CEE:4850 Project Design and Management

Jefferson County Conservation Area Campground Redevelopment Design Report

May 16th, 2025

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Section I: Executive Summary

Since 1972, Jefferson County Conservation has stewarded more than 1,400 acres and 12 different sites in southwest Iowa. Its most popular park is Jefferson County Park, which has a network of trails and a campground. Our team of University of Iowa Civil and Environmental Engineering students is excited to share this proposal for the Jefferson County Conservation Area Campground Redevelopment. This is a multi-faceted project that aims to meet the need of Jefferson County Park's many visitors.

Our design can be broken down into singular design elements. The scope of the design our engineers provided is as follows: two year-round cabins, a multi-use lodge, extension of key connecting roads, parking lots to serve the new buildings, a playscape area, extension of utilities for updated structures, and a grading plan.

Year-Round Cabin Design

Our design includes two identical cabins for year-round rental, both incorporating a green 24-gauge metal gable roof at a 9"/12" pitch. These roofs will incorporate a 1' overhang, and a 2' overhang over the largest windows, which are designed to emphasize the beauty of the park. The West and North cabins face toward their respective views of the park. The exterior walls will utilize a darker wood siding to blend well with the park's natural environment. Each cabin will feature a two-bedroom space, kitchen, living room, bathroom, mechanical room, and a lower walk out patio. They are also designed with a wraparound porch that has an ADA accessible ramp. Each cabin is 1818 square foot, large enough for parties of 8 to 10 guests.

Multi-Use Lodge Design

Our client requested we focus on a multi-use lodge that can host over 300 visitors for special events, including weddings. It can also serve as a nature educational center. The lodge design is a 2-story 24-gauge metal roof building, with a wood siding exterior and multiple wood columns, including two craftsman columns with stone facade at the front entryway. The lodge's main focal point is the 4,600 square foot event center, that emphasizes the natural views of its surroundings via large windows. We included ADA-compliant bathrooms and a front entry that allows visitors to access any portion of the building. The design also includes an elevated wood on at the upper floor and another porch for the lower floor.

The elevated porch is designed with multiple structural support columns. For practical purposes, a storage room was designed to hold tables and chairs for events, as well as other items. The lower floor includes a second, smaller event space, a bathroom and a mechanical room to house utilities.

Key Connecting Road Extensions and Design

Another focal point in this design is the extension of paths and roadways within the site itself to connect the new amenities our engineers designed. A 24’ concrete road connecting from the existing Prairie Trail campground extends west all the way to Jefferson County Co. Park Dr. This road was designed so that visitors could enter either from Key Blvd. or from Jefferson County Co. Park Dr. so that accessibility and travel were improved for the site. Another extension road in a teardrop shape was designed for the proposed cabins at the southern portion of the site. In addition to providing cabin visitors with easy access, this road incorporates connections to the parking lot .

Parking Lots for Proposed Amenities

The cabin parking lots are designed for a five-vehicle lot, including a space for ADA accommodations. The multi-use lodge parking lot accommodates 80 parking spaces in total, with four ADA parking spaces. The parking lot will incorporate green space islands to bring in more natural areas with the parking lot. The parking lot for the lodge features a main lot and a side lot.

Playscape Design

The playscape area design proposes a rubber foundation with features including seesaws, natural stones, small wooden structures, a wood balance beam, a sand pit, and a spider-web climbing net. These structures help emphasize ground play activities, and aim to meet ADA accommodations. The playscape will also include benches for parents to sit at to watch their kids, as well as trees and plants for a more natural environment. Finally, some light posts were recommended so that during low-light hours the playscape can be lit up for practical use.

Drainage Plan Design

It is imperative that the design includes a drainage plan for the new proposed amenities. The parking stalls will incorporate permeable pavers so water will flow to these spaces with a sloped surface. The green island spaces will incorporate a curb inlet to allow water to flow into them. Some subdrains are existing in the Prairie Trail Campground, and our engineers proposed adding some subdrains along the proposed roads.

Grading Design

One challenge our engineers dealt with was the slopes on the site. To deal with this, a grading design was proposed where new amenities would be placed, and an outlined estimate of cut and fill for the site was determined. More of these calculations and reports can be found in Appendix F.

Project Cost

Through analysis of the current market and other factors impacting material cost, our final total cost estimate for the Jefferson County Conservation Area Campground Redevelopment was $10,726,000.00. This estimate covers the entirety of the project including construction labor costs. The analysis was done using labor and material cost estimation from RSMeans, and site and utility cost used Iowa Public Works Service Bureau. More of this analysis can be found in Appendix H.

Construction Phasing

A construction phasing plan is provided to help alleviate the total cost of the project's entirety. In Phase I, the playscape, east parking lot, Park Rd., and lodge utilities will be constructed. The estimated timeline for this phase is five months, excluding any change orders during construction. This phase will cost $1,700,000.00, which includes a 20% contingency to account for any issues found in the field and the terrain challenges for the construction of Park Rd. Phase II will account for Cabin Rd. and the two cabins, including utilities, which will take about a year to complete if both cabins are built simultaneously. Phase II will cost approximately $1,903,000.00, with a contingency of 20%. This contingency is the same as Phase I, once again due to the complexity of the terrain in the area. Finally, Phase III includes the west parking lot and the multi-use lodge. This part of the project will take the longest; we estimate a two-year construction-season schedule to finish. Due to the footprint of the building and terrain concerns, this phase will have the highest contingency at 25% and has an estimated cost of $6,769,000.00.

Section II: Organization Qualifications and Experience

We are a team of students from the University of Iowa in our capstone design course. Our members are highly skilled in many different types of technical engineering work as well as organizational and communication skills.

As our group's project manager, Gabe Baird brings leadership experience and project coordination skills. His role as a Site Superintendent Intern at Turner Construction has given him hands-on experience overseeing large-scale projects, managing subcontractors, and ensuring on time budgets and schedules which are all critical skills for keeping our design project on track. His ability to communicate with design professionals makes him able to coordinate our team and take on many design challenges. His background in transportation design at Foth has strengthened his technical experience with CAD design, roadway geometry, and civil infrastructure planning. Additionally, his internship at HNTB gave him environmental assessment experience, which is valuable for regulatory considerations and site impact analysis of our project. Beyond his professional experience, Gabe has demonstrated strong leadership as the President of the American Society of Civil Engineers (ASCE), Concrete Canoe Captain, and Corn Monument Captain. He has led executive board meetings and coordinated with industry professionals to organize all sorts of events. His ability to lead teams, manage projects, and collaborate with clients makes him an outstanding project manager for our team.

Our Structural Designer Joseph Tippett brings experience in construction engineering and project documentation, making him a valuable team member especially to the structural aspects of our project. His internship with FOXXSTEM provided him with experience in document review, RFP submissions, and client interactions, making him well-equipped in project planning and client communication. His exposure to large infrastructure projects, such as DC PLUG and Capital Grid, has allowed him to solve real-world challenges. His role as a Mechanical/Electrical Assistant at the University of Iowa's National Advanced Driving Simulator has given him experience with AutoCAD and mechanical system design. Additionally, his experience in construction with RJT Builders has given him practical knowledge of blueprint reading, structural work, and site safety which will be great for understanding the implementation of our design. With expertise in engineering software like CAD, Civil 3D, and ArcMaps, as well as his FEMA certifications, Joseph brings many structural capabilities to our team.

Our Water Resource Expert and Architectural Designer, Alex Gates, has a strong background in water resource engineering and infrastructure design, making him a huge asset to our project. His experience designing water main alignments for sanitary and storm sewers proves his ability to create and implement essential hydraulic infrastructure. His work at Foth reviewing and evaluating stormwater pollution protection plans shows his understanding of environmental considerations and regulatory compliance which will be especially important for this project. Alex's ability to develop property displays for stakeholder meetings highlights his communication and presentation skills, which will be essential for engaging with our project's client. His experience observing sanitary sewer construction and assisting in culvert design evaluation gives him a practical understanding of the construction processes. He has worked with a specialized water resources team on projects that prove his teamwork and problem-solving skills, which are critical for our project's success.

Our Survey Expert and Water Resource Designer, Matt Pauling, brings a strong background in water resource design, environmental engineering, and civil infrastructure modeling, making him a crucial contributor to our project. His experience creating hydraulic system designs using industry-standard software like HEC-HMS and HEC-RAS ensures he has the technical expertise to analyze water-related infrastructure. His work developing water and wastewater treatment systems shows his understanding of environmental efficiency. He has experience in Civil3D software through coursework as well as in his internship at McClure Engineering, which allowed him to design detailed infrastructure models and consider constraints like elevation and existing utilities. Matt's ability to analyze survey data and evaluate cost-effective solutions will further support our project. As the ASCE Student Chapter Secretary and leader of the UESI Surveying Team, he brings strong organizational and leadership skills, ensuring strong collaboration within our team.

Our Transportation and Water Resource Designer, Sydney Parks, brings a well-rounded background in civil and environmental engineering, combining technical and leadership experience, as well as a passion for community impact. Her role as a Materials Engineering Intern at Terracon Consultants has provided her with hands-on experience in construction materials testing, geotechnical observation, and project management, including proposal drafting and budgeting. Her internship with Hunter Companies allowed her to engage with clients and learn about property development which translates directly to our design project's planning. Beyond her technical experience, Sydney has demonstrated strong leadership through her involvement as Outreach Chair for ASCE and a Peer Advisor for the Iowa Engineering Student Success Team. With experience in AutoCAD Civil 3D, ArcGIS, and field testing, along with her passion for conservation and community engagement, Sydney will contribute to our team's success.

Section III: Design Services

Throughout the project, we delivered many different services to provide a well-rounded design. A summary of the full scope of work and services completed are detailed below.

Key Changes:

- The recreational pond was excluded from our scope of design, as it had already been completed by another consulting firm prior to the start of our project.
- Trails were not included in the final deliverables due to time constraints. Also, the existing site already included basic grass paths, which were considered sufficient for the current phase of design.
- Only one conceptual site plan and one building layout were submitted for client review and selection, as the team focused on refining a single, cohesive, and ADA-compliant design.
- Topography and cut/fill optimization became a key design driver, influencing building orientation, grading, and roadway alignment.

- A phasing plan was not developed, as the client prioritized cost estimation and complete design documentation over construction sequencing.

Tasks Completed:

1. Preliminary Planning and Analysis:

- Reviewed site topography, contours, and aerial imagery using GIS and Civil 3D.
- Researched zoning and building codes, ADA requirements, and environmental considerations.
- Conducted a walk-through of the site and participated in biweekly meetings with the client to gather feedback and refine direction.
- Developed a Gantt chart and two-week look-ahead schedules for internal project management.

2. Conceptual Design:

- Created one comprehensive site plan and building concept that reflected client priorities for conservation, visitor engagement, and accessibility.
- Focused on grading efficiency and road alignment, particularly in the northern portion of the site, to minimize earthwork and construction disruption.
- Incorporated two cabins to the south of the lodge/nature center to support overnight stays.
- Developed a site layout that ensured ADA compliance across the entire project.

3. Building and Architectural Design:

- Designed the lodge/nature center in Revit, including:
    - An event space for community functions and educational programs
    - A classroom for conservation programming
    - A kitchen and ADA-accessible restrooms
- Modeled structural components and layouts, including foundations, walls, and roof structure.
- Completed architectural drawing sets to meet client and instructor standards.

4. Civil Site Design and Infrastructure:

- Developed a roadway alignment that minimized cut and fill, particularly at the northern access point, based on topographic analysis.
- Designed grading across the site to ensure ADA-compliant slopes for building access, parking, and key circulation routes.
- Modeled stormwater flow and incorporated drainage features into the site using Civil 3D and Infraworks.
- Created a utility layout that included water, wastewater, and electrical services based on site constraints and building orientation.

- Ensured all sidewalk and path designs met ADA accessibility standards.
- Generated full drawing sets including cross-sections, plan and profile views, utility locations, and drainage sheets.

5. Cost Estimation:

- Developed a detailed cost estimate for all elements included in the final design, identifying key material quantities and typical unit costs.

6. Final Deliverables:

- Design Report
- Final Poster
- PowerPoint Presentation
- Weekly work logs and a finalized project Gantt chart

Work Plan

Milestone description	Assigned to	Start	Duration
Kickoff Meeting	All	1/22/2025	4
Site Analysis and Data Collection	All	1/27/2025	9
Stakeholder Engagement	All	2/3/2025	1
Create Initial Concepts	All	2/5/2025	10
Develop Design Alternatives	All	2/15/2025	7
Alternative Evaluation	All	2/22/2025	7
Environmental and Impact Studies	Gabe and Sydney	2/5/2025	14
Refine Selected Design Concept	All	2/19/2025	7
Infrastructure Layouts	Matt	3/5/2025	7
Preliminary Cost Estimation	Alex	3/12/2025	7
Prepare Overall Site Layout	All	3/19/2025	7
Address Accessibility, Parking, and Flow	Gabe	3/26/2025	7
Structural Design	Alex and Joseph	4/1/2025	27
Utility Design	Matt	4/1/2025	27
Grading, Drainage, and Stormwater Plans	Alex	4/1/2025	27
Develop Construction Phasing Plan	Gabe and Joseph	4/8/2025	20
Final Cost Estimation	Alex	4/15/2025	13
Design Report	All	4/27/2025	14
Drawing Set	All	4/27/2025	14
Presentation	All	4/11/2025	10
Poster	All	4/11/2025	10

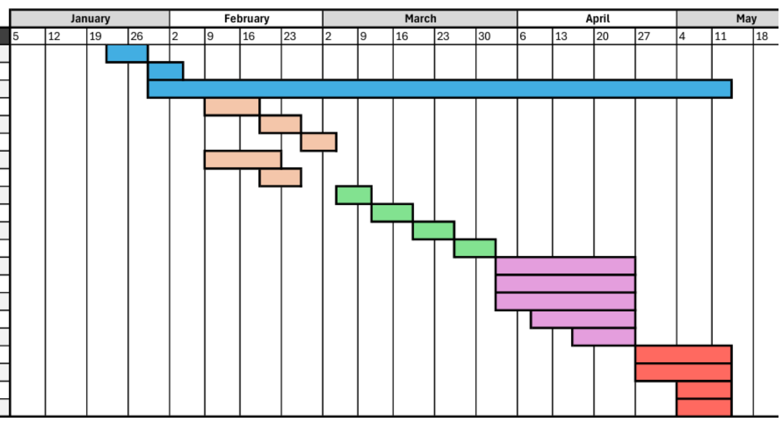


Figure 1: Project Schedule

Section IV: Constraints, Challenges, and Impacts

The Nature Center and Conservation Area project is subject to several constraints, which must be acknowledged throughout the design process. One major constraint is jurisdictional: the site falls under the City of Fairfield as well as conservation-focused governing bodies, meaning that regulatory compliance will involve multiple organizations. The timeline is also a non-negotiable constraint, with the completed design required to be submitted by April 27, 2025. Additionally, all design elements must remain within the existing boundaries of property owned by the Jefferson County Conservation Board. These constraints shape the scope of the project, guiding what is possible in terms of location, timeline, and regulatory compliance.

The project has presented several challenges, many of which have emerged more clearly throughout the design process. The team had limited prior experience with design, which necessitated steep learning curves and proactive problem-solving. One notable challenge is the lack of comprehensive soil data, which could result in unforeseen expenses if site preparation becomes necessary to support new structures. Environmental regulations, particularly those protecting local ecosystems, add complexity to the design and permitting process. Topography also presents a significant challenge, specifically the planned connecting road between Key Blvd. and Libertyville Rd must traverse steep terrain. Past concerns about construction site runoff entering the eastern pond highlight the need for robust erosion- and sediment-control strategies. Positive relationships with both the City of Fairfield and adjacent property owners are vital to avoiding conflict and building long-term support. Stakeholder engagement is another critical component. Gaining backing is essential for securing funding and ensuring continued momentum. The lodge/nature center must be spacious enough to accommodate large events, while the presence of endangered species on-site may lengthen the permit acquisition timeline. Aesthetics are also a priority as the design must complement the surrounding natural landscape. Lastly, access to the lower level of the lodge/nature center must be both ADA compliant and financially feasible, which poses an additional design challenge.

The societal impacts of the project on the Fairfield community and the broader state of Iowa are expected to be significant and largely positive. The development of the lodge/nature center and conservation area is projected to increase tourism, thereby generating additional revenue for both the park and the City of Fairfield. This growth may stimulate demand for short-term rental housing options, benefiting local property owners and the hospitality sector. Educational opportunities for local students will expand through enhanced programming and access to the site. New amenities and rentable event spaces will improve visitor experience and offer new venues for community engagement. However, these benefits will come with some impacts that must be managed. Increased traffic along Key Blvd. and Libertyville Road is anticipated, which may necessitate infrastructure updates and safety improvements. Construction activities will generate sound pollution, potentially affecting park visitors and nearby residents. The planned road connection will intersect existing trails, requiring the installation of crosswalks to maintain safe access. Additionally, this connection will narrow the drainage area east of the Libertyville Road Park entrance, necessitating careful hydrologic planning. Finally, the introduction of new facilities will place additional demands on local electrical, water, and sewer systems, requiring infrastructure assessments and potential upgrades to maintain service levels.

Section V: Alternative Solutions Considered

During the timeline for the design, our team and Jefferson County Conservation staff met bi-weekly to discuss design concepts and ideas. Throughout this process, there were multiple design options, some of which were abandoned during the selection.

Site Alternative Designs

The first decision made was the site design for the project. Our engineers proposed three different design options for the location of amenities. The first option is detailed in *Figure 2* below:

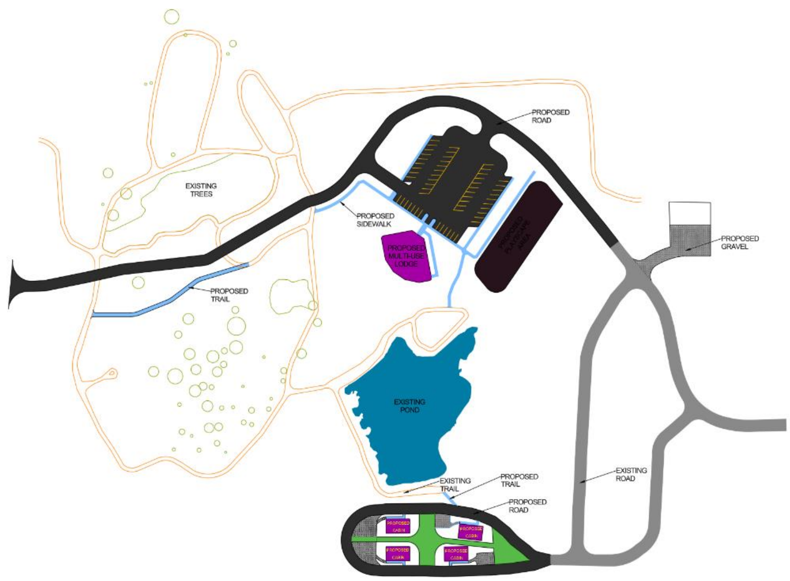


Figure 2: Site Layout Alternative Design 1

In this design, our engineers considered a lodge in a southwest orientation, and four different cabins, with a green space in between and connections to the existing trails around the pond. The parking lot was designed in the same southwest orientation, with the playscape on the southeast side. It was decided that four cabins would be too costly, so it was confirmed that only two cabins would be on site. It was also noted that the parking lot and lodge should face a north-south orientation, since the grading in southwest orientation would be much more costly and harder to design to. Alternative Site Layout 2, shown in *Figure 3,* was the next discussed site layout.

Figure 3: Site Layout Alternative Design 2

This design also proposed four cabins, which was discussed before deciding on two cabins. Alternative Site Layout 2 proposed some new ideas, including a north-south orientation for the multi-use lodge as well as the payscape on the west of the lodge. It proposed a parking lot north of the lodge. From this alternative, our firm and client agreed on having the playscape on the east side of the lodge, and a parking lot located further north for grading cost purposes. The final design, Alternative Site Layout 3, is shown below in *Figure 4:*

4: Site Layout Alternative Design 3

The final design utilized a large parking lot, and the lodge located on the northwest corner of the site. The playscape was also designed southwest of the parking lot, and two cabins were proposed on the south end of the site. Trees and bushes were proposed for the site. Both groups agreed that landscaping would be important to the design, and that the lodge would need to be closer to the pond for better views. It was reiterated that the playscape would be east, and that green space should be designed with the parking lot.

Multi-Use Lodge Alternative Designs

The multi-use lodge went through different design modifications. The main modifications highlighted through meetings with Jefferson County Conservation were room layouts and prioritization. Our engineers had three different design layouts for the multi-use lodgee, the first one is shown in *Figure 5:*

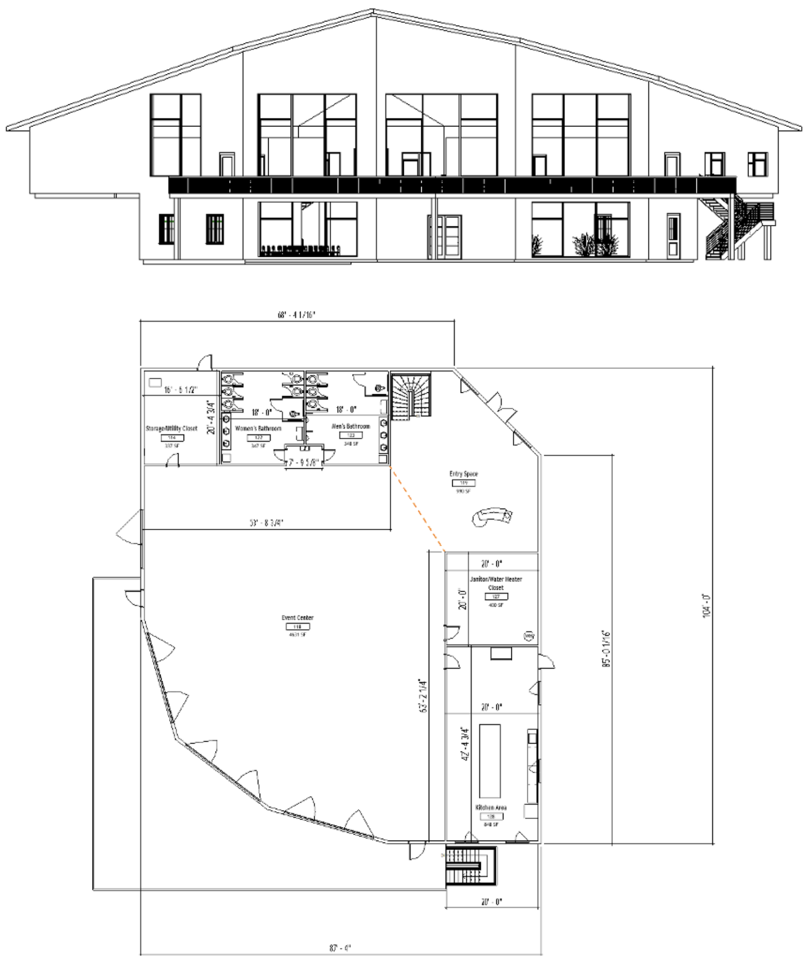


Figure 5: Multi-Use Lodge Alternative Design 1

In the first design, a large event center was emphasized including amenities like bathrooms, extra storage, a full kitchen, and mechanical rooms. We determined that it would be better to move the bathrooms out of the event center space, and that the mechanical room should be close to the rooms that need water. Because of this, the bathrooms and kitchen were moved closer together. The incorporation of the elevated porch was well-liked, as were the large windows.

Another design incorporated similar ideas with the lodge, but different floor layouts. *Figure 6* shows the second layout option:

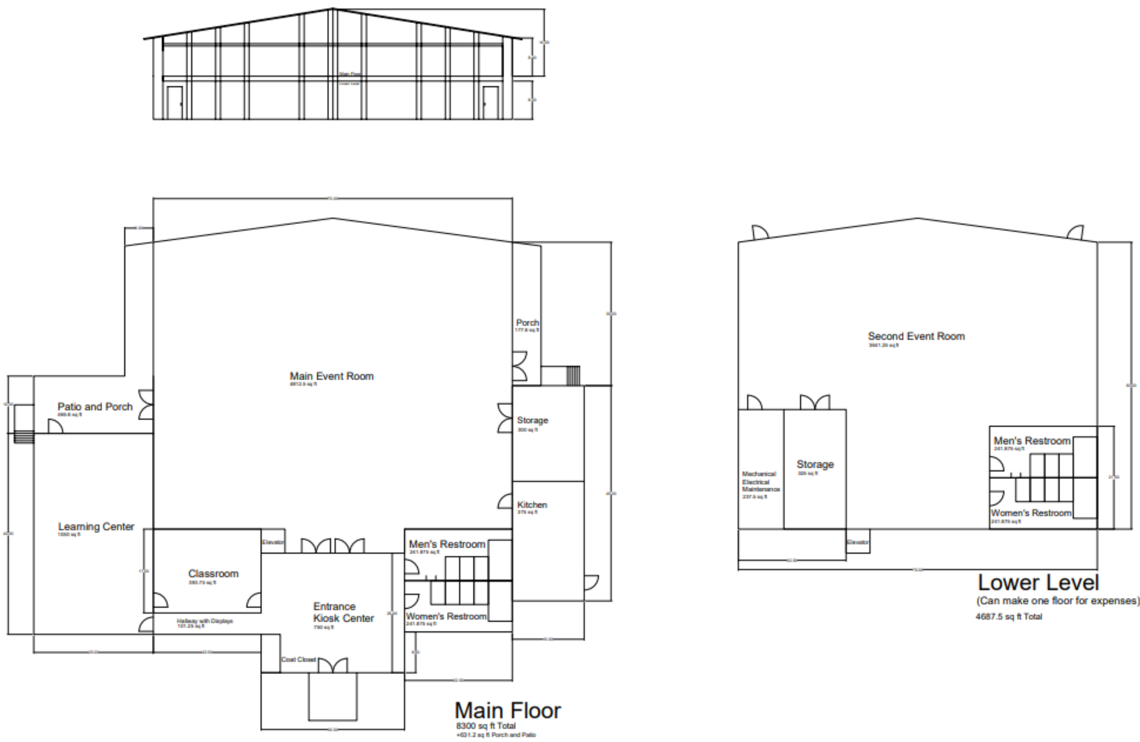


Figure 6: Multi-Use Lodge Alternative Design 2

In *Figure 6,* the event space was emphasized again, with a walkout elevated porch and large windows. A learning center, classroom, entry space, restrooms, kitchen, and storage room were included with this design as well. A lower floor space was added, which provided more bathrooms and a second event room. The storage and mechanical rooms were moved to the lower floor. From this design, it was agreed that a lower floor be added to help reduce footprint. It was recommended to move the mechanical room to a location beneath the restrooms on the upper floor. This design is the lodge design ultimately selected, with several small adjustments.

Cabin Alternative Designs

We created two design alternatives for the cabins. Our designers came up with uniquely different designs to choose from. One design, Cabin 1, was proposed as shown in *Figure 7:*

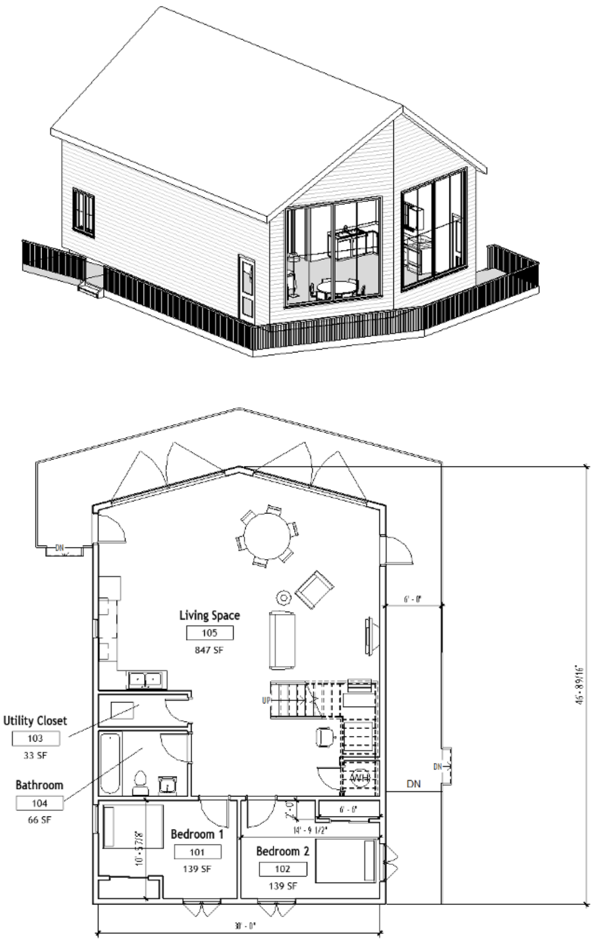


Figure 7: Cabin Alternative Design 1

This cabin included a porch with an ADA ramp, as well as a two bed, one bath design. A utility closet, bathroom, and living space was designed, featuring a lofted floor with two bunk beds for more sleeping space. The overall design was agreed upon, but the loft was removed, and a basement was added for more space and to account for the terrain change. The second cabin, Cabin Alt 2, was the second alternative as show in *Figure 8.*

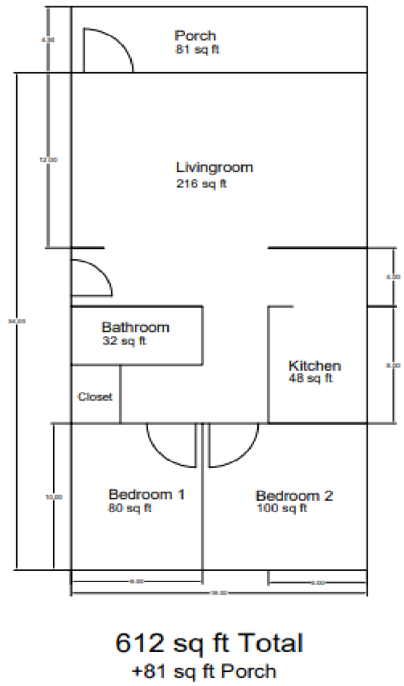


Figure 8: Cabin Alternative Design 2

The cabin features a two bed, one bathroom design, with a living room, kitchen, closet, and porch. The main concern with this design is that at 612 square feet, the space may be crammed with a constraint of 8-10 people. There was also no location for a mechanical room, and it was decided that a larger cabin would be needed.

