SKY HIGH STRUCTURES

2025-2026 TIMBER-STRONG DESIGN BUILD COMPETITION REPORT

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Final Design Report

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Abbreviations

ASCE	American Society of Civil Engineers
APA	The Engineered Wood Association
AWC	American Wood Council
BIM	Building Information Model
DF-L	Douglas Fir Larch
FS	Factor of Safety
ISWS	Intermountain Southwest Student Symposium
LFRS	Lateral Force Resisting System
MWFRS	Main Wind Force Resisting System
NAU	Northern Arizona University
NDS	National Design Specification for Wood Construction, 2017
OC	On Center
PSF	Pounds per Square Foot
SDPWS	Special Design Provisions for Wind and Seismic, 2021
SE2050 ECOM	Structural Engineering Institute 2050 Embodied Carbon Estimator
SST	Simpson Strong-Tie
SW	Shear Wall
TSDB	Timber-Strong Design Build

Acknowledgements

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1.0 Project Introduction

The 2026 Timber-Strong Design Build (TSDB) Competition is a student-based competition involving the Intermountain Southwest Student Symposium (ISWS). This competition will take place at the University of Utah, located in Salt Lake City, Utah. Teams are required to create a two-story, small-scale, light-framed wooden structure while also focusing on sustainability, structural, design and construction constraints. The design of this structure will follow the requirements outlined in the *TSDB 2026 Rules* [1].

Public welfare is taken into great consideration for this design, as teams are working to improve wood construction by providing structurally durable framing plans that ensure the safety of each structure. Teams are given exposure to management and building practices to gain hands-on experience for future engineers, paving the way globally for reliable and sustainable engineering design. Environmental constraints guide the team to develop a sustainable and renewable structure with efforts to help carbon footprint reduction. The plan of repurposing a project can economically enhance engineering designs with more effective solutions of reusing wood materials around the world.

2.0 Technical Work

2.1 Task 1: Research

2.1.1 Task 1.1: Research Past Competition Teams

The team researched NAU's 2024-2025 Timber-Strong Team as well as past competition teams throughout various competitions to evaluate their design choices, strengths and weaknesses.

- Takeaway: Help identify our schedule by seeing how previous teams have scheduled their work; how to account for more time in schedule based on their previous issues
- Finding: 2024 team had issues with the force distribution and load path in the design. They didn't use industry standard construction practices
    - Takeaway: ensure that load properly transfers between levels of the structure (roof, second floor, first floor, foundation); ensure that our load can be transferred through the connections/pick the right connection types
- Finding: 2023 team had lack of communication between construction members and design members of team; led to roof being built differently from structural plans; due to this being constructed incorrectly, there was an incomplete load path
    - Takeaway: coordinate tasks between members and meet often to ensure that everyone is on the same page with the design

2.1.2 Task 1.2: Research Competition Rules

The team researched the ASCE Timber-Strong Design Build 2026 Rules and reviewed any public Timber-Strong RFI submissions uploaded on the ASCE website to ensure that the following constraints are met in the final design:

- 90 Minute construction time constraint

- Dimension requirements (first floor, second floor, height, cantilever)
- Loadings for live, dead, and wind loads that are in 2026 rules
- Check ASCE website for rule changes/questions that other teams may be asking
- Cantilever load of 150 lb
- Cantilever deflection requirement of 0.5” to 1”; three possible cantilever loading locations (3’, 3.5’. 4’)
- Roof dead load and floor dead load are their self-weights
- Roof live load = 20 psf; floor live load = 50 psf
- Cantilever floor beam must be safely loaded at all given load spots without anchors or hold-downs that attach the structure to the floor
- No dead load allowed to resist uplift pressure
- MWFRS wind table provided in rules
- C&C pressures table provided in rules
- Structure must be clearly modeled in Revit to demonstrate load path
    - Connections must be provided to create a complete load path from roof to foundation
- Prefabricated roof elements cannot exceed 30 lb
- Design must be sustainable, relatively inexpensive, and have a low carbon footprint

2.1.3 Task 1.3: Research Structural Systems

The team researched wood construction specifications by reading the following resources, which provided guidance for the selected design:

- 2018 National Design Specifications (NDS) Supplement for Wood Construction
- 2018 National Design Specifications (NDS) for Wood Construction with Commentary
- 2021 Special Design Provisions for Wind and Seismic (SDPWS)
- Simpson Strong-Tie (SST) Metal Connector Plates for Wood Trusses
- 2024-2025 Simpson Strong-Tie (SST) Wood Construction Connectors
- The Engineered Wood Association (APA) Advanced Framing Construction Guide

These resources provided the team with load pathways for a continuous load path, load path to resist uplift and a load path to resist the in-plane loads. Lateral loading causes uplift, which must be transferred to the foundation through the shear walls and their connecting elements. Gravity load paths consist of a vertical downwards load being transferred from the top of the structure to its foundation due to its weight and other forces. The gravity load path begins with being distributed across the roof truss members and moving down to the wall studs. The path then continues with the load being spread across the floor joists and moving down the studs for the wall(s) below, finally ending at the foundation of the structure.

Figure 1 demonstrates the loads that need to be designed for the structure given from the competition.

