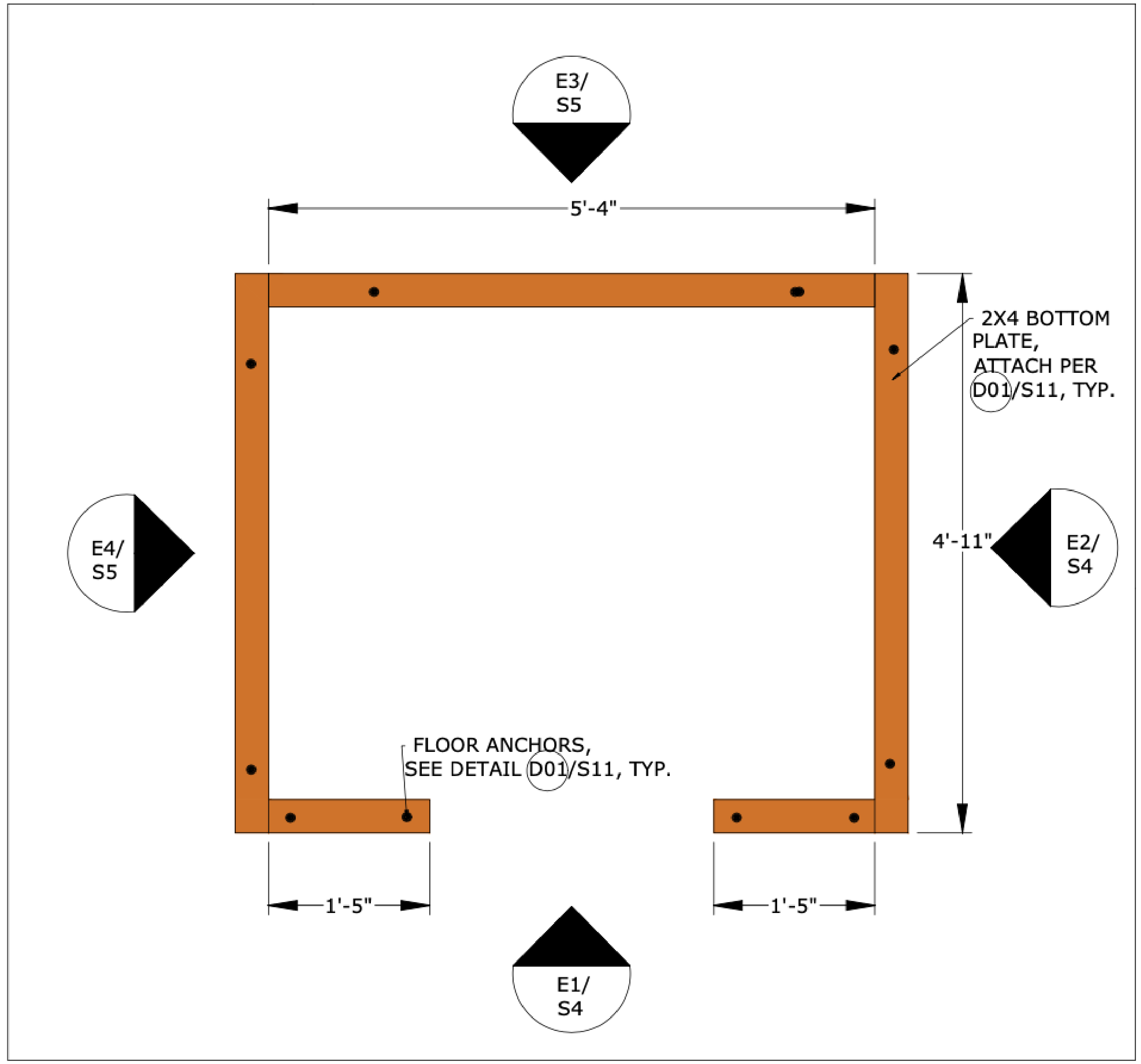
PONDEROSA TIMBERJACKS

Created By:
Mariah Boler
Mourtice Clitso
Jenna Hays

Submitted On:
February 23, 2024

PONDEROSA TIMBERJACKS	PROJECT: NAU TIMBERSTRONG	DATE: 02/23/2024	BY: PONDEROSA TIMBERJACKS	DRAWING NUMBER: S00
	COVER SHEET	DUE: 02/23/2024 REVISION: 0	0 N/A N/A N/A SCALE: N/A	PAGE: 1 OF: 14

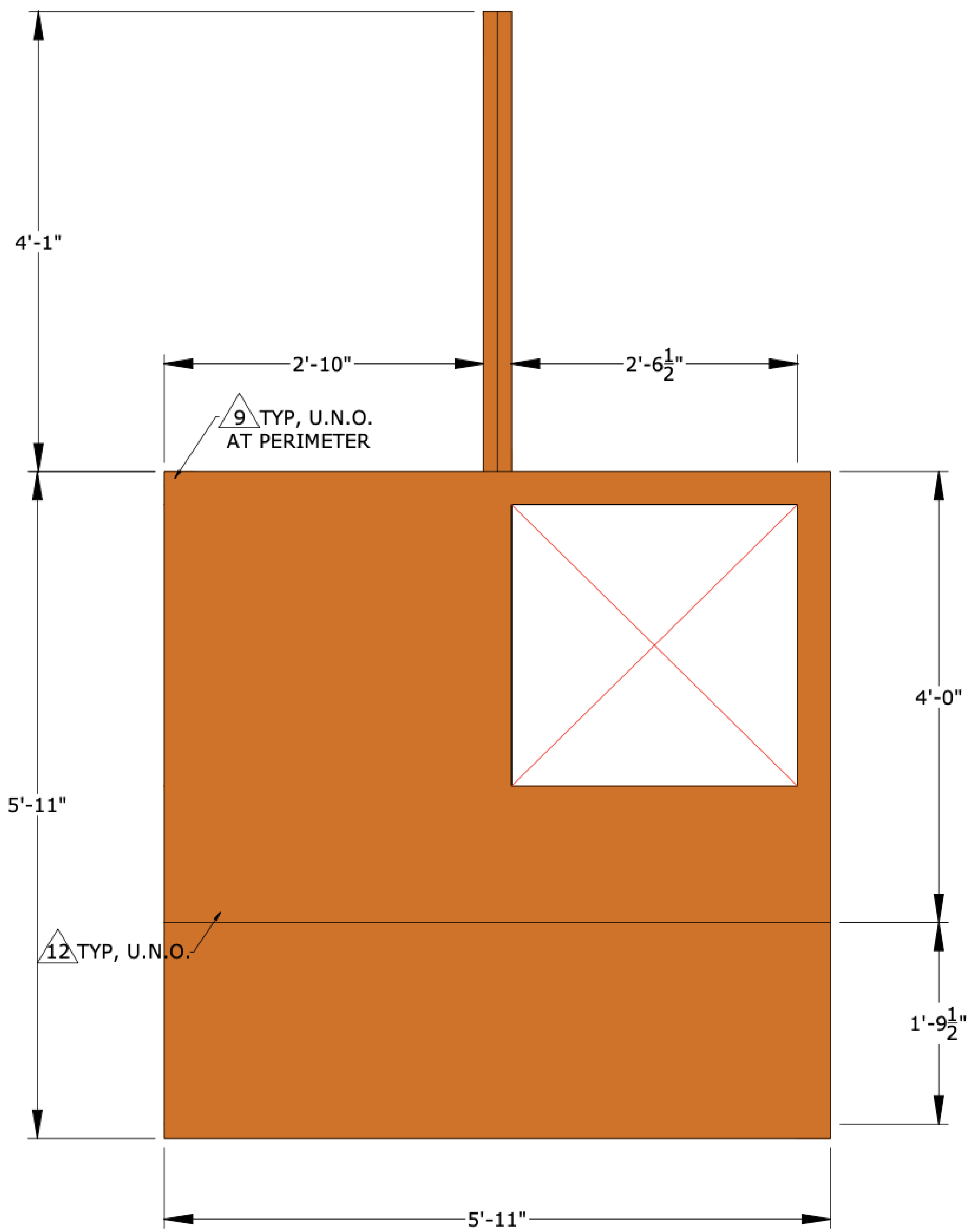
2024 TIMBERSTRONG COMPETITION FASTENER SCHEDULE FOR STRUCTURAL MEMBERS				
ITEM	DESCRIPTION OF BUILDING ELEMENTS	TYPE OF FASTENERS	NUMBER REQUIRED	SPACING
WALL				
1	TOP OR BOTTOM PLATE TO STUD, END NAIL	16d	3	–
2	BLOCKING BETWEEN STUDS, END NAIL	16d	3	–
3	ABUTTING STUDS AT DOORWAY, FACE NAIL	16d	–	6" OC
4	BUILT-UP HEADER, (2) PIECES	16d	–	4" OC
5	CONTINUOUS HEADER TO STUD, TOE NAIL	16d	2	–
6	DOUBLE TOP PLATE, FACE NAIL	16d	–	12" OC
7	ABUTTING STUDS AT INTERSECTIONS OF WALL CORNERS, FACE NAIL	16d	–	12" OC
8	SILL PLATE BETWEEN TRIMMER STUDS, END NAIL	16d	3	–
9	SHEATHING TO WALL PANELS, FACE NAIL	10d	–	4" OC U.N.O
FLOOR				
10	JOIST TO RIM JOIST, END NAIL	16d	3	–
11	BLOCKING BETWEEN JOISTS, END NAIL	16d	3	–
12	SUBFLOOR TO JOIST, FACE NAIL	10d	–	6" OC
13	RIM JOIST TO FIRST FLOOR, TOE NAIL	16d	–	6" OC
14	FLOOR CANTILEVER BEAMS, FACE NAIL	16d	–	4" OC
15	SECOND STORY TO FIRST FLOOR, FACE NAIL	16d	–	6" OC
ROOF				
16	RAFTER TO TOP PLATE, TOE NAIL	16d	3	–
17	RAFTER TO RIDGE BEAM, TOE NAIL	16d	3	–
18	STRUCTURAL SUPPORT TO TOP PLATE OR RIDGE BEAM, TOE NAIL	16d	3	–
19	RIDGE BEAM COMPONENTS, FACE NAIL	16d	–	6" OC
20	FACIA TO RAFTER, FACE NAIL	16d	3	–
21	STRUCTURAL SUPPORT COMPONENTS, FACE NAIL	16d	–	6" OC
22	SHEATHING TO ROOF PANELS, FACE NAIL	10d	–	6" OC



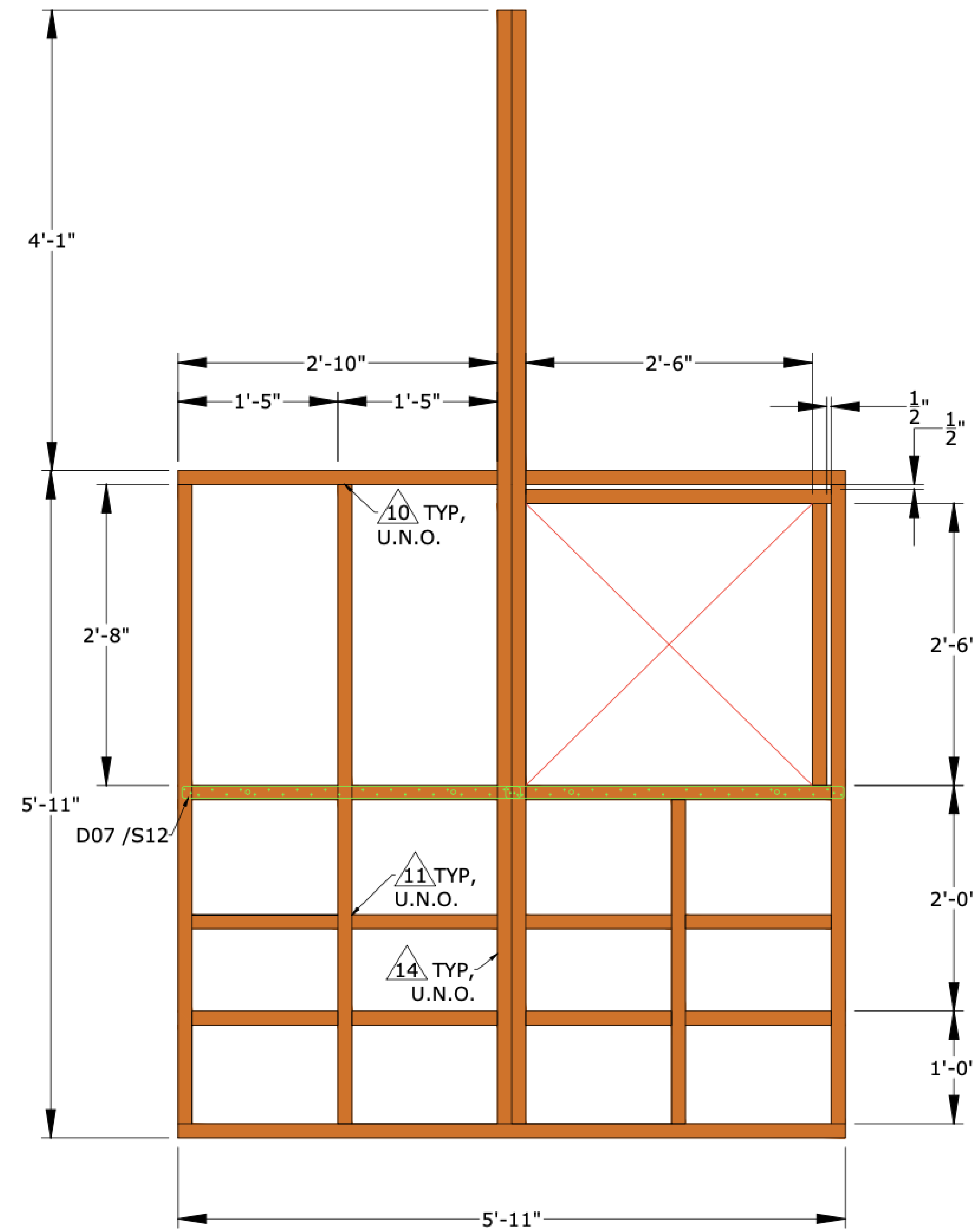
FOUNDATION PLAN

PONDEROSA TIMBERJACKS	PROJECT: NAU TIMBERSTRONG	DATE: 02/23/2024	BY: PONDEROSA TIMBERJACKS	DRAWING NUMBER: S1
	FOUNDATION ANCHORAGE	DUE: 02/23/2024 REVISION: 0	SCALE: 1-1/2"=1'-0"	PAGE: 3 OF: 14