Nature Playscape Alternative Design

Our team explored a variety of creative concepts for the alternative nature playscape design, including a unique bubble-shaped layout made from poured-in-place rubber shown in *Figure 9*.

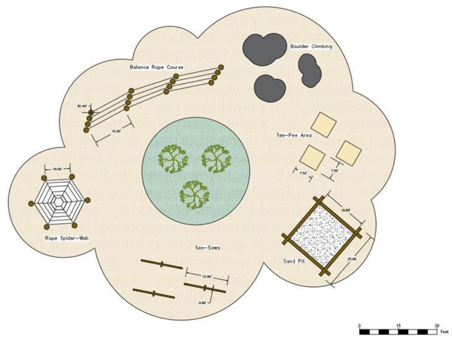


Figure 9: Nature Playscape Alternative Design

This design featured soft, flowing curves and different play zones to provide a safe, accessible play surface. While the idea offered a modern approach, we ultimately decided against moving forward with it. The cost of installing poured-in-place rubber at the desired scale and with the unique layout would have exceeded feasible budget costs. The synthetic material lacked the natural aesthetic we were aiming to create. We didn’t include seating for parents in this design or light fixtures to keep the area lit when the sun goes down.

Section VI: Final Design Details

Following alternative design selections for the design elements, we created a final design submission. All elements were designed regarding SUDAS and IBC codes.

Multi-Use Lodge Design

After going through multiple alternative designs, the final design was produced for the multi-use lodge as shown in *Figure 10:*

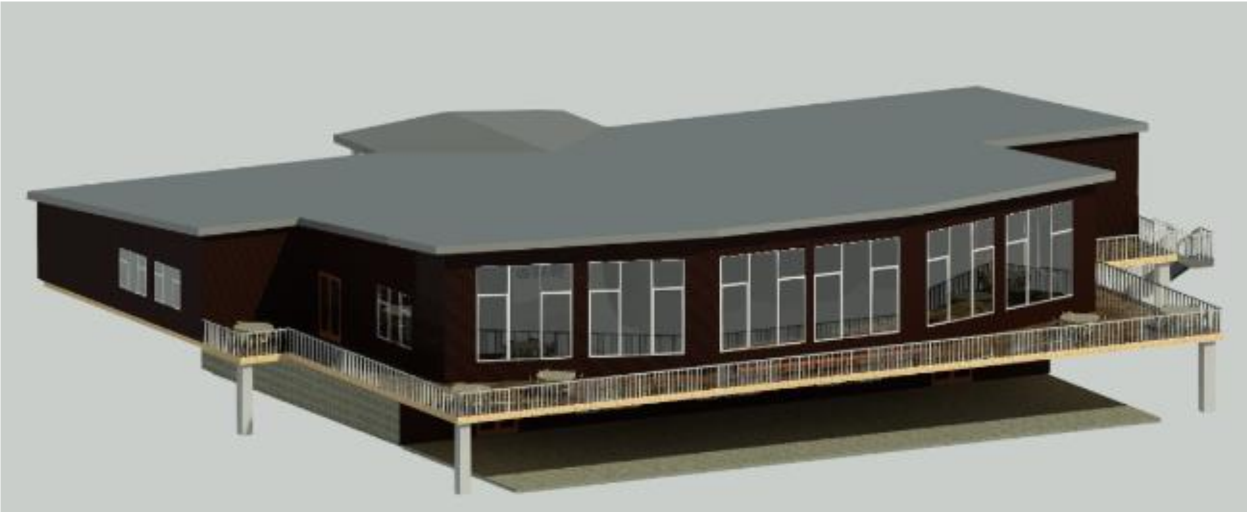


Figure 10: Final Multi-Use Lodge Design

The multi-use lodge design encompasses a large event center for special events, as well as bathrooms and a kitchen area for catering. The event space has two doors out to an elevated porch area for visitors to enjoy views of the park, as the building is situated towards the pond. The lodge incorporates large windows, two on each wall facing the south of the site. These windows will provide great views for visitors enjoying the event space. In addition to a large event space, the upper floor design also includes a classroom and nature center space for educational events and camps. The nature center has a garage door that allows for quick access. The upper floor also features a storage closet and entryway, with a connecting hallway from the entryway to the classroom and nature center rooms. The upper floor utilizes a staircase in the entryway, which leads to a lower level. The lower floor is designed with a smaller event space, to allow more visitors to rent the space, as well as a mechanical room for utilities and bathrooms. The front entryway on the exterior consists of a gable roof, with two craftsman columns. The two columns have a rock façade base, with exposed wood columns to the top of the roof at the entry roof. The roof from the main event space to the entryway will be a mono-slope design.

The multi-use lodge is designed with a 24-gauge metal-panel roof, with a light steel color to accent the dark wood siding on the exterior walls. The slope on the gable roof and mono-slope roof is 1:12.

The porch will incorporate a light-stained wood and includes a staircase on the east side of the lodge to access the lower-level patio outside. Structural columns will be placed beneath the main event space and elevated porch. The columns will be designed following ASCE 7-22 standards as to account for dead and live loads above. Steel beams with wood paneling are designed to account for roof live and dead loads in the lodge, and bracing will be provided at connections for support against shear and torsional affects. To determine what sizing of columns and beams will be necessary, structural analysis has been performed as outlined in Appendix C. The lodge will utilize a concrete-poured foundation to help support any loading and soil loading in the surrounding area.

Year-Round Cabin Design

Two cabins were designed with identical features. Each cabin features a two bed, one bath design, with a living space, kitchen, and storage closets all located on the first floor. The cabin also features a basement area, with a living space for extra room as a multi-functional option, and a mechanical closet for utility considerations. The cabin's exterior features a first-floor wrap-around porch with an ADA ramp. The basement has a walkout concrete patio to give visitors another place to relax and see the views of the park. The cabin also features two 12' 6" windows to highlight the natural views of the park. The west cabin is positioned so the windows focus on the west side of the park, and the east is positioned towards the pond on site. The final design of the cabins can be seen in *Figure 11*.

Figure 11: Final Cabin Design

The cabin has three exits, which uphold IBC code 1006.3.3 rules (see Appendix B). The ramp attached to the wrap-around porch will be 1"/12", upholding to IBC code 1012.1 (see Appendix B) The cabins will also feature an exposed King Post truss system, spanned at 7' apart along the length of the cabin. The truss system will utilize hurricane ties to brace chords together, as well as incorporate wood bracing across the truss system. The truss system is designed with Douglas Fir 5” square studs, ensuring a strong and supportive truss for the roof. The roof will be made from green 24-gauge metal paneling and insulation, ensuring that the cabin will be able to resist cold and windy Iowa weather. The roof will incorporate a gable design, with slopes of 9”/12” and include a gutter system for drainage. The exterior walls will utilize wood siding with a dark color finish, and the porch will be stained with a lighter color for a more elegant design. It is recommended that the porch wood be treated to have less maintenance needs and less weathering and damage issues over time. The foundation will be made from CMU block.

To determine what sizing would be needed for the truss system and the type of wood, structural analysis was performed by our engineers. Appendix B outlines the necessary calculations for determining the proper sizing, which is outlined in ASCE 7-22.

Roadway Design

With the additions of new amenities on site, it was imperative that surrounding infrastructure was designed to access every part of the site. To accommodate this challenge, our engineers designed two key roadways, Cabin Rd. and Park Rd., providing access locations to the cabins and the lodge. Park Rd. ties into the existing road from the Prairie Trail Campground and extends west until it ties into Jefferson County Park Dr. (see Appendix D). Park Rd. will be a 24’ PCC pavement design with a 2% cross slope. A bituminous seal coat single course is designed to help protect the underlying asphalt and will incorporate a 6” modified subbase and 6” subgrade preparation. This road also includes a 4’ type B granular shoulder and utilizes a 5’ flat ditch with a 4:1 fore slope and backslope, as specified in SUDAS guidelines. A 4” subdrain is provided underneath granular shoulder to help with drainage on the site. This subdrain is 1’ beneath the edge of pavement as specified in Iowa Department of Transportation DR-301 (see Appendix D). Cabin Rd. will be a 24’ road designed to the same standards as Park Rd. The road will incorporate a teardrop loop. This design envelopes a functional path for visitors to both enter and exit the cabin area safely and efficiently.

Nature Playscape Design

The final design for the nature playscape is shown below in *Figure 12* and includes a blend of natural materials and thoughtful layout to create a safe, engaging, and sustainable play environment. Divided into six distinct zones, the playscape features prefabricated play elements made from Robina which is a hard acacia wood known for its extreme durability, resistance to water absorption, and natural aesthetic. This material choice enhances the overall natural charm of the area while ensuring long-lasting performance and minimal maintenance.

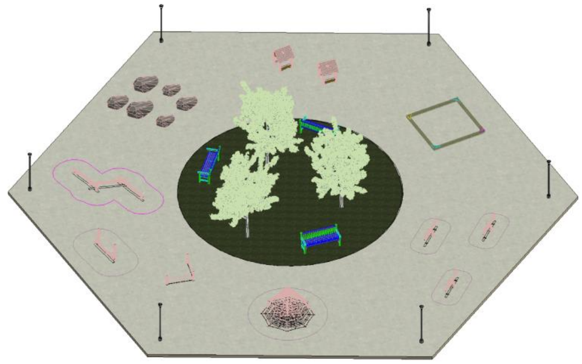


Figure 12: Final Playscape Design

The ground throughout the playscape is covered in bark mulch, offering a soft, natural look with excellent shock absorption. This material supports safe play, cushioning falls while maintaining the integrity of the natural design. Bark mulch is also more pervious than other playground materials, so it was designed to help benefit the drainage plan. To promote sustainability and manage stormwater, native grasses have been planted in a designated rain garden area. These grasses aid in drainage and filtration, reduce runoff, and create a small ecological habitat within the park.

At the center of the playscape, a shaded gathering area has been incorporated beneath the canopy of mature trees. This space features park benches for parents and caregivers, providing a comfortable and cool spot to relax while observing children at play. Altogether, the design encourages nature-based play and social interaction while supporting environmental health and durability.

Parking Lot Design

The addition of the new park amenities meant adding new parking lots. Four parking lots were designed in total for the site: a west and east lot for the lodge and playscape area, as well as two lots for the cabins. The east and west parking lots are designed to accommodate 97 vehicles in total. As per SUDAS Chapter 8C, the number of ADA accessible parking spaces in this area was 4 units, one of which is a van accessible spot. The west parking lot is designed with a front row of spaces at 90 degrees, with 9' width projection. This design upholds the regulations outlined in SUDAS Chapter 8B. There will be 45-degree spaces as well, which will incorporate 26' two-way aisles. The length of each parking space will be 18'. At each end of the two middle angled parking areas, islands with green space will be incorporated. The pavement will be made of 7" PCC, and both parking lots will include a 6" standard curb as shown in *Figure 13* from the Iowa DOT manual.

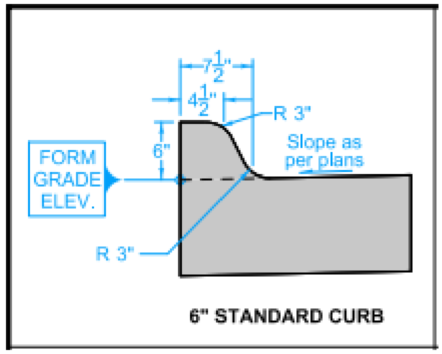


Figure 13: PCC Curb Details

The east parking lot will also use 45-degree spots, with two rows of 15 spots at 9' wide each. All parking spaces are designed with permeable pavers for drainage support. The parking lot will need cut and fill, which can be found in the "Grading Design" section and Appendix G. The east and west parking lot designs can be seen in Figure 14.

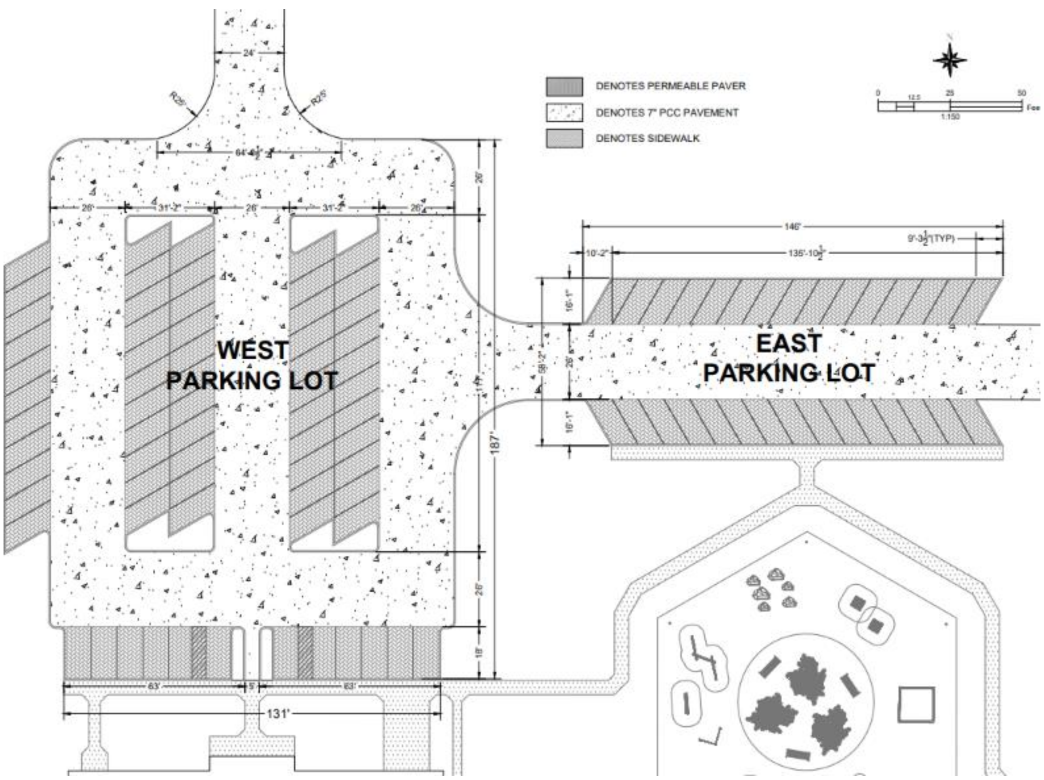


Figure 14: East and West Parking Lots

Accompanied with these two lots are two more parking lots for the cabins on the south side of the pond. These two parking lots are to be constructed from gravel to help reduce cost to the project.

Drainage Design

Drainage was an important detail considered throughout the design process. The drainage was designed using The Rational Method as outlined in Chapter 2 of the Statewide Urban Design and Specifications Manual (SUDAS).

There are different drainage designs implemented on the site. One of these is the use of permeable pavers in the parking lot. The permeable pavers will help reduce runoff thanks to their infiltrating properties and therefore have a lower runoff coefficient. The design for the permeable pavers will follow SUDAS standard specifications outline in Section 7080. *Figure 15* shows the design our engineers will utilize on site.

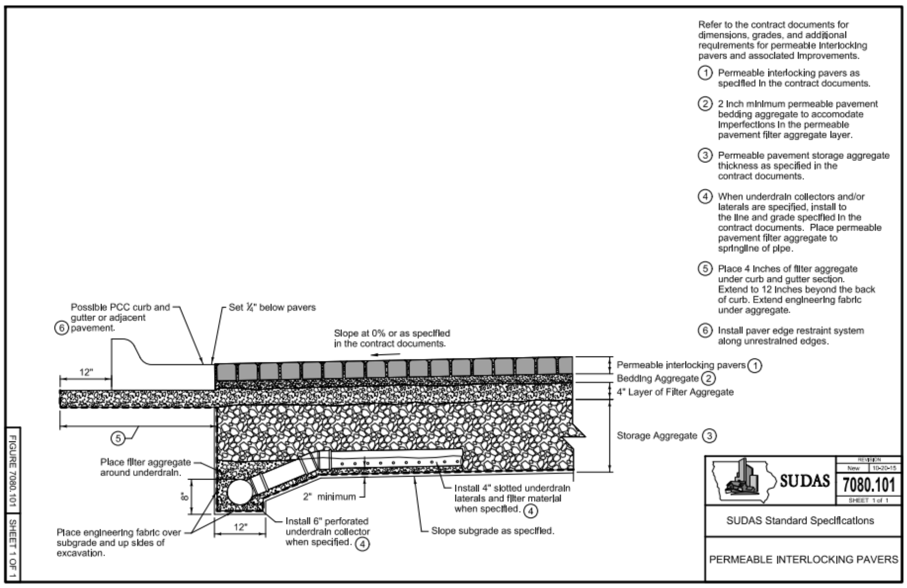


Figure 15: Permeable Paver Typical Design

For more information regarding the calculations for drainage with the pavers, please see Appendix F. Another design element that was implemented on the site for drainage is curb cuts leading into the rain gardens on the islands inside of the west parking lot. These islands will utilize native grasses to act as a conservational rain garden, improving soil stability. The rain gardens will also provide a pervious area within the parking lot design, so the stormwater will be drained to them to allow proper drainage on the site. In addition to these design elements, a bioswale is also designed in the middle of the Cabin Rd. roundabout. This area not only promotes a more natural view but also provides vegetation that can allow stormwater to infiltrate better on the south side of the site. Finally, storm drains were added throughout the site for water to exit from the surface. The locations and slopes of the site can be seen in Appendix F.

Grading Design

Terrain and elevation changes for the site were a reoccurring challenge that our engineers faced throughout the design process. It was proposed that Park Rd be built on the site , therefore proper cut and fill was needed. It was also proposed that cut and fill were needed for the parking lots and Cabin Rd. The total amount needed for both cut and fill, as shown in calculations done in Appendix G, is 21,000 cubic yards.

Section VII: Engineers Cost Estimate

Quantities were estimated based on the preliminary site plan developed for the Jefferson County Conservation project area. Key project components include the construction of two cabins, a lodge, and the installation of recreational site furnishings, such as benches, lighting poles, and playground equipment.

Dimensional takeoffs were performed using scaled plan drawings to determine approximate areas for pavement, building and playscape footprints, as well as counts for individual furnishings and fixtures. Additionally, the grading of the site requires both cut and fill of soils to accommodate site features and new structures. These measured quantities provided the basis for estimating volumes, lengths, and unit counts needed for construction.

Cost data for site design were obtained from the Iowa Public Works Service Bureau (IPWSB), which reflect unit prices submitted by contractors for similar items of work throughout the state. These values represent competitive market rates for materials, labor, and equipment within Iowa and were used as the primary basis for this preliminary cost estimate.

For the lodge and cabins, cost data was found through RSMeans data. The data provided outlines all the structural and architectural components of the design, with an inclusion of contingency and mobilization.

The playscape equipment does not align with standard bid items. These elements are categorized under special provision items, requiring unique vendor pricing and specifications not typically found in IDOT's standard bid schedule.

The general cost estimates for the Jefferson County Conservation Area Campground Redevelopment are outlined in Figure 16 through 18. Additional details on volume takeoffs, estimated unit costs, and breakdown on individual components can be found in Appendix H.

Site Design Estimate	
Item	Cost
Division 2	776290
Division 3	205008
Division 4	157050
Division 5	147168
Division 6	127050
Division 7	1033544
Division 8	49830
Division 9	115260
Division 11	501715
Special Provisions	1006800
TOTAL	4119715
20% Contingency	823943
FINAL COST	4943658

Figure 16: SUDAS Quantity Takeoffs and Cost Estimation

For the site design, a contingency of 20% was provided. This contingency accounts for unknown obstacles that could be encountered in the construction process, such as the grading issues for the roadway, unknown utility locations throughout the site, and any weather implications experienced throughout the project.

Figure 17 shows the cost estimate for the cabins and utilizes a contingency of 20%, with the same concerns as addressed in the site design. The lodge will factor in a contingency of 25% a more conservative cost estimate due to the size of the building and the more challenging terrain.

Multi-Use Lodge Cost Estimate	
Item	Cost
Structural	765748
Utilities	703595
Foundation	1681983
Architectural	163549
Excavation	152748
Mobilization	300000
25% Contingency	941906
TOTAL COST	**4710000**

Figure 17: Cost Estimation for Multi-Use Lodge

Multi-Use Lodge Cost Estimate	
Item	Cost
Structural	123323
Utilities	62009
Foundation	48816
Architectural	58326
Excavation	22912
Mobilization	48000
1 Cabin	375603
TOTAL	751206
20% Contingency	150241
TOTAL COST	**901448**

Figure 18: Cost Estimation for the Year-Round Cabins

Section IX: Phasing Plan

Due to the amount of total cost needed for this design, our firm recommends a phasing plan for the entire site design. The phasing plan is broken into three parts, which will be easier to fund for rather than an upfront cost. These three parts are shown in Figure 19.

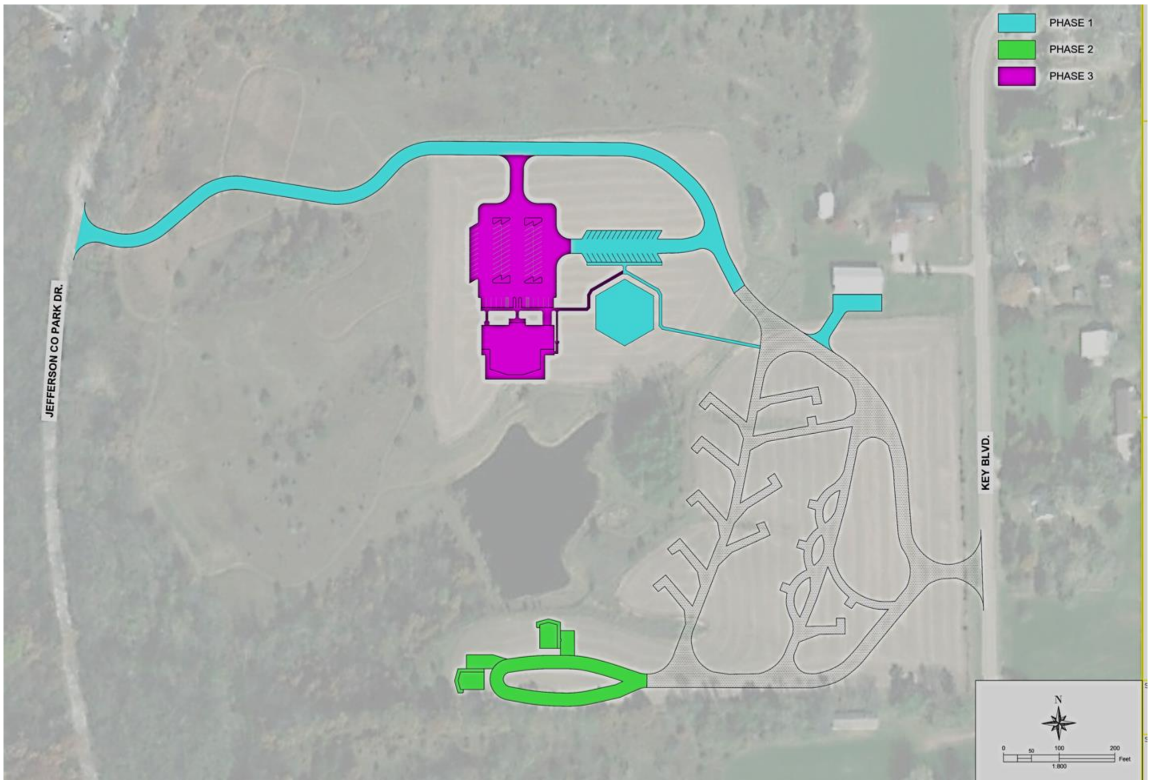


Figure 19: Phasing Plan for Jefferson County Conservation Area Campground Redevelopment

Phase I

Phase I incorporates the east parking lot, natural playscape, Park Rd., and gravel maintenance lot. These features were phased together to implement key connections for the park and increase foot traffic from the RV area. The playscape was also phased with this plan to help increase funding possibilities, as play structures are often easier to fund. This phase will also help with accessing the existing maintenance building. This phase will take one construction season to complete, and the cost to build will be $1,700,000.00. This includes a 20% contingency to account for any obstacles found during construction.

Phase II

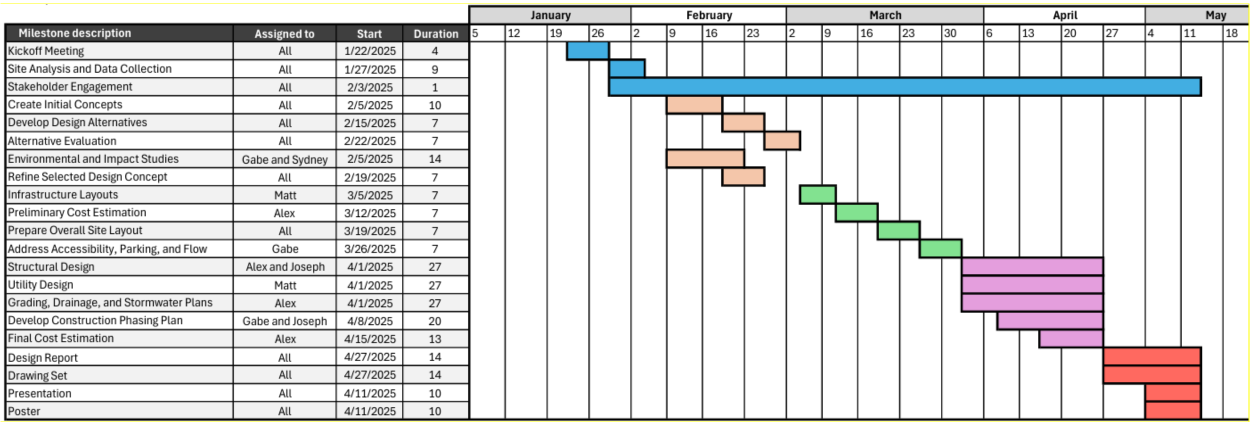
During Phase II, the two cabins and Cabin Rd. are set to be completed. These items were chosen with a goal of boosting revenue for the park. Having these cabins will not only allow visitors to stay year round but will increase foot traffic. The increase in foot traffic will help bring in more money for the park. This phase is expected to take one construction season to complete and will cost $1,903,000.00 to build. This includes a 20% contingency to account for any obstacles found during construction.

Phase III

Phase III is the most expensive and includes the cost of the multi-use lodge and the west parking lot. Due to the building's size and concerns with grading, a 25% contingency was utilized, for a final cost of $6,769,000.00. This phase will take 2-3 construction seasons to finish, barring any setbacks during construction. Although this Phase will cost more and take longer than the other two, the lodge will be the biggest provider of income from the design. The lodge itself will help bring in a lot of tourist attraction and will play a huge factor of increased engagement from the community.