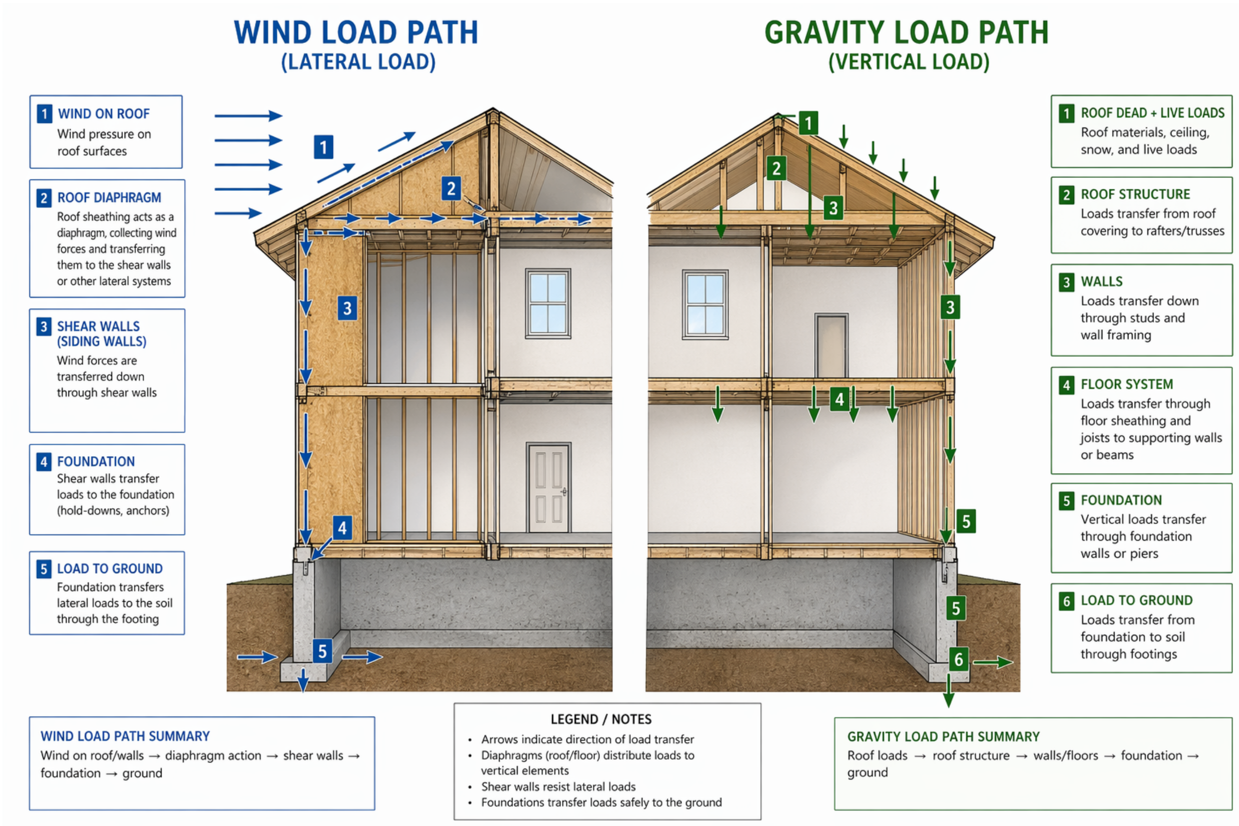


Figure 1: Typical Residential Structure Load Path Diagram

2.1.4 Task 1.4: Research Material Availability

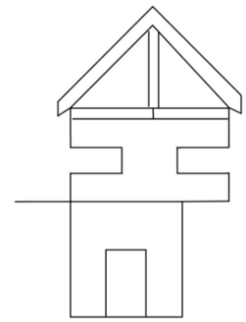
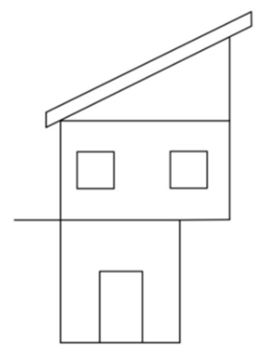
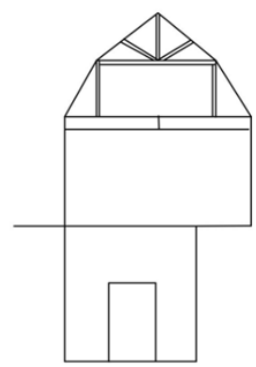
The team designed the structure according to the materials which the suppliers said were available; however, material availability changed several times during the design phase, requiring multiple redesigns. Final lumber selection included DF #1 2x6's and 2x4's for the trusses, DF#2 for the rest of the framing, and 15/32" sheathing, which is summarized in the following Shear Wall Design task. Availability of SST connectors was initially unconstrained, but that changed following the team's original competition submittal. Required reductions and changes in SST connectors are outlined in the following sections.

2.2 Task 2: Design Selection

2.2.1 Task 2.1: Brainstorm Structural Systems

Following the brainstorming process, the team narrowed the design options to three viable alternatives. Table 1, below, presents the three roof design options to be considered and compare their key characteristics to support the final selection.

Table 1: Structural Design Alternatives

Varied Structural Designs		
Design 1	Gable Roof trusses with cantilevered corner window headers	
Design 2	Monosloped with windows	
Design 3	Mansard trusses with large windows	

2.2.2 Task 2.2: Identify Viable Design Options

Design 1: The advantages of this option include the potential to earn higher design points in the competition for complexity, creativity, and durability. These points stem primarily from the roof truss system, which requires specialized fabrication techniques, advanced design methods, and additional research. Roof trusses are rarely used in this competition, making this approach both unique and technically challenging. Additionally, the cantilevered headers require design considerations and structural checks beyond those of conventional window headers, further demonstrating a higher level of engineering difficulty. The disadvantages of this design include a moderate cost, relative to other designs. Design alternative 1 also has a more time-consuming assembly process due to its truss design, when compared to other alternative designs.

There are no deflection criteria for the competition, meaning this design is governed by strength requirements. In a real, full-scale structure, deflection requirements would likely govern the design of headers, and possibly trusses or floor members, which means those members would need to be sized up significantly. This means that none of these alternatives are perfectly proportional to a scaled-up structure. Since this structure is built at a smaller scale, the team benefits greatly by adding on excess material to provide more strength than the structure needed.

Design 2: The advantages of this design include the lowest cost among the considered options, as well as a less tedious and less time-consuming assembly process. This is due to the lowest quantity of needed materials compared to the other options. These benefits are largely due to the simplicity of the monoslope roof design and its straightforward application. This roof type has been used numerous times in past competitions, and results in high efficiency and reliability, though it offers limited originality. Therefore, the disadvantages of this design include fewer opportunities to earn design and creativity points, as well as less appealing aesthetics compared to the other options.

Design 3: The advantages of this design include the potential to earn high scores in design and creativity, as well as strong performance in durability. These benefits stem from the roof being the most complex and innovative application among the options considered. Due to the complexity, this option is a highly original and distinctive choice. However, this originality also contributes to its disadvantages. This design has the highest cost of all the options, requiring more lumber and other materials, such as hardware, compared to the other design options. It also requires the most complex and time-consuming assembly process and is the least sustainable compared to the other designs.

Ultimately, the design choices were placed into a decision matrix, as seen below in Section 2.2.3, to make the decision-making process more efficient.

2.2.3 Task 2.3: Select Design

Table 2, below, presents the decision matrix developed to support the structural design selection process. The matrix applies objective weights that are directly proportional to the points allocated to each criterion in the TSDC Rules scoring rubric. Scoring of each design alternative was based solely on criteria that directly contribute to the overall competition score. While the weighting framework is objective, some judgment is inherent in the scoring process and is acknowledged as a potential source of subjectivity.

Table 2: Design Selection Decision Matrix

Design Selection Decision Matrix							
		Design 1		**Design 2**		**Design 3**	
Criteria	**Weight (%)**	Score	Weighted Score	Score	Weighted Score	Score	Weighted Score
Cost	15%	2.5	0.375	3	0.45	1	0.15
Assembly Speed	20%	2.5	0.5	3	0.6	2	0.4
Creativity/Aesthetic	15%	3	0.45	1	0.15	2.5	0.375
Sustainability	15%	2.5	0.375	2	0.3	1	0.15
Durability	35%	3	1.05	3	1.05	3	1.05
Total	100%		2.75		2.55		2.125

The Cost criteria in this table is measured by the amount of money it would cost, in total, for the amount of lumber and connectors needed to build the structure. A design with the highest score would have the cheapest total design cost, while a design with the lowest score would have the most expensive design cost.

The Assembly Speed criteria in this table is measured by the amount of total time it would take to build the designed roof of the structure. A design with the highest score would have the fastest assembly speed, while a design with a low score would have a slower assembly speed.

The Creativity/Aesthetic criteria in this table is measured by how unique of a design the entire structure is in relation to previous teams' designs in the TSDB competition. A design with a higher score would be the most unique, while a design with a low score would be a common design.

The Sustainability criteria in this table is measured by how much wood lumber and steel connectors are used in total for this design. A design with a higher score would be using less material and more environmentally friendly, regarding its carbon footprint. A design with a lower score would be less sustainable.

The Durability criteria in this table is measured by how well a design can withstand the given design loads from the competition such as wind load. With the help of quick calculations helped the team determine the resistibility of wind load forces on a design. A design with a higher score would be preferable regarding the resisting uplift forces and factor of safety it can provide.

This decision matrix was developed to help identify the most feasible design option for the team. It allowed the team to systematically evaluate each design based on defined criteria and score them according to their respective advantages and disadvantages. Based on this evaluation, the team selected Design 1 to move forward with for the final design.

2.3 Task 3: Final Structural Analysis & Design

2.3.1 Task 3.1: Draft Framing Plan

A gable roof truss with cantilevered second-level headers was selected to be the leading design option for this build. A rough-draft framing plan of this structure is constructed using AutoCAD which is below in Figure 2: Drafted Framing Plan. This framing plan draft includes the overall geometry of the structure, including roof truss dimensions and spacing, wall panel dimensions, anticipated stud spacing, and locations and dimensions of doors and windows. This draft was continuously updated during the design process as geometry and materials were altered to meet design requirements.