△ KEYNOTE, SEE S0



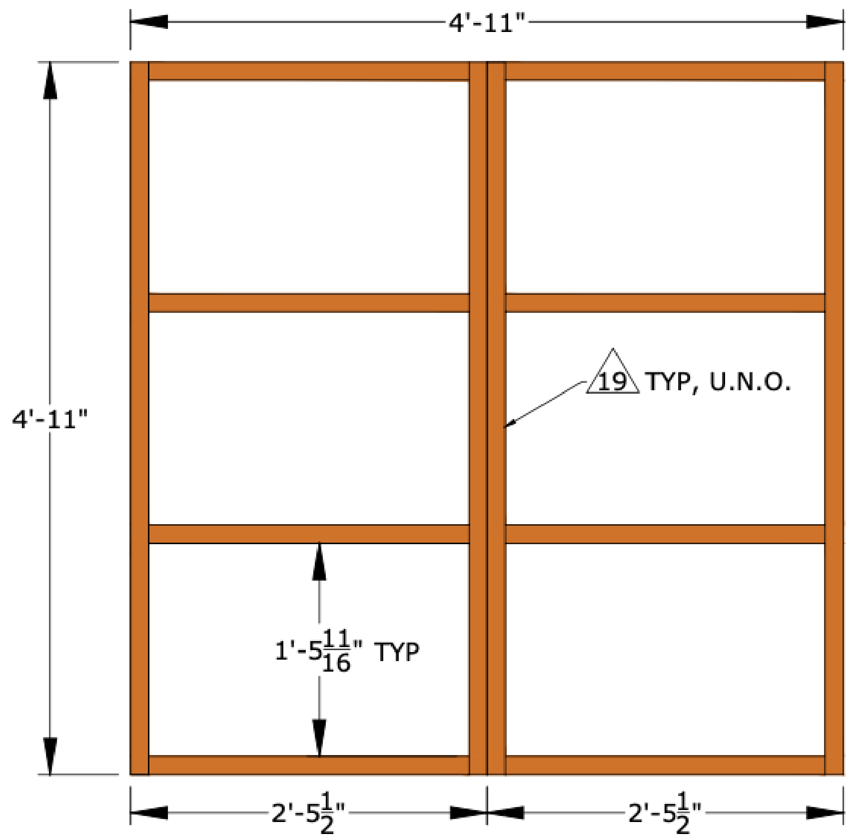
FIRST FLOOR SHEATHING PLAN



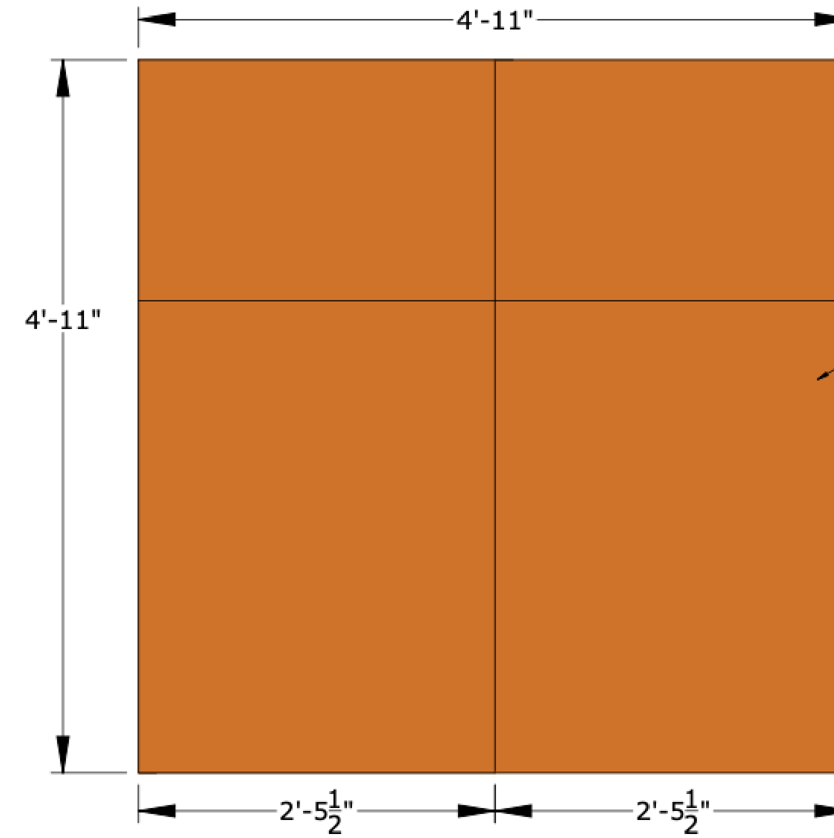
FIRST FLOOR FRAMING PLAN

PONDEROSA TIMBERJACKS	PROJECT: NAU TIMBERSTRONG	DATE: 02/23/2024	BY: PONDEROSA TIMBERJACKS	DRAWING NUMBER: S2
	FLOOR PLAN	DUE: 02/23/2024 REVISION: 0	0 3 6 12 SCALE: 1-1/2"=1'-0"	PAGE: 4 OF: 14

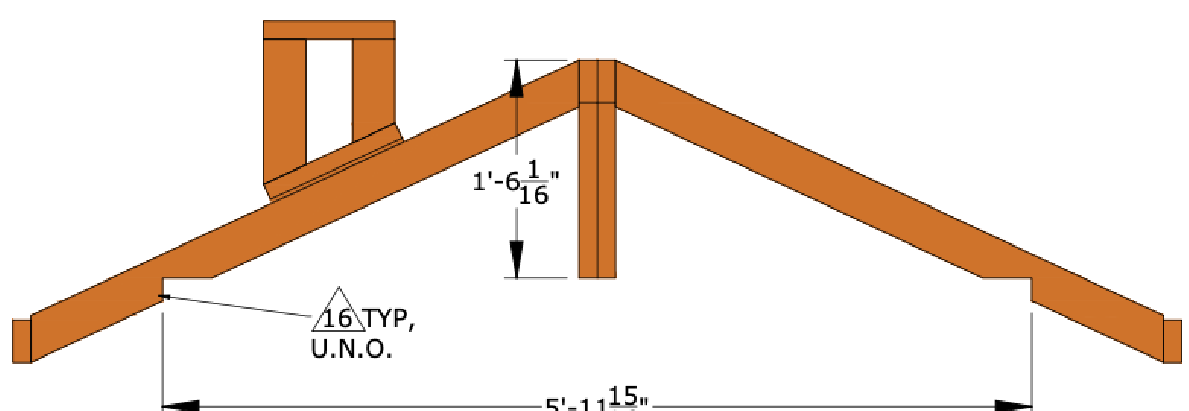
KEYNOTE, SEE S0



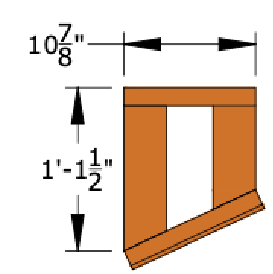
ROOF FRAMING PLAN



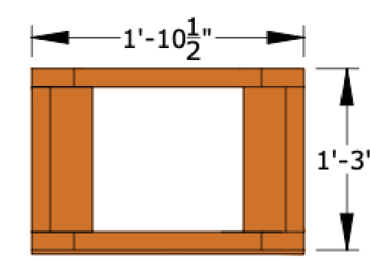
ROOF SHEATHING PLAN



ROOF PANEL



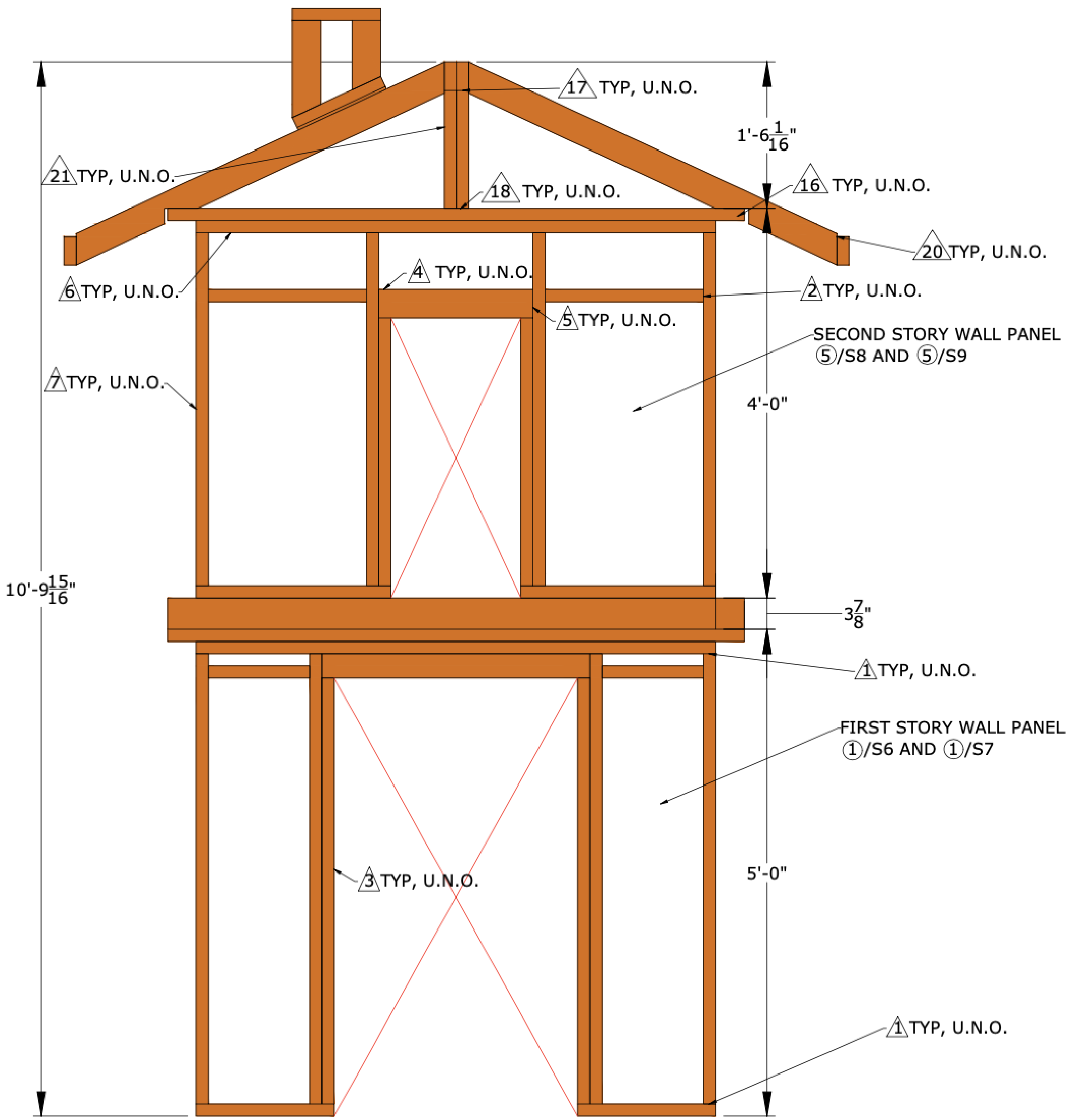
CHIMNEY NORTH FACE



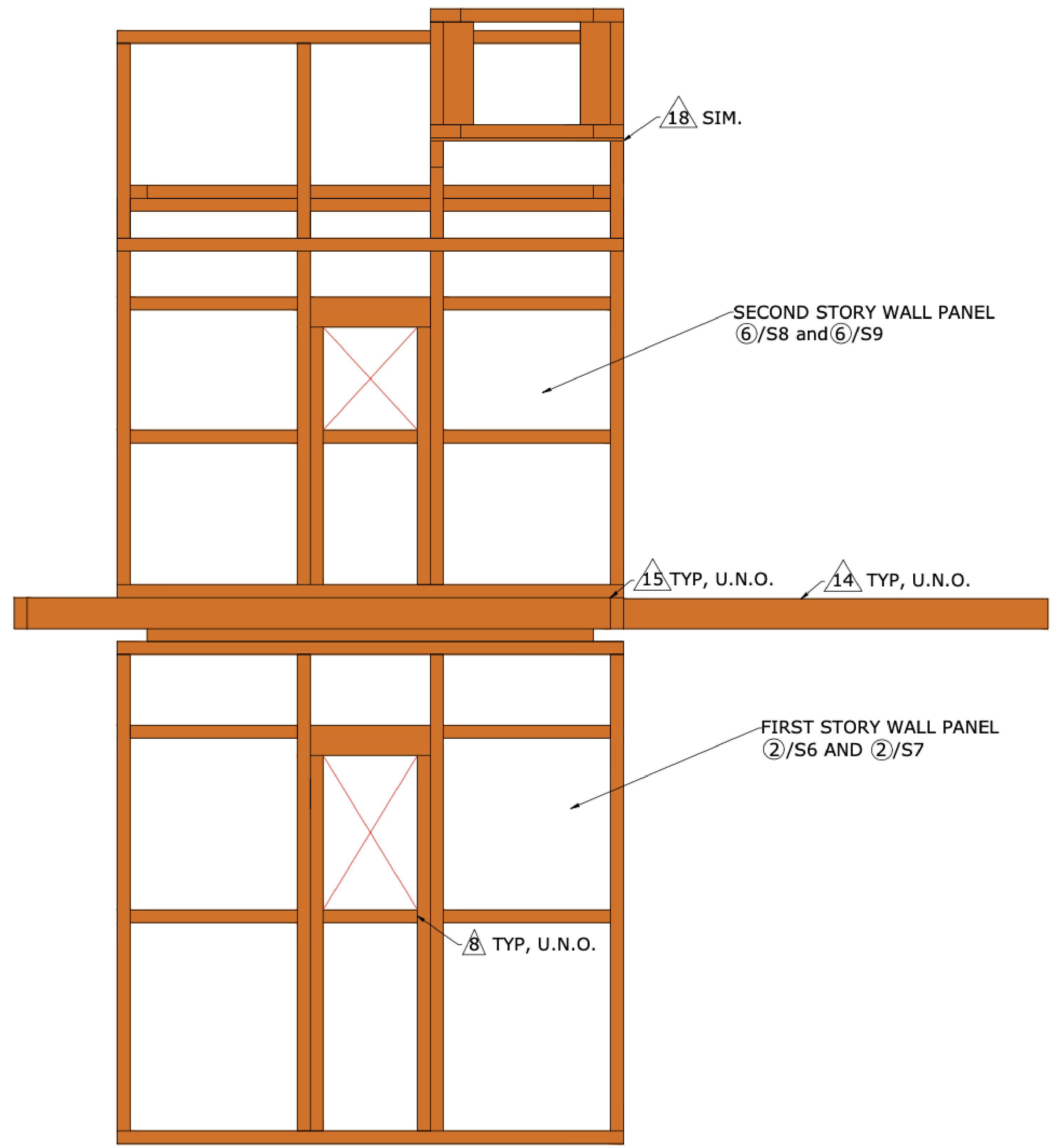
CHIMNEY EAST FACE

PONDERSOSA TIMBERJACKS	PROJECT: NAU TIMBERSTRONG	DATE: 02/23/2024	BY: PONDEROSA TIMBERJACKS	DRAWING NUMBER: S3
	ROOF PLAN	DUE: 02/23/2024 / REVISION: 0	SCALE: 1-1/12"=1'-0"	PAGE: 5 OF: 14