Section VII: Appendices

Appendix A: Project Scheduling



Milestone description	Assigned to	Start	Duration
Kickoff Meeting	All	1/22/2025	4
Site Analysis and Data Collection	All	1/27/2025	9
Stakeholder Engagement	All	2/3/2025	1
Create Initial Concepts	All	2/5/2025	10
Develop Design Alternatives	All	2/15/2025	7
Alternative Evaluation	All	2/22/2025	7
Environmental and Impact Studies	Gabe and Sydney	2/5/2025	14
Refine Selected Design Concept	All	2/19/2025	7
Infrastructure Layouts	Matt	3/5/2025	7
Preliminary Cost Estimation	Alex	3/12/2025	7
Prepare Overall Site Layout	All	3/19/2025	7
Address Accessibility, Parking, and Flow	Gabe	3/26/2025	7
Structural Design	Alex and Joseph	4/1/2025	27
Utility Design	Matt	4/1/2025	27
Grading, Drainage, and Stormwater Plans	Alex	4/1/2025	27
Develop Construction Phasing Plan	Gabe and Joseph	4/8/2025	20
Final Cost Estimation	Alex	4/15/2025	13
Design Report	All	4/27/2025	14
Drawing Set	All	4/27/2025	14
Presentation	All	4/11/2025	10
Poster	All	4/11/2025	10

Figure 2: Project Schedule

Appendix B: Site Design

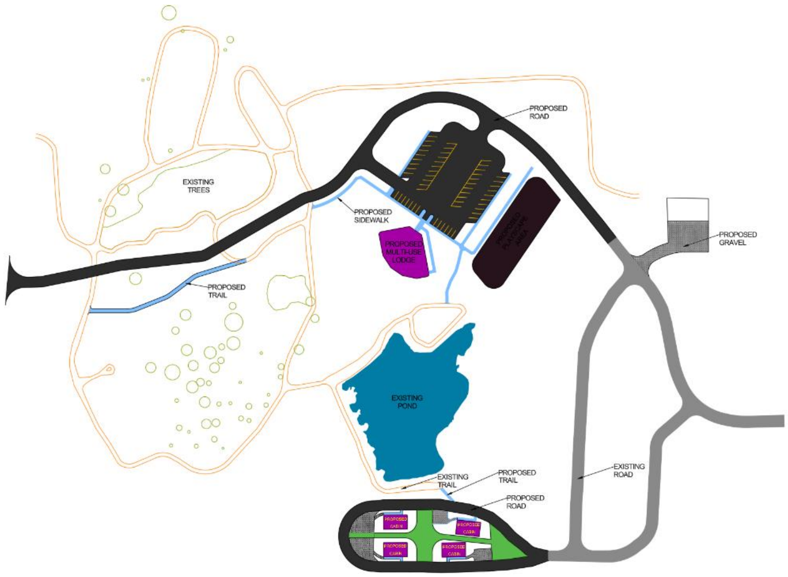


Figure 2: Site Layout Alternative Design 1

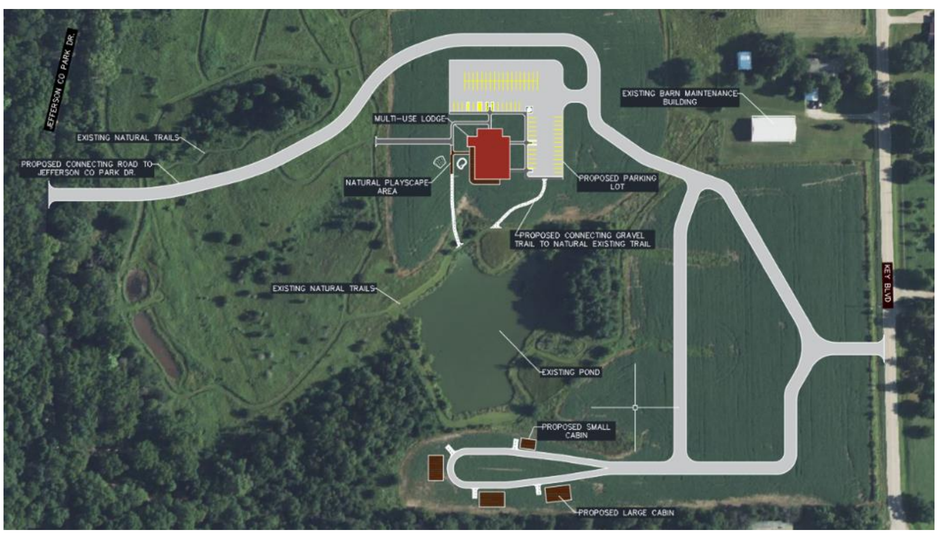


Figure 3: Site Layout Alternative Design 2

Figure 4: Site Layout Alternative Design 3

Appendix C: Multi-Use Lodge

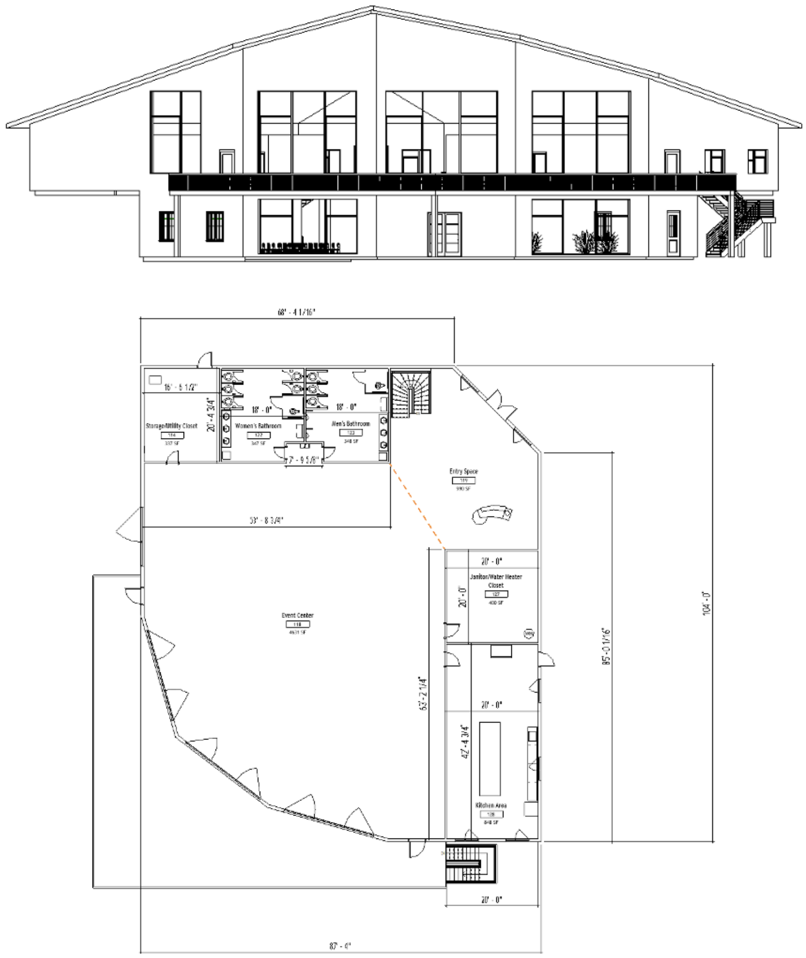


Figure 5: Multi-Use Lodge Alternative Design 1

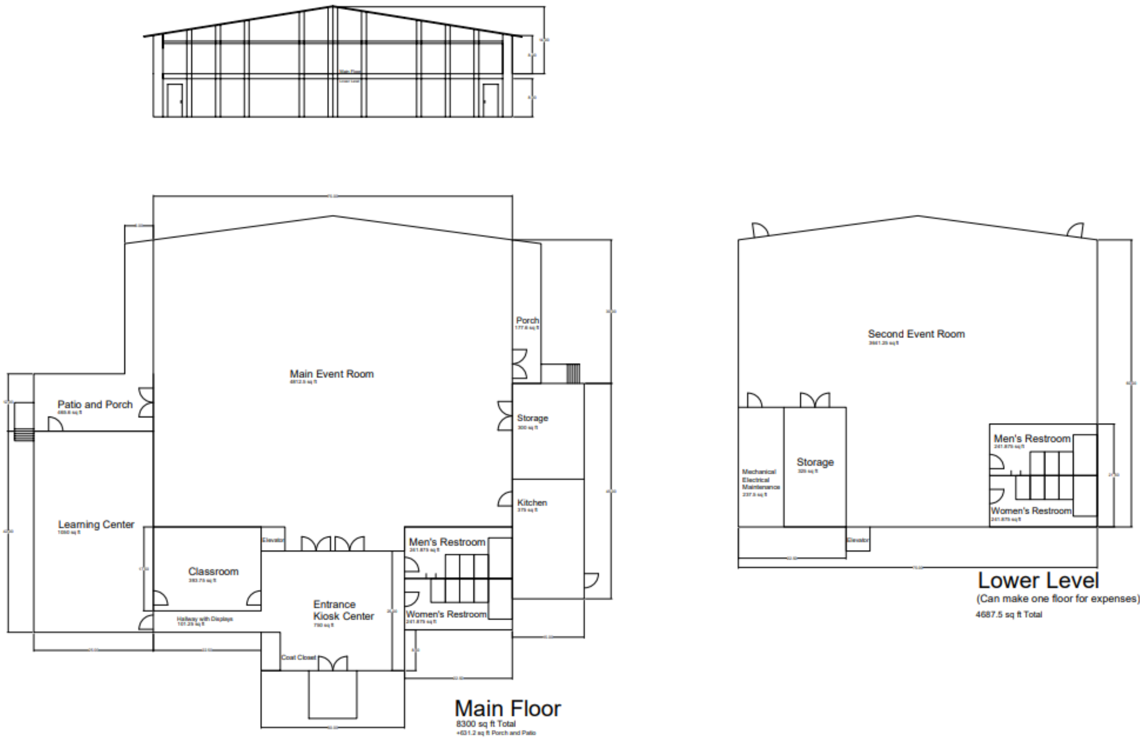


Figure 6: Multi-Use Lodge Alternative Design 1

Figure 10: Multi-Use Lodge Final Design

Table 1B Section Properties of Standard Dressed (S4S) Sawn Lumber

Nominal Size b x d	Standard Dressed Size (S4S) b x d in. x in.	Area of Section A in.^{2}	X-X AXIS Section Modulus S_{xx} in.^{3}	X-X AXIS Moment of Inertia I_{xx} in.^{4}	Y-Y AXIS Section Modulus S_{yy} in.^{3}	Y-Y AXIS Moment of Inertia I_{yy} in.^{4}	Approximate weight in pounds per linear foot (lbs/ft) of piece when density of wood equals: 25 lbs/ft^3	30 lbs/ft^3	35 lbs/ft^3	40 lbs/ft^3	45 lbs/ft^3	50 lbs/ft^3
Boards[1]												
1 x 3	3/4 x 2-1/2	1.875	0.781	0.977	0.234	0.088	0.326	0.391	0.456	0.521	0.586	0.651
1 x 4	3/4 x 3-1/2	2.625	1.531	2.680	0.328	0.123	0.456	0.547	0.638	0.729	0.820	0.911
1 x 6	3/4 x 5-1/2	4.125	3.781	10.40	0.516	0.193	0.716	0.859	1.003	1.146	1.289	1.432
1 x 8	3/4 x 7-1/4	5.438	6.570	23.82	0.680	0.255	0.944	1.133	1.322	1.510	1.699	1.888
1 x 10	3/4 x 9-1/4	6.938	10.70	49.47	0.867	0.325	1.204	1.445	1.686	1.927	2.168	2.409
1 x 12	3/4 x 11-1/4	8.438	15.82	88.99	1.055	0.396	1.465	1.758	2.051	2.344	2.637	2.930
Dimension Lumber (see NDS 4.1.3.2) and Decking (see NDS 4.1.3.5)												
2 x 3	1-1/2 x 2-1/2	3.750	1.56	1.953	0.938	0.703	0.651	0.781	0.911	1.042	1.172	1.302
2 x 4	1-1/2 x 3-1/2	5.250	3.06	5.359	1.313	0.984	0.911	1.094	1.276	1.458	1.641	1.823
2 x 5	1-1/2 x 4-1/2	6.750	5.06	11.39	1.688	1.266	1.172	1.406	1.641	1.875	2.109	2.344
2 x 6	1-1/2 x 5-1/2	8.250	7.56	20.80	2.063	1.547	1.432	1.719	2.005	2.292	2.578	2.865
2 x 8	1-1/2 x 7-1/4	10.88	13.14	47.63	2.719	2.039	1.888	2.266	2.643	3.021	3.398	3.776
2 x 10	1-1/2 x 9-1/4	13.88	21.39	98.93	3.469	2.602	2.409	2.891	3.372	3.854	4.336	4.818
2 x 12	1-1/2 x 11-1/4	16.88	31.64	178.0	4.219	3.164	2.930	3.516	4.102	4.688	5.273	5.859
2 x 14	1-1/2 x 13-1/4	19.88	43.89	290.8	4.969	3.727	3.451	4.141	4.831	5.521	6.211	6.901
3 x 4	2-1/2 x 3-1/2	8.75	5.10	8.932	3.646	4.557	1.519	1.823	2.127	2.431	2.734	3.038
3 x 5	2-1/2 x 4-1/2	11.25	8.44	18.98	4.688	5.859	1.953	2.344	2.734	3.125	3.516	3.906
3 x 6	2-1/2 x 5-1/2	13.75	12.60	34.66	5.729	7.161	2.387	2.865	3.342	3.819	4.297	4.774
3 x 8	2-1/2 x 7-1/4	18.13	21.90	79.39	7.552	9.440	3.147	3.776	4.405	5.035	5.664	6.293
3 x 10	2-1/2 x 9-1/4	23.13	35.65	164.9	9.635	12.04	4.015	4.818	5.621	6.424	7.227	8.030
3 x 12	2-1/2 x 11-1/4	28.13	52.73	296.6	11.72	14.65	4.883	5.859	6.836	7.813	8.789	9.766
3 x 14	2-1/2 x 13-1/4	33.13	73.15	484.6	13.80	17.25	5.751	6.901	8.051	9.201	10.35	11.50
3 x 16	2-1/2 x 15-1/4	38.13	96.90	738.9	15.89	19.86	6.619	7.943	9.266	10.59	11.91	13.24
4 x 4	3-1/2 x 3-1/2	12.25	7.15	12.51	7.146	12.51	2.127	2.552	2.977	3.403	3.828	4.253
4 x 5	3-1/2 x 4-1/2	15.75	11.81	26.58	9.188	16.08	2.734	3.281	3.828	4.375	4.922	5.469
4 x 6	3-1/2 x 5-1/2	19.25	17.65	48.53	11.23	19.65	3.342	4.010	4.679	5.347	6.016	6.684
4 x 8	3-1/2 x 7-1/4	25.38	30.66	111.1	14.80	25.90	4.405	5.286	6.168	7.049	7.930	8.811
4 x 10	3-1/2 x 9-1/4	32.38	49.91	230.8	18.89	33.05	5.621	6.745	7.869	8.993	10.12	11.24
4 x 12	3-1/2 x 11-1/4	39.38	73.83	415.3	22.97	40.20	6.836	8.203	9.570	10.94	12.30	13.67
4 x 14	3-1/2 x 13-1/4	46.38	102.41	678.5	27.05	47.34	8.051	9.661	11.27	12.88	14.49	16.10
4 x 16	3-1/2 x 15-1/4	53.38	135.66	1034	31.14	54.49	9.266	11.12	12.97	14.83	16.68	18.53
Timbers (5" x 5" and larger)[2]												
Post and Timber (see NDS 4.1.3.4 and NDS 4.1.5.3)												
5 x 5	4-1/2 x 4-1/2	20.25	15.19	34.17	15.19	34.17	3.516	4.219	4.922	5.625	6.328	7.031
6 x 6	5-1/2 x 5-1/2	30.25	27.73	76.26	27.73	76.26	5.252	6.302	7.352	8.403	9.453	10.50
6 x 8	5-1/2 x 7-1/2	41.25	51.56	193.4	37.81	104.0	7.161	8.594	10.03	11.46	12.89	14.32
8 x 8	7-1/2 x 7-1/2	56.25	70.31	263.7	70.31	263.7	9.766	11.72	13.67	15.63	17.58	19.53
8 x 10	7-1/2 x 9-1/2	71.25	112.8	535.9	89.06	334.0	12.37	14.84	17.32	19.79	22.27	24.74
10 x 10	9-1/2 x 9-1/2	90.25	142.9	678.8	142.9	678.8	15.67	18.80	21.94	25.07	28.20	31.34
10 x 12	9-1/2 x 11-1/2	109.3	209.4	1204	173.0	821.7	18.97	22.76	26.55	30.35	34.14	37.93
12 x 12	11-1/2 x 11-1/2	132.3	253.5	1458	253.5	1458	22.96	27.55	32.14	36.74	41.33	45.92
12 x 14	11-1/2 x 13-1/2	155.3	349.3	2358	297.6	1711	26.95	32.34	37.73	43.13	48.52	53.91
14 x 14	13-1/2 x 13-1/2	182.3	410.1	2768	410.1	2768	31.64	37.97	44.30	50.63	56.95	63.28
14 x 16	13-1/2 x 15-1/2	209.3	540.6	4189	470.8	3178	36.33	43.59	50.86	58.13	65.39	72.66
16 x 16	15-1/2 x 15-1/2	240.3	620.6	4810	620.6	4810	41.71	50.05	58.39	66.74	75.08	83.42
16 x 18	15-1/2 x 17-1/2	271.3	791.1	6923	700.7	5431	47.09	56.51	65.93	75.35	84.77	94.18
18 x 18	17-1/2 x 17-1/2	306.3	893.2	7816	893.2	7816	53.17	63.80	74.44	85.07	95.70	106.3
18 x 20	17-1/2 x 19-1/2	341.3	1109	10813	995.3	8709	59.24	71.09	82.94	94.79	106.6	118.5
20 x 20	19-1/2 x 19-1/2	380.3	1236	12049	1236	12049	66.02	79.22	92.4	105.6	118.8	132.0
20 x 22	19-1/2 x 21-1/2	419.3	1502	16150	1363	13285	72.79	87.34	101.9	116.5	131.0	145.6
22 x 22	21-1/2 x 21-1/2	462.3	1656	17806	1656	17806	80.25	96.30	112.4	128.4	144.5	160.5
22 x 24	21-1/2 x 23-1/2	505.3	1979	23252	1810	19463	87.72	105.3	122.8	140.3	157.9	175.4
24 x 24	23-1/2 x 23-1/2	552.3	2163	25415	2163	25415	95.88	115.1	134.2	153.4	172.6	191.8

Depth	Area	X-X Axis			Y-Y Axis	
d (in.)	A (in.^{2})	I_x (in.^{4})	S_x (in.^{3})	r_x (in.)	I_y (in.^{4})	S_y (in.^{3})
			8-1/2 in. Width		**(r_y = 2.454 in.)**	
9-5/8	81.81	631.6	131.2	2.778	492.6	115.9
11	93.50	942.8	171.4	3.175	562.9	132.5
12-3/8	105.2	1342	216.9	3.572	633.3	149.0
13-3/4	116.9	1841	267.8	3.969	703.7	165.6
15-1/8	128.6	2451	324.1	4.366	774.1	182.1
16-1/2	140.3	3182	385.7	4.763	844.4	198.7
17-7/8	151.9	4046	452.6	5.160	914.8	215.2
19-1/4	163.6	5053	525.0	5.557	985.2	231.8
20-5/8	175.3	6215	602.6	5.954	1056	248.4
22	187.0	7542	685.7	6.351	1126	264.9
23-3/8	198.7	9047	774.1	6.748	1196	281.5
24-3/4	210.4	10740	867.8	7.145	1267	298.0
26-1/8	222.1	12630	966.9	7.542	1337	314.6
27-1/2	233.8	14730	1071	7.939	1407	331.1
28-7/8	245.4	17050	1181	8.335	1478	347.7
30-1/4	257.1	19610	1296	8.732	1548	364.3
31-5/8	268.8	22400	1417	9.129	1618	380.8
33	280.5	25460	1543	9.526	1689	397.4
34-3/8	292.2	28770	1674	9.923	1759	413.9
35-3/4	303.9	32360	1811	10.32	1830	430.5
37-1/8	315.6	36240	1953	10.72	1900	447.0
38-1/2	327.3	40420	2100	11.11	1970	463.6
39-7/8	338.9	44910	2253	11.51	2041	480.2
41-1/4	350.6	49720	2411	11.91	2111	496.7
42-5/8	362.3	54860	2574	12.30	2181	513.3
44	374.0	60340	2743	12.70	2252	529.8
45-3/8	385.7	66170	2917	13.10	2322	546.4
46-3/4	397.4	72370	3096	13.50	2393	562.9
48-1/8	409.1	78950	3281	13.89	2463	579.5
49-1/2	420.8	85910	3471	14.29	2533	596.1
50-7/8	432.4	93270	3667	14.69	2604	612.6

Species and commercial grade	Size classification	Bending F_b	Tension parallel to grain F_t	Shear parallel to grain F_v	Compression perpendicular to grain $F_{c\perp}$	Compression parallel to grain F_c	Modulus of Elasticity E	Modulus of Elasticity E_{min}	Specific Gravity G	Grading Rules Agency
		Design values in pounds per square inch (psi)								
SOUTHERN PINE										
Dense Select Structural	2" - 4" wide	2,700	1,900	175	660	2,050	1,900,000	690,000	0.55	SPIB
Select Structural		2,350	1,650	175	565	1,900	1,800,000	660,000		
Non-Dense Select Structural		2,050	1,450	175	480	1,800	1,600,000	580,000		
No.1 Dense		1,650	1,100	175	660	1,750	1,800,000	660,000		
No.1		1,500	1,000	175	565	1,650	1,600,000	580,000		
No.1 Non-Dense		1,300	875	175	480	1,550	1,400,000	510,000		
No.2 Dense		1,200	750	175	660	1,500	1,600,000	580,000		
No.2		1,100	675	175	565	1,450	1,400,000	510,000		
No.2 Non-Dense		1,050	600	175	480	1,450	1,300,000	470,000		
No.3 and Stud		650	400	175	565	850	1,300,000	470,000		
Construction	4" wide	875	500	175	565	1,600	1,400,000	510,000	0.55	
Standard		475	275	175	565	1,300	1,200,000	440,000		
Utility		225	125	175	565	850	1,200,000	440,000		
Dense Select Structural	5" - 6" wide	2,400	1,650	175	660	1,900	1,900,000	690,000	0.55	
Select Structural		2,100	1,450	175	565	1,800	1,800,000	660,000		
Non-Dense Select Structural		1,850	1,300	175	480	1,700	1,600,000	580,000		
No.1 Dense		1,500	1,000	175	660	1,650	1,800,000	660,000		
No.1		1,350	875	175	565	1,550	1,600,000	580,000		
No.1 Non-Dense		1,200	775	175	480	1,450	1,400,000	510,000		
No.2 Dense		1,050	650	175	660	1,450	1,600,000	580,000		
No.2		1,000	600	175	565	1,400	1,400,000	510,000		
No.2 Non-Dense		950	525	175	480	1,350	1,300,000	470,000		
No.3 and Stud		575	350	175	565	800	1,300,000	470,000		
Dense Select Structural	8" wide	2,200	1,550	175	660	1,850	1,900,000	690,000	0.55	
Select Structural		1,950	1,350	175	565	1,700	1,800,000	660,000		
Non-Dense Select Structural		1,700	1,200	175	480	1,650	1,600,000	580,000		
No.1 Dense		1,350	900	175	660	1,600	1,800,000	660,000		
No.1		1,250	800	175	565	1,500	1,600,000	580,000		
No.1 Non-Dense		1,100	700	175	480	1,400	1,400,000	510,000		
No.2 Dense		975	600	175	660	1,400	1,600,000	580,000		
No.2		925	550	175	565	1,350	1,400,000	510,000		
No.2 Non-Dense		875	500	175	480	1,300	1,300,000	470,000		
No.3 and Stud		525	325	175	565	775	1,300,000	470,000		
Dense Select Structural	10" wide	1,950	1,300	175	660	1,800	1,900,000	690,000	0.55	
Select Structural		1,700	1,150	175	565	1,650	1,800,000	660,000		
Non-Dense Select Structural		1,500	1,050	175	480	1,600	1,600,000	580,000		
No.1 Dense		1,200	800	175	660	1,550	1,800,000	660,000		
No.1		1,050	700	175	565	1,450	1,600,000	580,000		
No.1 Non-Dense		950	625	175	480	1,400	1,400,000	510,000		
No.2 Dense		850	525	175	660	1,350	1,600,000	580,000		
No.2		800	475	175	565	1,300	1,400,000	510,000		
No.2 Non-Dense		750	425	175	480	1,250	1,300,000	470,000		
No.3 and Stud		475	275	175	565	750	1,300,000	470,000		
Dense Select Structural	12" wide	1,800	1,250	175	660	1,750	1,900,000	690,000	0.55	
Select Structural		1,600	1,100	175	565	1,650	1,800,000	660,000		
Non-Dense Select Structural		1,400	975	175	480	1,550	1,600,000	580,000		
No.1 Dense		1,100	750	175	660	1,500	1,800,000	660,000		
No.1		1,000	650	175	565	1,400	1,600,000	580,000		
No.1 Non-Dense		900	575	175	480	1,350	1,400,000	510,000		
No.2 Dense		800	500	175	660	1,300	1,600,000	580,000		
No.2		750	450	175	565	1,250	1,400,000	510,000		
No.2 Non-Dense		700	400	175	480	1,250	1,300,000	470,000		
No.3 and Stud		450	250	175	565	725	1,300,000	470,000		

Species and commercial grade	Size classification	Design values in pounds per square inch (psi)							Specific Gravity[5]	Grading Rules Agency
		Bending	Tension parallel to grain	Shear parallel to grain	Compression perpendicular to grain	Compression parallel to grain	Modulus of Elasticity			
		F_b	F_t	F_v	$F_{c\perp}$	F_c	E	E_{min}	G	
SOUTHERN PINE		(Surfaced Dry - Used in dry service condtions - 19% or less moisture content)								
Dense Structural 86	2" & wider	2,600	1,750	175	660	2,000	1,800,000	660,000	0.55	SPIB
Dense Structural 72		2,200	1,450	175	660	1,650	1,800,000	660,000		
Dense Structural 65		2,000	1,300	175	660	1,500	1,800,000	660,000		
SOUTHERN PINE		(Surfaced Green - Used in any service condtion)								
Dense Structural 86	2-1/2" & wider 2-1/2"-4" thick	2,100	1,400	165	440	1,300	1,600,000	580,000	0.55	SPIB
Dense Structural 72		1,750	1,200	165	440	1,100	1,600,000	580,000		
Dense Structural 65		1,600	1,050	165	440	1,000	1,600,000	580,000		
MIXED SOUTHERN PINE										
Select Structural	2" - 4" wide	2,050	1,200	175	565	1,800	1,600,000	580,000	0.51	SPIB
No.1		1,450	875	175	565	1,650	1,500,000	550,000		
No.2		1,100	675	175	565	1,450	1,400,000	510,000		
No.3 and Stud		650	400	175	565	850	1,200,000	440,000		
Construction	4" wide	850	500	175	565	1,600	1,300,000	470,000	0.51	
Standard		475	275	175	565	1,300	1,200,000	440,000		
Utility		225	125	175	565	850	1,100,000	400,000		
Select Structural	5" - 6" wide	1,850	1,100	175	565	1,700	1,600,000	580,000	0.51	
No.1		1,300	750	175	565	1,550	1,500,000	550,000		
No.2		1,000	600	175	565	1,400	1,400,000	510,000		
No.3 and Stud		575	350	175	565	775	1,200,000	440,000		
Select Structural	8" wide	1,750	1,000	175	565	1,600	1,600,000	580,000	0.51	
No.1		1,200	700	175	565	1,450	1,500,000	550,000		
No.2		925	550	175	565	1,350	1,400,000	510,000		
No.3 and Stud		525	325	175	565	800	1,200,000	440,000		
Select Structural	10" wide	1,500	875	175	565	1,600	1,600,000	580,000	0.51	
No.1		1,050	600	175	565	1,450	1,500,000	550,000		
No.2		800	475	175	565	1,300	1,400,000	510,000		
No.3 and Stud		475	275	175	565	750	1,200,000	440,000		
Select Structural	12" wide	1,400	825	175	565	1,550	1,600,000	580,000	0.51	
No.1		975	575	175	565	1,400	1,500,000	550,000		
No.2		750	450	175	565	1,250	1,400,000	510,000		
No.3 and Stud		450	250	175	565	725	1,200,000	440,000		