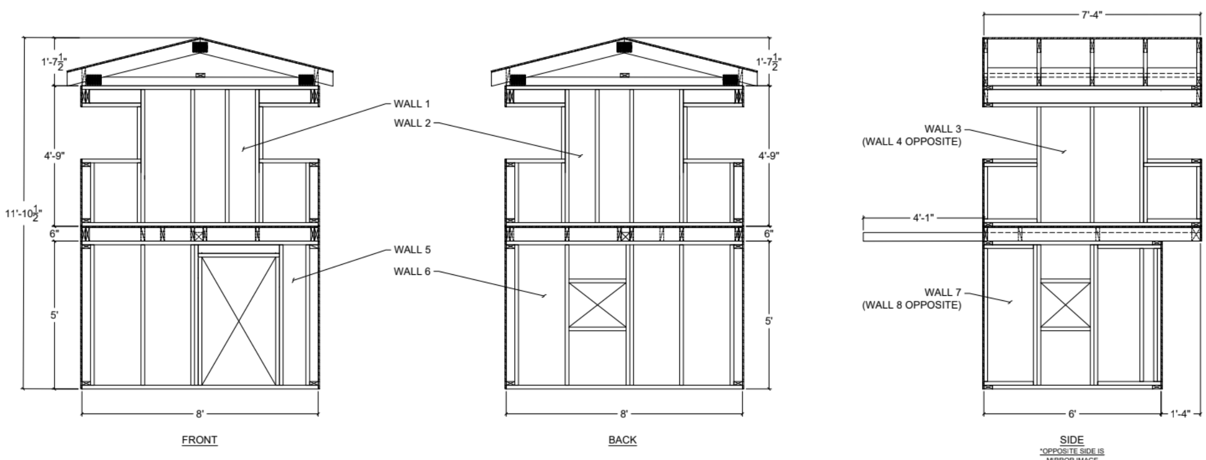


Figure 2: Drafted Framing Plan

2.3.2 Task 3.2: Gravity Design

The gravity design process has been completed by starting design at the top of the structure and following the gravity load path down the structure to the foundation. Roof member size and material selection depended on the availability from wood sponsors, as well as required member strengths, making Douglas Fir Larch (DF-L) #1 2x4 and 2x6 wood members ideal for the bottom chords and top chords, respectively. The current design as the first and second floor spanning up to 10 feet, leaving 2 feet left of allowable height for the roof design. This placed the roof truss members to be at a 14-degree angle.

The truss design has been split into truss top chord design, bottom chord design and truss plate design. After member stresses were calculated, the 2018 National Design Specification for Wood Construction with Commentary (NDS) and the 2018 National Design Specification Supplement

(NDS Supplement) was used to compare the calculated stresses in each member to adjusted allowable design values.

Language in the competition rules suggests that typical adjustment factors for wood design were not required. Accordingly, all structural members were originally designed using unadjusted design values from the NDS Supplement. A clarification to the rules was published a few weeks before the submittal deadline, and that clarification stated that adjustment factors should in fact be used for design throughout the structure. The use of adjustment factors increased allowable stresses for all members in the structure. Rather than redesign, re-draw, and re-model the whole structure, the team preferred to accept the overly conservative results of the original calculations. The low ratio of demand to allowable stress can be seen in both of the following tables and is typical of about half of the members in the structure.

Below in Table 3: Top Chord Roof Design Values is the top chord members roof design values compared to the NDS allowable adjusted stress to the calculated demand values of the design.

Table 3: Top Chord Roof Design Values

Top Chord Member	NDS Adjusted Allowable Stress	Calculated Demand
Bending	2392 psi	428 psi
Tension	1620 psi	337 psi
Shear	288 psi	52 psi

Below in Table 4: Bottom Chord Roof Design Values are the roof compression design values compared to the NDS allowable adjusted stress to the calculated demand values of the design.

Table 4: Bottom Chord Roof Design Values

Bottom Chord Member	NDS Adjusted Allowable Stress	Calculated Demand
Compression	2379 psi	428 psi

The self-weight and applied load in each member of the truss was calculated for the uplift loading of -75 pounds per square foot (psf). The Simpson Strong-Tie (SST) Metal Connector Plates for Wood Trusses Connector Plate Tables, located in Appendix C:SST Allowable Lateral, Tension and Shear Design Values were used to determine which truss plate would be best for the calculated tension and shear values at truss panel points. The allowable values given from the tables prompted the team to use 4”x6”x20 gauge plates. Roof sheathing of 1/2” was determined to have sufficient out-of-plane capacity for roof live loads and wind loads based on 2021 Special Design Provisions for Wind and Seismic (SDPWS) design tables; the lateral capacity of the roof diaphragm was confirmed in following design tasks.

The second level bearing walls of the structure were designed based on the proposed stud spacing from the drafted framing plan. Due to the high design wind loads and because some of the studs are shear wall boundary members, it was suspected that chord forces in the lateral system would control stud design. For that reason, detailed stud design was not carried out in this phase. Based on research of larger structures, it was assumed that stud spacing of no greater than 24” OC would suffice for this project. Following principles of Advanced Framing, trusses, studs, and floor joists were designed to be vertically aligned throughout the structure. This results in a direct gravity load path from the roof to the foundation and eliminates the need to design wall top plates and bottom plates for bending. This permits the use of single top and bottom plates without sacrificing strength or durability, which also reduces the cost and carbon footprint of the structure compared to double top and bottom plates.

The second level floor was designed to cantilever over the front wall per the TSDC Rules. Based on the given floor loading of 50 psf, 1/2” 32/16 floor decking was selected from SDPWS gravity load tables, and the lateral capacity of this decking was confirmed during the following design task. 2x6 HF #2 joists were determined to be more than strong enough for the application. Joist spacing varies due to the incorporation of the cantilever beam and the required 2’-6” square floor opening.

2.3.3 Task 3.3: Diaphragm Design

Per guidance given in the SDPWS, the wood roof and floor diaphragms were idealized as being flexible. Diaphragm demand was calculated using the geometry of the structure, the MWFRS loading given in the competition rules, and SDPWS formulas. SDPWS formulas and principles were then combined with the competition-required range for Factor of Safety (FS) of 1.5 +/- 10% to calculate the required adjusted capacity for each diaphragm.

Diaphragm parameters such as structural panel grade, rating and nailing pattern were selected from the applicable SDPWS tables. Selection of these parameters was largely influenced by the calculated demand, framing size and species, presence or absence of blocking, geometry, and local availability of structural panel sizes and grades. The parameters selected for each diaphragm are summarized in Table 5, below.

Table 5: Diaphragm Construction Parameters

Diaphragm Construction Parameters					
Diaphragm	**Diaphragm Type**	**Sheathing Grade**	**Nominal Panel Thickness**	**Edge Fastening**	**Nominal Capacity (plf)**
Roof	Unblocked	32/16	1/2"	WSV134S at 6” OC	505
Floor	Unblocked	32/16	1/2"	WSV134S at 6” OC	505

Unfortunately, the FS of the roof diaphragm is too high to receive maximum points in the competition. This is due to the low tributary area and low demand on the roof diaphragm combined with a lack of local availability of lower-capacity structural sheathing such as 3/8" or 5/16" thickness sheathing. The most feasible solution considered was the possibility of cutting large holes in the roof to increase demand on the diaphragm, but this was ultimately deemed to be unrealistic and impractical. Fortunately, being above the competition-required FS still results in a safe and durable design; the only downside is that maximum points cannot be earned for a FS over 1.65. The floor diaphragm was not subject to these same issues because it has twice as much tributary area, and because the competition rules required a large hole to be cut in it.

Factors of Safety for the roof and floor diaphragms are summarized in Table 6, below. Nominal capacities were reduced using an ASD adjustment factor of 2.0.

Table 6: Diaphragm Factor of Safety Summary Table

c			
Diaphragm	**Adjusted Capacity (plf)**	**Demand (plf)**	**FS**
Roof	252.2	73.1	3.45
Floor	252.2	184.9	1.37
Average			**2.41**

Diaphragm chord and collector stresses were calculated using formulas in the SDPWS and were then compared to allowable stresses from the NDS. Stresses in these members were calculated to be far below adjusted design values, which is unsurprising considering the small scale of this structure. Where the rear floor rim joist was cut to allow passage of the cantilever beam, an LSTA15 tension strap and a small piece of 2x blocking were used to mend the rim joist.

2.3.4 Task 3.4: Shear Wall Design

Shear wall demand was taken directly from the diaphragms. The load path is as follows: The roof diaphragm attaches to the second level shear walls, which then transfer the shear forces down to the second floor. There, shear forces from the second level shear wall are combined with the demand from the floor diaphragm before being transferred to the first level shear walls. From there, the total combined shear forces on the structure are passed to the foundation. Between each of these components are designed connections and fasteners, which will be discussed in following sections.