△ KEYNOTE, SEE S0



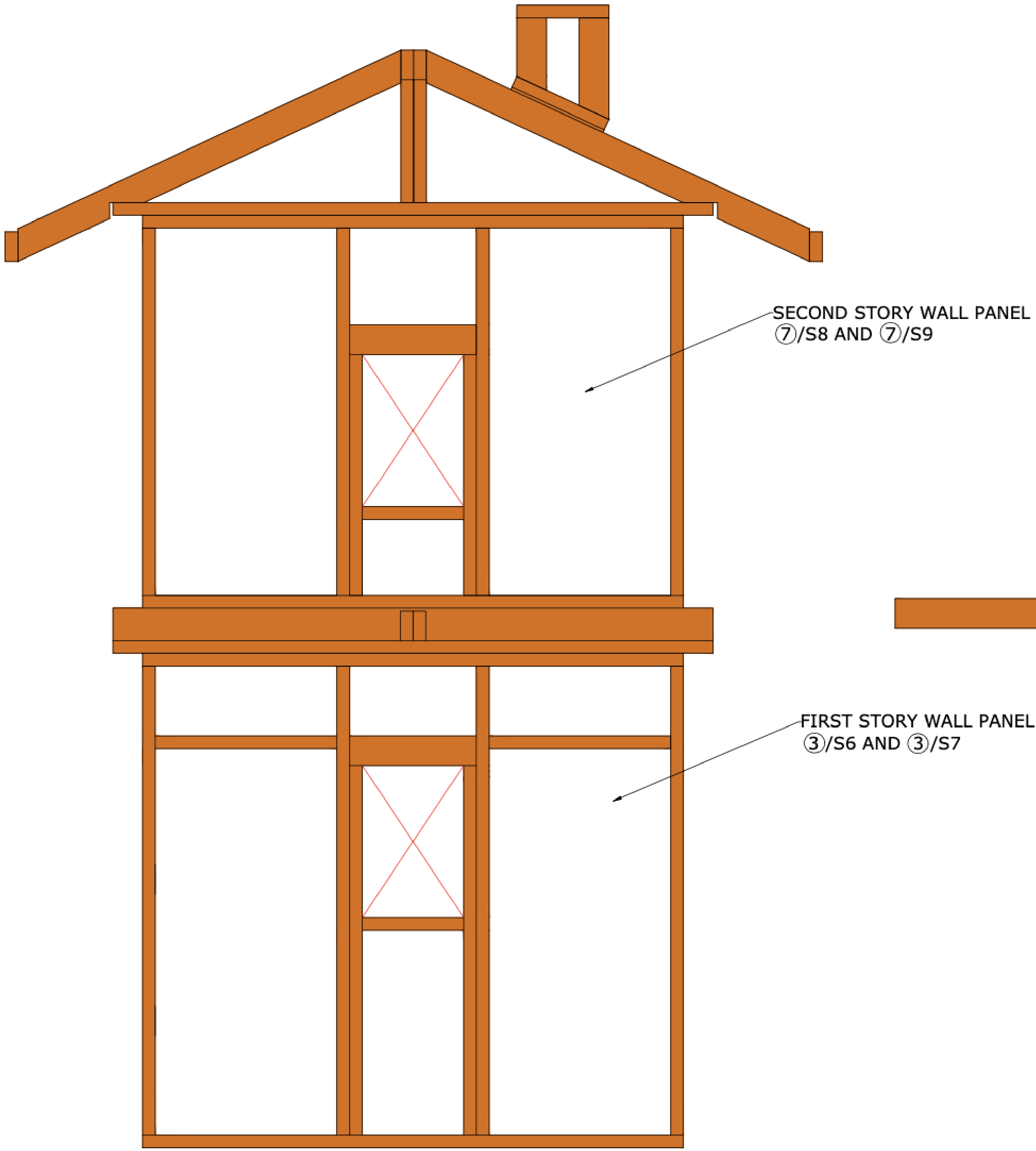
E1: ELEVATION NORTH FACE



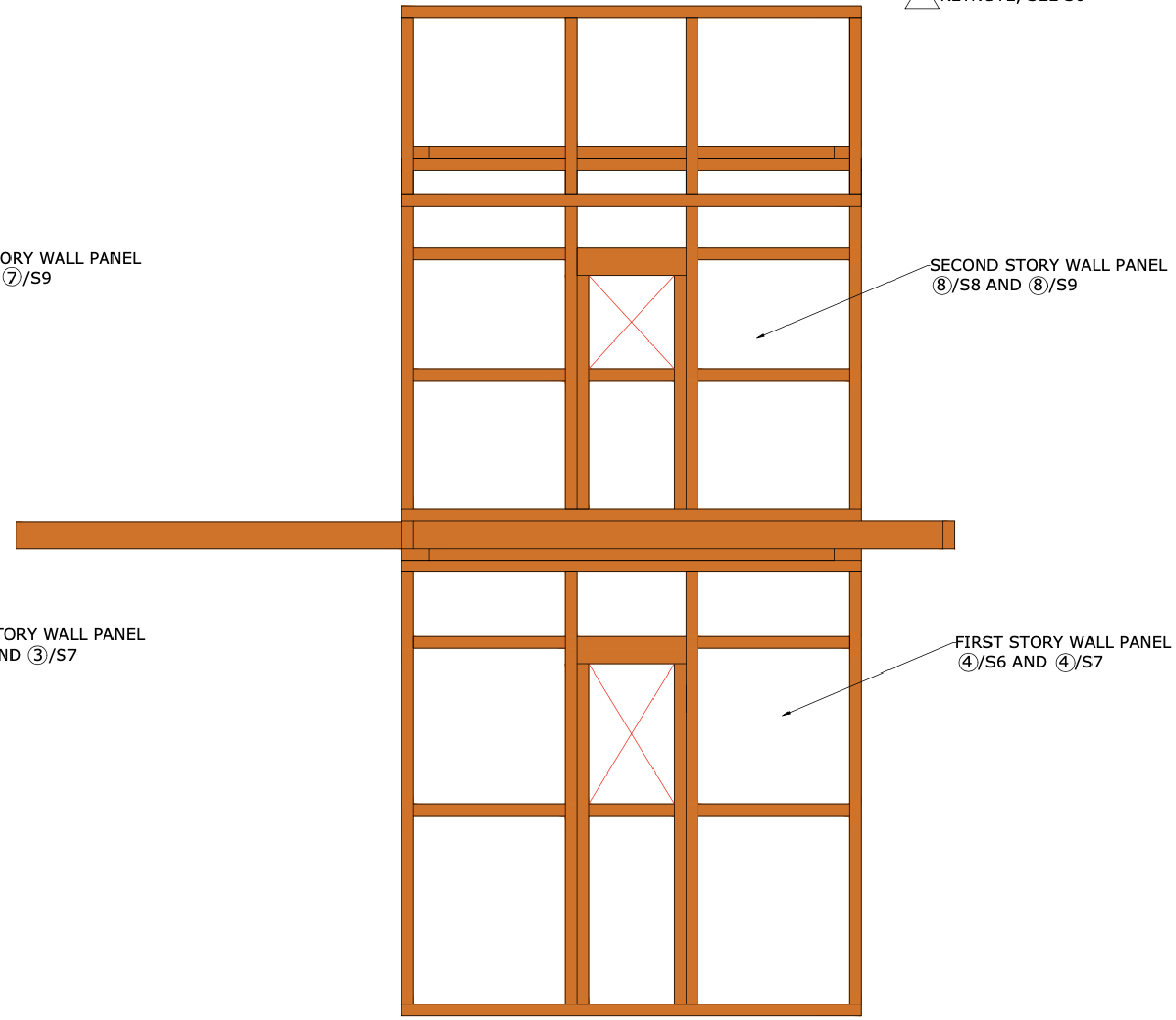
E2: ELEVATION EAST FACE

PONDEROSA TIMBERJACKS	PROJECT: NAU TIMBERSTRONG	DATE: 02/23/2024	BY: PONDEROSA TIMBERJACKS	DRAWING NUMBER: S4
	NORTH AND EAST ELEVATIONS	DUE: 02/23/2024 REVISION: 0	0 3 6 12 SCALE: 1-1/2"=1'-0"	PAGE: 8 OF: 14