Species and commercial Grade	Size classification	Design values in pounds per square inch (psi)							Specific Gravity[4]	Grading Rules Agency
		Bending	Tension parallel to grain	Shear parallel to grain	Compression perpendicular to grain	Compression parallel to grain	Modulus of Elasticity			
		F_b	F_t	F_v	$F_{c\perp}$	F_c	E	E_{min}	G	
RED MAPLE										
Select Structural	Beams and Stringers	1,500	875	195	615	900	1,500,000	550,000	0.58	NELMA
No.1		1,250	625	195	615	750	1,500,000	550,000		
No.2		800	400	195	615	475	1,200,000	440,000		
Select Structural	Posts and Timbers	1,400	925	195	615	950	1,500,000	550,000		
No.1		1,150	750	195	615	825	1,500,000	550,000		
No.2		650	425	195	615	375	1,200,000	440,000		
RED OAK										
Select Structural	Beams and Stringers	1,350	800	155	820	825	1,200,000	440,000	0.67	NELMA
No.1		1,150	550	155	820	700	1,200,000	440,000		
No.2		725	375	155	820	450	1,000,000	370,000		
Select Structural	Posts and Timbers	1,250	850	155	820	875	1,200,000	440,000		
No.1		1,000	675	155	820	775	1,200,000	440,000		
No.2		575	400	155	820	350	1,000,000	370,000		
RED PINE										
Select Structural	Beams and Stringers	1,050	625	130	440	725	1,100,000	400,000	0.44	NLGA
No.1		875	450	130	440	600	1,100,000	400,000		
No.2		575	300	130	440	375	900,000	330,000		
Select Structural	Posts and Timbers	1,000	675	130	440	775	1,100,000	400,000		
No.1		800	550	130	440	675	1,100,000	400,000		
No.2		475	325	130	440	475	900,000	330,000		
REDWOOD										
Select Structural	5" x 5" and Larger	1,100	750	145	420	900	1,000,000	370,000	0.37	RIS
No. 1		950	650	145	420	800	1,000,000	370,000		
No. 2		750	400	145	420	650	900,000	330,000		
SITKA SPRUCE										
Select Structural	Beams and Stringers	1,200	675	140	435	825	1,300,000	470,000	0.43	WCLIB
No.1		1,000	500	140	435	675	1,300,000	470,000		
No.2		650	325	140	435	450	1,000,000	370,000		
Select Structural	Posts and Timbers	1,150	750	140	435	875	1,300,000	470,000		
No.1		925	600	140	435	750	1,300,000	470,000		
No.2		550	350	140	435	525	1,000,000	370,000		
Select Structural	Beams and Stringers	1,200	675	140	435	825	1,300,000	470,000	0.43	WWPA
No.1		1,000	500	140	435	675	1,300,000	470,000		
No.2		650	325	140	435	450	1,100,000	400,000		
Select Structural	Posts and Timbers	1,150	750	140	435	875	1,300,000	470,000		
No.1		925	600	140	435	750	1,300,000	470,000		
No.2		550	350	140	435	525	1,100,000	400,000		
SOUTHERN PINE		(Wet Service Conditons)								
Dense Select Structural	5" x 5" and Larger	1,750	1,200	165	440	1,100	1,600,000	580,000	0.55	SPIB
Select Structural		1,500	1,000	165	375	950	1,500,000	550,000		
No. 1 Dense		1,550	1,050	165	440	975	1,600,000	580,000		
No. 1		1,350	900	165	375	825	1,500,000	550,000		
No. 2 Dense		975	650	165	440	625	1,300,000	470,000		
No. 2		850	550	165	375	525	1,200,000	440,000		
Dense Select Structural 86		2,100	1,400	165	440	1,300	1,600,000	580,000		
Dense Select Structural 72		1,750	1,200	165	440	1,100	1,600,000	580,000		
Dense Select Structural 65		1,600	1,050	165	440	1,000	1,600,000	580,000		
SPRUCE-PINE-FIR										
Select Structural	Beams and Stringers	1,100	650	125	425	775	1,300,000	470,000	0.42	NLGA
No.1		900	450	125	425	625	1,300,000	470,000		
No.2		600	300	125	425	425	1,000,000	370,000		
Select Structural	Posts and Timbers	1,050	700	125	425	800	1,300,000	470,000		
No.1		850	550	125	425	700	1,300,000	470,000		
No.2		500	325	125	425	500	1,000,000	370,000		

Combination Symbol	Species Outer/ Core	Bending About X-X Axis (Loaded Perpendicular to Wide Faces of Laminations): Bending, Bottom of beam Stressed in tension (Positive Bending) F_{bx}^{+} (psi)	Bending, Top of Beam Stressed in Tension (Negative Bending) F_{bx}^{-} (psi)	Compression Perpendicular to Grain, Tension Face $F_{c\perp x}$ (psi)	Compression Perpendicular to Grain, Compression Face $F_{c\perp x}$ (psi)	Shear Parallel to Grain F_{vx} (2) (psi)	Modulus of Elasticity, For Deflection Calculations(8) $E_{x\ true}$ (10^6 psi)	$E_{x\ app}$ (10^6 psi)	For Stability Calculations $E_{x\ min}$ (10^6 psi)	Bending About Y-Y Axis (Loaded Parallel to Wide Faces of Laminations): Bending F_{by} (psi)	Compression Perpendicular to Grain $F_{c\perp y}$ (psi)	Shear Parallel to Grain F_{vy} (3) (psi)	Modulus of Elasticity, For Deflection Calculations(8) $E_{y\ true}$ (10^6 psi)	$E_{y\ app}$ (10^6 psi)	For Stability Calculations $E_{y\ min}$ (10^6 psi)	Axially Loaded: Tension Parallel to Grain F_t (psi)	Compression Parallel to Grain F_c (psi)	Fasteners: Specific Gravity for Fastener Design G, Top or Bottom Face	Side Face
16F-1.3E		**1600**	**925**	**315**		**195**	**1.4**	**1.3**	**0.69**	**800**	**315**	**170**	**1.2**	**1.1**	**0.58**	**675**	**925**	**0.41**	
16F-V3	DF/DF	1600	1250	560	560	265	1.6	1.5	0.79	1450	560	230	1.6	1.5	0.79	975	1500	0.50	0.50
16F-V6	DF/DF	1600	1600	560	560	265	1.7	1.6	0.85	1450	560	230	1.6	1.5	0.79	1000	1600	0.50	0.50
16F-E2	HF/HF	1600	1050	375	375	215	1.5	1.4	0.74	1200	375	190	1.4	1.3	0.69	825	1150	0.43	0.43
16F-E3	DF/DF	1600	1200	560	560	265	1.7	1.6	0.85	1400	560	230	1.6	1.5	0.79	975	1600	0.50	0.50
16F-E6	DF/DF	1600	1600	560	560	265	1.7	1.6	0.85	1550	560	230	1.6	1.5	0.79	1000	1600	0.50	0.50
16F-E7	HF/HF	1600	1600	375	375	215	1.5	1.4	0.74	1350	375	190	1.4	1.3	0.74	875	1250	0.43	0.43
16F-V2	SP/SP	1600	1400	740	650	300	1.6	1.5	0.79	1450	650	260	1.5	1.4	0.74	1000	1300	0.55	0.55
16F-V3	SP/SP	1600	1450	740	740	300	1.5	1.4	0.74	1450	650	260	1.5	1.4	0.74	975	1400	0.55	0.55
16F-V5	SP/SP	1600	1600	650	650	300	1.7	1.6	0.85	1600	650	260	1.6	1.5	0.79	1000	1550	0.55	0.55
16F-E1	SP/SP	1600	1250	650	650	300	1.7	1.6	0.85	1400	650	260	1.7	1.6	0.85	1050	1550	0.55	0.55
16F-E3	SP/SP	1600	1600	650	650	300	1.8	1.7	0.90	1650	650	260	1.7	1.6	0.85	1100	1550	0.55	0.55
20F-1.5E		**2000**	**1100**	**425**		**195**	**1.6**	**1.5**	**0.79**	**800**	**315**	**170**	**1.3**	**1.2**	**0.63**	**725**	**925**	**0.41**	
20F-V3	DF/DF	2000	1450	650	560	265	1.7	1.6	0.85	1450	560	230	1.6	1.5	0.79	1000	1550	0.50	0.50
20F-V7	DF/DF	2000	2000	650	650	265	1.7	1.6	0.85	1450	560	230	1.7	1.6	0.85	1050	1600	0.50	0.50
20F-V12	AC/AC	2000	1400	560	560	265	1.6	1.5	0.79	1250	470	230	1.5	1.4	0.74	925	1500	0.46	0.46
20F-V13	AC/AC	2000	2000	560	560	265	1.6	1.5	0.79	1250	470	230	1.5	1.4	0.74	950	1550	0.46	0.46
20F-V14	POC/POC	2000	1450	560	560	265	1.6	1.5	0.79	1300	470	230	1.5	1.4	0.74	900	1600	0.46	0.46
20F-V15	POC/POC	2000	2000	560	560	265	1.6	1.5	0.79	1300	470	230	1.5	1.4	0.74	900	1600	0.46	0.46
20F-E2	HF/HF	2000	1400	500	500	215	1.7	1.6	0.85	1200	375	190	1.5	1.4	0.74	925	1350	0.43	0.43
20F-E3	DF/DF	2000	1200	560	560	265	1.8	1.7	0.90	1400	560	230	1.7	1.6	0.85	1050	1600	0.50	0.50
20F-E6	DF/DF	2000	2000	560	560	265	1.8	1.7	0.90	1550	560	230	1.7	1.6	0.85	1150	1650	0.50	0.50
20F-E7	HF/HF	2000	2000	500	500	215	1.8	1.6	0.85	1450	375	190	1.5	1.4	0.74	1050	1450	0.43	0.43
20F-E8	ES/ES	2000	1300	450	450	200	1.6	1.5	0.79	1000	315	175	1.5	1.4	0.74	825	1100	0.41	0.41
24F-E/SPF1	SPF/SPF	2400	2400	560	560	215	1.7	1.6	0.85	1150	470	190	1.7	1.6	0.85	1150	2000	0.42	0.42
24F-E/SPF3	SPF/SPF	2400	1550	560	650	215	1.7	1.6	0.85	1200	470	195	1.6	1.5	0.79	900	1750	0.42	0.42
20F-V2	SP/SP	2000	1550	740	650	300	1.6	1.5	0.79	1450	650	260	1.5	1.4	0.74	1000	1400	0.55	0.55
20F-V3	SP/SP	2000	1450	650	650	300	1.6	1.5	0.79	1600	650	260	1.6	1.5	0.79	1000	1400	0.55	0.55
20F-V5	SP/SP	2000	2000	740	740	300	1.7	1.6	0.85	1450	650	260	1.5	1.4	0.74	1050	1500	0.55	0.55
20F-E1	SP/SP	2000	1300	650	650	300	1.8	1.7	0.90	1400	650	260	1.7	1.6	0.85	1050	1550	0.55	0.55
20F-E3	SP/SP	2000	2000	650	650	300	1.8	1.7	0.90	1700	650	260	1.7	1.6	0.85	1150	1600	0.55	0.55
24F-1.7E		**2400**	**1450**	**500**		**210**	**1.8**	**1.7**	**0.90**	**1050**	**315**	**185**	**1.4**	**1.3**	**0.69**	**775**	**1000**	**0.42**	
24F-V5	DF/HF	2400	1600	650	650	215	1.8	1.7	0.90	1350	375	200	1.6	1.5	0.79	1100	1450	0.50	0.43
24F-V10	DF/HF	2400	2400	650	650	215	1.9	1.8	0.95	1450	375	200	1.6	1.5	0.79	1150	1550	0.50	0.43
24F-E11	HF/HF	2400	2400	500	500	215	1.9	1.8	0.95	1550	375	190	1.6	1.5	0.79	1150	1550	0.43	0.43
24F-E15	HF/HF	2400	1600	500	500	215	1.9	1.8	0.95	1200	375	190	1.6	1.5	0.79	975	1500	0.43	0.43
24F-V1	SP/SP	2400	1750	740	650	300	1.8	1.7	0.90	1450	650	260	1.6	1.5	0.79	1100	1500	0.55	0.55
24F-V4 (4)	SP/SP	2400	1650	740	650	210	1.8	1.7	0.90	1350	470	230	1.6	1.5	0.79	975	1350	0.55	0.43
24F-V5	SP/SP	2400	2400	740	740	300	1.8	1.7	0.90	1700	650	260	1.7	1.6	0.85	1150	1600	0.55	0.55

Figure 19: NDS Wood Member Properties

Design Properties (100% Load Duration)

Depth	TJI®	Basic Properties: Joist Weight (lbs/ft)	Maximum Resistive Moment(1) (ft-lbs)	Joist Only EI x 10^6 (in.²-lbs)	Maximum Vertical Shear (lbs)	Reaction Properties: 1¾" End Reaction (lbs)	3½" End Reaction (lbs)	3½" Intermediate Reaction (lbs): No Web Stiffeners	With Web Stiffeners(2)	5¼" Intermediate Reaction (lbs): No Web Stiffeners	With Web Stiffeners(2)
9½"	110	2.3	2,500	157	1,220	910	1,220	1,935	N.A.	2,350	N.A.
	210	2.6	3,000	186	1,330	1,005	1,330	2,145	N.A.	2,565	N.A.
	230	2.7	3,330	206	1,330	1,060	1,330	2,410	N.A.	2,790	N.A.
11⅞"	110	2.5	3,160	267	1,560	910	1,375	1,935	2,295	2,350	2,705
	210	2.8	3,795	315	1,655	1,005	1,460	2,145	2,505	2,565	2,925
	230	3.0	4,215	347	1,655	1,060	1,485	2,410	2,765	2,790	3,150
	360	3.0	6,180	419	1,705	1,080	1,505	2,460	2,815	3,000	3,360
	560	4.0	9,500	636	2,050	1,265	1,725	3,000	3,475	3,455	3,930
14"	110	2.8	3,740	392	1,860	910	1,375	1,935	2,295	2,350	2,705
	210	3.1	4,490	462	1,945	1,005	1,460	2,145	2,505	2,565	2,925
	230	3.3	4,990	509	1,945	1,060	1,485	2,410	2,765	2,790	3,150
	360	3.3	7,335	612	1,955	1,080	1,505	2,460	2,815	3,000	3,360
	560	4.2	11,275	926	2,390	1,265	1,725	3,000	3,475	3,455	3,930
16"	110	3.0	4,280	535	2,145	910	1,375	1,935	2,295	2,350	2,705
	210	3.3	5,140	629	2,190	1,005	1,460	2,145	2,505	2,565	2,925
	230	3.5	5,710	691	2,190	1,060	1,485	2,410	2,765	2,790	3,150
	360	3.5	8,405	830	2,190	1,080	1,505	2,460	2,815	3,000	3,360
	560	4.5	12,925	1,252	2,710	1,265	1,725	3,000	3,475	3,455	3,930

Figure 20: TJI Wood I-Joist Properties

Type	T_F	W	A	d	d_{det}	b_f	b_{fdet}	t_w	t_{wdet}	$t_{wdet}/2$	t_f	t_{fdet}	k_{des}	k_{det}	k_1	$b_f/2t_f$	h/t_w	I_x	Z_x	S_x	r_x	I_y	Z_y	S_y	r_y	J	C_w	W_{no}	S_{w1}	Q_f	Q_w	r_{ts}	h_o	P_A	P_B	P_C	P_D	T	WG_i
W6X25 A36 Steel Beam	F	25	7.34	6.38	6 3/8	6.08	6 1/8	0.32	5/16	3/16	0.455	7/16	0.705	15/16	9/16	6.68	15.5	53.4	18.9	16.7	2.7	17.1	8.56	5.61	1.52	0.461	150	9.01	6.23	3.88	9.39	1.74	5.93	28.9	36	18.8	24.9	4 1/2	3 1/2
W12X50 A36 Steel Beam	F	50	14.6	12.2	12 1/4	8.08	8 1/8	0.37	3/8	3/16	0.64	5/8	1.14	1 1/2	15/16	6.31	26.8	391	71.9	64.2	5.18	56.3	21.3	13.9	1.96	1.71	1880	23.4	30.2	14.3	35.4	2.25	11.6	47	55.1	32.5	40.6	9 1/4	5 1/2
W14X22 A36 Steel Beam	F	22	6.49	13.7	13 3/4	5	5	0.23	1/4	1/8	0.335	5/16	0.735	1 1/16	3/4	7.46	53.3	199	33.2	29	5.54	7	4.39	2.8	1.04	0.208	314	16.7	7	5.34	16.1	1.27	13.4	41.3	46.3	32.4	37.4	11 5/8	2 3/4
W14X38 A36 Steel Beam	F	38	11.2	14.1	14 1/8	6.77	6 3/4	0.31	5/16	3/16	0.515	1/2	0.915	1 1/4	13/16	6.57	39.6	385	61.5	54.6	5.87	26.7	12.1	7.88	1.55	0.798	1230	23	20	11.3	30.3	1.82	13.6	47.2	54	35	41.7	11 5/8	3 1/2
W16X45 A36 Steel Beam	F	45	13.3	16.1	16 1/8	7.04	7	0.345	3/8	3/16	0.565	9/16	0.967	1 1/4	13/16	6.23	41.1	586	82.3	72.7	6.65	32.8	14.5	9.34	1.57	1.11	1990	27.3	27.2	14.7	40.6	1.87	15.5	52	59	39.2	46.3	13 5/8	3 1/2
W12X65 A992 Steel Beam	F	65.0	19.1	12.1	12 1/8	12.0	12	0.390	3/8	3/16	0.605	5/8	1.20	1 1/2	1	9.92	24.9	533	96.8	87.9	5.28	174	44.1	29.1	3.02	2.18	5780	34.5	62.6	20.2	47.5	3.38	11.5	58.4	70.4	36.2	48.2	9 1/8	5 1/2

Figure 21: AISC Steel Member Properties

Boise Cascade
Engineered Wood Products

Technical Note

Weights of Building Materials – Pounds Per Square Foot [PSF]

CEILING	
Acoustical fiber board [(1)]	1
Suspended steel channel system [(1)]	2
Suspended wood channel system	2.5
2x8 ceiling joists @ 16" o.c., R-49 insulation, 1/2" gypsum board	7
1" Plaster	8
1/2" gypsum board [(1)]	2.2
5/8" gypsum board [(1)]	2.75

ROOF	
Fiberglass shingles	3
Asphalt shingles [(1)]	2
Wood shingles [(1)]	3
Spanish clay tile [(1)]	19
Concrete roof tile	12
Lightweight clay tile	6
Composition Roofing:	
Three-ply ready roofing [(1)]	1
Four-ply felt and gravel [(1)]	5.5
Five-ply felt and gravel [(1)]	6
20 gage metal deck [(1)]	2.5
18 gage metal deck [(1)]	3
0.05" thick polyvinyl chloride polymer membrane[(4)]	0.35
1" fiberglass batt insulation	0.04
1" loose fiberglass insulation	0.04
1" loose cellulose insulation	0.14
1" rigid insulation [(1)]	1.5
Blowing wool insulation R-38 (16"deep)	0.62
3/16" slate [(1)]	7
1/4" slate [(1)]	10
Single-ply (no ballast) [(1)]	0.7
Single-ply (ballasted)	11
Dry gravel [(1)]	8.7
2x8 rafters @ 16" o.c., fiberglass shingles, 15# felt, 3/8" sheathing	8
Skylight: metal frame w/ 3/8" wire glass [(1)]	8

FLOOR	
1" reinforced regular weight concrete	12.5
1" plain lightweight concrete [(1)]	8
7/16" cementitious backerboard	3
Ceramic or quarry tile (3/4") on 1/2" mortar bed [(1)]	16
Ceramic or quarry tile (3/4") on 1" mortar bed [(1)]	23
1" mortar bed	12
1" slate [(1)]	15
3/8" marble tile	6
3/8" ceramic floor tile [(1)]	4.7

FLOOR (cont.)	
Hardwood flooring, 7/7-in [(1)]	4
1/4" linoleum or asphalt tile [(1)]	1
BCI/AJS® joists @ 16" o.c., 3/4" sheathing, 1/2" gypsum board	10
3/4" Gyp-Crete® topping	6.5
Carpet & Pad	2.0
Waterproofing Membranes	
Bituminous, smooth surface [(1)]	1.5
Liquid applied [(1)]	1

SHEATHING	
11/32" or 3/8" Plywood – OSB[(3)]	1.0 - 1.2
15/32" or 1/2" Plywood - OSB[(3)]	1.4 - 1.7
19/32" or 5/8" Plywood - OSB[(3)]	1.8 - 2.1
23/32" or 3/4" Plywood - OSB[(3)]	2.2 - 2.5
7/8" Plywood - OSB[(3)]	2.6 - 2.9
1 1/8" Plywood - OSB[(3)]	3.3 - 3.6
1/2" cementitious backerboard	3
1-1/2" softwood T & G decking	4.6

FRAMING	
2x4 @ 16" o.c.	1.1
2x6 @ 16" o.c.	1.7
2x8 @ 16" o.c.	2.2
2x10 @ 16" o.c.	2.9
2x12 @ 16" o.c.	3.5
BCI® 4500s, 5000 or 5000s @ 12" o.c.	2.0 – 2.9
BCI® 4500s, 5000 or 5000s @ 16" o.c.	1.5 – 2.2
BCI® 4500s, 5000 or 5000s @ 19.2" o.c.	1.3 – 2.8
BCI® 4500s, 5000 or 5000s @ 24" o.c	1.0 – 1.5
BCI® 6000 or 6000s @ 12" o.c.	2.2 – 3.4
BCI® 6000 or 6000s @ 16" o.c.	1.7 – 2.6
BCI® 6000 or 6000s @19.2" o.c.	1.4 - 2.1
BCI® 6000 or 6000s @ 24" o.c.	1.1 - 1.7
BCI® 60, 60s, 6500 or 6500s @ 12" o.c.	2.3 – 3.8
BCI® 60, 60s, 6500 or 6500s @ 16" o.c.	1.7 – 2.9
BCI® 60, 60s, 6000 or 6500s @19.2" o.c.	1.4 – 2.4
BCI® 60, 60s, 6500 or 6500s @ 24" o.c.	1.2 – 1.9
BCI® 90 or 90s @ 12" o.c.	3.9 – 4.9
BCI® 90 or 90s @ 16" o.c.	2.9 – 3.7
BCI® 90 or 90s @ 19.2" o.c.	2.4 – 3.1
BCI® 90 or 90s @ 24" o.c.	1.9 – 2.5
AJS® 140, 150, 190 or 20 @ 12" o.c.	2.2 – 3.3
AJS® 140, 150, 190 or 20 @ 16" o.c.	1.7 – 2.5
AJS® 140, 150, 190 or 20 @ 19.2" o.c.	1.4 – 2.1
AJS® 140, 150, 190 or 20 @ 24" o.c.	1.1 – 1.7
AJS® 25 or 30 @ 12" o.c.	3.1 – 3.9
AJS® 25 or 30 @ 16" o.c.	2.3 – 2.9
AJS® 25 or 30 @ 19.2" o.c.	1.9 – 2.4
AJS® 25 or 30 @ 24" o.c.	1.6 – 2.0

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Boise Cascade
Engineered Wood Products

Technical Note

WALL	
5/16" x 7-1/2" fiber cement lap siding	3
4" clay brick [(1)]	39
1/4" ceramic wall tile [(1)]	3.1
1 ¾" Cultured Stone®	12
2x4 studs @ 16" o.c., 5/8" gypsum, insulation, 3/8" siding [(1)]	11
2x6 studs @ 16" o.c., 5/8" gypsum, insulation, 3/8" siding [(1)]	12
Wood or steel studs, 1/2" gypsum board each side [(1)]	8
Exterior stud walls w/ brick veneer [(1)]	48
Windows: glass, frame and sash [(1)]	8
Stucco	10
Log Wall: 10" diameter	26
Glass Block	
4" thick - standard (hollow)	20
3" thick - standard (hollow)	16
4" thick - thick face	30
3" thick - solid glass block	40
MISCELLANEOUS	
1" of sand	8
1" of water	5.2
Hay: baled (dry) [(2)]	15 PCF[(2)]
Straw: baled (dry) [(2)]	8 PCF[(2)]
Saturated soil (garden/landscaped roof)	135 PCF
Grand Piano	1000 LB
Hot Tub (tub & water weight)	150

Include at least 1.5 psf in all dead load summations to account for incidentals such as plumbing, ducts, light fixtures, etc.

(1) *Minimum Design Loads for Buildings and Other Structures*, ASCE 7-05.
(2) *National Farm Building Code (Canada) 1995. Value in pounds per cubic foot (PCF), multiply by maximum height to obtain PSF.*
(3) *Approximate Engineering Dead Load Weight of Wood Structural Panels, APA EWS TT-019, 1998.*
(4) *Duro-Last General Specifications, Duro-Last Roofing, Inc. 2005*

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Figure 22: Boise Cascade Weights of Building Materials

Wind

Snow

$w_{snow} := 46\ psf$

Risk Category III Site Soil Class DE

$V := 116\ mph$ (ASCE Hazard Tool)

$V := 116$

$z := 28\ ft + 7.25\ in = 28.604\ ft$

Rain

$m15 := 7.87\ \frac{in}{hr}$

$m60 := 3.73\ \frac{in}{hr}$

$K_{zt} := 1.0$ Under 1000ft

$K_d := 0.85$

Exposure C

$K_z := 0.94 + (z - 25\ ft) \cdot \frac{(0.98 - 0.94)}{(30\ ft - 25\ ft)} = 0.969$

$q_z := 0.002568\ K_z \cdot K_{zt} \cdot K_d \cdot V^2\ psf = 28.456\ psf$

$C_{pw} := 0.8$

$C_{ps} := -0.7$

$G := 0.85$

$GC_{pi} := 0.18$ +or- (enclosed)

$\theta := \text{atan}\left(\frac{1}{12}\right) \cdot \frac{180}{\pi} = 4.764$

North Wind

Roof Pressures

$\theta = 4.764$

$C_{pwr} := 0.3 + (\theta - 35) \cdot \frac{(0.4 - 0.3)}{(45 - 35)} = -0.002$

$C_{plr} := -0.3$

$h := z = 28.604\ ft$

$L := 93\ ft + 10.75\ in = 93.896\ ft$ $B := 91\ ft + 2\ in + \frac{3}{8}\ in = 91.198\ ft$

Leeward

$$p_{leewardroof} := q_z \cdot K_d \cdot C_{plr} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -2.134 \\ -12.379 \end{bmatrix} psf$$

Overhang

$C_{po} := 0.8$

$p_{overhang} := q_z \cdot K_d \cdot C_{po} = 19.35\ psf$

South Wind

Roof Pressures

Windward

$$p_{windwardroof} := q_z \cdot K_d \cdot C_{pwr} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} 5.065 \\ -5.179 \end{bmatrix} \; psf$$

East Wind

Roof Pressures

$C_{psr1} := -0.9$ $\quad C_{psr2} := -0.9$ $\quad C_{psr3} := -0.5$ $\quad C_{psr4} := -0.3$

Side

0 to $\frac{h}{2} = 14.302 \; ft$

$$p_{side1roof} := q_z \cdot K_d \cdot C_{psr1} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -16.647 \\ -26.891 \end{bmatrix} \; psf$$

$\frac{h}{2}$ to $h = 28.604 \; ft$

$$p_{side2roof} := q_z \cdot K_d \cdot C_{psr2} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -16.647 \\ -26.891 \end{bmatrix} \; psf$$

h to $2\ h = 57.208 \; ft$

$$p_{side3roof} := q_z \cdot K_d \cdot C_{psr3} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -6.972 \\ -17.216 \end{bmatrix} \; psf$$

$2\ h$ to end

$$p_{side4roof} := q_z \cdot K_d \cdot C_{psr4} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -2.134 \\ -12.379 \end{bmatrix} \; psf$$

$$F_{wMax} := p_{side1roof}(1) = -26.891 \; psf$$

Rain Load

$\rho := 62.4 \; pcf$

$d_s :=$ $\quad d_h :=$ $\quad d_p :=$

$$R := 5.2 \cdot (d_s + d_h + d_p) = ?$$

$$f_r := \begin{bmatrix} 0.08 \\ 0.25 \end{bmatrix}$$

$b := 92 \; ft + 2.5 \; in$

$L_r := 95 \; ft + 2 \; in + \frac{3}{8} \; in$ $\quad A := L_r \cdot b = 8778.041 \; ft^2$ $\quad i := 7.85 \; \frac{in}{hr}$

$A := \frac{L_r \cdot b}{ft^2} = 8778.041$ $\quad i := 7.85$ $\quad \frac{A \cdot i}{400} = 172.269$

Length of free flow off edge is $\geq \frac{A \cdot i}{400}$ therefor d_h is negligible.