Once the loading for each shear wall was calculated, the design process proceeded exactly the same as for the diaphragms by implementing SDPWS formulas and tables. All shear walls except for 7 and 8 were designed as single full-height panels for simplicity. Additional SDPWS combined uplift and shear checks were performed on shear walls 1 and 2 to reduce the quantity of steel connectors required in the uplift load path and reduce the carbon footprint of the structure. Shear walls 7 and 8 experienced the highest demand; they were designed as perforated to meet the required FS while also incorporating the lower-level windows. Though other valid design options were considered for SW 7&8, the addition of another type of shear wall to the project was deemed to be desirable for competition creativity points. Table 7, below, summarizes the selected parameters for each shear wall.

Table 7: Shear Wall Construction Parameters

Shear Wall Construction Parameters					
Shear Wall	**Shear Wall Type**	**Sheathing Grade**	**Nominal Panel Thickness**	**Edge Fastening**	**Nominal Capacity (plf)**
1&2	Single Full-Height Panel	32/16	1/2"	WSV134S at 6" OC	730
3&4	Single Full-Height Panel	32/16	1/2"	WSV134S at 4" OC	1065
5&6	Single Full-Height Panel	32/16	1/2"	WSV134S at 4" OC	1065
7&8	Perforated	32/16	1/2"	WSV134S at 4" OC	1065

Each shear wall was designed to have an ideal FS for the competition. This was possible because the demand on a shear wall depends partially on its width; by iterating different combinations of stud spacing and nailing, the capacity of each shear wall was either increased or decreased to give an ideal FS. Adjustments to shear wall capacity included an ASD adjustment factor of 2.0 and an aspect ratio factor of 0.92 for walls 1-4. Shear wall Factors of Safety are summarized in Table 8, below.

Table 8: Shear Wall Factors of Safety

Shear Wall Factors of Safety			
Shear Wall ID	**Adjusted Capacity (plf)**	**Demand (plf)**	**FS**
1&2	365.0	211.3	1.59
3&4	490.0	304.2	1.61
5&6	490.0	332.0	1.60
7&8	532.5	352.2	1.51
Average			**1.58**

Shear wall chord stresses were calculated using SDPWS formulas and were compared to allowable stresses from the NDS. Multiple checks were performed on these members, which experienced axial loading from gravity loads, vertical wind loads, and LFRS chord stresses. In the most conservative and comprehensive check, the induced tensile and compressive stresses in the first level chords were combined with MWFRS out-of-plane bending stresses for a combined axial and bending check per the NDS. In all applicable load combinations, stresses in these members were calculated to be well below adjusted design values and code checks; however, crushing of the sill plate controlled the design and required double studs in some locations to reduce crushing pressure.

Shear forces are transferred between diaphragm collectors to shear wall collectors using SDWS16300 screws. The second level wall top plate is connected to roof blocking and truss bottom chords, and the bottom plate is connected to the floor rim joists. The first level top plate is also connected to the floor rim joists. These connections were designed for both in-plane and out-of-plane shear forces. Though these screws do have significant withdrawal capacity, this was conservatively neglected when designing uplift connections.

2.3.5 Task 3.5: Uplift Connection Design

Competition rules and sound engineering design both require a complete and continuous load path for roof uplift forces to transfer to the foundation. Uplift connections were required to connect roof trusses to the second level wall and to connect the upper-level wall to the lower-level wall.

Installation speed was a key factor in the selection of uplift connectors, and preference was given to connectors with a minimum of screws. For the roof-to-wall connection, SST SDWC15600 screws were chosen rather than typical hurricane tie connections such as the H2.5A. Using the SDWC screws means that only two screws need to be installed per truss rather than 8-12 for typical hurricane ties. The SDWC screws are installed from the inside face of the studs and create a direct uplift load path to the base of the second level studs.

For the connection between levels, the second level studs were connected to the first level studs using SST MSTA tension straps. These straps are only applied to the trusses' bearing walls; the other walls receive very little tributary area from the roof and were designed to transfer that load through the floor using screws.

The MSTA straps were designed to transfer both pure C&C uplift as well as combined MWFRS uplift and shear wall chord forces. Where uplift and chord forces combined, MSTA36 straps were required; MSTA18 straps were sufficient for pure uplift.

The MSTA straps are cheap to manufacture, but they do require dozens of screws each, which makes them very slow to install in competition. Although the team originally designed tension ties between the levels which would require the installation of a single bolt per connection in competition, SST required the team to reduce cost after reviewing the plan set and construction documents. Several design changes were made as a result of this unforeseen budgeting issue.

Uplift connector demand and allowable loads are summarized in Table 9, below. All allowable loads are for connection to DF-L framing, and allowable loads for MSTA straps include a reduction factor for fastening over 1/2" sheathing.

Table 9: Uplift Connection Summary Table

Uplift Connection Summary Table		
Connector	Allowable Tension Load (lb)	Demand (lb)
SDWC15600	715	675
MSTA36	2050	2019
MSTA18	1315	574

The SDWC screws and MSTA straps are visible in Figure 3, below, along with a bulleted summary of the uplift load path.

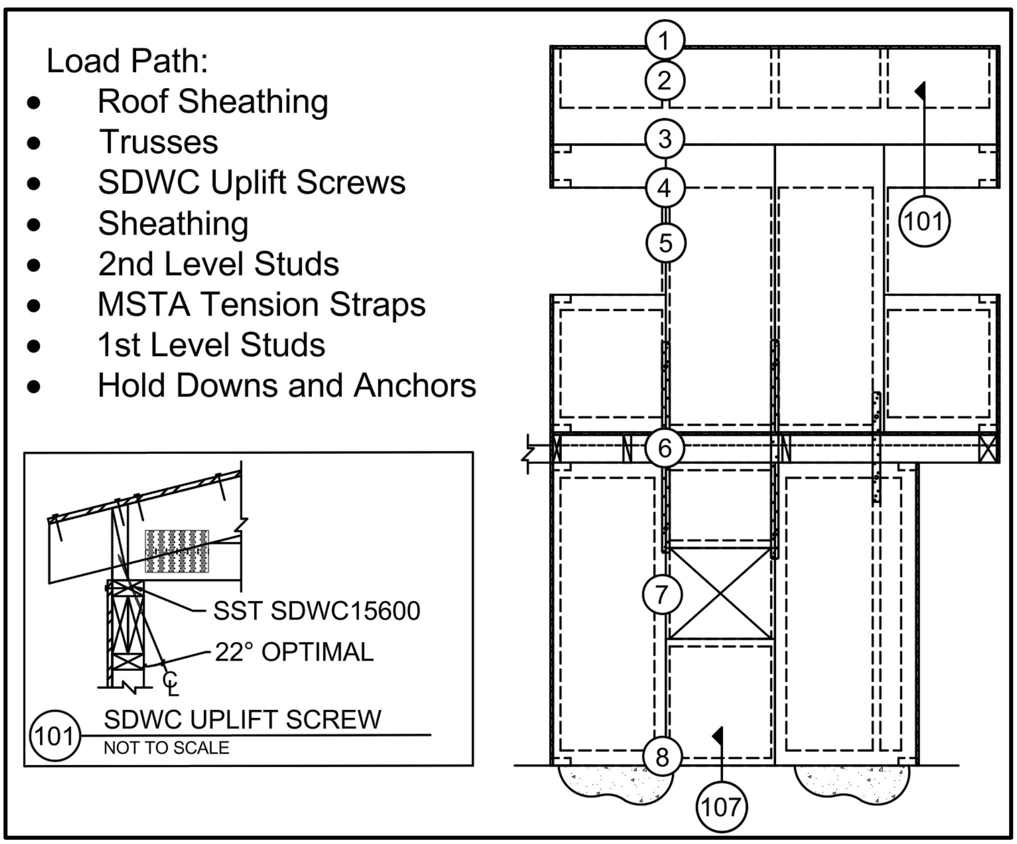


Figure 3: Uplift Load Path and Connectors.

2.3.6 Task 3.6: Tie-down and Anchor Design

Tie-downs were designed to carry combined uplift and overturning forces from each wall down to the foundation. Tie-downs were connected with SDS screws to any stud that carried uplift or shear wall chord forces. SST DTT2Z, LTTP2, and HDUE-SDS3 tie-downs were selected for this purpose. Due to Simpson's requirement to reduce cost, preference was given for these tie-downs in the order listed. DTT2Z's are the cheapest, and HDUE's are the most expensive. Figure 4, below, is excerpted from the structural plan set and depicts each of these connectors.