△ KEYNOTE, SEE S0

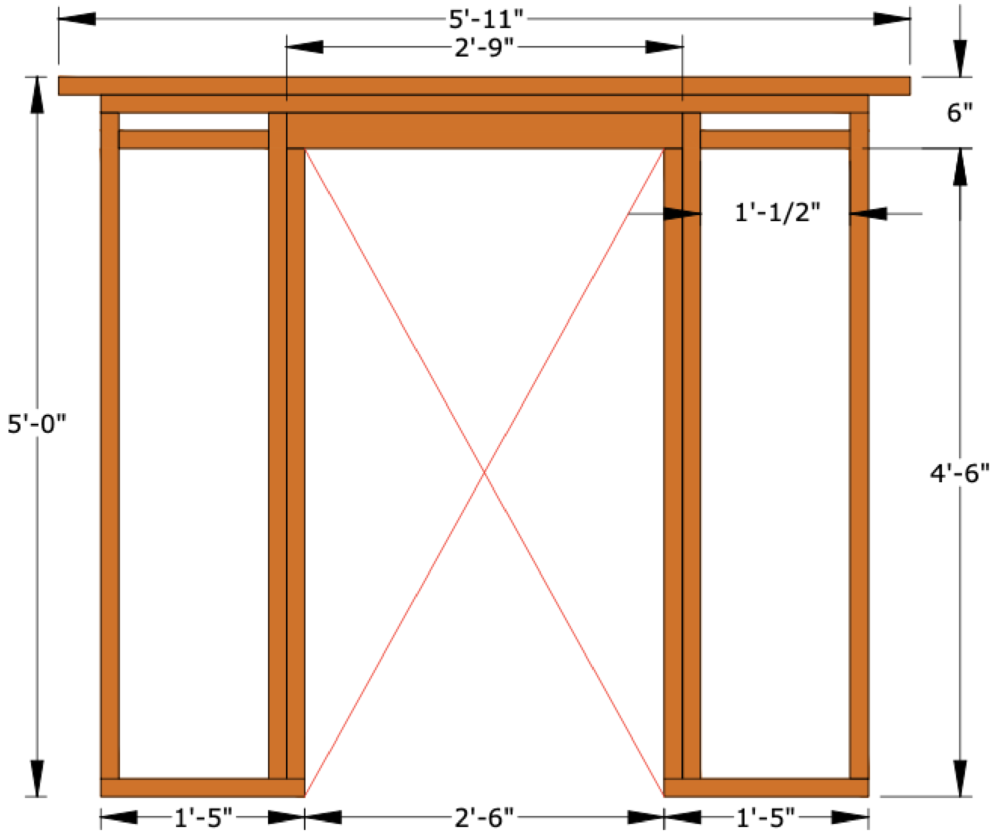


E3: ELEVATION SOUTH FACE

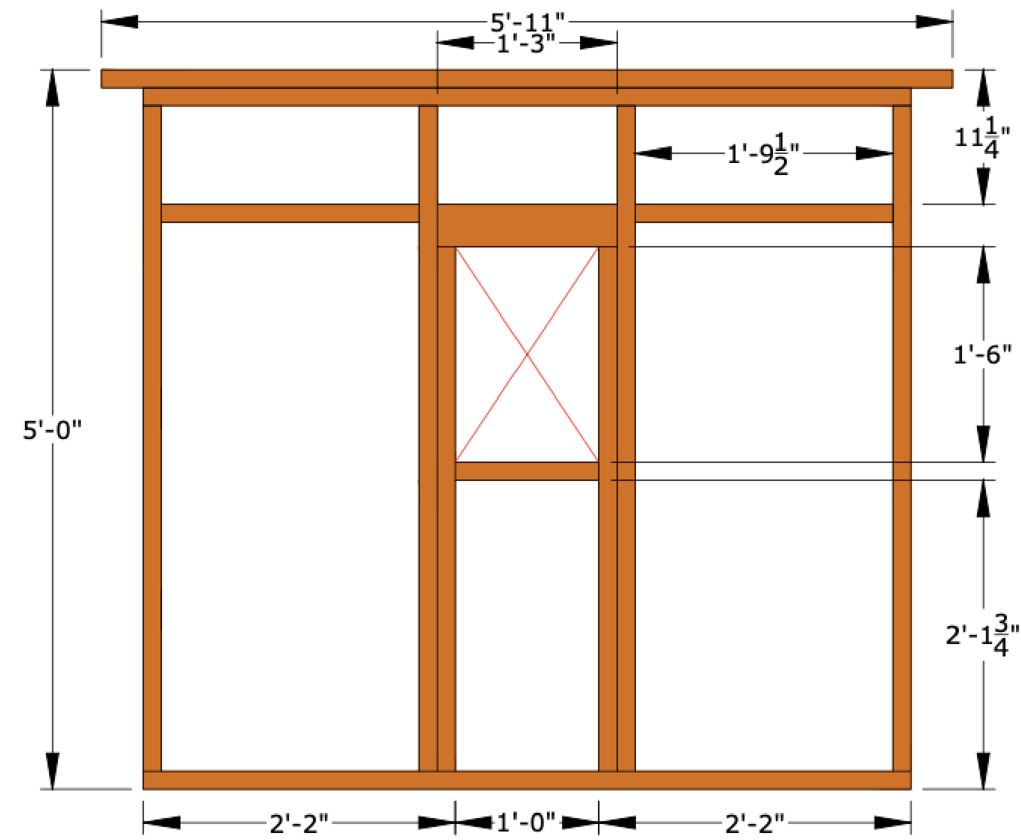


E4: ELEVATION WEST FACE

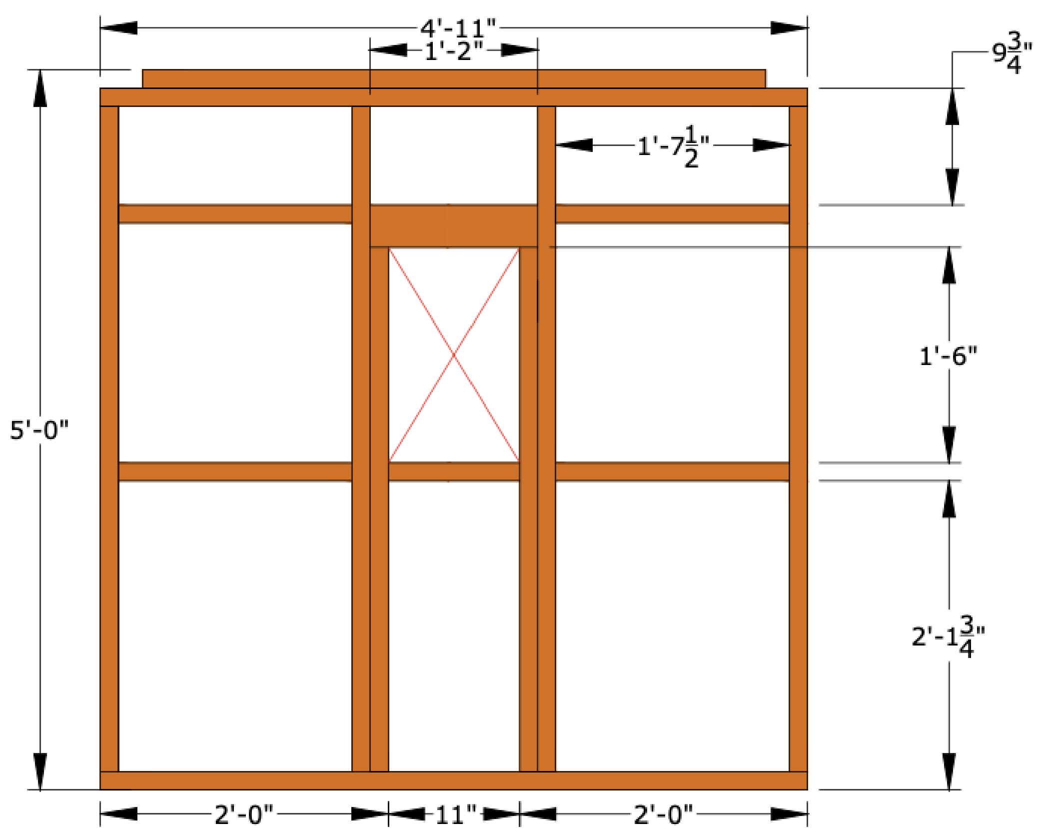
PONDEROSA TIMBERJACKS	PROJECT: NAU TIMBERSTRONG	DATE: 02/23/2024	BY: PONDEROSA TIMBERJACKS	DRAWING NUMBER: S5
	SOUTH AND WEST ELEVATIONS	DUE: 02/23/2024 REVISION: 0	SCALE: 1-1/2"=1'-0"	PAGE: 7 OF: 14



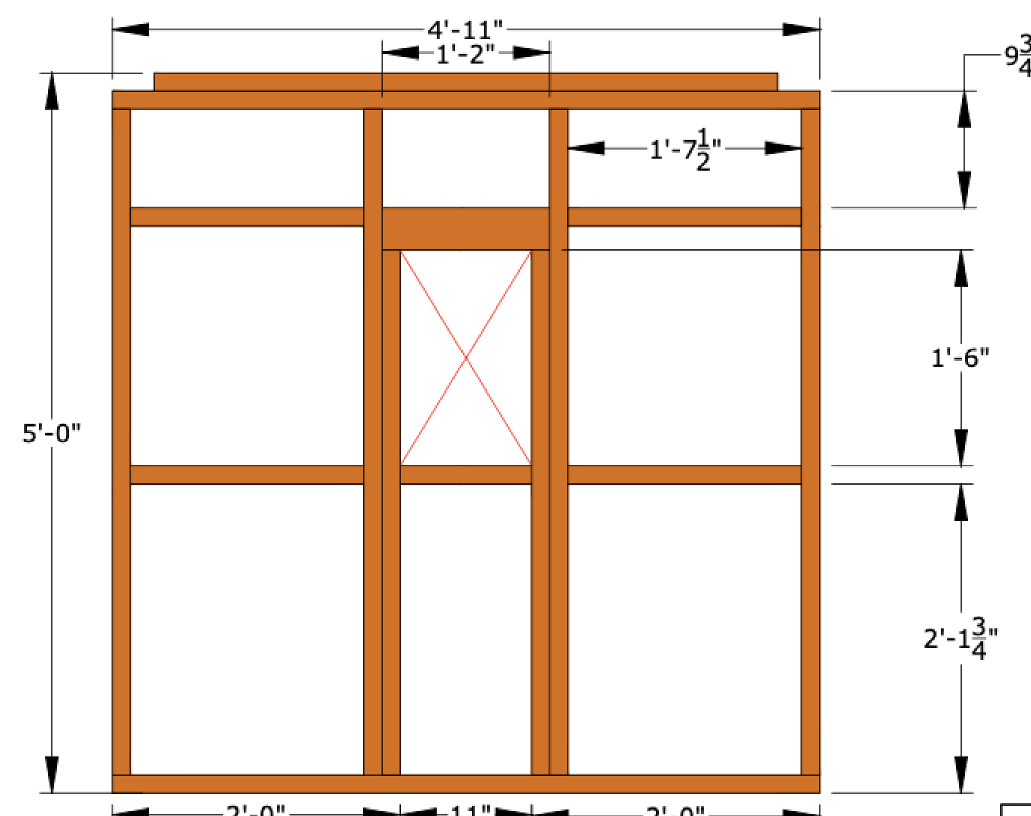
①: FIRST STORY NORTH FACE



③: FIRST STORY SOUTH FACE

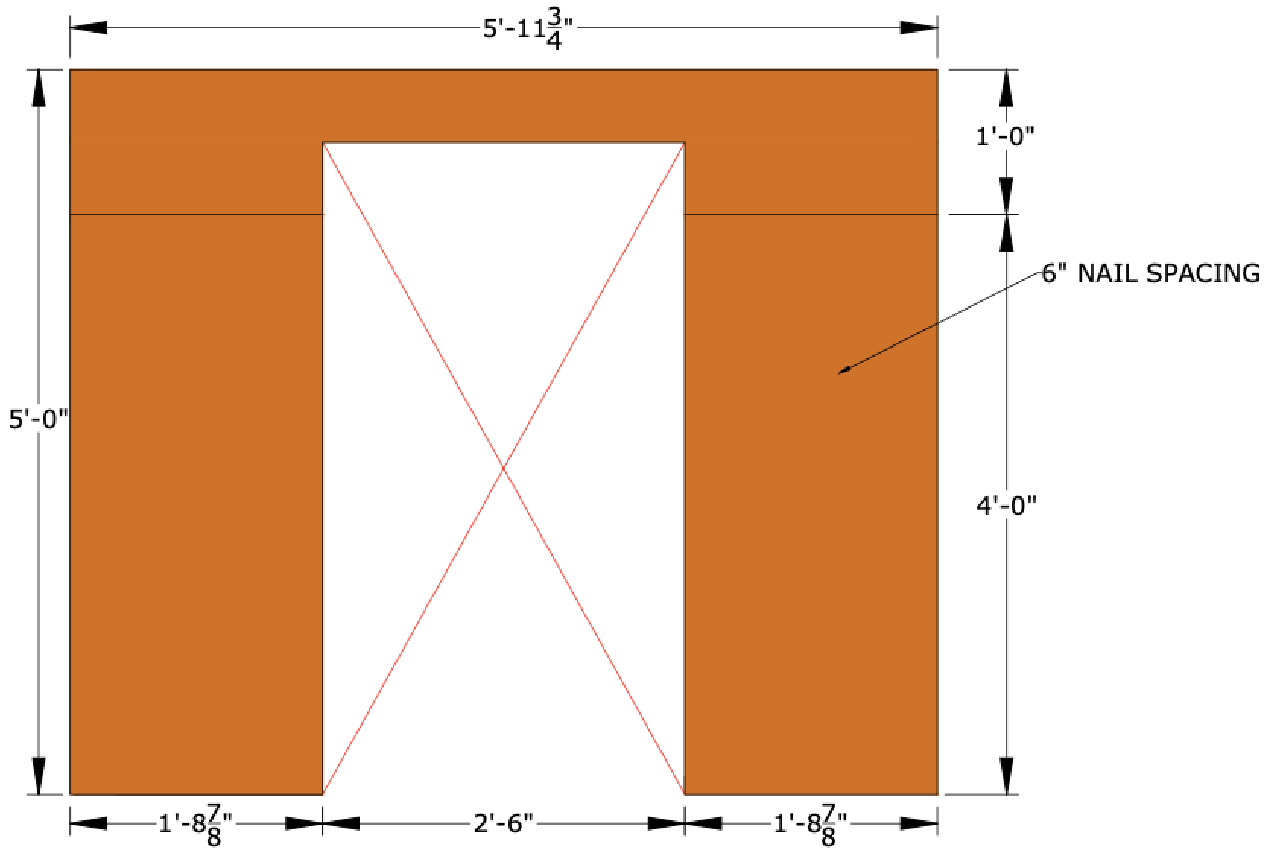


②: FIRST STORY EAST FACE

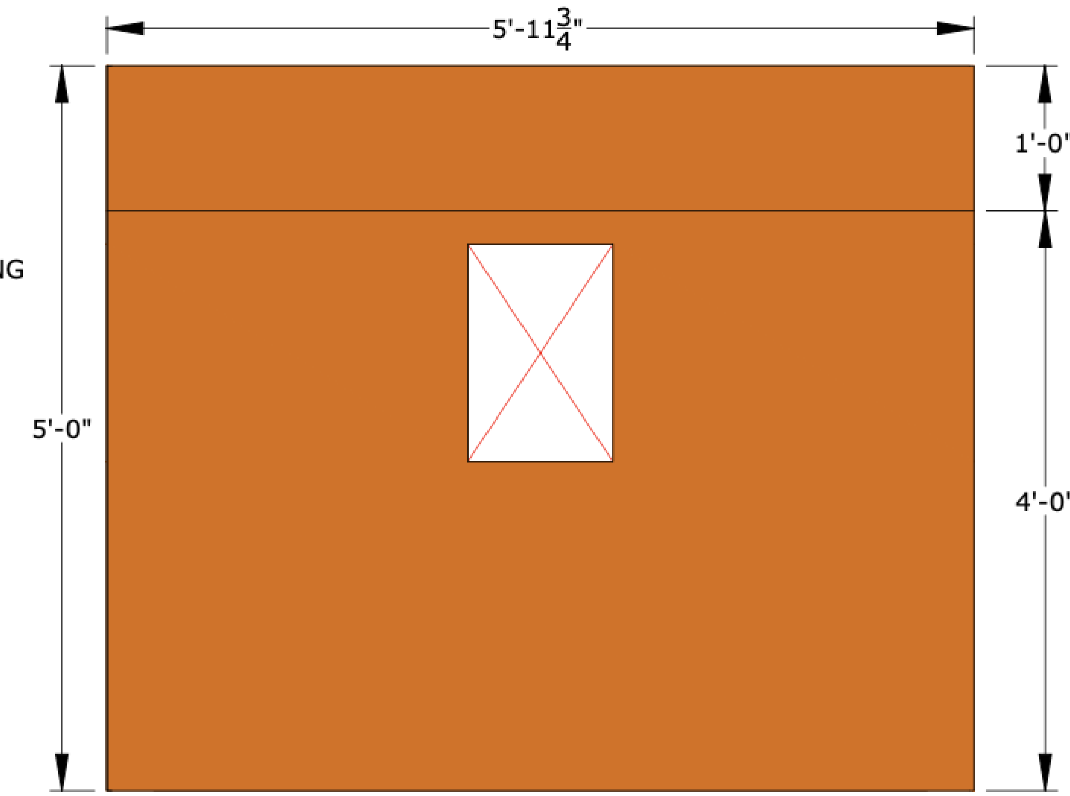


④: FIRST STORY WEST FACE

PONDEROSA TIMBERJACKS	PROJECT: NAU TIMBERSTRONG	DATE: 02/23/2024	BY: PONDEROSA TIMBERJACKS	DRAWING NUMBER: S6
	FIRST STORY FRAMING PLAN	DUE: 02/23/2024 REVISION: 0	0 3 6 12 SCALE: 1-1/2"=1'-0"	PAGE: 8 OF: 14