$Q := 0.0104 \cdot A \cdot i \cdot \frac{gal}{min} = 716.639\ gpm$

Snow Load

<u>Balanced Load</u> Risk Category III

Ground Load

$P_g := 46\ psf$ (ASCE Hazard Tool)

Surface Roughness

Category C (Rural)

$C_e := 1.0$

Exposure

Partially Exposed (Forested Rural Location)

Thermal Factor

$C_t := 1.16$

Slope Factor

$C_s := 1$ $\theta = 4.764$ Unobstructed slippery surface

$P_s := 0.7 \cdot C_s \cdot C_e \cdot C_t \cdot P_g = 37.352\ psf$

<u>Unbalanced Snow Load</u>

$l_{rs} := b$

$l_u := l_{rs} = 92.208\ ft$ $L_u := 92.208$ $P_g = 46\ psf$ $p_g := 46$

$h_D := 0.43 \cdot \sqrt[3]{L_u} \cdot \sqrt[4]{p_g + 10} - 1.5 = 3.814$

$\gamma := 0.13 \cdot p_g + 14 = 19.98$ $\gamma := 19.98\ pcf$

Assuming standard residential area for other cabin, S=12ft

$P_{un} := h_D \cdot ft \cdot \frac{\gamma}{\sqrt[2]{12}} = 21.999\ psf$ $W_{un} := \frac{8}{3} \cdot h_D \cdot ft \cdot \sqrt[2]{12} = 35.234\ ft$

$S_{max} := P_s = 37.352\ psf$

Dead Loads

Load Calculation	From Boise Cascade	
Suspended wood channel system	$D_1 := 2.5\ psf$	
18 gauge metal decking roofing	$D_2 := 3\ psf$	
Membrane	$D_3 := 0.35\ psf$	
1/2" osb plywood sheathing	$D_4 := 1.7\ psf$	
1" Spray Foam Insulation (6")	$D_5 := 1\ psf \cdot 6$	(West Roofing Systems Inc)
		(First American Roofing & Siding)
HVAC Ducts	$D_6 := 0.25\ psf$	(Johns Manville)
Standard Roof	$L_1 := 20\ psf$	(ASCE)

$$D_{purlin} := D_1 + D_2 + D_3 + D_4 + D_5 = 13.55\ psf$$

$$L_{rupper} := L_1 = 20\ psf \qquad L_{lower} := 0\ psf$$

$$l := 12\ ft \qquad l_{upper} := \sqrt{(1\ ft)^2 + l^2} = 12.042\ ft$$

$$D_{upperCorrection} := D_{purlin} \cdot \frac{l}{l_{upper}} = 13.503\ psf \qquad L_{rupperCorrection} := L_{rupper} \cdot \frac{l}{l_{upper}} = 19.931\ psf$$

$$S_{maxCorrection} := S_{max} \cdot \frac{l}{l_{upper}} = 37.223\ psf \qquad W_{maxCorrection} := \left|F_{wMax}\right| \cdot \frac{l}{l_{upper}} = 26.798\ psf$$

Roof

$$T_w := 24\ in$$

$$w_{upper1} := T_w \cdot \left(1.4 \cdot D_{upperCorrection}\right) = 37.809\ plf$$

$$w_{upper2} := T_w \cdot \left(1.2 \cdot D_{upperCorrection} + 1.6 \cdot 0\ psf + \max\left(0.5 \cdot L_{rupperCorrection}, 0.5 \cdot S_{maxCorrection}\right)\right) = 69.631\ plf$$

$$w_{upper3} := T_w \cdot \left(1.2 \cdot D_{upperCorrection} + \max\left(1.6 \cdot L_{rupperCorrection}, S_{maxCorrection}\right) + \max\left(L_{lower}, 0.5 \cdot W_{maxCorrection}\right)\right) = 133.652\ plf$$

$$w_{upper4} := T_w \cdot \left(1.2 \cdot D_{upperCorrection} + W_{maxCorrection} + L_{lower} + \max\left(0.5 \cdot L_{rupperCorrection}, 0.3 \cdot S_{maxCorrection}\right)\right) = 108.338\ plf$$

$$w_{upper5} := T_w \cdot \left(0.9 \cdot D_{upperCorrection} + W_{maxCorrection}\right) = 77.902\ plf$$

$$MaxLoad := w_{upper3}$$

$D_{upperCorrection}$ $\quad S_{maxCorrection}$ $\quad W_{maxCorrection}$ to be used in calculations

Roof Truss

$$D_{purlin} := \frac{10.5\ \frac{lbf}{ft}}{24\ in} = 5.25\ psf$$

$$D_{upper} := D_2 + D_3 + D_4 + D_5 + D_{purlin} = 16.3\ psf \qquad D_{lower} := D_1 + D_6 = 2.75\ psf$$

$$L_{rupper} := L_1 = 20\ psf \qquad L_{lower} := 0\ psf$$

$$l := 12\ ft \qquad l_{upper} := \sqrt{(1\ ft)^2 + l^2} = 12.042\ ft$$

$$D_{upperCorrection} := D_{upper} \cdot \frac{l}{l_{upper}} = 16.244\ psf \qquad D_{lowerCorrection} := D_{lower} \cdot \frac{l}{l_{upper}} = 2.741\ psf$$

$$L_{rupperCorrection} := L_{rupper} \cdot \frac{l}{l_{upper}} = 19.931\ psf$$

$$S_{maxCorrection} := S_{max} \cdot \frac{l}{l_{upper}} = 37.223\ psf \qquad W_{maxCorrection} := \left|F_{wMax}\right| \cdot \frac{l}{l_{upper}} = 26.798\ psf$$

$$D_{maxC} := D_{upperCorrection} + D_{lowerCorrection} = 18.984\ psf$$

$$T_w := 20\ ft$$

$$w_{upper1} := T_w \cdot \left(1.4 \cdot D_{upperCorrection}\right) = 454.823\ plf$$

$$w_{upper2} := T_w \cdot \left(1.2 \cdot D_{upperCorrection} + 1.6 \cdot 0\ psf + \max\left(0.5 \cdot L_{rupperCorrection}, 0.5 \cdot S_{maxCorrection}\right)\right) = 762.078\ plf$$

$$w_{upper3} := T_w \cdot \left(1.2 \cdot D_{upperCorrection} + \max\left(1.6 \cdot L_{rupperCorrection}, S_{maxCorrection}\right) + \max\left(L_{lower}, 0.5 \cdot W_{maxCorrection}\right)\right) = 1402.292\ plf$$

$$w_{upper4} := T_w \cdot \left(1.2 \cdot D_{upperCorrection} + W_{maxCorrection} + L_{lower} + \max\left(0.5 \cdot L_{rupperCorrection}, 0.3 \cdot S_{maxCorrection}\right)\right) = 1149.154\ plf$$

$$w_{upper5} := T_w \cdot \left(0.9 \cdot D_{upperCorrection} + W_{maxCorrection}\right) = 828.354\ plf$$

Roof Alternative
Perlin

$T_w := 40\ in \qquad span := 20\ ft$

Load Calculation	From Boise Cascade	
Suspended wood channel system	$D_1 := 2.5\ psf$	
18 gauge metal decking roofing	$D_2 := 3\ psf$	
Membrane	$D_3 := 0.35\ psf$	
1/2" osb plywood sheathing	$D_4 := 1.7\ psf$	
1" Rigid Insulation (3")	$D_5 := 1.5\ psf \cdot 3$	(West Roofing Systems Inc)
		(First American Roofing & Siding)
HVAC Ducts	$D_6 := 0.25\ psf$	(Johns Manville)
Standard Roof	$L_1 := 20\ psf$	(ASCE)

$$D_{upper} := D_1 + D_2 + D_5 = 10\ psf$$

$$L_{rupper} := L_1 = 20\ psf \qquad L_{lower} := 0\ psf$$

$$l_{upper} := \sqrt{(1\ ft)^2 + l^2} = 12.042\ ft$$

$$D_{upperCorrection} := \left(D_{purlin} + D_{upper}\right) \cdot \frac{l}{l_{upper}} = 15.197\ psf \qquad L_{rupperCorrection} := L_{rupper} \cdot \frac{l}{l_{upper}} = 19.931\ psf$$

$$S_{maxCorrection} := S_{max} \cdot \frac{l}{l_{upper}} = 37.223\ psf \qquad W_{maxCorrection} := \left|F_{wMax}\right| \cdot \frac{l}{l_{upper}} = 26.798\ psf$$

$$w_{upper1} := T_w \cdot \left(1.4 \cdot D_{upperCorrection}\right) = 70.921\ plf$$

$$w_{upper2} := T_w \cdot \left(1.2 \cdot D_{upperCorrection} + 1.6 \cdot 0\ psf + \max\left(0.5 \cdot L_{rupperCorrection}, 0.5 \cdot S_{maxCorrection}\right)\right) = 122.828\ plf$$

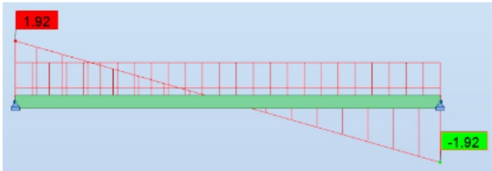
$$w_{upper3} := T_w \cdot \left(1.2 \cdot D_{upperCorrection} + \max\left(1.6 \cdot L_{rupperCorrection}, S_{maxCorrection}\right) + \max\left(L_{lower}, 0.5 \cdot W_{maxCorrection}\right)\right) = 229.53\ plf$$

$$w_{upper4} := T_w \cdot \left(1.2 \cdot D_{upperCorrection} + W_{maxCorrection} + L_{lower} + \max\left(0.5 \cdot L_{rupperCorrection}, 0.3 \cdot S_{maxCorrection}\right)\right) = 187.34\ plf$$

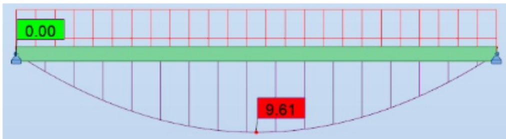
$$w_{upper5} := T_w \cdot \left(0.9 \cdot D_{upperCorrection} + W_{maxCorrection}\right) = 134.92\ plf$$

$$D_{upperCorrection} \cdot T_w = 0.051\ \frac{1}{ft} \cdot kip \qquad S_{maxCorrection} \cdot T_w = 0.124\ \frac{kip}{ft}$$

$$W_{maxCorrection} \cdot T_w = 0.089\ \frac{kip}{ft}$$



Shear



Bending

$L = 93.896\ ft$

$T_w = 40\ in$

4x12 SP no 2

$E := 1400\ ksi$ $I := 415.2832\ in^4$ $d := 11.25\ in$ $b := 3.5\ in$

Deflection

$$w_{ST} := 0.5 \cdot L_{rupperCorrection} \cdot T_w = 0.033\ \frac{kip}{ft}$$

$$w_{LT} := \left(D_{upperCorrection} + 0.5 \cdot L_{rupperCorrection}\right) \cdot T_w = 0.084\ \frac{kip}{ft}$$

$$\delta_{ST} := \frac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot (span)^4}{E \cdot I} = 0.109\ in \quad < \quad \Delta_{ST} := \frac{(span)}{360} = 0.667\ in$$

$$\delta_{LT} := \frac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot (span)^4}{E \cdot I} = 0.275\ in \quad < \quad \Delta_{LT} := \frac{(span)}{360} = 0.667\ in$$

$$\delta_{tot} := \left(1.5 \cdot \delta_{LT}\right) + \delta_{ST} = 0.522\ in \quad < \quad \Delta_{tot} := \frac{(span)}{240} = 1\ in$$

Tension

$A := b \cdot d$

$C_D := 1$ $C_m := 1$ $C_F := 1$ $C_r := 1.15$ $P_i := 1.92\ kip$

$C_t := 1$ $C_{fu} := 1$ $C_i := 1$ $C_L := 1$

$F_b := 750\ psi$

$w_l := L_{rupperCorrection}$

$F_v := 175\ psi$

$W_{tj1} := 40\ in$ $G := 0.55$

$w_{lj} := w_l \cdot W_{tj1} = 0.066\ \frac{kip}{ft}$ $w_d := D_{upperCorrection}$

Design for Bending Stress

$F'_b := F_b \cdot C_D \cdot C_m \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 862.5\ psi$

$F'_v := F_v \cdot C_D \cdot C_m \cdot C_t = 175\ psi$

$\gamma := 62.4\ pcf$

$w_{dj} := A \cdot G \cdot \gamma + W_{tj1} \cdot w_d = 60.042\ \frac{lbf}{ft}$

$M := 9.61\ kip \cdot ft$

$w_{dlj} := w_{dj} + w_{lj}$

$S := \frac{I}{d} = 0.16\ gal$

$M := \frac{w_{dlj} \cdot l^2}{8} = 2276.613\ lbf \cdot ft$

$f_b := \frac{M}{S} = 740.08\ psi$

$\frac{F'_b}{f_b} = 1.165$ DCR value appropriate.

Design for Shear Stress

$V := \frac{w_{dlj} \cdot l}{2} = 758.871\ lbf$

$F'_v := F_v \cdot C_D \cdot C_m \cdot C_t = 175\ psi$

$f_v := \frac{3 \cdot V}{2 \cdot A} = 28.909\ psi$ $f_v < F'_v$

Purlins designed as 4x12 Southern Pine No. 2 at 40" spacing o.c.

Glulam Truss

$T_w := 40\ in$ $span := 20\ ft$

Load Calculation	From Boise Cascade	
Suspended wood channel system	$D_1 := 2.5\ psf$	
18 gauge metal decking roofing	$D_2 := 3\ psf$	
Membrane	$D_3 := 0.35\ psf$	
1/2" osb plywood sheathing	$D_4 := 1.7\ psf$	
1" Rigid Insulation (3")	$D_5 := 1.5\ psf \cdot 3$	(West Roofing Systems Inc) (First American Roofing & Siding)
HVAC Ducts	$D_6 := 0.25\ psf$	(Johns Manville)
Purlin	$D_7 := \frac{A \cdot G \cdot \gamma}{T_w} = 2.815\ psf$	
Standard Roof	$L_1 := 20\ psf$	(ASCE)

$D_{upper} := D_1 + D_2 + D_5 + D_6 + D_7 = 13.065\ psf$ $T_w := 20\ ft$

$$L_{rupper} := L_1 = 20\ psf \qquad L_{lower} := 0\ psf$$

$$l_{upper} := \sqrt{(1\ ft)^2 + l^2} = 12.042\ ft$$

$$D_{upperCorrection} := D_{purlin} \cdot \frac{l}{l_{upper}} = 5.232\ psf \qquad L_{rupperCorrection} := L_{rupper} \cdot \frac{l}{l_{upper}} = 19.931\ psf$$

$$S_{maxCorrection} := S_{max} \cdot \frac{l}{l_{upper}} = 37.223\ psf \qquad W_{maxCorrection} := \left|F_{wMax}\right| \cdot \frac{l}{l_{upper}} = 26.798\ psf$$

$$w_{upper1} := T_w \cdot \left(1.4 \cdot D_{upperCorrection}\right) = 146.492\ plf$$

$$w_{upper2} := T_w \cdot \left(1.2 \cdot D_{upperCorrection} + 1.6 \cdot 0\ psf + \max\left(0.5 \cdot L_{rupperCorrection}, 0.5 \cdot S_{maxCorrection}\right)\right) = 497.795\ plf$$

$$w_{upper3} := T_w \cdot \left(1.2 \cdot D_{upperCorrection} + \max\left(1.6 \cdot L_{rupperCorrection}, S_{maxCorrection}\right) + \max\left(L_{lower}, 0.5 \cdot W_{maxCorrection}\right)\right) = 1138.008\ plf$$

$$w_{upper4} := T_w \cdot \left(1.2 \cdot D_{upperCorrection} + W_{maxCorrection} + L_{lower} + \max\left(0.5 \cdot L_{rupperCorrection}, 0.3 \cdot S_{maxCorrection}\right)\right) = 884.87\ plf$$

$$w_{upper5} := T_w \cdot \left(0.9 \cdot D_{upperCorrection} + W_{maxCorrection}\right) = 630.141\ plf$$

Solid Glulam Beam for Truss

$$F_t := 1150\ psi \qquad F_c := 1600\ psi \qquad F_b := 2400\ psi$$

$$b := 8.5\ in \qquad d := 44\ in \qquad A := b \cdot d = 374\ in^2 \qquad I := \frac{b \cdot d^3}{12} = 60338.667\ in^4$$

$$S := \frac{I}{\frac{d}{2}} = 2742.667\ in^3 \qquad E := 1800000\ psi \qquad E_{min} := 660000\ psi$$

$$L := 58.6\ ft \qquad T_w := \frac{19.3\ ft + 20\ ft}{2} = 19.65\ ft$$

$$D_{selfGL} := \frac{62.4\ pcf \cdot G \cdot A}{T_w}$$

$$w := T_w \cdot \left(1.2 \cdot \left(D_{upperCorrection} + D_{selfGL}\right) + \max\left(1.6 \cdot L_{rupperCorrection}, S_{maxCorrection}\right) + \max\left(L_{lower}, 0.5 \cdot W_{maxCorrection}\right)\right) = 1225.057\ plf$$

$$w_{correction} := w \cdot \sin\left(\left(90 - \operatorname{atan}\left(\frac{1}{12}\right) \cdot \frac{180}{\pi}\right) \cdot \frac{\pi}{180}\right) = 1220.825\ plf$$

$$w_p := w \cdot \cos\left(\left(90 - \operatorname{atan}\left(\frac{1}{12}\right) \cdot \frac{180}{\pi}\right) \cdot \frac{\pi}{180}\right) = 101.735\ plf$$

$$M_u := \frac{w_{correction} \cdot L^2}{8} = 524.033\ kip \cdot ft \qquad P_u := \frac{w_p \cdot L}{2} = 2.981\ kip \qquad V := \frac{w_{correction} \cdot L}{2} = 35.77\ kip$$

Adjustment Factors

$$C_D := 1.0 \quad C_{fu} := 1.00 \quad C_i := 1.0 \quad C_M := 1.0 \quad C_r := 1.00 \quad C_P := 1.0 \quad C_{vr} := 1.0$$

$C_t := 1.0$ $C_F := 1.0$ $C_T := 1.0$

$x := 20$ $t := 5.5\ in$

$$C_V := \left(\frac{5.125\ in}{b}\right)^{\frac{1}{x}} \cdot \left(\frac{12\ in}{d}\right)^{\frac{1}{x}} \cdot \left(\frac{21\ ft}{L}\right)^{\frac{1}{x}} = 0.868 \quad > 1$$

$C_V := 1$

$f_c := \frac{P_u}{A} = 7.97\ psi$ $f_b := \frac{M_u}{S} = 2.293\ ksi$

Bending

$l_u := L = 58.6\ ft$ $\frac{l_u}{d} = 15.982 \quad \geq 7$ $l_e := 1.63 \cdot l_u + 3 \cdot d = 106.518\ ft$

$R_B := \sqrt{\frac{l_e \cdot d}{b^2}} = 27.9$ $F°_b := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \cdot C_r = 2400\ psi$

$E'_{min} := E_{min} \cdot C_M \cdot C_t \cdot C_i = 660\ ksi$ $F_{bE} := \frac{1.2 \cdot E'_{min}}{R_B} = 28.387\ ksi$

$E'_{min} := 660\ ksi$

$$C_L := \frac{1 + \left(\frac{F_{bE}}{F°_b}\right)}{1.9} - \sqrt{\left(\frac{1 + \left(\frac{F_{bE}}{F°_b}\right)}{1.9}\right)^2 - \frac{\left(\frac{F_{bE}}{F°_b}\right)}{0.95}} = 0.995$$

$F'_b := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 2.389\ ksi$

$\frac{F'_b}{f_b} = 1.042$ $f_b < F'_b$

$F'_b = 2.389\ ksi$ > $f_b = 2.293\ ksi$

Compression

$F'_c := F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i = 1600\ psi$

$R_B := \sqrt{\frac{\left(\frac{L}{3}\right) \cdot d}{b^2}} = 11.948$ $F_{cStar} := \frac{F'_c}{C_P} = 1600\ psi$

$E'_{min} := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = 660000\ psi$

$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\frac{L}{d}\right)^2} = 2124.043\ psi$ $F_{bE} := \frac{1.2 \cdot E'_{min}}{{R_B}^2} = 5548.208\ psi$

$c := 0.8$

$\alpha := \frac{1 + \frac{F_{cE}}{F_{cStar}}}{2 \cdot c} = 1.455$ $\beta := \frac{\frac{F_{cE}}{F_{cStar}}}{c} = 1.659$

$$C_P := \alpha - \sqrt{\alpha^2 - \beta} = 0.779$$

$F'_c := F_{cStar} \cdot C_P = 1246.187\ psi$ > $f_c = 7.97\ psi$

Shear

$$F'_v := F_v \cdot C_D \cdot C_m \cdot C_t \cdot C_{vr} = 175\ psi$$

$$f_v := \frac{3 \cdot V}{2 \cdot A} = 143.463\ psi$$

$$F'_v > f_v$$

$\frac{f_c}{F_{cE}} + \left(\frac{f_b}{F_{bE}}\right)^2 = 0.175$ < 1

Design is adequate

Deflection

$$w_{ST} := 0.5 \cdot L_{rupperCorrection} \cdot T_w = 0.196\ \frac{kip}{ft}$$

$$w_{LT} := \left(D_{upperCorrection} + D_{selfGL} + 0.5 \cdot L_{rupperCorrection}\right) \cdot T_w = 0.388\ \frac{kip}{ft}$$

$\delta_{ST} := \frac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot (L)^4}{E \cdot I} = 0.253\ in$ < $\Delta_{ST} := \frac{(L)}{360} = 1.953\ in$

$\delta_{LT} := \frac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot (L)^4}{E \cdot I} = 0.502\ in$ < $\Delta_{LT} := \frac{(L)}{360} = 1.953\ in$

$\delta_{tot} := \left(1.5 \cdot \delta_{LT}\right) + \delta_{ST} = 1.006\ in$ < $\Delta_{tot} := \frac{(L)}{240} = 2.93\ in$

Member composed of 24F-V5 SP/SP 8 1/2 x 44 Design is adequate

Main Event Space Floor Plan

Joist

$b := 3.5\ in$ $\quad d := 7.25\ in$ $\quad A := b \cdot d = 25.375\ in^2$

$l_{max} := 12.4\ ft$ $\quad G := 0.55$

$D_{self} := G \cdot \gamma \cdot A = 6.048\ plf$

$F_b := 925\ psi$

$D_{hardwood} := 4\ psf$

$F_v := 175\ psi$

$D_{gypsum} := 2.2\ psf$

$D_{sub} := 2.5\ psf$

$E := 1400\ ksi$

$T_w := 24\ in$

$E_{min} := 510\ ksi$

$L_f := 100\ psf$ Assembly area with movable seating

$$D := 1.2 \cdot \left(\left(D_{hardwood} + D_{gypsum} + D_{sub}\right) \cdot T_w + D_{self}\right) = 28.137\ plf$$

$$L := 1.6 \cdot L_f \cdot T_w = 320\ plf$$

$$I := \frac{1}{12} \cdot 3 \cdot b \cdot d^3 = 333.443\ in^4 \qquad S := \frac{I}{\left(\frac{d}{2}\right)} = 91.984\ in^3$$

$$w_{dl} := D + L$$

$$M := \frac{w_{dl} \cdot l_{max}^{\ 2}}{8} = 6.691\ kip \cdot ft$$

$$f_b := \frac{M}{S} = 0.873\ ksi$$

$C_M := 1$ $\quad C_F := 1$ $\quad C_i := 1$

$C_t := 1$ $\quad C_{fu} := 1$

$C_r := 1.15$ $\quad C_D := 1$ $\quad C_L := 1$

Table 4.3.1 Applicability of Adjustment Factors for Sawn Lumber

		ASD only	ASD and LRFD										LRFD only		
		Load Duration Factor	Wet Service Factor	Temperature Factor	Beam Stability Factor	Size Factor	Flat Use Factor	Incising Factor	Repetitive Member Factor	Column Stability Factor	Buckling Stiffness Factor	Bearing Area Factor	Format Conversion Factor K_F	Resistance Factor ϕ	Time Effect Factor
$F_b' = F_b$	x	C_D	C_M	C_t	C_L	C_F	C_{fu}	C_i	C_r	-	-	-	2.54	0.85	λ
$F_t' = F_t$	x	C_D	C_M	C_t	-	C_F	-	C_i	-	-	-	-	2.70	0.80	λ
$F_v' = F_v$	x	C_D	C_M	C_t	-	-	-	C_i	-	-	-	-	2.88	0.75	λ
$F_c' = F_c$	x	C_D	C_M	C_t	-	C_F	-	C_i	-	C_P	-	-	2.40	0.90	λ
$F_{c\perp}' = F_{c\perp}$	x	-	C_M	C_t	-	-	-	C_i	-	-	-	C_b	1.67	0.90	-
$E' = E$	x	-	C_M	C_t	-	-	-	C_i	-	-	-	-	-	-	-
$E_{min}' = E_{min}$	x	-	C_M	C_t	-	-	-	C_i	-	-	C_T	-	1.76	0.85	-

$l_u := l_{max} = 12.4\ ft$ $\quad \frac{l_u}{d} = 20.524$ $\quad \geq 7$ $\quad l_e := 1.63 \cdot l_u + 3 \cdot d = 22.025\ ft$

$$R_B := \sqrt{\frac{l_e \cdot d}{b^2}} = 12.507 \qquad F^{\circ}_b := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \cdot C_r = 1063.75\ psi$$

$$E'_{min} := E_{min} \cdot C_M \cdot C_t \cdot C_i = 510\ ksi \qquad F_{bE} := \frac{1.2 \cdot E'_{min}}{R_B} = 48.934\ ksi$$

$$C_L := \frac{1+\left(\frac{F_{bE}}{F°_b}\right)}{1.9} - \sqrt{\left(\frac{1+\left(\frac{F_{bE}}{F°_b}\right)}{1.9}\right)^2 - \frac{\left(\frac{F_{bE}}{F°_b}\right)}{0.95}} = 0.999$$

$$F'_b := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 1062.571\ psi$$

$$f_b < F'_b$$

$$\frac{F'_b}{f_b} = 1.217$$

Shear

$$V := \frac{D \cdot l_{max}}{2} = 174.451\ lbf$$

$$F'_v := F_v \cdot C_D \cdot C_m \cdot C_t \cdot C_i = 175\ psi$$

$$f_v := \frac{3 \cdot V}{2 \cdot A} = 10.312\ psi \qquad f_v < F'_v$$

Deflection

$$w_{ST} := 0.5 \cdot L = 0.16\ \frac{kip}{ft}$$

$$w_{LT} := D = 0.028\ \frac{kip}{ft}$$

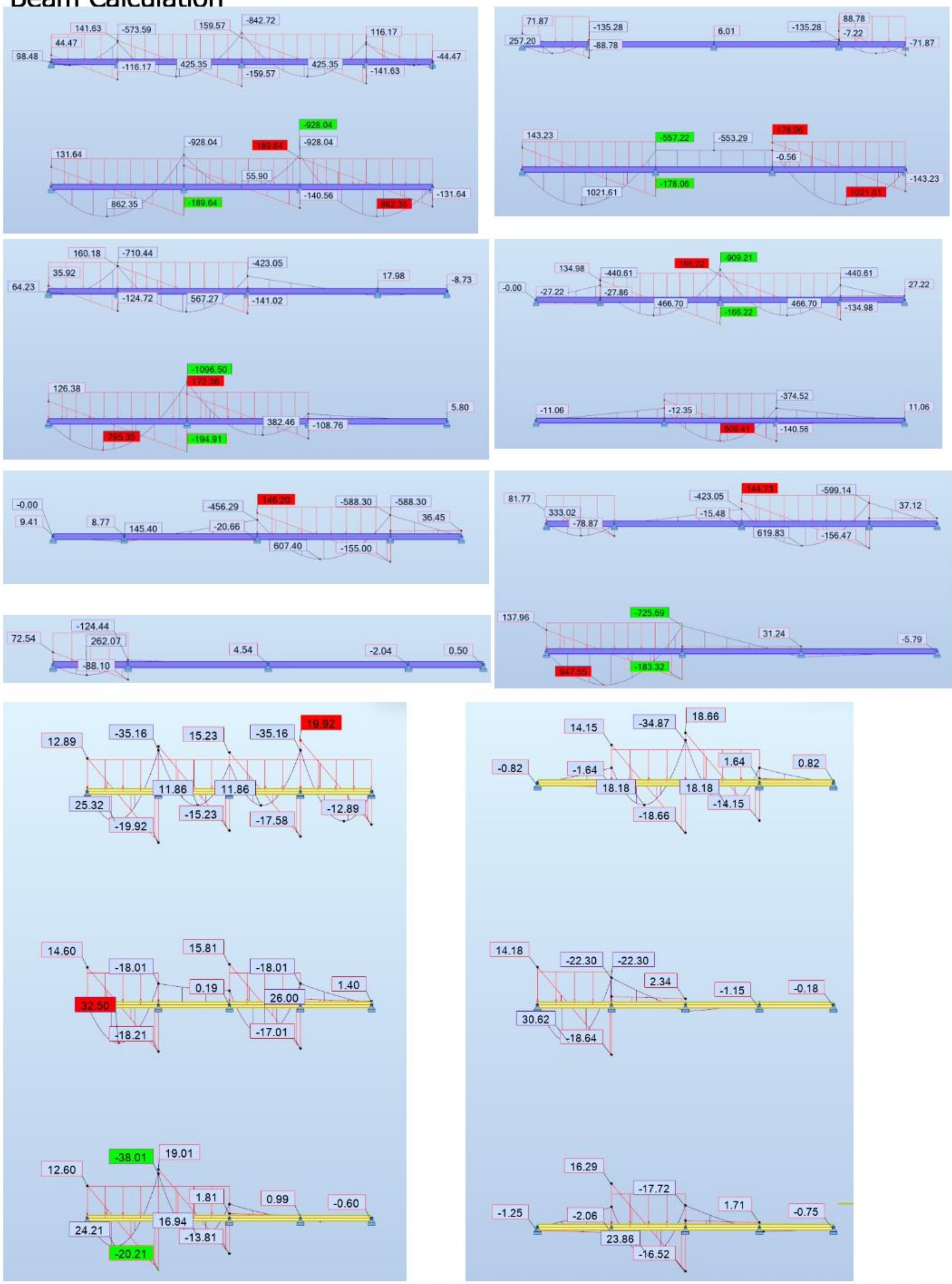
$$\delta_{ST} := \frac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot \left(l_{max}\right)^4}{E \cdot I} = 0.097\ in \quad < \quad \Delta_{ST} := \frac{\left(l_{max}\right)}{360} = 0.413\ in$$

$$\delta_{LT} := \frac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot \left(l_{max}\right)^4}{E \cdot I} = 0.017\ in \quad < \quad \Delta_{LT} := \frac{\left(l_{max}\right)}{360} = 0.413\ in$$

$$\delta_{tot} := \left(1.5 \cdot \delta_{LT}\right) + \delta_{ST} = 0.122\ in \quad < \quad \Delta_{tot} := \frac{\left(l_{max}\right)}{240} = 0.62\ in$$