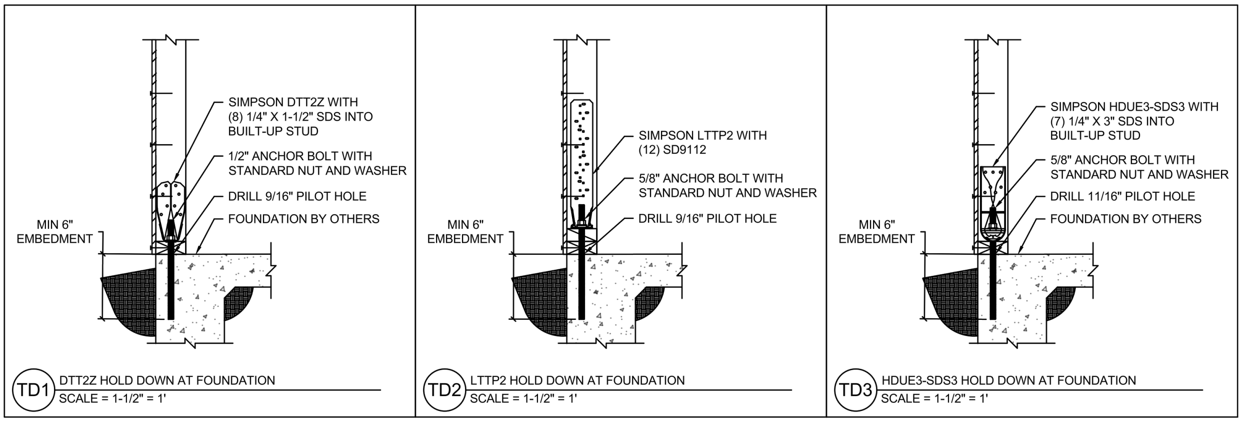


Figure 4: Tiedown Connection Details.

DTT2Z connectors were specified both at the foundation level and at Shear Wall 1, which bears on a 4x6 beam at the cantilevered end of the second level. Straps could not be used in this location due to the vertical offset of the first and second levels. The DTT2Z were connected to the 4x6 beam using lag screws, which were designed per the NDS; the lag screw withdrawal capacity exceeded the DTT2Z uplift capacity with 5" of thread penetration. The 4x6 beam was designed for the bending induced by the shear wall overturning forces, and the beam's hangers were designed both for downforce and uplift. Loads from the beam transfer into the cantilevered ends of built-up rim joists before reaching the lower-level walls.

At the foundation level, LTTP2 hold-downs were used where DTT2Z's would not suffice. The HDUE connector was only required in one location where the combined chord forces from the first and second level shear walls were above the capacity of the LTTP2.

1/2" anchor bolts were determined to have far more than enough shear and tensile capacity for all hold-down locations, but 5/8" anchor bolts were selected for the LTTP2's because their rated capacity is higher when attached using a 5/8" bolt. HDUE's require 5/8" minimum anchor bolts and must also be connected to a double-2x stud at a minimum.

Spacing between anchor bolts at tie-downs was greater than 4' in two locations, so two additional anchor bolts were designed to meet the ASCE 7 minimum requirements for anchor bolt spacing. Shear load on these bolts was negligible, but omission of these bolts would have required that the walls be designed for bending between the tie-downs.

Tie-down and anchor selections are summarized in Table 10, below.

Table 10: Tie-down and Anchor Summary Table

Tie-down and Anchor Summary Table			
Tie-down	**Anchorage**	**Allowable Tension Load (lb)**	**Demand (lb)**
DTT2Z	1/2" x 7" Lag Screw	1825	1057
DTT2Z	1/2" Bolt	1825	1712
LTTP2	5/8" Bolt	2570	2335
HDUE-SDS3	5/8" Bolt	3790	2716

The demand on these tie-downs is unusually high for a structure of this size due to two factors. Firstly, the wind pressures given in the competition rules are very high—approximately 3x greater than typical wind pressures in the Flagstaff area. Secondly, due to the relatively high capacity of available sheathing and low lateral demand from a structure of this size, contrivances were made to meet the arbitrary FS of 1.5 for shear walls. To meet this requirement, shear walls were designed to have very narrow base widths, which increased demand while also resulting in unusually high chord forces. For example, Shear Walls 3&4 have a base width of only 21 5/8" with a height-to-width aspect ratio of 2.63.

2.3.7 Task 3.7: Cantilever Beam Design

The cantilever beam consists of a 4x4 DF#2 member that has 4' 1" of the member hanging out from the back of the structure and spans across under the second floor to the front of the structure. The primary span of this cantilever is 7.33 ft, which is used in the cantilever deflection calculations. The cantilever is required to sustain a 150 lb point load at 3 possible overhang locations: 3' (L1), 3' 6" (L2), and 4' (L3). In the calculations, the beam was subjected to deflection limits, stress checks, and global stability analysis to ensure that the beam was structurally sound upon loading and met the deflection requirements. Figure 5, below, shows the visual for the cantilever beam loadings.

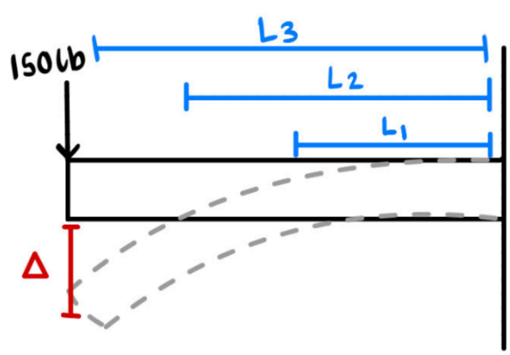


Figure 5: Cantilever Beam Deflection

The cantilever is required to maintain a member deflection between 0.5” and 1.0”. The deflection was calculated by applying the 150 lb point load (DL_C) to the cantilever deflection formula, which was then modified for the span geometry of the member. Table 11, below, shows the predicted cantilever deflections for each of the three load locations.

Table 11: Predicted deflection for each cantilever loading point.

Load Location	Predicted Deflection
L1	0.40”
L2	0.57”
L3	0.70”

Bending and shear checks were also performed to ensure that the structure could withstand the cantilever loading. The bending stress, f_b, was calculated to be 1008 psi, which was below the adjusted design value, F'_b , of 2790 psi. The shear stress, f_v, was calculated to be 18.37 psi, which was also lower than its adjusted design value, F'_v, of 288 psi. These results prove that the 4x4 cantilever beam can resist the applied loads without failure. The uplift at the opposite rim board was found by dividing the uplift, which was the 150 lb load multiplied by the load location, by the length of the rim joist. This was checked for all lengths, with the highest uplift of 82 lb at L3.

Lastly, a global overturning stability check was performed to ensure that the rest of the structure remains stable under the 150 lb point load. The total structure self-weight was calculated to be 1782.8 lb, which provided a resisting moment of 5348.3 lb·ft. The maximum moment from the 150 lb point load applied to the cantilever was 600 lb ft. This maximum moment was found by multiplying the load with the furthest possible loading location.

2.3.8 Task 3.8: Final Calculation Packet

A final calculation packet for the entire structural design is required by the competition and was submitted on the deadline of February 20th. This includes hand calculations for section sets, loads, roof truss, the cantilever beam, lateral loads, joists, headers, studs, and connection designs. Each section of the packet includes cited equations from the NDS as well as allowable design limits labeled from the SST tables. Hand drawn diagrams are included to illustrate applicable loading, spacing or load path of the design. Multiple design alterations were made due to material availability, requiring hand calculation updates throughout the design process. A combined PDF of these calculations is located in Appendix F: Hand Calculations.

2.3.9 Task 3.9: Final Structural Drawings

The structural plan set confirms that the final design adheres to all 2026 Timber-Strong Design Build rules [1] and was submitted on February 20th. These final structural drawings demonstrate a continuous load path for gravity, uplift, and lateral loads through detailed frame plans, section views, and the fastening schedule.

Table 12, below, provides a list of the sheets in the structural plan set.