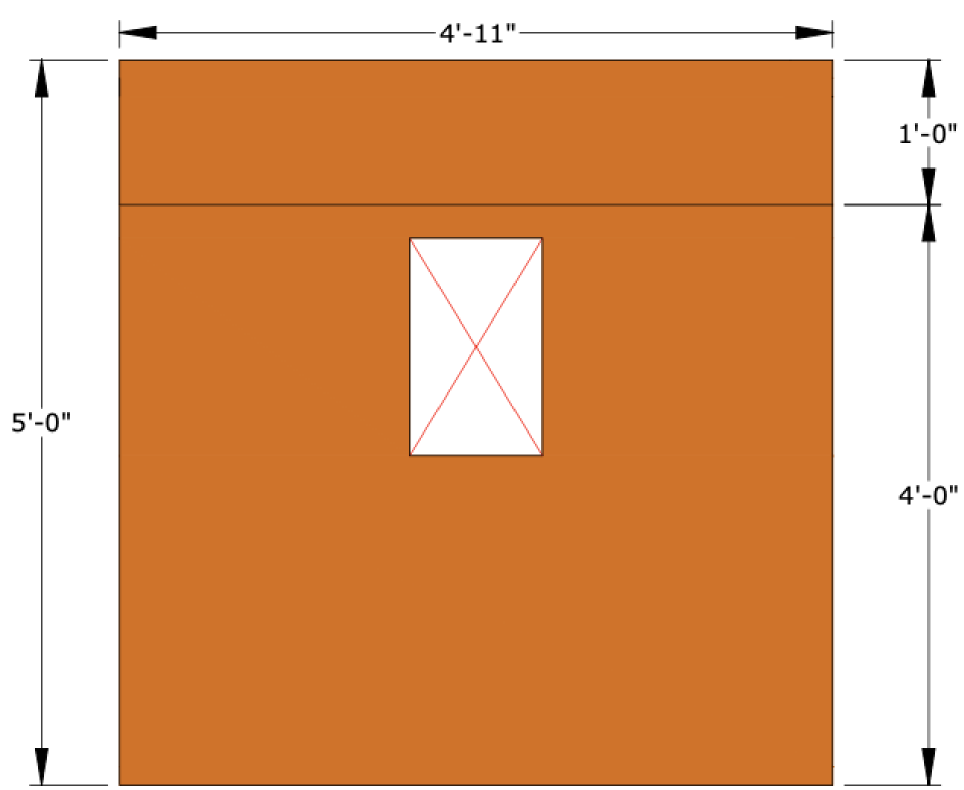
①: FIRST STORY NORTH FACE



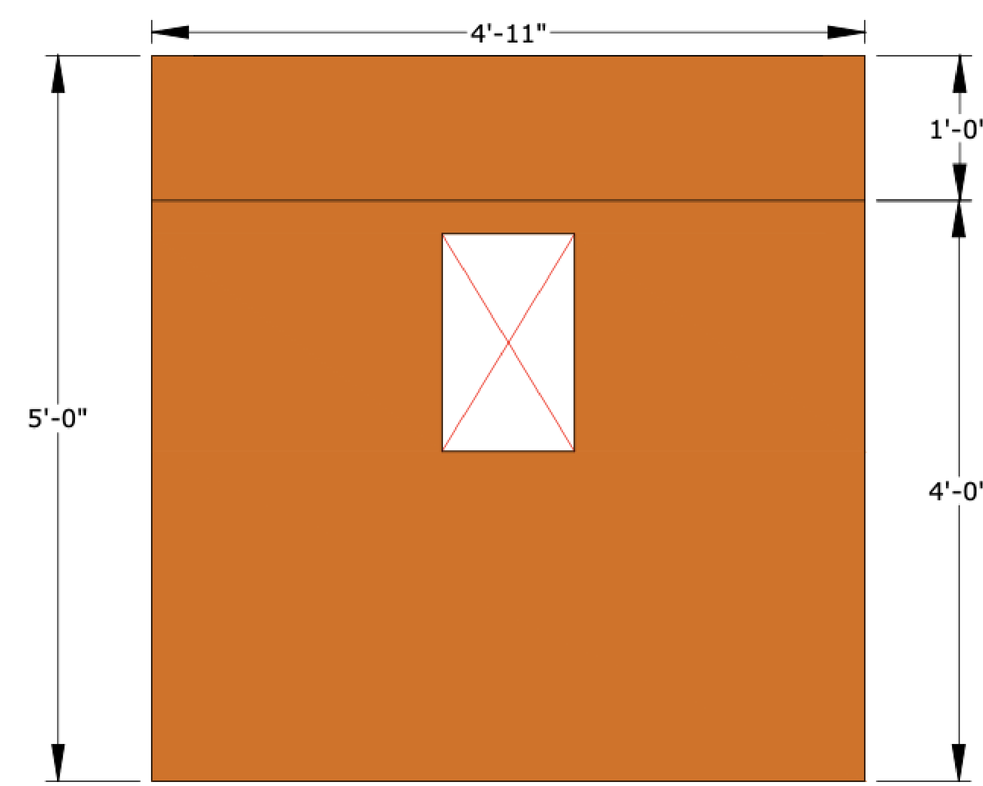
③: FIRST STORY SOUTH FACE

NOTES:

- $\frac{3}{8}$" Sheathing
- 6d Common Nails
- 4" Nail Spacing U.N.O On Panel Edges

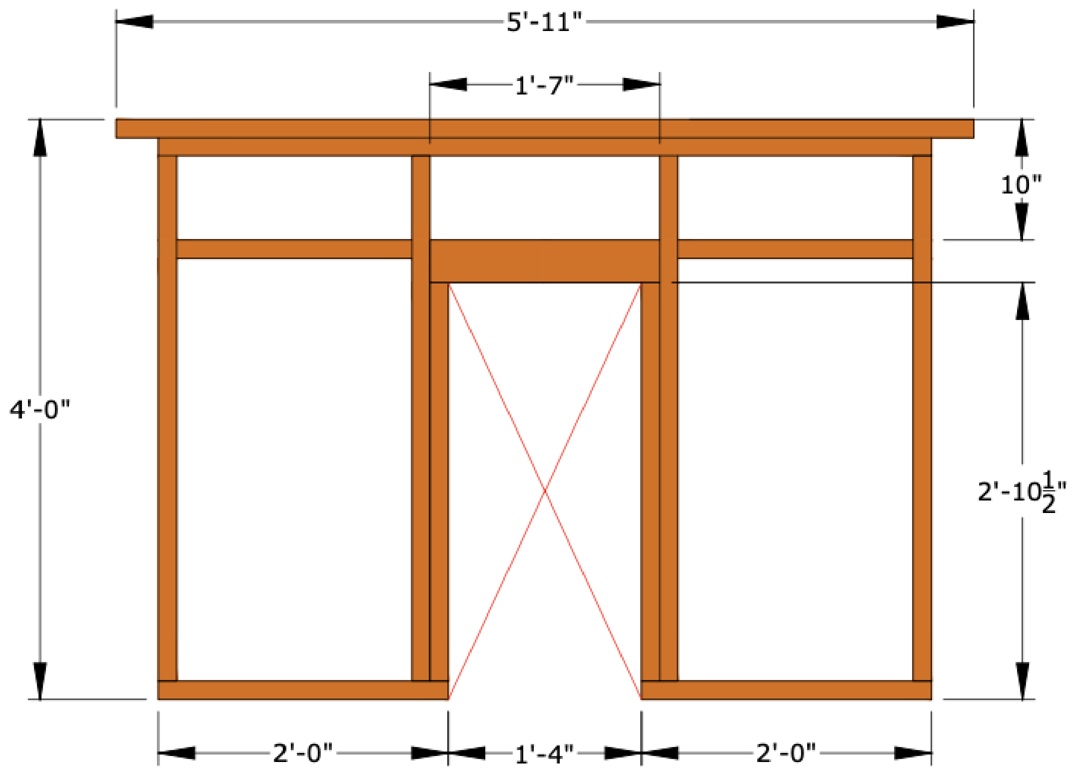


②: FIRST STORY EAST FACE

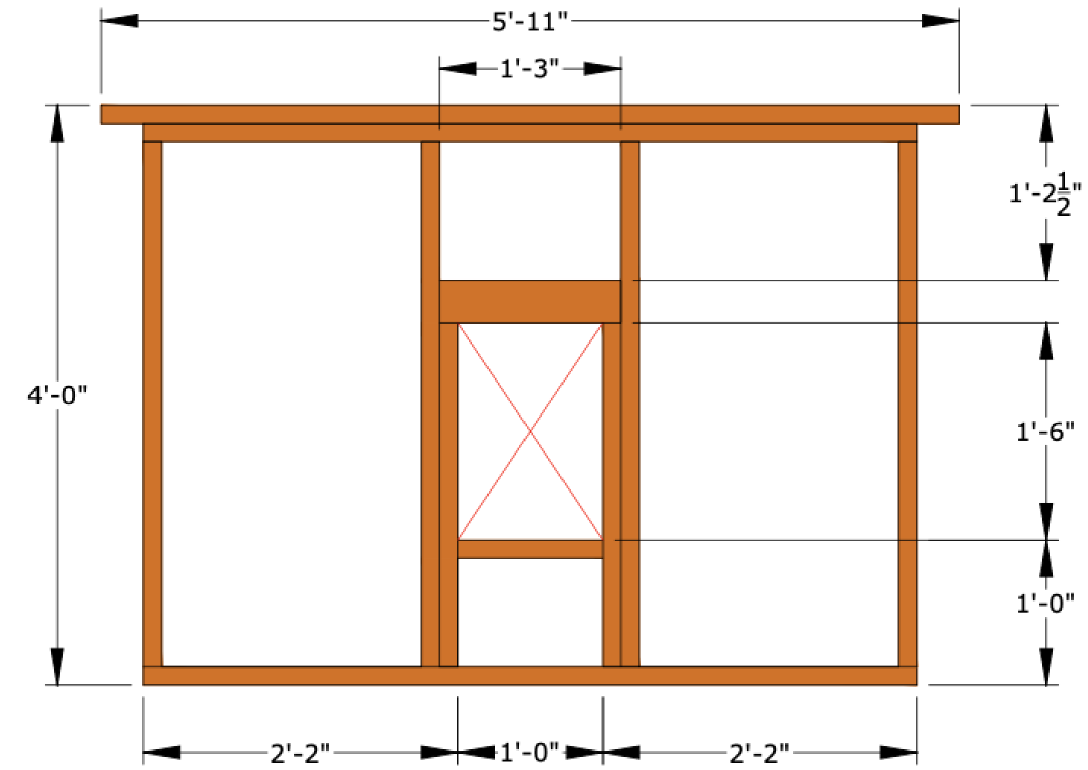


④: FIRST STORY WEST FACE

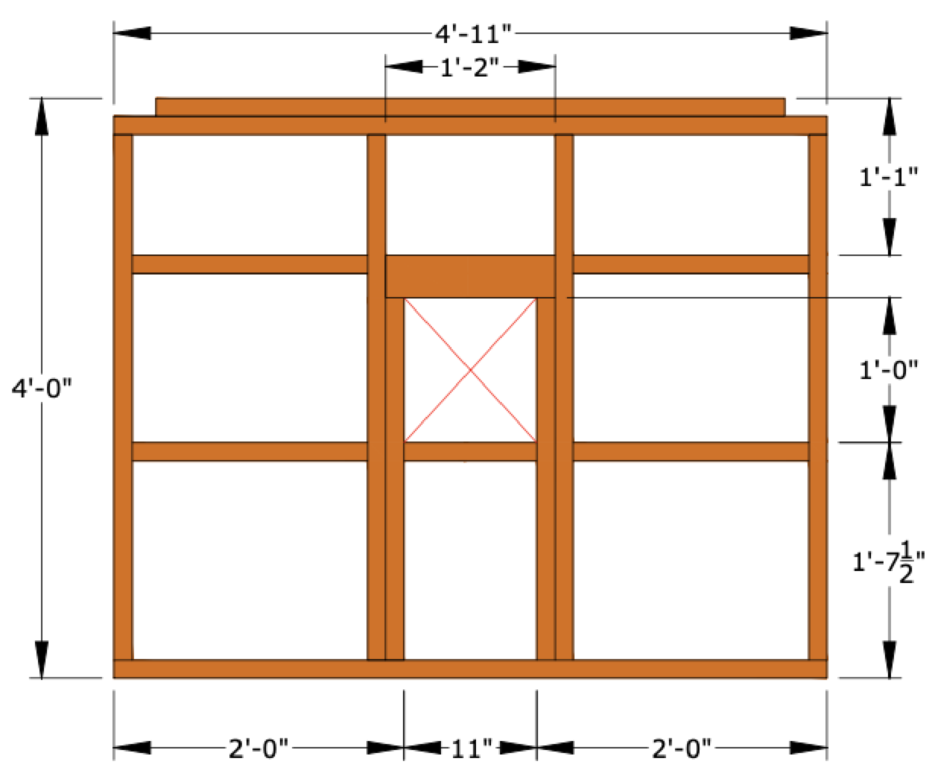
PONDEROSA TIMBERJACKS	PROJECT: NAU TIMBERSTRONG	DATE: 02/23/2024	BY: PONDEROSA TIMBERJACKS	DRAWING NUMBER: S7
	FIRST STORY SHEATHING PLAN	DUE: 02/23/2024 REVISION: 0	SCALE: 1-1/2"=1'-0"	PAGE: 9 OF: 14