3x10 or 4x8 can be used (SP No. 2 Dense)

Beam Calculation



-M: Adjacent Side
+M: Pattern
V: Adjacent Side

5 support tested

W12x50 A36

$W := 50\ plf$ $A := 14.6\ in^2$

$l := 90\ ft$ $l_{max} := 22.5\ ft$ $T_w := 12\ ft$

$E := 29000\ ksi$ $d := 12.2\ in$

$F_y := 36\ ksi$

$D_{self} := G \cdot \gamma \cdot A \cdot \frac{T_w}{24\ in} = 20.878\ plf$

$F_u := 58\ ksi$ $b_f := 8.08\ in$

$t_f := 0.64\ in$

$D_{hardwood} := 4\ psf$

$D_{gypsum} := 2.2\ psf$

$D_{sub} := 2.5\ psf$

$D_{beam} := W$

$L_f := 100\ psf$

Assembly area with movable seating

$Z_x := 71.9\ in^3$ $t_w := 0.37\ in$

$r_y := 1.96\ in$ $h_{tw} := 26.8$

$I_y := 56.3\ in^4$ $I_x := 391\ in^4$

$h := h_{tw} \cdot t_w = 9.916\ in$ $S_x := 64.2\ in^3$

$J := 1.71\ in^4$ $h_0 := 11.6\ in$

$C_w := 1880\ in^6$ $r_{ts} := 2.25\ in$

$D := 1.2 \cdot (((D_{hardwood} + D_{gypsum} + D_{sub}) \cdot T_w + D_{self}) + D_{beam}) = 210.334\ plf$

$L := 1.6 \cdot L_f \cdot T_w = 1920\ plf$

$D_{rob} := D - (1.2 \cdot D_{beam}) = 150.334\ plf$

$w_{dl} := D + L$

+M $M_{maxp} := 161.66\ kip \cdot ft$

-M $M_{max} := 189.98\ kip \cdot ft$

$V := -44.97\ kip$

Deflection Check

$w_{ST} := 0.5 \cdot L = 0.96\ \frac{kip}{ft}$

$w_{LT} := D = 0.21\ \frac{kip}{ft}$

$\delta_{ST} := \frac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot (l_{max})^4}{E \cdot I_x} = 0.259\ in$ < $\Delta_{ST} := \frac{(l_{max})}{360} = 0.75\ in$

$\delta_{LT} := \frac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot (l_{max})^4}{E \cdot I_x} = 0.057\ in$ < $\Delta_{LT} := \frac{(l_{max})}{360} = 0.75\ in$

$\delta_{tot} := (1.5 \cdot \delta_{LT}) + \delta_{ST} = 0.344\ in$ < $\Delta_{tot} := \frac{(l_{max})}{240} = 1.125\ in$

Design is adequate

Flexure

$$M_{xYield} := F_y \cdot S_x = 192.6\ kip \cdot ft \quad > 189.98\ kip \cdot ft$$

FLB

$$\lambda_f := 0.5 \cdot \frac{b_f}{t_f} = 6.313 \qquad \lambda_w := \frac{h}{t_w} = 26.8 \quad \le \quad 3.76 \cdot \sqrt{\frac{E}{F_y}} = 106.717$$

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 10.785 \qquad \lambda_{rf} := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 28.382$$

$$\lambda_f = 6.313 \quad \le \quad \lambda_{pf} = 10.785$$

$$M_p := Z_x \cdot F_y = 215.7\ kip \cdot ft \qquad \phi M_{nFLB} := 0.9 \cdot M_p = 194.13\ kip \cdot ft \quad > 189.98\ kip \cdot ft$$

LTB

$$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 8.159\ ft \qquad c := 1$$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_0} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_0}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 31.254\ ft$$

$$r_{ts} := \sqrt{\frac{\sqrt{I_y \cdot C_w}}{S_x}} = 2.251\ in \qquad L_b := 22.5\ ft$$

correction, $L_b := 2\ ft$ due to joists.

$$L_p > L_b$$

$$\phi M_{nFLB} := 0.9 \cdot M_p = 194.13\ kip \cdot ft \quad > 189.98\ kip \cdot ft$$

Shear Check

$$A_w := d \cdot t_w = 4.514\ in^2 \qquad \phi := 0.9$$

$$\phi V_n := \phi \cdot A_w \cdot 0.6 \cdot F_y = 87.752\ kip \quad > 44.97\ kip$$

Floor beams of main event space composed of W12x50 A36 steel beams.

Overhang floor plan design

Side Joist (Wood I Beam) from TrusJoist Weyerhaeuser

$T_w := 16\ in$

$L_s := 16\ ft$

11 7/8" 230

$D_{self} := 4\ plf$ $L_f := 100\ psf$

$D := 1.2 \cdot (((D_{hardwood} + D_{gypsum} + D_{sub}) \cdot T_w + D_{self})) = 18.72\ plf$

$L := 1.6 \cdot L_f \cdot T_w = 213.333\ plf$

$w := D + L = 232.053\ \frac{lbf}{ft}$

Properties

$M_{max} := 9500\ lbf \cdot ft$

$V_{max} := 2050\ lbf$

$EI := (636 \cdot 10^6)\ in^2 \cdot lbf$

$d := 11\ in + \frac{7}{8}\ in$

$M_s := \frac{w \cdot L_s^2}{8} = 7.426\ kip \cdot ft$ $V_s := \frac{w \cdot L_s}{2} = 1.856\ kip$

$\Delta := \frac{22.5 \cdot 232 \cdot 16^4}{636 \cdot 10^6} + \frac{2.29 \cdot 232 \cdot 16^2}{11.85 \cdot 10^5} = 0.653$ in

$\Delta_{tot} := \frac{L_s}{240} = 0.8\ in$

Side floor joists of main event space composed of 11 7/8" TJI 560 wood I-Joists.

Center Joists (W Steel)

$L_m := 26\ ft$ $T_w := 32\ in$

$D_{self} := W$ $L_f := 100\ psf$

$D := 1.2 \cdot (((D_{hardwood} + D_{gypsum} + D_{sub}) \cdot T_w + D_{self})) = 0.058\ \frac{kip}{ft}$

$L := 1.6 \cdot L_f \cdot T_w = 0.427\ \frac{kip}{ft}$

$w := D + L = 0.485\ \frac{kip}{ft}$

W6X25 A36

$W := 25\ plf$ $A := 7.34\ in^2$

$E := 29000\ ksi$ $d := 6.38\ in$

$F_y := 36\ ksi$

$F_u := 58\ ksi$ $b_f := 6.08\ in$

$t_f := 0.455\ in$

$Z_x := 18.9\ in^3$ $t_w := 0.32\ in$

$r_y := 1.52\ in$ $h_{tw} := 26.8$

$I_y := 17.1\ in^4$ $I_x := 53.4\ in^4$

$h := h_{tw} \cdot t_w = 8.576\ in$ $S_x := 16.7\ in^3$

$J := 0.461\ in^4$ $h_0 := 5.93\ in$

$C_w := 150\ in^6$ $r_{ts} := 1.74\ in$

$M_{max} := \frac{w \cdot L_m^2}{8} = 40.941\ kip \cdot ft$

$V_{max} := \frac{w \cdot L_m}{2} = 6.299\ kip$

Deflection Check

$w_{ST} := 0.5 \cdot L = 0.213 \ \frac{kip}{ft}$

$w_{LT} := D = 0.058 \ \frac{kip}{ft}$

$\delta_{ST} := \frac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot (L_m)^4}{E \cdot I_x} = 0.751 \ in$ < $\Delta_{ST} := \frac{(L_m)}{360} = 0.867 \ in$

$\delta_{LT} := \frac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot (L_m)^4}{E \cdot I_x} = 0.204 \ in$ < $\Delta_{LT} := \frac{(L_m)}{360} = 0.867 \ in$

$\delta_{tot} := (1.5 \cdot \delta_{LT}) + \delta_{ST} = 1.056 \ in$ < $\Delta_{tot} := \frac{(L_m)}{240} = 1.3 \ in$

Flexure

$M_{xYield} := F_y \cdot S_x = 50.1 \ kip \cdot ft \ > 43.78 \ (kip \cdot ft)$

FLB

$\lambda_f := 0.5 \cdot \frac{b_f}{t_f} = 6.681$ $\lambda_w := \frac{h}{t_w} = 26.8 \ \le \ 3.76 \cdot \sqrt{\frac{E}{F_y}} = 106.717$

$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 10.785$ $\lambda_{rf} := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 28.382$

$\lambda_f = 6.681 \ \le \ \lambda_{pf} = 10.785$

$M_p := Z_x \cdot F_y = 56.7 \ kip \cdot ft$ $\phi M_{nFLB} := 0.9 \cdot M_p = 51.03 \ kip \cdot ft \ > 40.941 \ (kip \cdot ft)$

LTB

$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 6.327 \ ft$ $c := 1$

$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_0} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_0}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 32.26 \ ft$

$r_{ts} := \sqrt{\frac{\sqrt{I_y \cdot C_w}}{S_x}} = 1.741 \ in$ $L_b := 26 \ ft$ correction, $L_b := 2 \ ft$ due to joists.

$L_p < L_b < L_r$

$$M(x) := V_{max} \cdot x - \frac{1}{2} \cdot w \cdot x^2 \qquad M_A := M\left(L_m \cdot \frac{1}{4}\right) \qquad M_B := M\left(L_m \cdot \frac{2}{4}\right) \qquad M_C := M\left(L_m \cdot \frac{3}{4}\right)$$

$$C_b := min\left(3, \frac{12.5 \cdot M_{max}}{2.5 \cdot M_{max} + 3 \cdot M_A + 4 \cdot M_B + 3 \cdot M_C}\right) = 1.136$$

$$\phi M_{nFLB} := 0.9 \cdot min\left(M_p, C_b \cdot \left(M_p - \left(M_p - 0.7 \cdot F_y \cdot S_x\right) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right)\right) = 51.03 \; kip \cdot ft$$

$$> 40.941 \; (kip \cdot ft)$$

Shear Check

$$A_w := d \cdot t_w = 2.042 \; in^2 \qquad \phi := 0.9$$

$$\phi V_n := \phi \cdot A_w \cdot 0.6 \cdot F_y = 39.689 \; kip \qquad > 6.299 \; kip$$

Center floor joists of main event space composed of W6X25 A36 steel beams.

Floor Beam Side

$$l_1 := \frac{16}{2} \; ft \qquad l_2 := \frac{26}{2} \; ft \qquad T_w := l_1 + l_2 = 21 \; ft$$

$$D_{wj} := \frac{W \cdot l_2}{32 \; in} = 121.875 \; plf \qquad D_{ij} := \frac{4 \; plf \cdot l_1}{16 \; in} = 24 \; plf$$

W14X22 A36

(14 chosen to allow for joists to fit in web to reduce floor depth, W6X25 may be used if depth is not a concern or is preferred)

$W := 22 \; plf$ $\quad A := 6.49 \; in^2$

$E := 29000 \; ksi$ $\quad d := 13.7 \; in$

$F_y := 36 \; ksi$

$F_u := 58 \; ksi$ $\quad b_f := 5 \; in$

$t_f := 0.335 \; in$

$Z_x := 33.2 \; in^3$ $\quad t_w := 0.32 \; in$

$r_y := 1.04 \; in$ $\quad h_{tw} := 53.3$

$I_y := 7 \; in^4$ $\quad I_x := 199 \; in^4$

$h := h_{tw} \cdot t_w = 17.056 \; in$ $\quad S_x := 29 \; in^3$

$J := 0.208 \; in^4$ $\quad h_0 := 13.4 \; in$

$C_w := 314 \; in^6$ $\quad r_{ts} := 1.27 \; in$

Single Span

$L_{si} := 18\ ft$ $\quad T_{ws} := 16\ ft$

Multi Span

$L_{mi} := 23\ ft$ $\quad T_{wm} := \frac{26\ ft + 16\ ft}{2} = 21\ ft$

$L_{m1} := 11\ ft$ $\quad L_{m2} := 12\ ft$

$$D_s := 1.2 \cdot \left(\left(\left(D_{hardwood} + D_{gypsum} + D_{sub}\right) \cdot T_{ws} + D_{self} + 2 \cdot D_{ij}\right)\right) = 0.255\ \frac{kip}{ft}$$

$$D_m := 1.2 \cdot \left(\left(\left(D_{hardwood} + D_{gypsum} + D_{sub}\right) \cdot T_{wm} + D_{self} + D_{wj} + D_{ij}\right)\right) = 0.424\ \frac{kip}{ft}$$

$$L_s := 1.6 \cdot L_f \cdot T_{ws} = 2.56\ \frac{kip}{ft} \qquad D_{robS} := \frac{D_s - D_{self}}{1.2} = 0.191\ \frac{kip}{ft}$$

$$L_m := 1.6 \cdot L_f \cdot T_{wm} = 3.36\ \frac{kip}{ft} \qquad L_{robS} := L_f \cdot T_{ws} = 1.6\ \frac{kip}{ft}$$

$$w_s := D_s + L = 0.681\ \frac{kip}{ft} \qquad D_{robM} := \frac{D_m - D_{self}}{1.2} = 0.333\ \frac{kip}{ft}$$

$$w_m := D_s + L = 0.681\ \frac{kip}{ft} \qquad L_{robM} := L_f \cdot T_{wm} = 2.1\ \frac{kip}{ft}$$

$$M_s := \frac{L_{si}^{\ 2} \cdot w_s}{8} = 27.593\ kip \cdot ft \qquad M_p := F_y \cdot Z_x = 99.6\ kip \cdot ft$$

$$V_s := \frac{L_{si} \cdot w_s}{2} = 6.132\ kip$$

Deflection Check

$$w_{ST} := 0.5 \cdot L = 0.213\ \frac{kip}{ft}$$

$$w_{LT} := D = 0.058\ \frac{kip}{ft}$$

$$\delta_{ST} := \frac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot \left(L_{si}\right)^4}{E \cdot I_x} = 0.046\ in \quad < \quad \Delta_{ST} := \frac{\left(L_{si}\right)}{360} = 0.6\ in$$

$$\delta_{LT} := \frac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot \left(L_{si}\right)^4}{E \cdot I_x} = 0.013\ in \quad < \quad \Delta_{LT} := \frac{\left(L_{si}\right)}{360} = 0.6\ in$$

$$\delta_{tot} := \left(1.5 \cdot \delta_{LT}\right) + \delta_{ST} = 0.065\ in \quad < \quad \Delta_{tot} := \frac{\left(L_{si}\right)}{240} = 0.9\ in$$

Flexure

$M_{xYield} := F_y \cdot S_x = 87\ kip \cdot ft \quad > 28.808\ (kip \cdot ft)$

FLB

$\lambda_f := 0.5 \cdot \frac{b_f}{t_f} = 7.463 \qquad \lambda_w := \frac{h}{t_w} = 53.3 \quad \le \quad 3.76 \cdot \sqrt{\frac{E}{F_y}} = 106.717$

$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 10.785 \qquad \lambda_{rf} := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 28.382$

$\lambda_f = 7.463 \quad \le \quad \lambda_{pf} = 10.785$

$M_p := Z_x \cdot F_y = 99.6\ kip \cdot ft \qquad \phi M_{nFLB} := 0.9 \cdot M_p = 89.64\ kip \cdot ft \quad > 28.808\ (kip \cdot ft)$

LTB

$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 4.329\ ft \qquad c := 1$

$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_0} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_0}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 12.695\ ft$

$r_{ts} := \sqrt{\frac{\sqrt{I_y \cdot C_w}}{S_x}} = 1.271\ in \qquad L_b := 18\ ft$ correction, $L_b := 2\ ft$
due to joists.

$L_b < L_p$

$\phi M_{nLTB} := 0.9 \cdot M_p = 89.64\ kip \cdot ft \quad > 62.94\ (kip \cdot ft)$

Shear Check

$A_w := d \cdot t_w = 4.384\ in^2 \qquad \phi := 0.9$

$\phi V_n := \phi \cdot A_w \cdot 0.6 \cdot F_y = 85.225\ kip \quad > 6.402\ kip$

Side floor beams of main event space composed of W14X22 A36 steel beams.

Floor Beam Middle

W14X22 A36

$W := 22\ plf$ $\quad A := 6.49\ in^2$

$E := 29000\ ksi$ $\quad d := 13.7\ in$

$F_y := 36\ ksi$

$F_u := 58\ ksi$ $\quad b_f := 5\ in$

$t_f := 0.335\ in$

$Z_x := 33.2\ in^3$ $\quad t_w := 0.32\ in$

$r_y := 1.04\ in$ $\quad h_{tw} := 53.3$

$I_y := 7\ in^4$ $\quad I_x := 199\ in^4$

$h := h_{tw} \cdot t_w = 17.056\ in$ $\quad S_x := 29\ in^3$

$J := 0.208\ in^4$ $\quad h_0 := 13.4\ in$

$C_w := 314\ in^6$ $\quad r_{ts} := 1.27\ in$

Multi Span

$L_{mi} := 23\ ft$ $\quad T_{wm} := \dfrac{26\ ft + 16\ ft}{2} = 21\ ft$

$L_{m1} := 11\ ft$ $\quad L_{m2} := 12\ ft$

$$D_m := 1.2 \cdot \left(\left(\left(D_{hardwood} + D_{gypsum} + D_{sub}\right) \cdot T_{wm} + D_{self} + D_{wj} + D_{ij}\right)\right) = 0.424\ \frac{kip}{ft}$$

$$L_m := 1.6 \cdot L_f \cdot T_{wm} = 3.36\ \frac{kip}{ft}$$

$$w_m := D_s + L = 0.681\ \frac{kip}{ft}$$

$$D_{robM} := \frac{D_m - D_{self}}{1.2} = 0.333\ \frac{kip}{ft}$$

$$L_{robM} := L_f \cdot T_{wm} = 2.1\ \frac{kip}{ft}$$

$M_{max} := 62.94\ kip \cdot ft$ $\quad V_{max} := 27.96\ kip$ $\quad M_p := F_y \cdot Z_x = 99.6\ kip \cdot ft$

Deflection Check

$$w_{ST} := 0.5 \cdot L = 0.213\ \frac{kip}{ft}$$

$$w_{LT} := D = 0.058\ \frac{kip}{ft}$$

$$\delta_{ST} := \frac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot \left(L_{m2}\right)^4}{E \cdot I_x} = 0.009\ in \quad < \quad \Delta_{ST} := \frac{\left(L_{m2}\right)}{360} = 0.4\ in$$

$$\delta_{LT} := \frac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot \left(L_{m2}\right)^4}{E \cdot I_x} = 0.002\ in \quad < \quad \Delta_{LT} := \frac{\left(L_{m2}\right)}{360} = 0.4\ in$$

$\delta_{tot} := (1.5 \cdot \delta_{LT}) + \delta_{ST} = 0.013\ in$ < $\Delta_{tot} := \dfrac{(L_{m2})}{240} = 0.6\ in$

Flexure

$M_{xYield} := F_y \cdot S_x = 87\ kip \cdot ft \quad > 28.808\ (kip \cdot ft)$

FLB

$\lambda_f := 0.5 \cdot \dfrac{b_f}{t_f} = 7.463 \qquad \lambda_w := \dfrac{h}{t_w} = 53.3 \quad \le \quad 3.76 \cdot \sqrt{\dfrac{E}{F_y}} = 106.717$

$\lambda_{pf} := 0.38 \cdot \sqrt{\dfrac{E}{F_y}} = 10.785 \qquad \lambda_{rf} := 1.0 \cdot \sqrt{\dfrac{E}{F_y}} = 28.382$

$\lambda_f = 7.463 \quad \le \quad \lambda_{pf} = 10.785$

$M_p := Z_x \cdot F_y = 99.6\ kip \cdot ft \qquad \phi M_{nFLB} := 0.9 \cdot M_p = 89.64\ kip \cdot ft \quad > 62.94\ (kip \cdot ft)$

LTB

$L_p := 1.76 \cdot r_y \cdot \sqrt{\dfrac{E}{F_y}} = 4.329\ ft \qquad c := 1$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_0} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_0}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 12.695\ ft$$

$r_{ts} := \sqrt{\dfrac{\sqrt{I_y \cdot C_w}}{S_x}} = 1.271\ in \qquad L_b := 12\ ft$ correction, $L_b := 2\ ft$ due to joists.

$L_b < L_p$

$\phi M_{nLTB} := 0.9 \cdot M_p = 89.64\ kip \cdot ft \quad > 62.94\ (kip \cdot ft)$

Shear Check

$A_w := d \cdot t_w = 4.384\ in^2 \qquad \phi := 0.9$

$\phi V_n := \phi \cdot A_w \cdot 0.6 \cdot F_y = 85.225\ kip \quad > 6.402\ kip$

Middle floor beams of main event space composed of W14X22 A36 steel beams.

Remaining Floor Joists

I-Joist 1

$span := 16\ ft$ $\qquad T_w := 16\ in$

$L := 100\ psf$ Corridor
$L := 40\ psf$ Classroom

Design for worse case scenario: $L := 100\ psf$

$D_1 := 2.2\ psf$ 1/2" gypsum board
$D_2 := 1.7\ psf$ 2x6 @ 16" o.c.
$D_3 := \max(16\ psf, 4\ psf)$ Tile on mortar or hard wood
$D_4 := 2.5\ psf$ Subfloor

14" TJI 560

$M_{max} := 11275\ lbf \cdot ft$
$V_{max} := 2390\ lbf$
$EI := (926 \cdot 10^6)\ in^2 \cdot lbf$
$d := 14\ in$

$D_{self} := 4.2\ plf$

$D := D_1 + D_2 + D_3 + D_4 = 22.4\ psf$ $\qquad L = 100\ psf$

$w := (1.2 \cdot D + 1.6 \cdot L) \cdot T_w + 1.2 \cdot D_{self} = 254.213\ plf$

$M := \frac{w \cdot span^2}{8} = 8134.827\ lbf \cdot ft$ $\qquad V := \frac{w \cdot span}{2} = 2033.707\ lbf$

$\Delta := \frac{22.5 \cdot 254.213 \cdot 16^4}{926 \cdot 10^6} + \frac{2.29 \cdot 254.213 \cdot 16^2}{14 \cdot 10^6} = 0.415$ $\qquad \Delta_{tot} := \frac{(span)}{360} = 0.533\ in$

Design Criteria Met

I-Joist 1 will be 14" TJI 560

I-Joist 2

$span := 19.5\ ft$ $\qquad T_w = 16\ in$
$D_{self} := 4.5\ plf$

14" TJI 560

$M_{max} := 12925\ lbf \cdot ft$
$V_{max} := 2710\ lbf$
$EI := (1252 \cdot 10^6)\ in^2 \cdot lbf$
$d := 16\ in$

$D_3 := 4\ psf$ Only hard wood

$D := D_1 + D_2 + D_3 + D_4 = 10.4\ psf$ $\qquad L = 100\ psf$

$w := (1.2 \cdot D + 1.6 \cdot L) \cdot T_w + 1.2 \cdot D_{self} = 235.373\ plf$

$M := \frac{w \cdot span^2}{8} = 11187.589\ lbf \cdot ft$ $\quad V := \frac{w \cdot span}{2} = 2294.89\ lbf$

$\Delta := \frac{22.5 \cdot 235.373 \cdot 19.5^4}{1252 \cdot 10^6} + \frac{2.29 \cdot 235.373 \cdot 19.5^2}{16 \cdot 10^6} = 0.624$ $\quad \Delta_{tot} := \frac{(span)}{360} = 0.65\ in$

Design Criteria Met

I-Joist 2 will be 16" TJI 560

Floor Beam

Due to shear loads, steel beams will be used instead

3 3x12s Southern Pine

$l_1 := 14\ ft$ $\quad l_2 := 16.5\ ft$

Worst Case Loading

$D = 10.4\ psf$ $\quad L = 100\ psf$ $\quad T_w := \frac{l_1 + l_2}{2} = 15.25\ ft$ $\quad l := 52\ ft$

$l_u := 13\ ft$

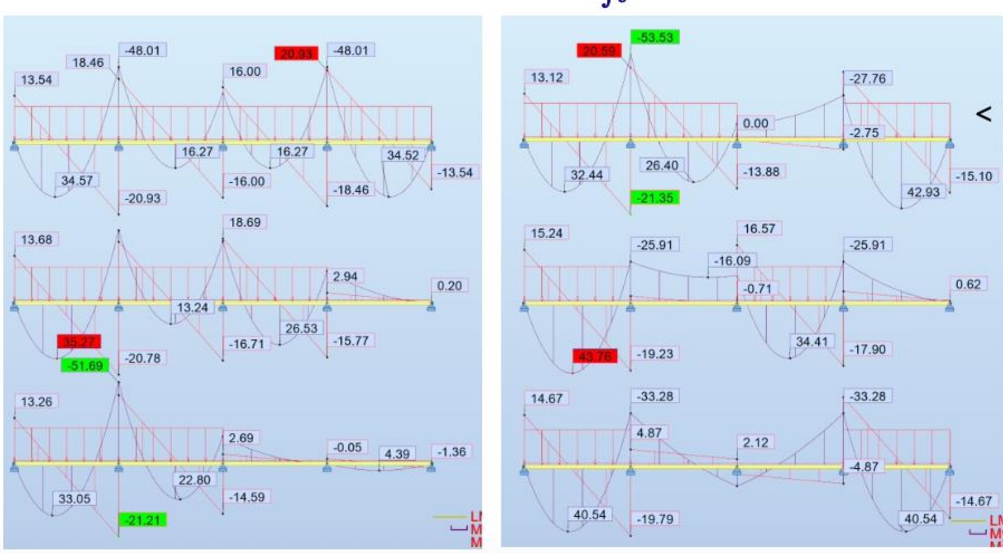
$G := 0.55$ $\quad \rho_w := \frac{62.4\ lbf}{ft^3}$ $\quad b := 2.5\ in$ $\quad d := 11.25\ in$

$D_{self} := G \cdot \rho_w \cdot (3 \cdot b) \cdot d = 20.109\ \frac{lbf}{ft}$ $\quad A := b \cdot d \cdot 3 = 84.375\ in^2$

$Live := L \cdot T_w = 1.525\ \frac{kip}{ft}$ $\quad I := \frac{1}{12} \cdot 3 \cdot b \cdot d^3 = 889.893\ in^4$

$Dead := D \cdot T_w + D_{self} = 0.179\ \frac{kip}{ft}$

$w := (1.2 \cdot D + 1.6 \cdot L) \cdot T_w + D_{self} = 2.65\ \frac{kip}{ft}$



< both cases

Change to steel beam

$l_1 = 14\ ft$ $\quad l_2 = 16.5\ ft$

$T_w := \dfrac{l_1 + l_2}{2} = 15.25\ ft$ $\quad l := 52\ ft$ $\quad l_u := 13\ ft$