Table 12: Structural Plan Set Sheet List

Structural Plan Set Sheet List		
SHEET NO.	**TITLE**	**DESCRIPTION**
1	CVR	COVER SHEET
2	GSN	GENERAL STRUCTURAL NOTES
3	ELEV	ELEVATION VIEWS
4	FDN	FOUNDATION PLAN
5	ROOF	ROOF PLAN AND CONSTRUCTION DETAILS
6	2LVL	FLOOR PLAN AND CONSTRUCTION DETAILS
7	WALL12	WALL PANELS 1 & 2 FRAMING PLAN AND SHEATHING DETAILS
8	WALL34	WALL PANELS 3 & 4 FRAMING PLAN AND SHEATHING DETALS
9	WALL56	WALL PANELS 5 & 6 FRAMING PLAN AND SHEATHING DETAILS
10	WALL78	WALL PANELS 7 & 8 FRAMING PLAN AND SHEATHING DETAILS
11	STRAP	STRAPPING DETAILS
12	DET	CONSTRUCTION DETAILS

The plan set confirms that the first-floor area is within the 6' x 8' requirement, and that the second-floor area is within the 7' 4" x 8' requirement. The floor diaphragm sheathing plan on page 6 of the plan set shows the required 4' 1" cantilever that extends from the back side of the second floor. The overall structure is within the 12' height limit and includes all required window, door, and floor openings.

The structural drawings also include schedules for the fasteners, diaphragms, headers, and shear walls. The schedule states the specific screw spacing for each wall panel, as well as the fastener type. The plans show the roof truss members, and how they are to be constructed, meeting the 30lb prefabrication requirement. Beyond technical compliance, the plan set also shows the visual details of the corner windows and roof truss, which Sky-High Structures intends to achieve creativity points from in the competition.

The final structural drawings have been subjected to change orders to ensure that all details are updated and accurate prior to Build Day. The final structural plan set drawings are located in Appendix D: Structural Plan Set.

2.3.10 Task 3.10: Building Information Model

The BIM model was developed in Revit to create a detailed 3D representation of the Timber Strong structure, including the load path, sheathing plan, and all connections and fasteners associated with the design. The model incorporated both architectural and structural components to fully represent the system and support visualization, coordination, and documentation.

The model was organized using multiple levels, including the first-floor walls, second floor, second floor walls, and the roof truss system representing the “third” level. Scope boxes and elevation views were used throughout the modeling process to improve viewing clarity and assist with placing detailed elements accurately. These tools allowed for better control of specific areas of the model when working on different components.

Most of the model was created from scratch, with the exception of imported plug ins used for fasteners and connectors. Structural members were developed by modifying existing load families and adjusting their dimensions to match the exact sizes and materials used in the design. All elements were modeled in accordance with the Timber Strong competition rules to ensure accuracy and compliance.

The structural system included walls, beams, floor systems, and roof elements, all modeled to clearly represent the intended load path. The cantilever was specifically modeled by extending a 4x4x12 ft member through the second-floor joists, accurately reflecting the design decision made by the team. Careful attention was given to alignment, spacing, and dimensions to ensure the model matched the structural plans.

Simpson Strong Tie connectors were also incorporated into the model, including hold downs, hangers, and other hardware. Placement of these components presented challenges due to the way some plug-ins were originally created, requiring adjustments to orientation and positioning to ensure they were correctly aligned with the structural members.

Multiple views were created for the submission of the file to the competition. This was to represent different aspects of the structure. These included 3D views showing all levels of the building, as well as separate architectural and structural views. An example of the structural view can be seen in Figure 6, below.

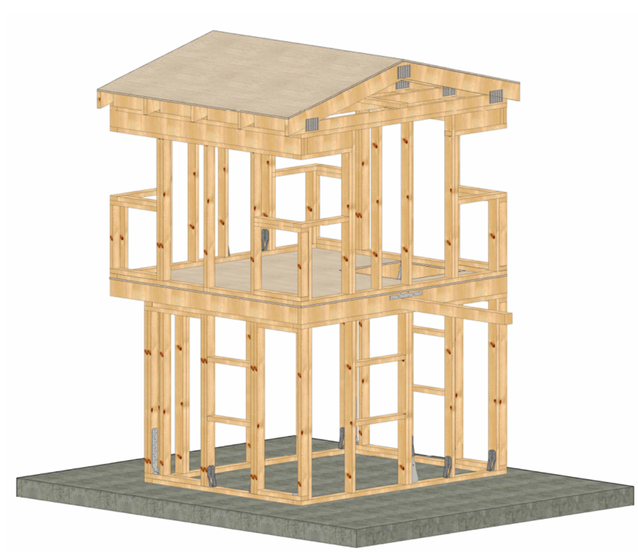


Figure 6: Structural View BIM Model

The architectural views highlighted the exterior sheathing, while the structural views focused on columns, joists, and wall panels, omitting sheathing to allow for clearer inspection of the load path and framing system.

Overall, the BIM model allowed for a complete and accurate representation of the design, improving visualization, and ensuring that both structural and architectural components were clearly coordinated.

2.4 Task 4: Pre-Competition Construction

2.4.1 Task 4.1: Material Procurement Phase

During the design process, the team had to choose members and other materials based on availability from their lumber source. HomCo Flagstaff kindly sold the material the team needed at cost, which played a significant role in guiding the overall design decisions. Rather than designing without constraints, the team had to be realistic about what materials could actually be sourced in time and within budget. In addition to basing the design on availability, the team also had to design within a strict budget of $1950. This required careful planning and prioritization to ensure that all necessary components were included without exceeding the cost limit. A full breakdown of these costs is provided in Section 4.0 of the report, which outlines the cost of engineering services and materials.

Once the design and material list were finalized, the team ordered the required lumber and coordinated a delivery with HomCo to have the materials dropped off at the Field Station at Northern Arizona University. In addition to the lumber, the team also needed to order fasteners and other hardware, which were specified to be sourced from Simpson Strong-Tie, one of the competition sponsors. However, the team encountered complications with this sponsor, as the allotted donation budget for each team was not clearly defined in the competition rules. Because of this, the team had to revise several aspects of the design after the first submission of construction documents to the sponsor.

This caused significant overages on the predicted design hours and also set the team back approximately two weeks in the pre-construction phase. The delay was not only due to the time required for approval, but also the additional wait time for the sponsor to process and ship the materials. This impacted required the team to adjust their schedule moving forward in order to meet deadlines.

2.4.2 Task 4.2: Prefabrication Phase

Despite the late start, the team made up for lost time, and the prefabrication proceeded faster than planned. The total prefabrication time was initially estimated to be 29 days, which would have left 18 days for mock assembly practice leading up to competition. The team worked hard to accelerate this schedule, and ultimately managed to complete the prefabrication in 19 days.

Although problem-solving and adaptation was required at every stage of this task, the team ultimately produced a structure which exactly matched the plan set. This accomplishment is a testament to both the quality of the plan set as well as the team's patience and attention to detail on the job site.

2.4.3 Task 4.3: Mock Assembly and Practice

Following the prefabrication, the team immediately began practice assemblies. Over the course of just one week leading up to transportation preparation, the team fully assembled the structure three times—approximately half of what was initially planned for.

Before the first assembly, an "assembly plans" document was drafted which showed dimensions of the competition assembly area and proposed materials layout, as well as a step-by-step process for each person on the team for every phase of assembly. Development of the assembly plans was not an anticipated task, but the written plans proved to be invaluable for the practice process.

The first assembly took approximately three total hours, with one hour and twenty-two minutes being "on the clock." During this assembly, the team constructed "helper blocks" which aided floor alignment and mounting of the final second floor wall panel. The team also incorporated a string of measured length into one corner of the structure which permitted rapid and accurate squaring of the lower level before the floor panel was lifted. Following this practice assembly, the assembly plans document was updated with new materials layout geometry, wall panel staging order, and individual tasks.

Following the updated assembly plans, the second practice assembly took nearly exactly the estimated build time: a total of fifty-five minutes and ten seconds. The team worked very fast in this assembly, and it proved that the goal time of fifty-five minutes was attainable. Only minor adjustments were made to the assembly plans following this practice assembly.

The final practice assembly was completed approximately ten minutes under the estimated build time. Building at that rate allowed the team extra time for troubleshooting and QC on competition day.