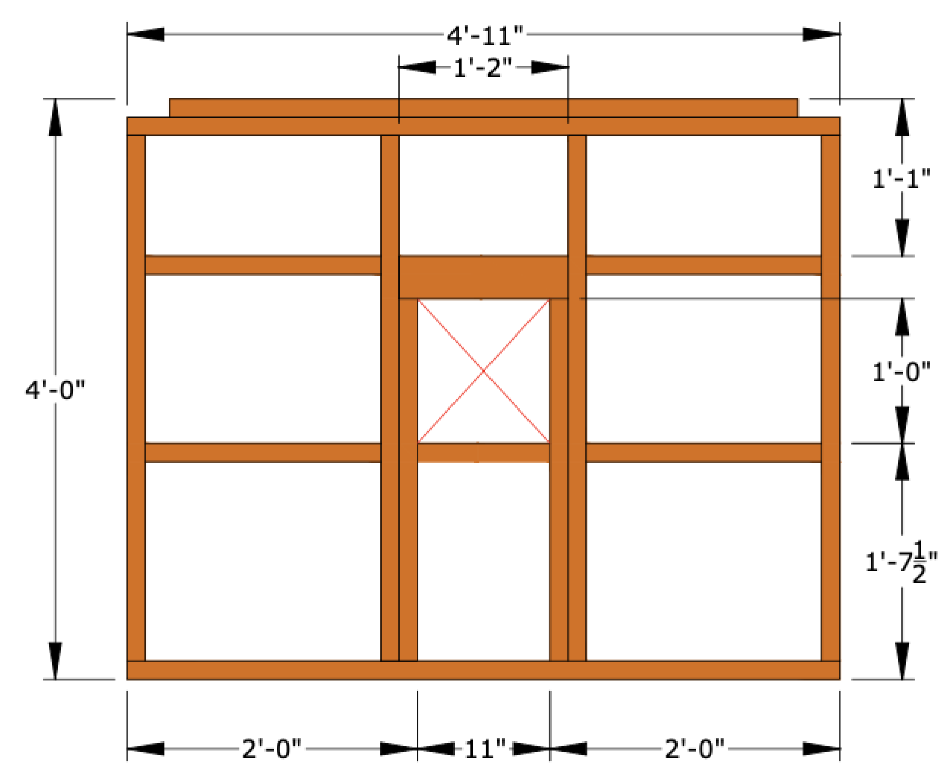
⑤: SECOND STORY NORTH FACE



⑦: SECOND STORY SOUTH FACE



⑥: SECOND STORY EAST FACE

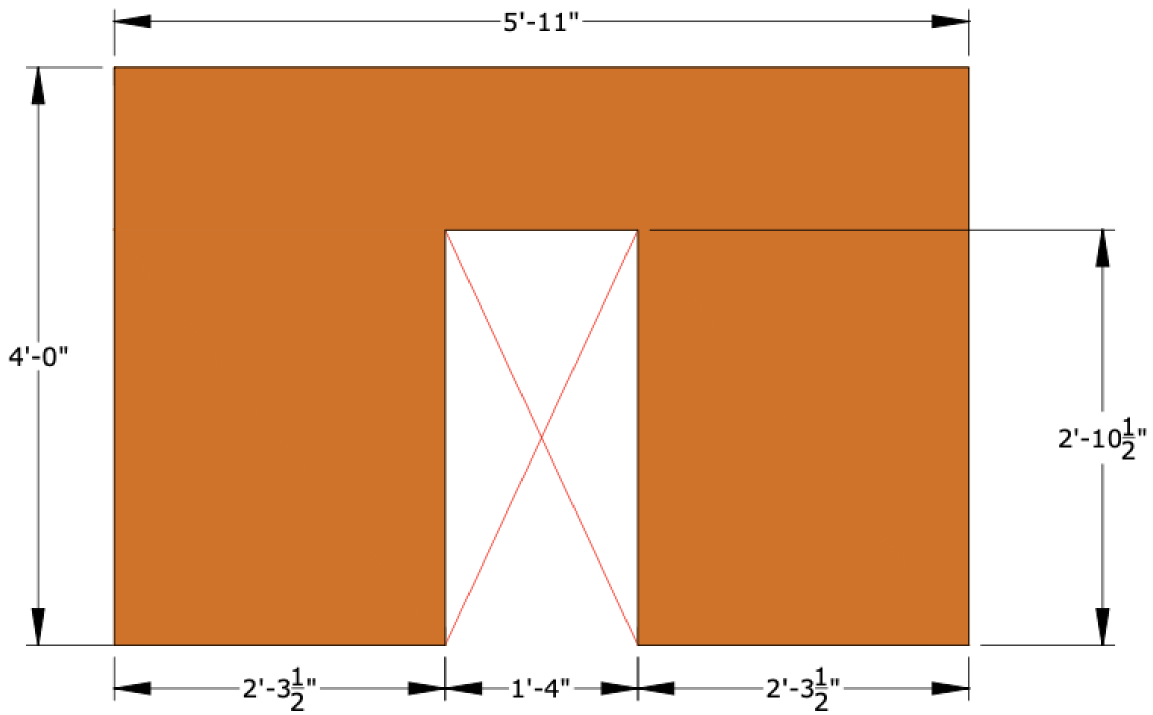


⑧: SECOND STORY WEST FACE

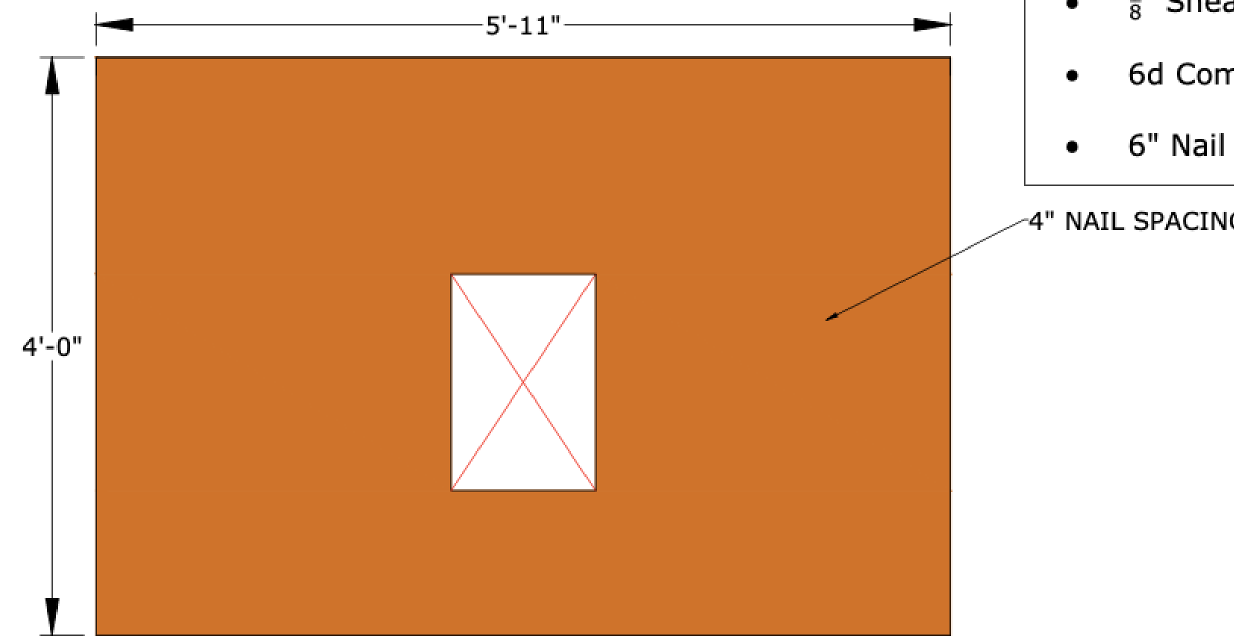
PONDEROSA TIMBERJACKS	PROJECT: NAU TIMBERSTRONG	DATE: 02/23/2024	BY: PONDEROSA TIMBERJACKS	DRAWING NUMBER: S8
	SECOND STORY FRAMING PLAN	DUE: 02/23/2024 REVISION: 0	0 3 6 12 SCALE: 1-1/2"=1'-0"	PAGE: 10 OF: 14

NOTES:

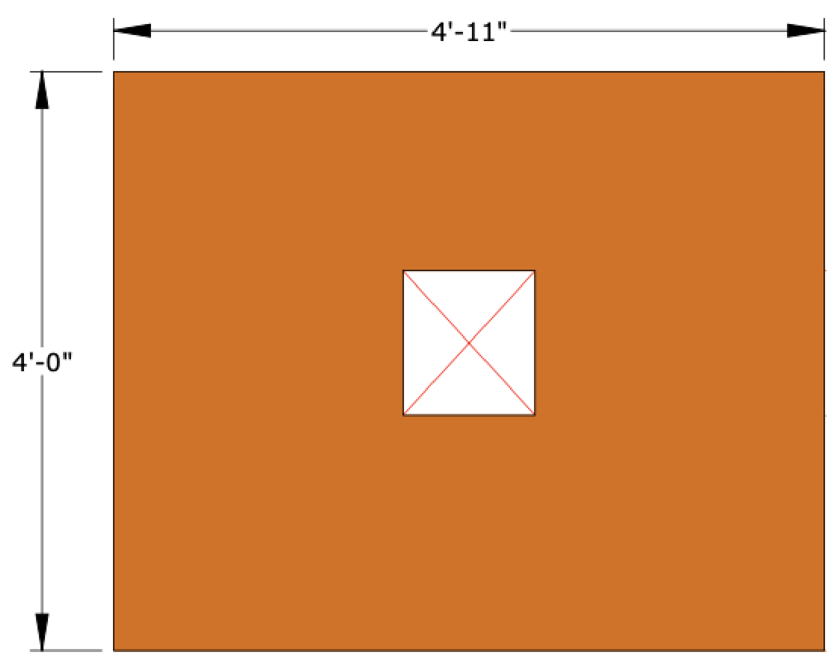
- $\frac{3}{8}$" Sheathing
- 6d Common Nails
- 6" Nail Spacing U.N.O On Panel Edges