$D_{self} := W = 22\ \dfrac{lbf}{ft}$

$D_{j1} := \dfrac{4.2\ plf \cdot \dfrac{l_1}{2}}{16\ in} = 22.05\ plf$

$D_{j2} := \dfrac{4.5\ plf \cdot \dfrac{l_2}{2}}{16\ in} = 27.844\ plf$

$W := 22\ plf$ $\quad A := 6.49\ in^2$

$E := 29000\ ksi$ $\quad d := 13.7\ in$

$F_y := 36\ ksi$

$F_u := 58\ ksi$ $\quad b_f := 5\ in$

$t_f := 0.335\ in$

$Z_x := 33.2\ in^3$ $\quad t_w := 0.32\ in$

$r_y := 1.04\ in$ $\quad h_{tw} := 53.3$

$I_y := 7\ in^4$ $\quad I_x := 199\ in^4$

$h := h_{tw} \cdot t_w = 17.056\ in$ $\quad S_x := 29\ in^3$

$J := 0.208\ in^4$ $\quad h_0 := 13.4\ in$

$C_w := 314\ in^6$ $\quad r_{ts} := 1.27\ in$

$Live := L \cdot T_w = 1.525\ \dfrac{kip}{ft}$

$Dead := D \cdot T_w + D_{self} + D_{j1} + D_{j2} = 0.23\ \dfrac{kip}{ft}$ $\qquad Dead_{rob} := D \cdot T_w + D_{j1} + D_{j2} = 0.208\ \dfrac{kip}{ft}$

$M_n := 54.7\ kip \cdot ft$ $\quad M_p := 44.59\ kip \cdot ft$ $\quad V := 21.04\ kip$

$M := 54.7\ kip \cdot ft$

Deflection Check

$w_{ST} := 0.5 \cdot Live = 0.763\ \dfrac{kip}{ft}$

$w_{LT} := Dead = 0.23\ \dfrac{kip}{ft}$

$\delta_{ST} := \dfrac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot (l_u)^4}{E \cdot I_x} = 0.045\ in$ $\quad < \quad$ $\Delta_{ST} := \dfrac{(l_u)}{360} = 0.433\ in$

$\delta_{LT} := \dfrac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot (l_u)^4}{E \cdot I_x} = 0.014\ in$ $\quad < \quad$ $\Delta_{LT} := \dfrac{(l_u)}{360} = 0.433\ in$

$\delta_{tot} := (1.5 \cdot \delta_{LT}) + \delta_{ST} = 0.065\ in$ $\quad < \quad$ $\Delta_{tot} := \dfrac{(l_u)}{240} = 0.65\ in$

Flexure

$M_{xYield} := F_y \cdot S_x = 87\ kip \cdot ft$ $\quad > 54.7\ (kip \cdot ft)$

Rood Design (not main space)

Load Calculation	From Boise Cascade
Suspended wood channel system	$D_1 := 2.5\ psf$
18 gauge metal decking roofing	$D_2 := 3\ psf$
Membrane	$D_3 := 0.35\ psf$
1/2" osb plywood sheathing	$D_4 := 1.7\ psf$
1" Rigid Insulation (3")	$D_5 := 1.5\ psf \cdot 3$
HVAC Ducts	$D_6 := 0.25\ psf$

Purlins to be used in order to remain at the same height: 2x12 Southern Pine No. 2

$G := 0.55 \qquad \gamma = 62.4\ pcf \qquad A_p := 1.5\ in \cdot 11.25\ in \qquad T_w := \frac{68.802\ in}{2}$

$D_{purlin} := \frac{G \cdot \gamma \cdot A_p}{40\ in} = 1.207\ psf \qquad l := 12\ ft \qquad l_r := \sqrt{12^2 + 1^2}\ ft = 12.042\ ft$

$Cor := \frac{l}{l_r} = 0.997$

$D_r := D_{purlin} + D_2 + D_2 + D_3 + D_4 + D_5 = 13.757\ psf \qquad D_{rCorrection} := D_r \cdot Cor = 13.709\ psf$

$D := D_1 + D_6 = 2.75\ psf$

$D_t := D_{rCorrection} + D = 16.459\ psf$

$L_r := 20\ psf \qquad L_{rCorrection} := L_r \cdot Cor = 19.931\ psf$

$L := 10\ psf$

$S_{maxCorrection} = 37.223\ psf$

$W_{maxCorrection} = 26.798\ psf$

$w_{upper1} := T_w \cdot (1.4 \cdot D_{rCorrection}) = 55.021\ plf$

$w_{upper2} := T_w \cdot (1.2 \cdot D_{rCorrection} + 1.6 \cdot L + \max(0.5 \cdot L_{rupperCorrection}, 0.5 \cdot S_{maxCorrection})) = 146.383\ plf$

$w_{upper3} := T_w \cdot (1.2 \cdot D_{rCorrection} + \max(1.6 \cdot L_{rupperCorrection}, S_{maxCorrection}) + \max(L, 0.5 \cdot W_{maxCorrection})) = 192.282\ plf$

$w_{upper4} := T_w \cdot (1.2 \cdot D_{rCorrection} + W_{maxCorrection} + L + \max(0.5 \cdot L_{rupperCorrection}, 0.3 \cdot S_{maxCorrection})) = 184.665\ plf$

$w_{upper5} := T_w \cdot (0.9 \cdot D_{rCorrection} + W_{maxCorrection}) = 112.195\ plf$

$w_{upper} := w_{upper3} \qquad 1.2 \quad T_w \cdot D_{rCorrection} = 0.039\ \frac{kip}{ft} \qquad 1.0 \quad T_w \cdot S_{maxCorrection} = 0.107\ \frac{kip}{ft}$

$0.5 \quad T_w \cdot W_{maxCorrection} = 0.077\ \frac{kip}{ft}$

FLB

$\lambda_f := 0.5 \cdot \frac{b_f}{t_f} = 7.463$ $\lambda_w := \frac{h}{t_w} = 53.3$ $\le$ $3.76 \cdot \sqrt{\frac{E}{F_y}} = 106.717$

$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 10.785$ $\lambda_{rf} := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 28.382$

$\lambda_f = 7.463$ $\le$ $\lambda_{pf} = 10.785$

$M_p := Z_x \cdot F_y = 99.6\ kip \cdot ft$ $\phi M_{nFLB} := 0.9 \cdot M_p = 89.64\ kip \cdot ft$ $> 54.7\ (kip \cdot ft)$

LTB

$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 4.329\ ft$ $c := 1$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_0} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_0}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 12.695\ ft$$

$r_{ts} := \sqrt{\frac{\sqrt{I_y \cdot C_w}}{S_x}} = 1.271\ in$ $L_b := 18\ ft$ correction, $L_b := 16\ in$ due to joists.

$L_b < L_p$

$\phi M_{nLTB} := 0.9 \cdot M_p = 89.64\ kip \cdot ft$ $> 54.7\ (kip \cdot ft)$

Shear Check

$A_w := d \cdot t_w = 4.384\ in^2$ $\phi := 0.9$

$\phi V_n := \phi \cdot A_w \cdot 0.6 \cdot F_y = 85.225\ kip$ $> 21.04\ kip$

Side floor beams of main event space composed of W14X22 A36 steel beams.

$w_{lower1} := T_w \cdot (1.4 \cdot D) = 11.037\ plf$
$w_{lower2} := T_w \cdot (1.2 \cdot D + 1.6 \cdot L) = 55.328\ plf$
$w_{lower} := w_{lower2}$ $\quad 1.2 \quad T_w \cdot D = 0.008\ \frac{kip}{ft}$

For Robot point locations

$1.6 \quad T_w \cdot L = 0.029\ \frac{kip}{ft}$

$x1 := 40\ in$

Point	x	y	y_u
1	0	0	0
2	$35.624\ in + x1$	0	$\frac{35.624\ in + x1}{12}$
3	$35.624\ in + 2 \cdot x1$	0	$\frac{35.624\ in + 2 \cdot x1}{12}$
4	$35.624\ in + 3 \cdot x1$	0	$\frac{35.624\ in + 3 \cdot x1}{12}$
5	$35.624\ in + 4 \cdot x1$	0	$\frac{35.624\ in + 4 \cdot x1}{12}$
6	$35.624\ in + 5 \cdot x1$	0	$\frac{35.624\ in + 5 \cdot x1}{12}$
7	$35.624\ in + 6 \cdot x1$	0	$\frac{35.624\ in + 6 \cdot x1}{12}$
8	$35.624\ in + 7 \cdot x1$	0	$\frac{35.624\ in + 7 \cdot x1}{12}$
9	$35.624\ in + 8 \cdot x1$	0	$\frac{35.624\ in + 8 \cdot x1}{12}$
10	$35.624\ in + 9 \cdot x1$	0	$\frac{35.624\ in + 9 \cdot x1}{12}$
11	$35.624\ in + 10 \cdot x1$	0	$\frac{35.624\ in + 10 \cdot x1}{12}$
12	$35.624\ in + 11 \cdot x1$	0	$\frac{35.624\ in + 11 \cdot x1}{12}$
13	$42.604\ ft$	0	$\frac{42.604\ ft}{12}$

$$x = \begin{bmatrix} 0 \\ 6.302 \\ 9.635 \\ 12.969 \\ 16.302 \\ 19.635 \\ 22.969 \\ 26.302 \\ 29.635 \\ 32.969 \\ 36.302 \\ 39.635 \\ 42.604 \end{bmatrix} ft \quad y_u = \begin{bmatrix} 0 \\ 0.525 \\ 0.803 \\ 1.081 \\ 1.359 \\ 1.636 \\ 1.914 \\ 2.192 \\ 2.47 \\ 2.747 \\ 3.025 \\ 3.303 \\ 3.55 \end{bmatrix} ft$$

$$\frac{35.624\ in}{12} = 0.247\ ft$$

$$42.604\ ft - 12.115\ ft = 30.489\ ft$$

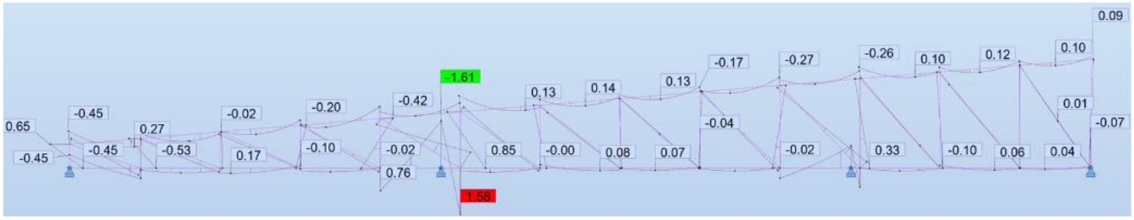
$$\frac{42.604\ ft - 12.115\ ft}{12} = 2.541\ ft$$

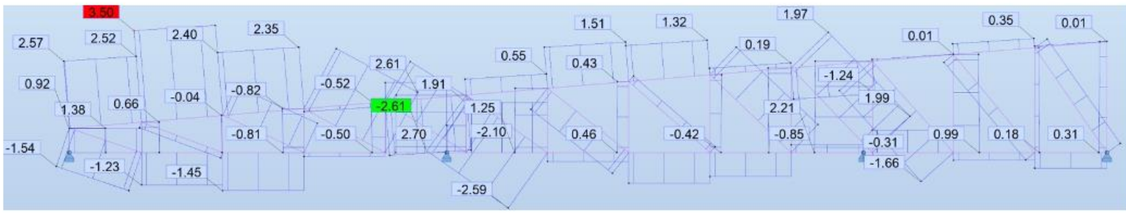
$$\frac{15.5\ ft}{12} = 1.292\ ft$$

$$\frac{32.604\ ft}{12} = 2.717\ ft$$

$$\frac{30.489\ ft}{12} = 2.541\ ft$$

$$l := 42.604\ ft$$





$M_l := 1.61\ kip \cdot ft$ $\quad P_l := 2.67\ kip$ $\quad M_i := 0.85\ kip \cdot ft$ $\quad P_i := 2.61\ kip$

$M_u := 0.45\ kip \cdot ft$ $\quad P_u := 3.5\ kip$ $\quad l_1 := 15.489\ ft$ $\quad l_2 := 17.104\ ft$

Bottom Chord Design

4x8 Section $\quad F_t := 600\ psi$ $\quad F_c := 1400\ psi$ $\quad F_b := 975\ psi$

$b := 3.5\ in$ $\quad d := 7.25\ in$ $\quad A := b \cdot d = 25.375\ in^2$ $\quad I := \dfrac{b \cdot d^3}{12} = 111.148\ in^4$

$S := \dfrac{I}{\frac{d}{2}} = 30.661\ in^3$ $\quad E := 1600000\ psi$ $\quad E_{min} := 580000\ psi$

Adjustment Factors

$C_D := 1.0$ $\quad C_{fu} := 1.00$ $\quad C_i := 1.0$ $\quad C_M := 1.0$ $\quad C_r := 1.00$

$C_t := 1.0$ $\quad C_F := 1.0$ (Implemented with Southern Pine Table)

$F°_b := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 975\ psi$

$l_e > 96$ $\quad l_{eT} := 96$ $\quad E_T := 1800000$

$COV_E := 0.25$ $\quad K_M := 1200$ $\quad K_T := 1 - 1.645 \cdot \left(COV_E\right)$ $\quad C_T := 1 + \dfrac{K_M \cdot l_{eT}}{K_T \cdot E_T} = 1.109$
(Table 4B)

$l_u := 17.104\ ft$ $\quad \dfrac{l_u}{d} = 28.31 \geq 7$ $\quad l_e := 0.9 \cdot l_u + 3 \cdot d = 17.206\ ft$

$E'_{min} := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = 643048.832\ psi$

$R_B := \sqrt{\dfrac{l_e \cdot d}{b^2}} = 11.054$ $\quad F_{bE} := \dfrac{1.2 \cdot E_{min}}{{R_B}^2} = 5695.654\ psi$

$$C_L := \frac{1+\left(\frac{F_{bE}}{F^{\circ}_b}\right)}{1.9} - \sqrt{\left(\frac{1+\left(\frac{F_{bE}}{F^{\circ}_b}\right)}{1.9}\right)^2 - \frac{\left(\frac{F_{bE}}{F^{\circ}_b}\right)}{0.95}} = 0.99$$

$$f_t := \frac{P_l}{A} = 105.222\ psi \qquad f_b := \frac{M_l}{S} = 630.107\ psi$$

$$F'_t := F_t \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i = 600\ psi$$

$$F'_b := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 965.154\ psi$$

Check

$$F'_t = 600\ psi \quad > \quad f_t = 105.222\ psi$$

$$F'_b = 965.154\ psi \quad > \quad f_b = 630.107\ psi$$

Design is adequate

$$w_{ST} := 0.5 \cdot L_{rCorrection} \cdot T_w = 0.029\ \frac{kip}{ft}$$

$$w_{LT} := \left(D_{rCorrection} + 0.5 \cdot L_{rCorrection}\right) \cdot T_w = 0.068\ \frac{kip}{ft}$$

$$\delta_{ST} := \frac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot \left(l_1\right)^4}{E \cdot I} = 0.11\ in \quad < \quad \Delta_{ST} := \frac{\left(l_1\right)}{360} = 0.516\ in$$

$$\delta_{LT} := \frac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot \left(l_1\right)^4}{E \cdot I} = 0.262\ in \quad < \quad \Delta_{LT} := \frac{\left(l_1\right)}{360} = 0.516\ in$$

$$\delta_{tot} := \left(1.5 \cdot \delta_{LT}\right) + \delta_{ST} = 0.503\ in \quad < \quad \Delta_{tot} := \frac{\left(l_1\right)}{240} = 0.774\ in$$

Design is adequate

$$\delta_{ST} := \frac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot (17.104\ ft)^4}{E \cdot I} = 0.164\ in \quad < \quad \Delta_{ST} := \frac{(17.104\ ft)}{360} = 0.57\ in$$

$$\delta_{LT} := \frac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot (17.104\ ft)^4}{E \cdot I} = 0.389\ in \quad < \quad \Delta_{LT} := \frac{(17.104\ ft)}{360} = 0.57\ in$$

$$\delta_{tot} := \left(1.5 \cdot \delta_{LT}\right) + \delta_{ST} = 0.748\ in \quad < \quad \Delta_{tot} := \frac{(17.104\ ft)}{240} = 0.855\ in$$

4x8s of Southern Pine No. 2 Dense used for Bottom Chord

Top Chord Design

SP No. 2 Dense

4x8 Section

$F_t := 600\ psi$ $F_c := 1400\ psi$ $F_b := 975\ psi$

$b := 3.5\ in$ $d := 7.25\ in$ $A := b \cdot d = 25.375\ in^2$ $I := \frac{b \cdot d^3}{12} = 111.148\ in^4$

$S := \frac{I}{\frac{d}{2}} = 30.661\ in^3$ $E := 1800000\ psi$ $E_{min} := 660000\ psi$

Adjustment Factors

$C_D := 1.15$ $C_L := 1.0$ $C_i := 1.0$ $C_M := 1.0$ $C_F := 1.0$ $C_r := 1.0$

$C_t := 1.0$ $C_{fu} := 1.0$ $C_P := 1.0$ $C_T := 1.0$

$f_c := \frac{P_u}{A} = 137.931\ psi$ $f_b := \frac{M_u}{S} = 176.117\ psi$

$F'_c := F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i = 1610\ psi$

$F'_b := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 1121.25\ psi$

Check

$F'_c = 1610\ psi$ > $f_c = 137.931\ psi$

$F'_b = 1121.25\ psi$ > $f_b = 176.117\ psi$

Design is adequate

$$R_B := \sqrt{\frac{\left(\begin{bmatrix} l_1 \\ l_2 \end{bmatrix}\right) \cdot d}{b^2}} = \begin{bmatrix} 10.488 \\ 11.021 \end{bmatrix}$$

$F_{cStar} := \frac{F'_c}{C_P} = 1610\ psi$

$E'_{min} := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = 660000\ psi$

$$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\frac{\begin{bmatrix} l_1 \\ l_2 \end{bmatrix}}{d}\right)^2} = \begin{bmatrix} 825.434 \\ 676.915 \end{bmatrix} psi$$

$$F_{bE} := \frac{1.2 \cdot E'_{min}}{{R_B}^2} = \begin{bmatrix} 7199.77 \\ 6519.951 \end{bmatrix} psi$$

$c := 0.8$

$$\alpha := \frac{1 + \frac{F_{cE}}{F_{cStar}}}{2 \cdot c} = \begin{bmatrix} 0.945 \\ 0.888 \end{bmatrix}$$

$$\beta := \frac{\frac{F_{cE}}{F_{cStar}}}{c} = \begin{bmatrix} 0.641 \\ 0.526 \end{bmatrix}$$

$$C_P := \alpha - \sqrt{\alpha^2 - \beta} = \begin{bmatrix} 0.442 \\ 0.375 \end{bmatrix}$$

$F'_c := F_{cStar} \cdot C_P = \begin{bmatrix} 712.367 \\ 604.295 \end{bmatrix}$ *psi* > $f_c = 137.931$ *psi*

$\frac{f_c}{F_{cE}} + \left(\frac{f_b}{F_{bE}}\right)^2 = \begin{bmatrix} 0.168 \\ 0.204 \end{bmatrix}$ < 1

Design is adequate

$w_{ST} = 0.029 \frac{kip}{ft}$

$w_{LT} = 0.068 \frac{kip}{ft}$

$\delta_{ST} := \frac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot \left(\begin{bmatrix} l_1 \\ l_2 \end{bmatrix}\right)^4}{E \cdot I} = \begin{bmatrix} 0.098 \\ 0.146 \end{bmatrix}$ *in* < $\Delta_{ST} := \frac{\left(\begin{bmatrix} l_1 \\ l_2 \end{bmatrix}\right)}{360} = \begin{bmatrix} 0.516 \\ 0.57 \end{bmatrix}$ *in*

$\delta_{LT} := \frac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot \left(\begin{bmatrix} l_1 \\ l_2 \end{bmatrix}\right)^4}{E \cdot I} = \begin{bmatrix} 0.233 \\ 0.346 \end{bmatrix}$ *in* < $\Delta_{LT} := \frac{\left(\begin{bmatrix} l_1 \\ l_2 \end{bmatrix}\right)}{360} = \begin{bmatrix} 0.516 \\ 0.57 \end{bmatrix}$ *in*

$\delta_{tot} := (1.5 \cdot \delta_{LT}) + \delta_{ST} = \begin{bmatrix} 0.447 \\ 0.665 \end{bmatrix}$ *in* < $\Delta_{tot} := \frac{\left(\begin{bmatrix} l_1 \\ l_2 \end{bmatrix}\right)}{240} = \begin{bmatrix} 0.774 \\ 0.855 \end{bmatrix}$ *in*

4x8s of Southern Pine No. 2 Dense used for Top Chord

Web Design

4x8 Section

$F_t := 600$ *psi* $F_c := 1400$ *psi* $F_b := 975$ *psi*

$b := 3.5$ *in* $d := 7.25$ *in* $A := b \cdot d = 25.375$ *in*^{2} $I := \frac{b \cdot d^3}{12} = 111.148$ *in*^{4}

$S := \frac{I}{\frac{d}{2}} = 30.661$ *in*^{3} $E := 1800000$ *psi* $E_{min} := 660000$ *psi*

Adjustment Factors

$C_D := 1.15$ $C_i := 1.0$ $C_M := 1.0$ $C_F := 1.0$ $C_t := 1.0$

$f_t := \frac{P_i}{A} = 102.857$ *psi*

$F'_t := F_t \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i = 690$ *psi*

Check

$F'_t = 690$ *psi* > $f_t = 102.857$ *psi*

$\frac{f_t}{F'_t} = 0.149$ < 1 Design is adequate

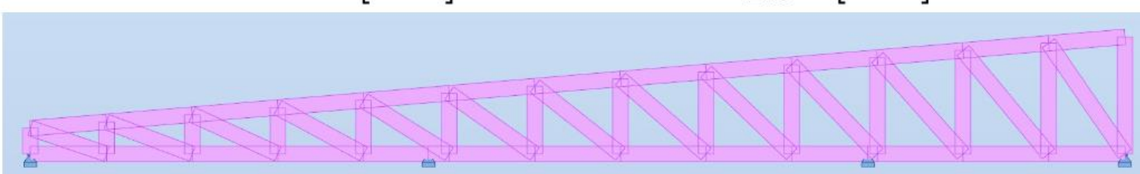
$w_{ST}=0.029\ \frac{kip}{ft}$

$w_{LT}=0.068\ \frac{kip}{ft}$

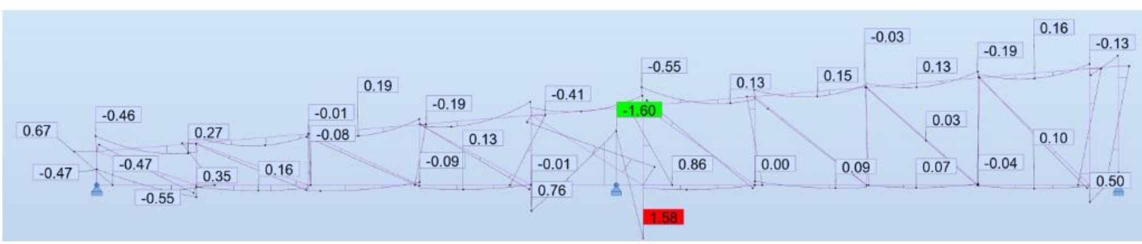
$$\delta_{ST}:=\frac{6.9\cdot10^{-3}\cdot w_{ST}\cdot\left(\begin{bmatrix}l_1\\l_2\end{bmatrix}\right)^4}{E\cdot I}=\begin{bmatrix}0.098\\0.146\end{bmatrix}\ in\quad<\quad\Delta_{ST}:=\frac{\left(\begin{bmatrix}l_1\\l_2\end{bmatrix}\right)}{360}=\begin{bmatrix}0.516\\0.57\end{bmatrix}\ in$$

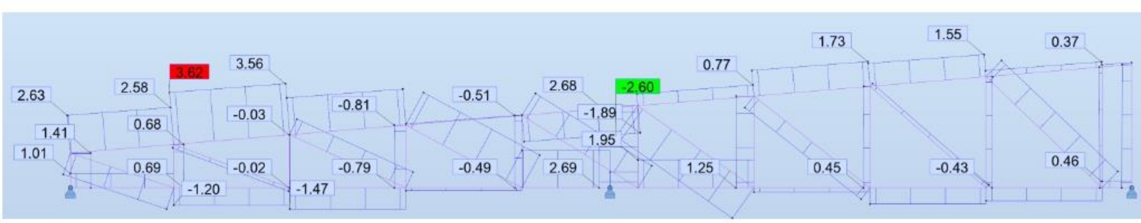
$$\delta_{LT}:=\frac{6.9\cdot10^{-3}\cdot w_{LT}\cdot\left(\begin{bmatrix}l_1\\l_2\end{bmatrix}\right)^4}{E\cdot I}=\begin{bmatrix}0.233\\0.346\end{bmatrix}\ in\quad<\quad\Delta_{LT}:=\frac{\left(\begin{bmatrix}l_1\\l_2\end{bmatrix}\right)}{360}=\begin{bmatrix}0.516\\0.57\end{bmatrix}\ in$$

$$\delta_{tot}:=\left(1.5\cdot\delta_{LT}\right)+\delta_{ST}=\begin{bmatrix}0.447\\0.665\end{bmatrix}\ in\quad<\quad\Delta_{tot}:=\frac{\left(\begin{bmatrix}l_1\\l_2\end{bmatrix}\right)}{240}=\begin{bmatrix}0.774\\0.855\end{bmatrix}\ in$$



Truss (Shorter)





$M_l:=1.6\ kip\cdot ft$ $\quad P_l:=2.59\ kip$ $\quad M_i:=0.86\ kip\cdot ft$ $\quad P_i:=2.6\ kip$

$M_u:=0.46\ kip\cdot ft$ $\quad P_u:=3.62\ kip$ $\quad l_1:=15\ ft$ $\quad l_2:=15.2445\ ft$

Bottom Chord Design

4x8 Section $F_t := 600\ psi$ $F_c := 1400\ psi$ $F_b := 975\ psi$

$b := 3.5\ in$ $d := 7.25\ in$ $A := b \cdot d = 25.375\ in^2$ $I := \frac{b \cdot d^3}{12} = 111.148\ in^4$

$S := \frac{I}{\frac{d}{2}} = 30.661\ in^3$ $E := 1600000\ psi$ $E_{min} := 580000\ psi$

Adjustment Factors

$C_D := 1.0$ $C_{fu} := 1.00$ $C_i := 1.0$ $C_M := 1.0$ $C_r := 1.00$

$C_t := 1.0$ $C_F := 1.0$ (Implemented with Southern Pine Table)

$F°_b := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 975\ psi$

$l_e > 96$ $l_{eT} := 96$ $E_T := 1800000$

$COV_E := 0.25$ $K_M := 1200$ $K_T := 1 - 1.645 \cdot (COV_E)$ $C_T := 1 + \frac{K_M \cdot l_{eT}}{K_T \cdot E_T} = 1.109$
(Table 4B)

$l_u := 17.104\ ft$ $\frac{l_u}{d} = 28.31$ ≥ 7 $l_e := 0.9 \cdot l_u + 3 \cdot d = 17.206\ ft$
$E'_{min} := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = 643048.832\ psi$

$R_B := \sqrt{\frac{l_e \cdot d}{b^2}} = 11.054$ $F_{bE} := \frac{1.2 \cdot E_{min}}{{R_B}^2} = 5695.654\ psi$

$$C_L := \frac{1 + \left(\frac{F_{bE}}{F°_b}\right)}{1.9} - \sqrt{\left(\frac{1 + \left(\frac{F_{bE}}{F°_b}\right)}{1.9}\right)^2 - \frac{\left(\frac{F_{bE}}{F°_b}\right)}{0.95}} = 0.99$$

$f_t := \frac{P_l}{A} = 102.069\ psi$ $f_b := \frac{M_l}{S} = 626.193\ psi$

$F'_t := F_t \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i = 600\ psi$

$F'_b := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 965.154\ psi$

Check

$F'_t = 600\ psi$ > $f_t = 102.069\ psi$

$F'_b = 965.154\ psi$ > $f_b = 626.193\ psi$

$w_{ST} := 0.5 \cdot L_{rCorrection} \cdot T_w = 0.029\ \frac{kip}{ft}$

$w_{LT} := \left(D_{rCorrection} + 0.5 \cdot L_{rCorrection}\right) \cdot T_w = 0.068\ \frac{kip}{ft}$