2.4.4 Table 4.4: Transportation Preparation

The team prepared the materials needed for the competition by creating a detailed checklist of everything required. This included but was not limited to: the pre-assembled pieces of the structure; all necessary hardware and equipment; and other miscellaneous tools. Having a checklist was important to make sure nothing was overlooked, especially given the time constraint during the competition and the need to stay organized and efficient on site. This preparation helped ensure that the team could focus on construction rather than dealing with missing materials or tools.

In addition to organizing the materials, the team also had to coordinate with the client and the lab manager to secure an appropriate trailer for transporting the structure and all associated materials. The originally available option was not suitable for safely transporting all components, so a different type of trailer had to be arranged to accommodate the size and quantity of the items. This required additional planning and communication to make sure everything could be transported in one trip and arrive safely.

The team also needed to obtain additional ratchet straps to properly secure the structure and its individual pieces to the trailer. This step was critical to prevent any movement or damage during transportation. Ensuring that all components were tightly secured not only protected the materials but also maintained safety during transit.

2.5 Task 5: Competition Construction

2.5.1 Task 5.1: 90-Minute Competition Construction Process

The 90-minute phase was the maximum amount of time a team was allowed to build their structure within. The Sky-High Structures team traveled to the University of Utah, and the team consisted of Heavenlee Seria, Rivka De Conto, Sydney Gibson, and the team's two mentees Tommy Strange and Landen Bell. Zac Timmons was unable to attend the competition, and after the team captains meeting with the ISWS and ASCE coordinators, the team realized they needed a sixth person to hold the bottom of the A Frame ladders whenever someone was climbing or occupying the ladder. To address this, the team asked another ASCE member to assist specifically with ladder holding during the competition.

The team successfully built their structure in their estimated time of 55 minutes. After construction was complete, the team moved into testing the deflection of their cantilever beam. The location for the deflection test was determined by a dice roll, and the number rolled was one, which corresponded to a load placement at 3.5 feet. The calculated deflection was 0.57 inches, and the actual measured deflection during the competition was 0.60 inches, showing very strong accuracy between design and performance.

The team did receive some deductions during the build phase. There were three instances where either equipment, team members, or parts of the structure went out of bounds of the 18 by 18-foot taped area, which was not allowed under the competition rules. Additionally, the team was deducted for not having a high visibility marker to indicate the end of the cantilever beam during construction.

Overall, the team placed first, with scoring based on the accuracy of calculations, the precision of the BIM model and structural plans, and the quality of the final report. The design also stood out as the most unique, among the eight competing schools, due to the cantilevered window systems that the team implemented into the design. Which helped the team earn additional points for creativity and overall aesthetics. The team was awarded not only a trophy but also a sum of $1000 that will be allocated to the budget of the Timber-Strong Team that competes next year.

2.5.2 Task 5.2 : Deconstruction Phase

After the deflection test, the team deconstructed the structure. Sydney Gibson was elected as the safety officer, and she was allowed to participate as long as she remained on the floor at all times to observe and ensure the safety of the team members working on the second floor during deconstruction.

Once the structure was fully deconstructed, the team, along with other ASCE members, assisted in transporting the materials out to the trailer. This allowed the team to clear the competition space efficiently, load everything onto the trailer, and prepare it to be towed off campus.

The structure was then transported back to Flagstaff, where it is planned to be donated to a team member. It will be reused as a community playhouse for the team member’s children and other neighbors, giving the project a continued purpose beyond the competition.

3.0 Project Impacts

3.1 Carbon Footprint Calculation

Per the competition rules, it is required that the net embodied carbon of the design shall be calculated to the scale of a full-sized building. This prompted the team to multiply all material and lumber used in the structure by 100 to simulate a larger scale. The amount of carbon stored in the structure was calculated using the WoodWorks Carbon Calculator tool. A total of 191 metric tons of carbon dioxide is predicted to be in the structure at a full-sized scale. This tool showed the total volume of wood products used, the replenishment rate from U.S and Canadian forests, weight of carbon stored in the wood, greenhouse gas emissions, and total potential carbon benefit. Below in Figure 7 shows the calculated carbon calculated data given.

Project: Timber-Strong x100
Date: March 3, 2026

Results

Volume of wood products used (m^3):
202 m^3 (7,130 ft^3) of lumber and sheathing

U.S. and Canadian forests grow this much wood in:
1 minutes

Carbon stored in the wood:
191 metric tons of CO_2

Avoided greenhouse gas emissions:
407 metric tons of CO_2

Total potential carbon benefit:
598 metric tons of CO_2

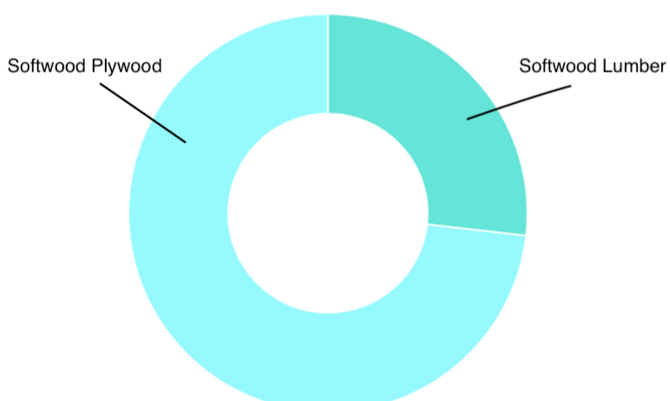
Equivalent to

126 cars off the road for a year

63 homes energy use for one year

Figure 7: WoodWorks Carbon Stored

The net embodied carbon was calculated using the Structural Engineering Institute 2050 (SE 2050 ECOM) Embodied Carbon Estimator. The total carbon stored in the structure is estimated to be 191 metric tons, with an embodied carbon total of 26.5 metric tons. Below in Figure 8 shows the carbon totals with a pie chart representing the makeup of materials in the structure.



Embodied Carbon Area	
Total Area (ft^2)	4,800
Total Area (m^2)	445.92

Embodied Carbon Totals	
Total Impact (lb CO2e)	58,477
Total Impact (kg CO2e)	26,524

Embodied Carbon Intensities	
Intensity (lb CO2e/ ft^2)	12.18
Intensity (kg CO2e/ m^2)	59.48

Figure 8: Embodied Carbon Diagram

The total net embodied carbon of the structure was determined by subtracting the total carbon stored from the estimated total embodied carbon. This resulted in a total net embodied carbon of -164.5 metric tons of carbon dioxide, leaving the design at a negative embodied carbon. These online calculators provided were used to produce an estimate of the embodied carbon, due to this estimate it is unclear whether or not this estimate is accurate.

3.2 Triple Bottom Line Assessment

The societal impact of this project includes bringing awareness to sustainable construction regarding using wood materials. Wood is a great resource that has a reduced carbon footprint in comparison to steel or concrete. The environmental impact of this project includes carbon sequestration which promotes the use of materials that can reduce humans overall carbon emissions to the environment. The economic impact of this project includes cost awareness and effectiveness. Wood is an easily accessible material and is the cheapest out of the construction materials. The global impact of this project includes pushing the idea of sustainable construction worldwide. Inspiring other engineers to design a building that promotes carbon mitigation and reducing the overall carbon footprint impact on the environment.

To fully assess the design's impact on the society, environment and economy, the team compared two scenarios of constructing this design for competition purposes versus not building this design at all. The positive and negative impacts of each alternative were taken into consideration, comparing its impacts on people, planet and price. These subjects were also further identified as global or cultural impacts.

Below in Table 13 is the impacts for each alternative identified.