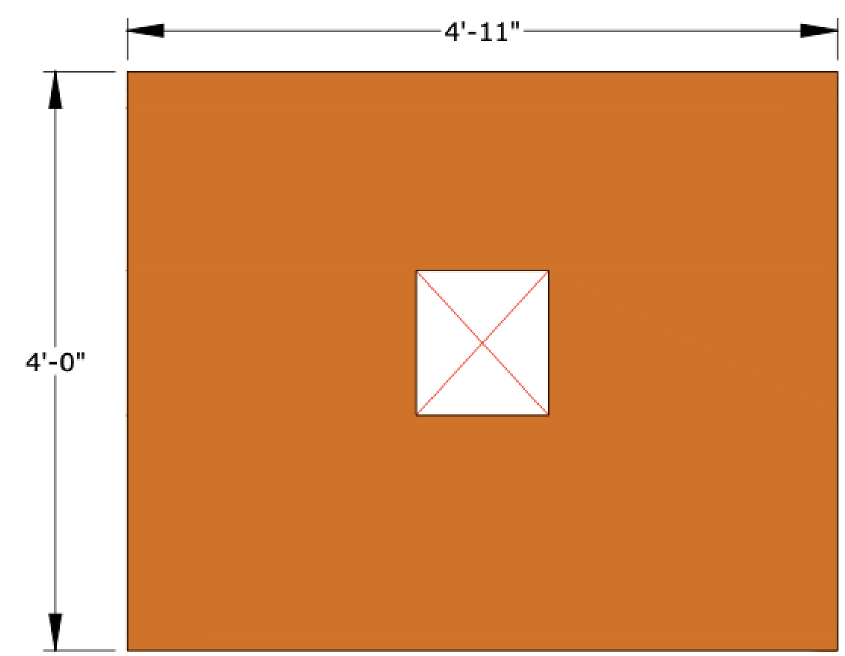
⑤: SECOND STORY NORTH FACE



⑦: SECOND STORY SOUTH FACE

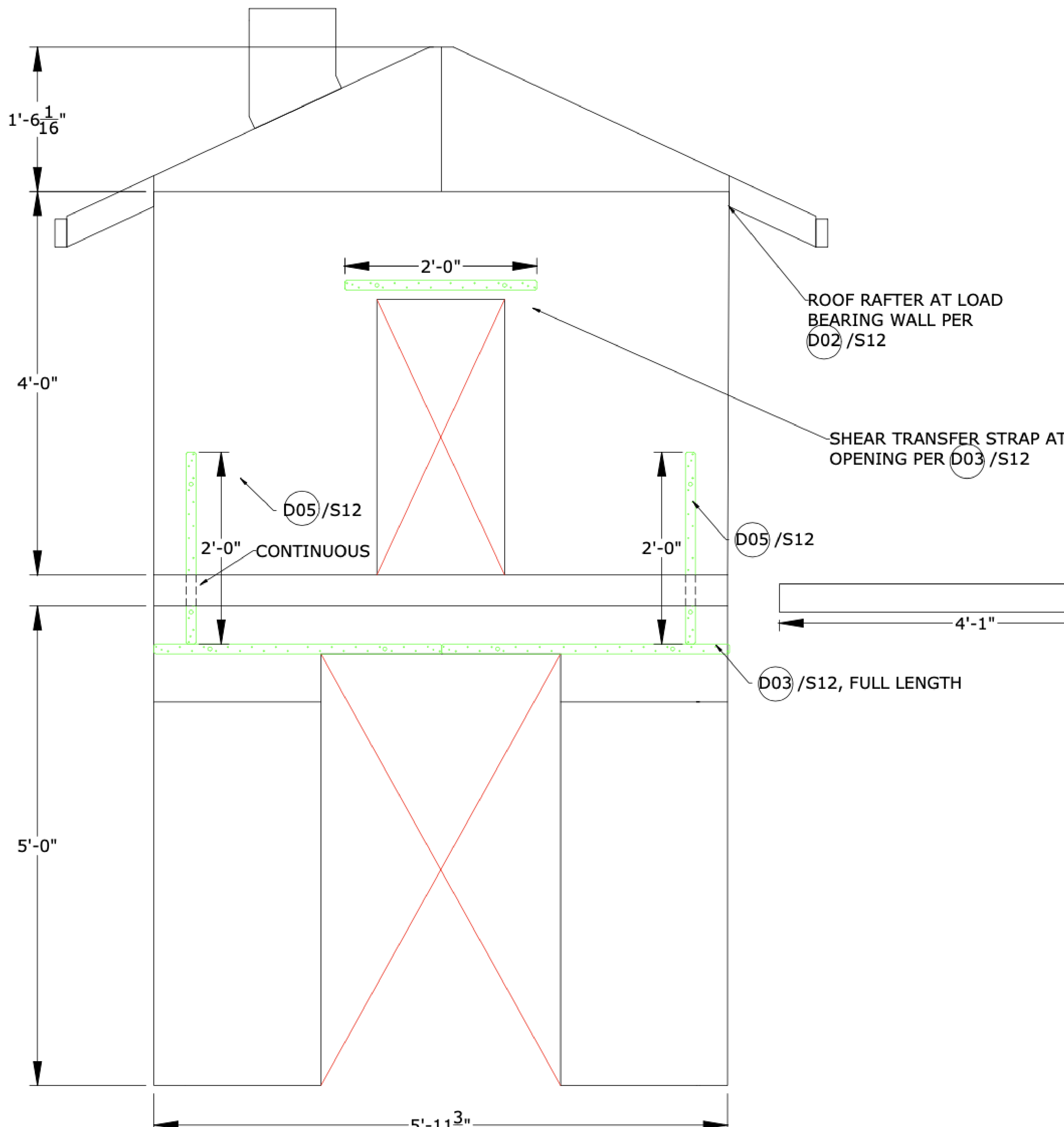


⑥: SECOND STORY EAST FACE

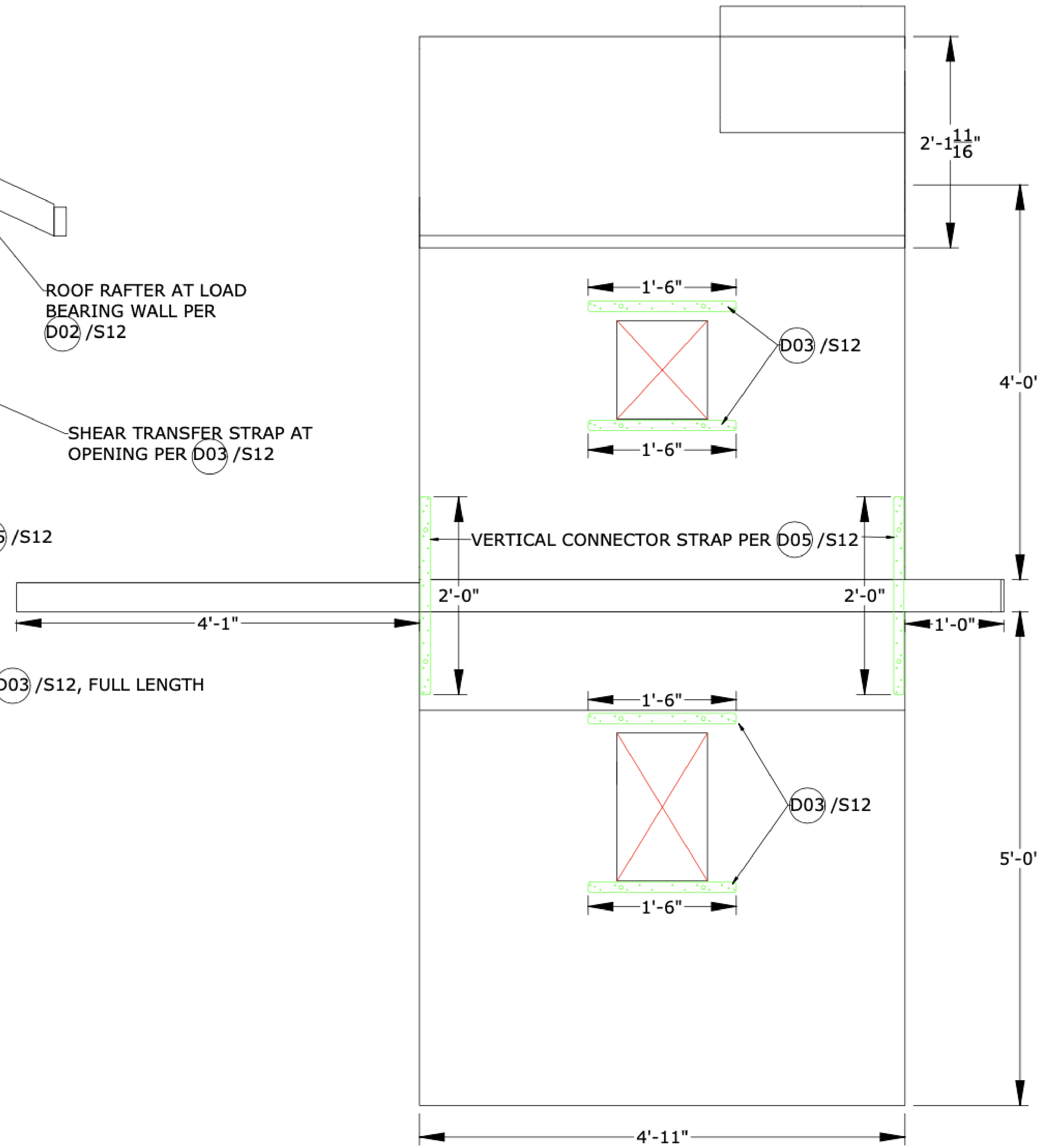


⑧: SECOND STORY WEST FACE

PONDEROSA TIMBERJACKS	PROJECT: NAU TIMBERSTRONG	DATE: 02/23/2024	BY: PONDEROSA TIMBERJACKS	DRAWING NUMBER: S9
	SECOND STORY SHEATHING PLAN	DUE: 02/23/2024 REVISION: 0	SCALE: 1-1/2"=1'-0"	PAGE: 11 OF: 14



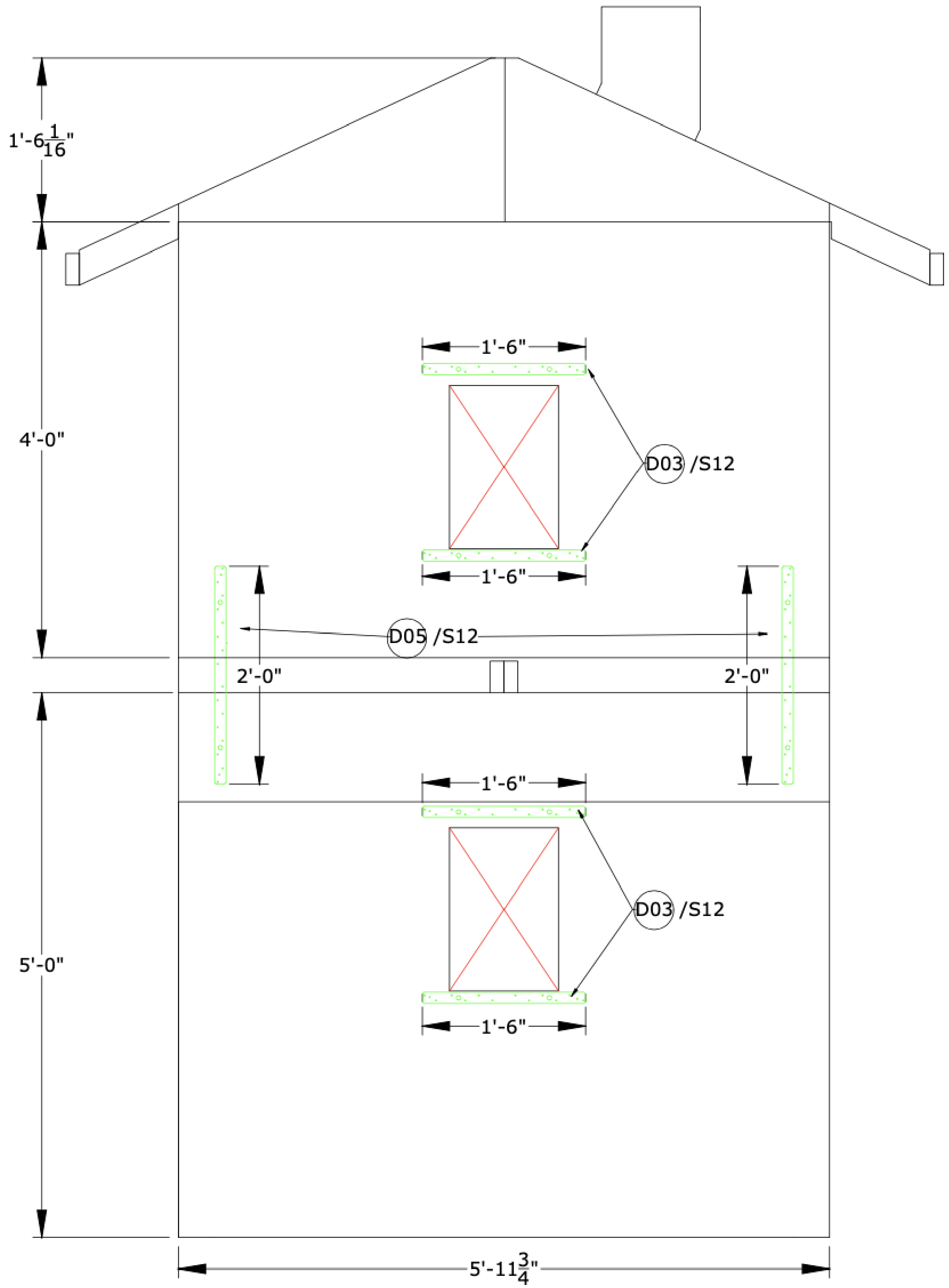
E1: ELEVATION NORTH FACE



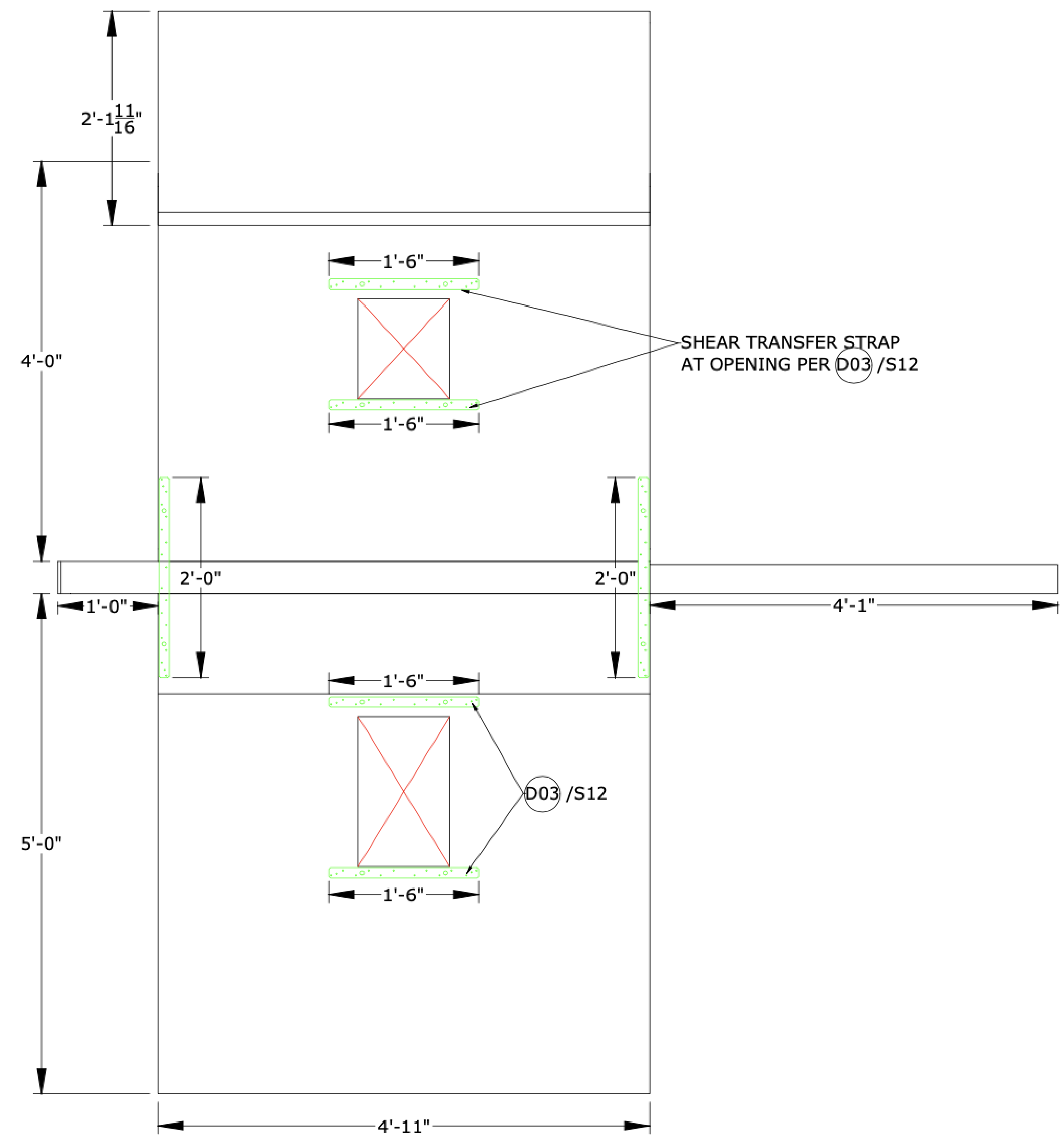
E2: ELEVATION EAST FACE

PROJECT: NAU TIMBERSTRONG	DATE: 02/23/2024	BY: PONDEROSA TIMBERJACKS	DRAWING NUMBER: S10
CONNECTOR PLACEMENT VIEW 1	DUE: 02/23/2024 / REVISION: 0	SCALE: 1-1/2"=1'-0"	PAGE: 12 OF: 14

PONDEROSA TIMBERJACKS

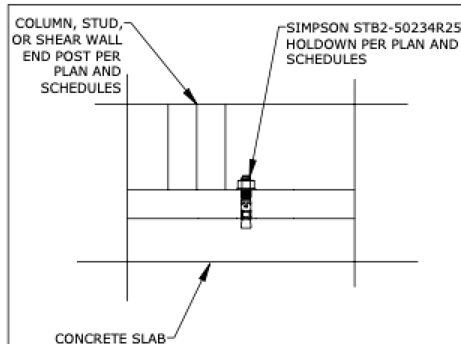


E3: ELEVATION SOUTH FACE

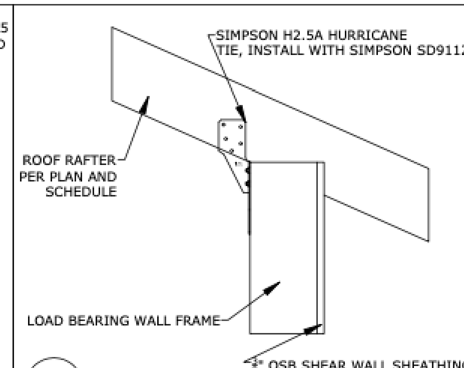


E4: ELEVATION WEST FACE

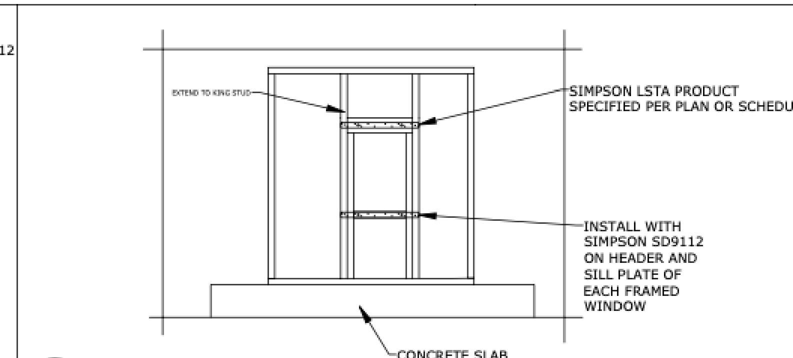
PONDEROSA TIMBERJACKS	PROJECT: NAU TIMBERSTRONG	DATE: 02/23/2024	BY: PONDEROSA TIMBERJACKS	DRAWING NUMBER: S11
	CONNECTOR PLACEMENT VIEW 2	DUE: 02/23/2024 REVISION: 0	SCALE: 1-1/2"=1'-0"	PAGE: 13 OF: 14



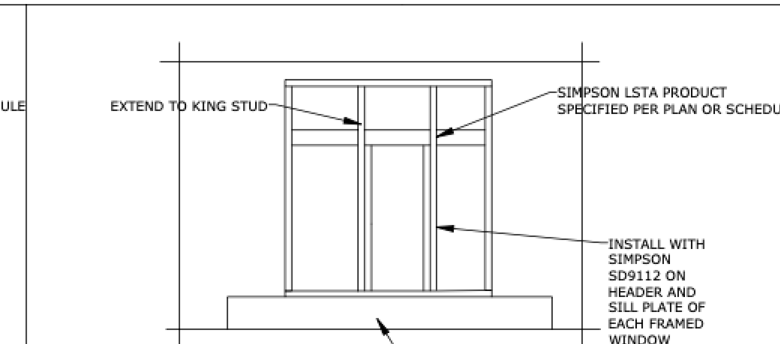
D01 SIMPSON STB2-50234R25 HOLDOWN ANCHORAGE
SCALE: NTS