Table 13: Project Impacts

Project Impacts				
Alternative		**People**	**Planet**	**Price**
[Alt 1] Design built for Competition	**Positive Impact**	Advanced techniques needed for construction process. Cultural impact as this provides creativity and design innovation.	Entire design is built using timber throughout construction. Global impact as it promotes climate change mitigation.	Prefabrication of wall panel, floor diaphragm and trusses. Cultural impact as it shortens construction time and reduces labor costs.
	Negative Impact	High risk of construction errors due to complexity. Global impact as design should rely on reliability and quality.	Competition requires more steel straps installed for hypothetical weather conditions. Global impact as it effects the total embodied carbon of design.	Design requires specific machinery for pressing truss plates. Global impact as this can increase the value of engineering design costs due to machinery needs.
[Alt 2] Design not built at all	**Positive Impact**	Advances structural techniques needed for construction process. Cultural impact as it provides students with design and creativity experience.	No resource extraction. Global impact as it does not create a carbon footprint.	No material procurement needed. Cultural impact as no extra costs would need to go into construction.
	Negative Impact	No physical build of the structural plan. Cultural impact as it doesn’t educate students in understanding the full construction processes.	Not using timber wood for material. Global impact as this could instead promote engineers to design with concrete or steel.	Increased design programming. Cultural impact as this increases an engineers design labor costs.

Each component of these scenarios is scored relative to one another. The total is then compared to the designs sustainability index (SI). A numerical score for each alternative as well as a explanation for each is displayed in Table 14 below.

Table 14: TBL Component Scoring

TBL Component Scoring						
	People	**Planet**	**Price**	**Total**	**Max-Min**	**SI**
Alt 1	**Score = 80** Provides advanced construction techniques and experiences for people.	**Score = 30** Promotes sustainable construction using wood Timber, however a lot more steel strap connectors than necessary.	**Score = 40** Prefabrication shortens labor costs. High equipped machinery for trusses is necessary.	**150**	**50**	**100**
Alt 2	**Score = 60** Provides technical experience for students with design and calculations.	**Score = 90** No build avoids complete environmental impact.	**Score = 80** Not constructing avoids construction material costs. Less construction experienced engineers may use more materials in future designs.	**230**	**30**	**200**

The alternative with a higher sustainability index is preferred for this design as it's the balance between each component. After further triple bottom line scoring it is analyzed that this design should instead be designed for competition, but not built.

4.0 Summary of Engineering Work

Table 15, below, show a summarized comparison of the proposed versus the actual work of each position.

Table 15: Summary of predicted versus actual hours completed by each position.

Task	TOTAL	
POSITION		
	Predicted	Actual
SPM	56	67
SUP	83	113
PrE	169	305
HSS	73	95
EIT	211	422
INT	86	103
Total	678	1105

The project began on September 2, 2025 and will be completed by May 5, 2026. The original Gantt chart, shown in Appendix B is a visual representation of the timeline, which includes the design brainstorming, structural analysis, prefabrication, construction, and submittal milestones. The updated Gantt chart in Appendix B reflects the changes discussed above and shows that all deadlines were still met on time.

Changes in the final design and challenges with material procurement caused overages in predicted personnel hours, altered the project's schedule, and significantly delayed the start of the prefabrication. Another factor which partially contributed to overages in the allotted design hours was that changes in lumber availability occurred in the later phases of the design process, which required some redesign work.

As mentioned in Section 2.4.1 delays in material procurement from HomCo Lumber & Hardware and Simpson Strong-Tie resulted in a shorter prefabrication time of 19 days, which was over a week shorter than initially planned. This avoidable delay was caused by lack of communication in the competition rules regarding the budget for the Simpson Strong-Tie materials. This led to the team having to redesign connections and fasteners in order to reduce cost of these materials by over 60%. The rework included recalculating structural elements, updating CAD drawings and structural plans, modifying the BIM model, and submitting a revised report. Additional overages in the design phase are a result of a complex design, suggestions from the TA for calculations beyond the scope of the competition, and design changes after the original submittal.

Due to the shortened prefabrication time, the team increased the frequency and duration of meetings for prefabrication. In addition to the regular meetings on Tuesdays and Thursdays, the team extended prefabrication to Saturdays and additional weekdays when most members were available. This also accounted for time needed for competition practice, as the team was able to complete three practice builds to ensure that the competition build was able to be completed within the 55-minute timeframe.

Throughout this period, quality assurance was used to ensure that all parts were fabricated correctly. This included measuring each member twice to ensure that it was only cut once. This ensured that the limited materials were used appropriately and not wasted. Quality control allowed the structure to be assembled according to the construction plans. This included another team member re-measuring all dimensions before final fasteners are placed. Despite these challenges and schedule changes, the client was kept informed throughout the project completion and all deliverables were submitted on time.

The only task which took less time than originally planned was project management, as the team was very organized and maintained consistent communication on project progress. Ultimately, the actual number of hours spent on this project for each position was approximately 60% greater than the predicted time. Included in Appendix G: Proposed and Actual Work Hours, is a full table comparing the proposed versus actual work.

5.0 Summary of Engineering Costs

The cost of engineering services is presented in a budget table included in Appendix E: Cost of Engineering Services. This table above provides a detailed breakdown of the overall project costs and is intended to clearly show how the team allocated resources throughout the project. The table includes, but is not limited to, the cost of all materials used for construction, ensuring that every purchased component is accounted for.

In addition to material costs, the table also outlines the cost of all types of labor and overhead associated with the project. This includes both direct labors involved in construction and any additional time spent on planning, coordination, and preparation. Overhead costs were also considered to reflect the full scope of effort required to complete the project.

Furthermore, the table includes the cost of travel, which was necessary for transporting materials, equipment, and team members to and from the project site and competition location. Including these costs ensures that the budget reflects a complete and realistic representation of the total engineering services required.

The predicted proposed cost for the project was $95,072. while the actual proposed cost came out to $157,551 and after donations of the SST connectors, a privilege specifically for the Competition Team, the cost is $156,685. This difference was primarily due to the actual cost of hours spent on the project compared to the proposed hours of personnel. The total proposed personnel hours were 678 and the actual was 1105 total personnel hours. In addition, the cost of lumber was also higher than what was originally estimated. Finally, the design of the structure required more expensive types of lumber than was originally planned for.

A condensed version of the cost of engineering services is shown below, while a more detailed breakdown including what was purchased and donated can be found in Appendix E: Cost of Engineering Services.

Table 16: Condensed Version: Actual Cost of Engineering Services

Category	Description	Quantity	Unit	Price Per	Cost
Personnel	Senior Project Manager	67	Hr	$220	$14,740
	Project Engineer	133	Hr	$165	$21,945
	Superintendent	305	Hr	$200	$61,000
	Safety Officer	95	Hr	$80	$7,600
	Engineer in Training	422	Hr	$85	$35,870
	Construction Intern	103	Hr	$60	$6,180
	Competition Labor	5.52	Hr	$50	$276
	Subtotal Personnel				**$147,611**
Travel	Rental Van	5	Days	$74	$368
	Driving Mileage	500	Miles	$0.4	$205
	Per Diem	20	People-Day	$60	$1,200
	Hotel Room	16	Room-Night	$300	$4,800
	Subtotal Travel				**$6,573**
Lab Use	"The Farm" Field Station	14	Days	$100	$1,400
	Subtotal Lab Use				**$1,400**
Materials	Lumber	1	Lump Sum	$422	$422
	4x8 15/32" Plywood Sheathing	25	EA	$27	$676
	Misc. Fasteners and Hardware	1	Lump Sum	$3	$3.44
	SST Connectors	1	Lump Sum	$865	$865
	Subtotal Material Cost				**$1,967**
Total Proposed Cost of Engineering Services					**$157,551**

6.0 Conclusion

The project successfully met the original objectives, which were to design and construct a two-story, small-scale, light-framed wooden structure that was cost-efficient, sustainable, and durable, all while satisfying the design and construction constraints. The final build also demonstrated high levels of originality, and creativity, and structural design difficulty, which made it stand out at the ISWS Conference. Through detailed structural analysis, the team was able to incorporate advanced framing concepts, premanufactured roof trusses, and cantilevered corner window headers to obtain an advantage in the competition. The final structure met all loading requirements, including dead loads, live loads, wind loads, and cantilever point loads, ensuring its safety and reliability. Despite challenges with material procurement and schedule compression, the team was able to complete the build exactly to specifications within 55 minutes, which was well under the 90-minute competition time limit. This efficient execution, combined with the design challenges and creativity mentioned above, allowed the team to stand out at the competition and win first place overall. This project not only fulfilled its technical and aesthetic goals, but also demonstrated the importance of teamwork, adaptability, quick problem solving, and organized project management to deliver a successful build.

7.0 References

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