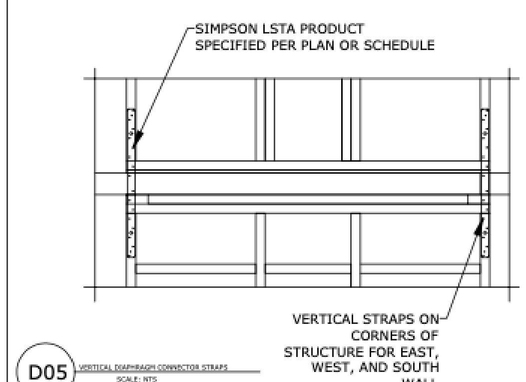
D02 ROOF RAFTER AT LOAD BEARING WALL
SCALE: NTS



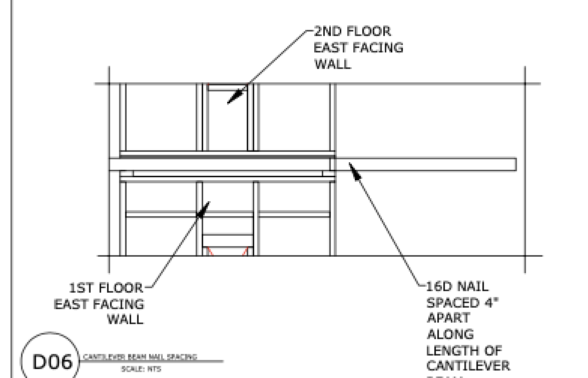
D03 SIMPSON LSTA AT WINDOW FRAME
SCALE: NTS



D04 SIMPSON LSTA AT DOOR FRAME
SCALE: NTS

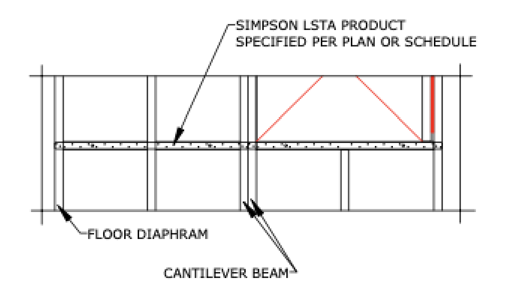


D05 VERTICAL DIAPHRAGM CONNECTOR STRAPS
SCALE: NTS



D06 CANTILEVER BEAM NAIL SPACING
SCALE: NTS

NOTE: SIMPSON LSTA PRODUCT WILL NEED TO BE CUT TO FIT TO FLOOR DIAPHRAGM

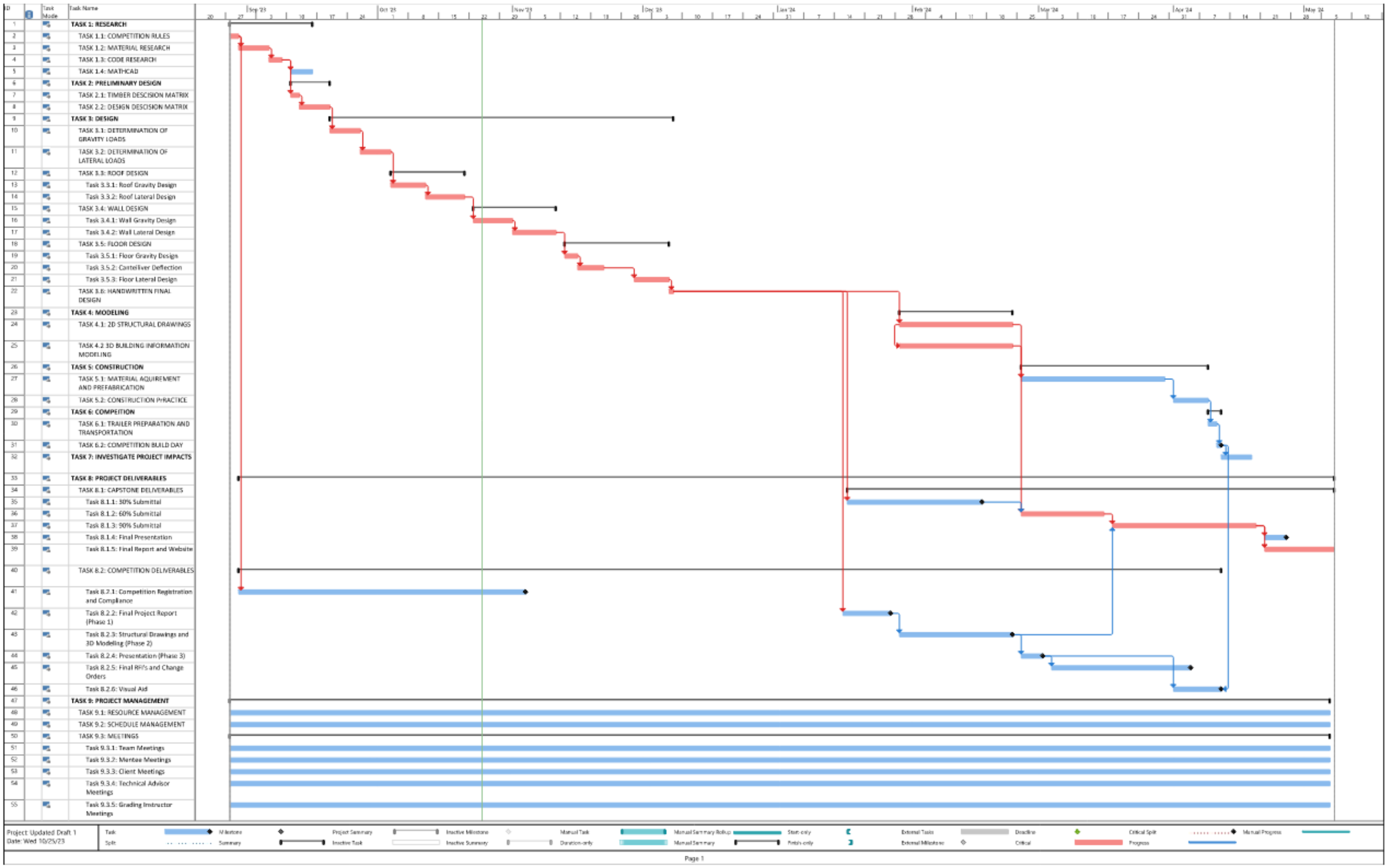


D07 FLOOR DIAPHRAGM OPENING STRAP
SCALE: NTS

PONDEROSA TIMBERJACKS	PROJECT: NAU TIMBERSTRONG	DATE: 02/23/2024	BY: PONDEROSA TIMBERJACKS	DRAWING NUMBER: S12
	COMPLETE LOAD PATH 1	DUE: 02/23/2024 REVISION: 0	0 N/A N/A N/A SCALE: NTS	PAGE: 14 OF: 14

APPENDIX E: GANTT CHART

The project Gantt Chart can be found on the following page, with the critical path highlighted in red.


ID
Task Mode
Task Name
1 TASK 1: RESEARCH
2 TASK 1.1: COMPETITION RULES
3 TASK 1.2: MATERIAL RESEARCH
4 TASK 1.3: CODE RESEARCH
5 TASK 1.4: MATHCAD
6 TASK 2: PRELIMINARY DESIGN
7 TASK 2.1: TIMBER DESCISION MATRIX
8 TASK 2.2: DESIGN DESCISION MATRIX
9 TASK 3: DESIGN
10 TASK 3.1: DETERMINATION OF GRAVITY LOADS
11 TASK 3.2: DETERMINATION OF LATERAL LOADS
12 TASK 3.3: ROOF DESIGN
13 Task 3.3.1: Roof Gravity Design
14 Task 3.3.2: Roof Lateral Design
15 TASK 3.4: WALL DESIGN
16 Task 3.4.1: Wall Gravity Design
17 Task 3.4.2: Wall Lateral Design
18 TASK 3.5: FLOOR DESIGN
19 Task 3.5.1: Floor Gravity Design
20 Task 3.5.2: Cantelliver Deflection
21 Task 3.5.3: Floor Lateral Design
22 TASK 3.6: HANDWRITTEN FINAL DESIGN
23 TASK 4: MODELING
24 TASK 4.1: 2D STRUCTURAL DRAWINGS
25 TASK 4.2 3D BUILDING INFORMATION MODELING
26 TASK 5: CONSTRUCTION
27 TASK 5.1: MATERIAL AQUIREMENT AND PREFABRICATION
28 TASK 5.2: CONSTRUCTION PrRACTICE
29 TASK 6: COMPEITION
30 TASK 6.1: TRAILER PREPARATION AND TRANSPORTATION
31 TASK 6.2: COMPETITION BUILD DAY
32 TASK 7: INVESTIGATE PROJECT IMPACTS
33 TASK 8: PROJECT DELIVERABLES
34 TASK 8.1: CAPSTONE DELIVERABLES
35 Task 8.1.1: 30% Submittal
36 Task 8.1.2: 60% Submittal
37 Task 8.1.3: 90% Submittal
38 Task 8.1.4: Final Presentation
39 Task 8.1.5: Final Report and Website
40 TASK 8.2: COMPETITION DELIVERABLES
41 Task 8.2.1: Competition Registration and Compliance
42 Task 8.2.2: Final Project Report (Phase 1)
43 Task 8.2.3: Structural Drawings and 3D Modeling (Phase 2)
44 Task 8.2.4: Presentation (Phase 3)
45 Task 8.2.5: Final RFI's and Change Orders
46 Task 8.2.6: Visual Aid
47 TASK 9: PROJECT MANAGEMENT
48 TASK 9.1: RESOURCE MANAGEMENT
49 TASK 9.2: SCHEDULE MANAGEMENT
50 TASK 9.3: MEETINGS
51 Task 9.3.1: Team Meetings
52 Task 9.3.2: Mentee Meetings
53 Task 9.3.3: Client Meetings
54 Task 9.3.4: Technical Advisor Meetings
55 Task 9.3.5: Grading Instructor Meetings
Sep '23
Oct '23
Nov '23
Dec '23
Jan '24
Feb '24
Mar '24
Apr '24
May '24
Project: Updated Draft 1
Date: Wed 10/25/23
Task
Split
Milestone
Summary
Project Summary
Inactive Task
Inactive Milestone
Inactive Summary
Manual Task
Duration-only
Manual Summary Rollup
Manual Summary
Start-only
Finish-only
External Tasks
External Milestone
Deadline
Critical
Critical Split
Progress
Manual Progress
Page 1

APPENDIX F: STAFFING

Task	SENG	ENG	TECH	INT	SO
TASK 1: RESEARCH	**3**	**8**	**3**	**5**	**1**
TASK 1.1: COMPETITION RULES	1	1	1	3	1
TASK 1.2: MATERIAL RESEARCH	0	2	0	1	0
TASK 1.3: CODE RESEARCH	0	3	0	1	0
TASK 1.4: MATHCAD	2	2	2	0	0
TASK 2: PRELIMINARY DESIGN	**2**	**6**	**0**	**2**	**0**
TASK 2.1: TIMBER DECISION MATRIX	1	3	0	1	0
TASK 2.2: DESIGN DECISION MATRIX	1	3	0	1	0
TASK 3: DESIGN AND ANALYSIS	**14**	**54**	**0**	**22**	**0**
TASK 3.1: DETERMINATION OF GRAVITY LOADS	1	2	0	1	0
TASK 3.2: DETERMINATION OF LATERAL LOADS	1	2	0	1	0
TASK 3.3: ROOF DESIGN	3	15	0	4	0
Task 3.3.1: Roof Gravity Design	1	7	0	2	0
Task 3.3.2: Roof Lateral Design	2	8	0	2	0
TASK 3.4: WALL DESIGN	3	17	0	4	0
Task 3.4.1: Wall Gravity Design	1	8	0	2	0
Task 3.4.2: Wall Lateral Design	2	9	0	2	0
TASK 3.5: FLOOR DESIGN	5	18	0	9	0
Task 3.5.1: Floor Gravity Design	1	7	0	2	0
Task 3.5.2: Cantilever Design	1	3	0	2	0
Task 3.5.3: Floor Lateral Design	2	8	0	2	0
TASK 3.6: HANDWRITTEN FINAL DRAFT	1	0	0	3	0
TASK 4: MODELING	**2**	**12**	**0**	**16**	**0**
TASK 4.1: 2D STRUCTURAL DRAWING	1	4	0	8	0
TASK 4.2: 3D BUILDING INFORMATIONAL MODELING	1	8	0	8	0
TASK 5: CONSTRUCTION	**0**	**4**	**25**	**25**	**19**
TASK 5.1: MATERIAL ACQUIREMENT AND PREFABRICATION	0	4	16	16	16
TASK 5.2: CONSTRUCTION PRACTICE	0	0	9	9	3
TASK 6: COMPETITION	**0**	**0**	**19**	**9**	**4**
TASK 6.1: TRAILER PREPARATION AND TRANSPORTATION	0	0	10	0	1
TASK 6.2: COMPETITION BUILD DAY	0	0	9	9	3
TASK 7: INVESTIGATE PROJECT IMPACTS	**3**	**0**	**3**	**0**	**1**
TASK 8: PROJECT DELIVERABLES	**18**	**48**	**3**	**12**	**0**
TASK 8.1: CAPSTONE DELIVERABLES	10	40	0	0	0
Task 8.1.1: 30% Submittal	2	10	0	0	0
Task 8.1.2: 60% Submittal	2	15	0	0	0
Task 8.1.3: 90% Submittal	2	15	0	0	0
Task 8.1.4: Final Presentation	2	0	0	0	0
Task 8.1.5: Final Report and Website	2	0	0	0	0
TASK 8.2: COMPETITION DELIVERABLES	8	8	3	12	0
Task 8.2.1: Competition Registration and Compliance	0	0	0	0	0
Task 8.2.2: Final Project Report (Phase 1)	2	2	0	0	0
Task 8.2.3: Structural Drawings & 3D Modeling (Phase 2)	1	0	0	0	0
Task 8.2.4: Presentation (Phase 3)	2	2	2	6	0
Task 8.2.5: Final RFIs and Change Orders	2	1	0	0	0
Task 8.2.6: Visual Aid	1	3	1	6	0
TASK 9: PROJECT MANAGEMENT	**30**	**53**	**3**	**25**	**1**
TASK 9.1: RESOURCE MANAGEMENT	5	3	0	0	0
TASK 9.2: SCHEDULE MANAGEMENT	10	8	3	0	0
TASK 9.3: MEETINGS	15	42	0	25	1
Task 9.3.1: Team Meetings	3	15	0	10	1
Task 9.3.2: Mentee Meetings	0	15	0	15	0

Task 9.3.3: Client Meetings	4	4	0	0	0
Task 9.3.4: Technical Advisor Meetings	4	4	0	0	0
Task 9.3.5: Grading Instructor Meetings	4	4	0	0	0