$\delta_{ST} := \frac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot \left(l_1\right)^4}{E \cdot I} = 0.097\ in$ < $\Delta_{ST} := \frac{\left(l_1\right)}{360} = 0.5\ in$

$\delta_{LT} := \frac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot \left(l_1\right)^4}{E \cdot I} = 0.23\ in$ < $\Delta_{LT} := \frac{\left(l_1\right)}{360} = 0.5\ in$

$\delta_{tot} := \left(1.5 \cdot \delta_{LT}\right) + \delta_{ST} = 0.443\ in$ < $\Delta_{tot} := \frac{\left(l_1\right)}{240} = 0.75\ in$

Design is adequate

$\delta_{ST} := \frac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot \left(l_2\right)^4}{E \cdot I} = 0.103\ in$ < $\Delta_{ST} := \frac{\left(l_2\right)}{360} = 0.508\ in$

$\delta_{LT} := \frac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot \left(l_2\right)^4}{E \cdot I} = 0.246\ in$ < $\Delta_{LT} := \frac{\left(l_2\right)}{360} = 0.508\ in$

$\delta_{tot} := \left(1.5 \cdot \delta_{LT}\right) + \delta_{ST} = 0.472\ in$ < $\Delta_{tot} := \frac{\left(l_2\right)}{240} = 0.762\ in$

4x8s of Southern Pine No. 2 Dense used for Bottom Chord

Top Chord Design

SP No. 2 Dense

4x8 Section

$F_t := 600\ psi$ $F_c := 1400\ psi$ $F_b := 975\ psi$

$b := 3.5\ in$ $d := 7.25\ in$ $A := b \cdot d = 25.375\ in^2$ $I := \frac{b \cdot d^3}{12} = 111.148\ in^4$

$S := \frac{I}{\frac{d}{2}} = 30.661\ in^3$ $E := 1800000\ psi$ $E_{min} := 660000\ psi$

Adjustment Factors

$C_D := 1.15$ $C_L := 1.0$ $C_i := 1.0$ $C_M := 1.0$ $C_F := 1.0$ $C_r := 1.0$

$C_t := 1.0$ $C_{fu} := 1.0$ $C_P := 1.0$ $C_T := 1.0$

$f_c := \frac{P_u}{A} = 142.66\ psi$ $\qquad f_b := \frac{M_u}{S} = 180.031\ psi$

$F'_c := F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i = 1610\ psi$

$F'_b := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 1121.25\ psi$

Check

$F'_c = 1610\ psi \quad > \quad f_c = 142.66\ psi$

$F'_b = 1121.25\ psi \quad > \quad f_b = 180.031\ psi$

Design is adequate

$R_B := \sqrt{\frac{\left(\begin{bmatrix} l_1 \\ l_2 \end{bmatrix}\right) \cdot d}{b^2}} = \begin{bmatrix} 10.321 \\ 10.405 \end{bmatrix}$ $\qquad F_{cStar} := \frac{F'_c}{C_P} = 1610\ psi$

$E'_{min} := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = 660000\ psi$

$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\frac{\begin{bmatrix} l_1 \\ l_2 \end{bmatrix}}{d}\right)^2} = \begin{bmatrix} 880.13 \\ 852.124 \end{bmatrix} psi$ $\qquad F_{bE} := \frac{1.2 \cdot E'_{min}}{{R_B}^2} = \begin{bmatrix} 7434.483 \\ 7315.244 \end{bmatrix} psi$

$c := 0.8$

$\alpha := \frac{1 + \frac{F_{cE}}{F_{cStar}}}{2 \cdot c} = \begin{bmatrix} 0.967 \\ 0.956 \end{bmatrix}$ $\qquad \beta := \frac{\frac{F_{cE}}{F_{cStar}}}{c} = \begin{bmatrix} 0.683 \\ 0.662 \end{bmatrix}$

$C_P := \alpha - \sqrt{\alpha^2 - \beta} = \begin{bmatrix} 0.466 \\ 0.454 \end{bmatrix}$

$F'_c := F_{cStar} \cdot C_P = \begin{bmatrix} 749.544 \\ 730.687 \end{bmatrix} psi \quad > \quad f_c = 142.66\ psi$

$\frac{f_c}{F_{cE}} + \left(\frac{f_b}{F_{bE}}\right)^2 = \begin{bmatrix} 0.163 \\ 0.168 \end{bmatrix} \quad < \quad 1$

$w_{ST} = 0.029\ \frac{kip}{ft}$

$w_{LT} = 0.068\ \frac{kip}{ft}$

$$\delta_{ST} := \frac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot \left(\begin{bmatrix} l_1 \\ l_2 \end{bmatrix}\right)^4}{E \cdot I} = \begin{bmatrix} 0.086 \\ 0.092 \end{bmatrix} \, in \quad < \quad \Delta_{ST} := \frac{\left(\begin{bmatrix} l_1 \\ l_2 \end{bmatrix}\right)}{360} = \begin{bmatrix} 0.5 \\ 0.508 \end{bmatrix} \, in$$

$$\delta_{LT} := \frac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot \left(\begin{bmatrix} l_1 \\ l_2 \end{bmatrix}\right)^4}{E \cdot I} = \begin{bmatrix} 0.205 \\ 0.218 \end{bmatrix} \, in \quad < \quad \Delta_{LT} := \frac{\left(\begin{bmatrix} l_1 \\ l_2 \end{bmatrix}\right)}{360} = \begin{bmatrix} 0.5 \\ 0.508 \end{bmatrix} \, in$$

$$\delta_{tot} := \left(1.5 \cdot \delta_{LT}\right) + \delta_{ST} = \begin{bmatrix} 0.393 \\ 0.42 \end{bmatrix} \, in \quad < \quad \Delta_{tot} := \frac{\left(\begin{bmatrix} l_1 \\ l_2 \end{bmatrix}\right)}{240} = \begin{bmatrix} 0.75 \\ 0.762 \end{bmatrix} \, in$$

4x8s of Southern Pine No. 2 Dense used for Top Chord

Web Design

4x8 Section $F_t := 600 \, psi$ $F_c := 1400 \, psi$ $F_b := 975 \, psi$

$b := 3.5 \, in$ $d := 7.25 \, in$ $A := b \cdot d = 25.375 \, in^2$ $I := \frac{b \cdot d^3}{12} = 111.148 \, in^4$

$S := \frac{I}{\frac{d}{2}} = 30.661 \, in^3$ $E := 1800000 \, psi$ $E_{min} := 660000 \, psi$

Adjustment Factors

$C_D := 1.15$ $C_i := 1.0$ $C_M := 1.0$ $C_F := 1.0$ $C_t := 1.0$

$$f_t := \frac{P_i}{A} = 102.463 \, psi$$

$$F'_t := F_t \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i = 690 \, psi$$

Check

$F'_t = 690 \, psi$ > $f_t = 102.463 \, psi$

$\frac{f_t}{F'_t} = 0.148$ < 1

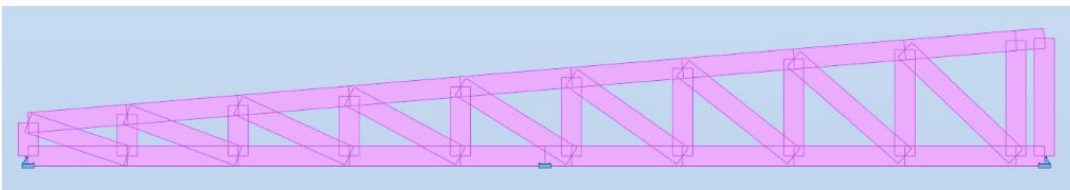
$$w_{ST} = 0.029 \, \frac{kip}{ft}$$

$$w_{LT} = 0.068 \, \frac{kip}{ft}$$

$$\delta_{ST} := \frac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot \left(\begin{bmatrix} l_1 \\ l_2 \end{bmatrix}\right)^4}{E \cdot I} = \begin{bmatrix} 0.086 \\ 0.092 \end{bmatrix} \textbf{ in} \quad < \quad \Delta_{ST} := \frac{\left(\begin{bmatrix} l_1 \\ l_2 \end{bmatrix}\right)}{360} = \begin{bmatrix} 0.5 \\ 0.508 \end{bmatrix} \textbf{ in}$$

$$\delta_{LT} := \frac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot \left(\begin{bmatrix} l_1 \\ l_2 \end{bmatrix}\right)^4}{E \cdot I} = \begin{bmatrix} 0.205 \\ 0.218 \end{bmatrix} \textbf{ in} \quad < \quad \Delta_{LT} := \frac{\left(\begin{bmatrix} l_1 \\ l_2 \end{bmatrix}\right)}{360} = \begin{bmatrix} 0.5 \\ 0.508 \end{bmatrix} \textbf{ in}$$

$$\delta_{tot} := \left(1.5 \cdot \delta_{LT}\right) + \delta_{ST} = \begin{bmatrix} 0.393 \\ 0.42 \end{bmatrix} \textbf{ in} \quad < \quad \Delta_{tot} := \frac{\left(\begin{bmatrix} l_1 \\ l_2 \end{bmatrix}\right)}{240} = \begin{bmatrix} 0.75 \\ 0.762 \end{bmatrix} \textbf{ in}$$



Beam For Trusses

$l_1 := 25 \textbf{ ft}$ $\quad l_2 := 26.813 \textbf{ ft}$ $\quad l_3 := 8.372 \textbf{ ft}$ $\quad l_4 := 20.779 \textbf{ ft}$ $\quad l_5 := 19.022 \textbf{ ft}$

$l_6 := 15.385 \textbf{ ft}$ $\quad l_7 := 15 \textbf{ ft}$

$T_w := 16.302 \textbf{ ft}$

$$D_{truss} := \frac{G \cdot \gamma \cdot 3.5 \textbf{ in} \cdot 7.25 \textbf{ in}}{34.401 \textbf{ in}} \cdot \frac{\left(\frac{1 \textbf{ ft} + 3.541 \textbf{ ft}}{2}\right) + \sqrt{\left(\frac{1 \textbf{ ft} + 3.541 \textbf{ ft}}{2}\right)^2 + (40 \textbf{ in})^2} + 40 \textbf{ in}}{40 \textbf{ in}} = 6.099 \textbf{ psf}$$

$D_{rCorrection} = 13.709 \textbf{ psf}$ $\quad D = 2.75 \textbf{ psf}$ $\quad D_t := D_{truss} + D + D_{rCorrection} = 22.558 \textbf{ psf}$

$L_r := 20 \textbf{ psf}$ $\quad L_{rCorrection} = 19.931 \textbf{ psf}$

$L := 10 \textbf{ psf}$

$S_{maxCorrection} = 37.223 \textbf{ psf}$

$W_{maxCorrection} = 26.798 \textbf{ psf}$

$$w_1 := T_w \cdot \left(1.4 \cdot D_t\right) = 514.839 \textbf{ plf}$$

$$w_2 := T_w \cdot \left(1.2 \cdot D_t + 1.6 \cdot L + \max\left(0.5 \cdot L_{rCorrection}, 0.5 \cdot S_{maxCorrection}\right)\right) = 1005.527 \textbf{ plf}$$

$$w_3 := T_w \cdot \left(1.2 \cdot D_t + \max\left(1.6 \cdot L_{rCorrection}, S_{maxCorrection}\right) + \max\left(L, 0.5 \cdot W_{maxCorrection}\right)\right) = 1266.533 \textbf{ plf}$$

$$w_4 := T_w \cdot \left(1.2 \cdot D_t + W_{maxCorrection} + L + \max\left(0.5 \cdot L_{rCorrection}, 0.3 \cdot S_{maxCorrection}\right)\right) = 1223.22 \textbf{ plf}$$

$$w_5 := T_w \cdot \left(0.9 \cdot D_t + W_{maxCorrection}\right) = 767.835 \textbf{ plf}$$

$w := w_3$ 1.2 $T_w \cdot D_{rCorrection} = 0.223\ \frac{kip}{ft}$ 1.0 $T_w \cdot S_{maxCorrection} = 0.607\ \frac{kip}{ft}$

0.5 $T_w \cdot W_{maxCorrection} = 0.437\ \frac{kip}{ft}$

Due to factoring loads, live loads are not considered. This means load patterns do not need to be considered as well. Full loading used.

W14X38 A36

$V_{max} := 17.71\ kip$ $M_{max} := 84.14\ kip \cdot ft$

$W := 38\ plf$ $A := 11.2\ in^2$ $E := 29000\ ksi$ $l_u := l_2$

$F_y := 36\ ksi$

$d := 14.1\ in$ $t_f := 0.515\ in$ $F_u := 58\ ksi$

$b_f := 6.77\ in$ $t_w := 0.31\ in$ $h_{tw} := 39.6$ $h := h_{tw} \cdot t_w = 12.276\ in$

$I_x := 385\ in^4$ $I_y := 26.7\ in^4$ $J := 0.798\ in^4$ $h_0 := 23.6\ in$

$C_w := 1230\ in^6$ $r_{ts} := 1.82\ in$

$Z_x := 61.5\ in^3$ $r_y := 1.55\ in$

$Live := (L_{rCorrection} + L) \cdot T_w = 0.488\ \frac{kip}{ft}$

$S_x := 54.6\ in^3$

$Dead := D_t \cdot T_w + W = 0.406\ \frac{kip}{ft}$

Deflection Check

$w_{ST} := 0.5 \cdot Live = 0.244\ \frac{kip}{ft}$

$w_{LT} := Dead = 0.406\ \frac{kip}{ft}$

$\delta_{ST} := \frac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot (l_u)^4}{E \cdot I_x} = 0.135\ in$ < $\Delta_{ST} := \frac{(l_u)}{360} = 0.894\ in$

$\delta_{LT} := \frac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot (l_u)^4}{E \cdot I_x} = 0.224\ in$ < $\Delta_{LT} := \frac{(l_u)}{360} = 0.894\ in$

$\delta_{tot} := (1.5 \cdot \delta_{LT}) + \delta_{ST} = 0.471\ in$ < $\Delta_{tot} := \frac{(l_u)}{240} = 1.341\ in$

Flexure

$M_{xYield} := F_y \cdot S_x = 163.8\ kip \cdot ft > 84.14\ (kip \cdot ft)$

FLB

$\lambda_f := 0.5 \cdot \frac{b_f}{t_f} = 6.573$ $\lambda_w := \frac{h}{t_w} = 39.6$ $\leq$ $3.76 \cdot \sqrt{\frac{E}{F_y}} = 106.717$

$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 10.785$ $\lambda_{rf} := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 28.382$

$\lambda_f = 6.573$ $\leq$ $\lambda_{pf} = 10.785$

$M_p := Z_x \cdot F_y = 184.5\ kip \cdot ft$ $\phi M_{nFLB} := 0.9 \cdot M_p = 166.05\ kip \cdot ft$ $> 84.14\ (kip \cdot ft)$

LTB

$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 6.452\ ft$ $c := 1$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_0} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_0}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 18.523\ ft$$

$r_{ts} := \sqrt{\frac{\sqrt{I_y \cdot C_w}}{S_x}} = 1.822\ in$ $L_b := l_u = 26.813\ ft$

$L_b > L_r$

Due to symmetry, $y_c := \frac{d}{2} = 7.05\ in$

$S_{xc} := \frac{I_x}{y_c}$ $M_{yc} := F_y \cdot S_{xc}$ $R_{pc} := \frac{M_p}{M_{yc}}$

$M_{max} := 84.14\ kip \cdot ft$ $M_A := 40\ kip \cdot ft$ $M_B := 47\ kip \cdot ft$ $M_C := 10\ kip \cdot ft$

$$C_b := min\left(3, \frac{12.5 \cdot M_{max}}{2.5 \cdot M_{max} + 3 \cdot M_A + 4 \cdot M_B + 3 \cdot M_C}\right) = 1.918$$

$$F_{cr} := \frac{C_b \cdot \pi^2 \cdot E}{\left(\frac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_0} \cdot \left(\frac{L_b}{r_{ts}}\right)^2}$$

$\phi M_{nLTB} := 0.9 \cdot min\left(R_{pc} \cdot M_{yc}, F_{cr} \cdot S_{xc}\right) = 114.13\ kip \cdot ft$ $> 84.14\ kip \cdot ft$

Shear Check

$A_w := d \cdot t_w = 4.371\ in^2$ $\phi := 0.9$

$\phi V_n := \phi \cdot A_w \cdot 0.6 \cdot F_y = 84.972\ kip$ $> 21.04\ kip$

Beam supporting non event spaced trusses composed of W14X38 A36 steel beams.

Beam For Over Front Window

$span := \frac{58.6\ ft}{2} = 29.3\ ft$ $T_w := \frac{19.3\ ft + 20\ ft}{2} = 19.65\ ft$

$l := 26.37\ ft$

Load Calculation	From Boise Cascade	
Suspended wood channel system	$D_1 := 2.5\ psf$	
18 gauge metal decking roofing	$D_2 := 3\ psf$	
Membrane	$D_3 := 0.35\ psf$	
1/2" osb plywood sheathing	$D_4 := 1.7\ psf$	
1" Rigid Insulation (3")	$D_5 := 1.5\ psf \cdot 3$	(West Roofing Systems Inc)
		(First American Roofing & Siding)
HVAC Ducts	$D_6 := 0.25\ psf$	(Johns Manville)
Purlin	$D_7 := \frac{A \cdot G \cdot \gamma}{T_w} = 0.136\ psf$	
Standard Roof	$L_1 := 20\ psf$	(ASCE)

$D_{upper} := D_1 + D_2 + D_5 + D_6 + D_7 = 10.386\ psf$ $T_w := 20\ ft$

$l_{upper} := \sqrt{(1\ ft)^2 + l^2} = 26.389\ ft$

$D := (D_{purlin} + D_{upper}) \cdot \frac{l}{l_{upper}} = 11.584\ psf$

$S_{maxCorrection} = 37.223\ psf$

$W_{maxCorrection} = 26.798\ psf$

$L_r := 20\ psf$ $L_{rCorrection} = 19.931\ psf$

$L := 0\ psf$

W16X45 A32

$W := 45\ plf$ $A := 13.3\ in^2$ $E := 29000\ ksi$ $l_u := 26.37\ ft$ $l_{LB} := 20\ ft$

$F_y := 36\ ksi$

$d := 16.1\ in$ $t_f := 0.565\ in$ $F_u := 58\ ksi$

$b_f := 7.04\ in$ $t_w := 0.345\ in$ $h_{tw} := 41.1$ $h := h_{tw} \cdot t_w = 14.18\ in$

$I_x := 586\ in^4$ $I_y := 32.8\ in^4$ $J := 1.11\ in^4$ $h_0 := 15.5\ in$

$C_w := 1990\ in^6$ $r_{ts} := 1.87\ in$

$Z_x := 82.3\ in^3$ $r_y := 1.57\ in$

$S_x := 72.7\ in^3$

$$A_{gl} := 374\ in^2 \qquad W_{gl} := A_{gl} \cdot G \cdot \gamma = 0.089\ \frac{kip}{ft}$$

$$T_l := \frac{span}{2}$$

$$Live := \left(L_{rCorrection} + L\right) \cdot T_w \cdot T_l = 5.84\ kip$$

$$Dead := \left(D_t \cdot T_w + W + W_{gl}\right) \cdot T_l = 8.575\ kip$$

$$D_t := D_t + \frac{W + W_{gl}}{T_w} = 29.265\ psf$$

$$w_1 := T_w \cdot \left(1.4 \cdot D_t\right) = 819.418\ plf$$

$$w_2 := T_w \cdot \left(1.2 \cdot D_t + 1.6 \cdot L + \max\left(0.5 \cdot L_{rCorrection}, 0.5 \cdot S_{maxCorrection}\right)\right) = 1074.588\ plf$$

$$w_3 := T_w \cdot \left(1.2 \cdot D_t + \max\left(1.6 \cdot L_{rCorrection}, S_{maxCorrection}\right) + \max\left(L, 0.5 \cdot W_{maxCorrection}\right)\right) = 1714.802\ plf$$

$$w_4 := T_w \cdot \left(1.2 \cdot D_t + W_{maxCorrection} + L + \max\left(0.5 \cdot L_{rCorrection}, 0.3 \cdot S_{maxCorrection}\right)\right) = 1461.663\ plf$$

$$w_5 := T_w \cdot \left(0.9 \cdot D_t + W_{maxCorrection}\right) = 1062.736\ plf$$

$$P := w_3 \cdot T_l = 25.122\ kip$$

$$P_{st} := Live \cdot 0.5 = 2.92\ kip$$

$$P_{lt} := Dead = 8.575\ kip$$

Deflection

$$a := \frac{l - l_{LB}}{2} = 3.185\ ft$$

$$\delta_{ST} := \max\left(\frac{23 \cdot P_{st} \cdot l^3}{648 \cdot E \cdot I_x}, \frac{P_{st} \cdot a}{24 \cdot E \cdot I_x} \cdot \left(3 \cdot l^2 - 4 \cdot a^2\right)\right) = 0.193\ in \quad < \quad \Delta_{ST} := \frac{(l)}{360} = 0.879\ in$$

$$\delta_{LT} := \max\left(\frac{23 \cdot P_{lt} \cdot l^3}{648 \cdot E \cdot I_x}, \frac{P_{lt} \cdot a}{24 \cdot E \cdot I_x} \cdot \left(3 \cdot l^2 - 4 \cdot a^2\right)\right) = 0.567\ in \quad < \quad \Delta_{LT} := \frac{(l)}{360} = 0.879\ in$$

$$\delta_{tot} := \left(1.5 \cdot \delta_{LT}\right) + \delta_{ST} = 1.044\ in \quad < \quad \Delta_{tot} := \frac{(l)}{240} = 1.319\ in$$

$$M_{max} := P \cdot a = 80.013\ kip \cdot ft \qquad V_{max} := P = 25.122\ kip$$

Flexure

$M_{xYield} := F_y \cdot S_x = 218.1 \; kip \cdot ft$ $> 80.013 \; (kip \cdot ft)$

FLB

$\lambda_f := 0.5 \cdot \frac{b_f}{t_f} = 6.23$ $\lambda_w := \frac{h}{t_w} = 41.1$ $\leq$ $3.76 \cdot \sqrt{\frac{E}{F_y}} = 106.717$

$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 10.785$ $\lambda_{rf} := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 28.382$

$\lambda_f = 6.23$ $\leq$ $\lambda_{pf} = 10.785$

$M_p := Z_x \cdot F_y = 246.9 \; kip \cdot ft$ $\phi M_{nFLB} := 0.9 \cdot M_p = 222.21 \; kip \cdot ft$ $> 80.013 \; (kip \cdot ft)$

LTB

$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 6.536 \; ft$ $c := 1$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_0} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_0}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 20.539 \; ft$$

$r_{ts} := \sqrt{\frac{\sqrt{I_y \cdot C_w}}{S_x}} = 1.875 \; in$ $L_b := l_{LB} = 20 \; ft$

$L_b > L_r$

Due to symmetry, $y_c := \frac{d}{2} = 8.05 \; in$

$S_{xc} := \frac{I_x}{y_c}$ $M_{yc} := F_y \cdot S_{xc}$ $R_{pc} := \frac{M_p}{M_{yc}}$

$M_{max} = 80.013 \; kip \cdot ft$ $M_A := M_{max}$ $M_B := M_{max}$ $M_C := M_{max}$

$$C_b := min\left(3, \frac{12.5 \cdot M_{max}}{2.5 \cdot M_{max} + 3 \cdot M_A + 4 \cdot M_B + 3 \cdot M_C}\right) = 1$$

$$F_{cr} := \frac{C_b \cdot \pi^2 \cdot E}{\left(\frac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_0} \cdot \left(\frac{L_b}{r_{ts}}\right)^2}$$

$L_p < L_b \leq L_r$

$$\phi M_{nLTB} := 0.9 \cdot min\left(M_p, C_b \cdot \left(M_p - \left(M_p - 0.7 \cdot F_y \cdot S_x\right) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right)\right) = 140.669 \; kip \cdot ft \; > 80.013 \; kip \cdot ft$$

Shear Check

$A_w := d \cdot t_w = 5.555\ in^2$ $\phi := 0.9$

$\phi V_n := \phi \cdot A_w \cdot 0.6 \cdot F_y = 107.979\ kip$ $> 25.122\ kip$

Beam over the center window of main event space designed as W16X45 A36 steel.

Decking

Joist

$spacing := 12\ in$ Assumed cantilever design for load consideration

$l := 6\ ft$ $L := 100\ psf$ $D_1 := \frac{3}{4}\ in \cdot G \cdot \gamma = 2.145\ psf$ 1x6 decking

2x4 SP No. 2 $D_2 := 4\ plf$ railing

$b := 1.5\ in$ $d := 3.5\ in$ $I := \frac{b \cdot d^3}{12} = 5.359\ in^4$ $S := \frac{I}{d} = 1.531\ in^3$

$D_{self} := b \cdot d \cdot G \cdot \gamma = 1.251\ plf$

$T_w := 12\ in$

$D := \left(D_1 \cdot T_w + D_{self}\right) \cdot 1.2 = 4.076\ plf$

$P := D_2 \cdot 1.2 \cdot T_w = 4.8\ lbf$

$L_t := L \cdot T_w \cdot 1.6 = 160\ plf$

$w := D + L_t = 164.076\ plf$

Columns

Non-Event Truss Supports (E, W, and Interior)

$h := 10\ ft + 11.25\ in$ (The truss and beams may result in a raised ceiling if desired to prevent low ceiling heights) (Floor beam added to height so column will reach foundation)

Interior Column

Design for largest load (largest tributary area)

$$A_t := 160.878\ in \cdot 182.934\ in + 195.624\ in \cdot 150\ in = 408.15\ ft^2$$

$$w_l = 19.931\ psf \qquad w_d = 15.197\ psf$$

$$D_{truss} := \frac{G \cdot \gamma \cdot 3.5\ in \cdot 7.25\ in}{34.401\ in} \cdot \frac{\left(\frac{1\ ft + 3.541\ ft}{2}\right) + \sqrt{\left(\frac{1\ ft + 3.541\ ft}{2}\right)^2 + (40\ in)^2} + 40\ in}{40\ in} = 6.099\ psf$$

$D_{rCorrection} = 13.709\ psf \qquad D := 2.75\ psf$

$L_r := 20\ psf \qquad L_{rCorrection} = 19.931\ psf \qquad D_{beam} := \frac{38\ plf \cdot (160.878\ in + 150\ in)}{A_t}$

$L := 10\ psf$

$D_t := D_{truss} + D + D_{rCorrection} + D_{beam} = 24.97\ psf$

$S_{maxCorrection} = 37.223\ psf$

$W_{maxCorrection} = 26.798\ psf$

$$P_1 := A_t \cdot (1.4 \cdot D_t) = 14.268\ kip$$

$$P_2 := A_t \cdot (1.2 \cdot D_t + 1.6 \cdot L + \max(0.5 \cdot L_{rCorrection}, 0.5 \cdot S_{maxCorrection})) = 26.357\ kip$$

$$P_3 := A_t \cdot (1.2 \cdot D_t + \max(1.6 \cdot L_{rCorrection}, S_{maxCorrection}) + \max(L, 0.5 \cdot W_{maxCorrection})) = 32.891\ kip$$

$$P_4 := A_t \cdot (1.2 \cdot D_t + W_{maxCorrection} + L + \max(0.5 \cdot L_{rCorrection}, 0.3 \cdot S_{maxCorrection})) = 31.807\ kip$$

$$P_5 := A_t \cdot (0.9 \cdot D_t + W_{maxCorrection}) = 20.11\ kip$$

$P_c := P_3 = 32.891\ kip \qquad c := 0.8$ (fixed, pinned)

$l := h \qquad K_e := 0.8 \qquad l_e := K_e \cdot l = 8.75\ ft$

$F_c := 1300\ psi \qquad E := 16000\ ksi \qquad E_{min} := 580\ ksi$

Column Section Selected: 6x8 Timber Dense Select Structural 86

(Selected to fit in wall system, can make larger to reduce pricing)

$b := 7.5\ in$ $d := 5.5\ in$ $A := b \cdot d = 0.286\ ft^2$

Factors

$C_D := 1$ $C_t := 1$ $C_i := 1$ $C_F := 1$ $C_M := 0.91$ $C_P := 1$ $C_T := 1$

$F^{\circ}_c := F_c \cdot C_D \cdot C_t \cdot C_M \cdot C_T = 1183\ psi$

$E'_{min} := E_{min} \cdot C_t \cdot C_i \cdot C_T = 580000\ psi$

$f_c := \frac{P_c}{A} = 797.365\ psi$

$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\frac{l_e}{d}\right)^2} = 1308.117\ psi$

$\alpha := \frac{1 + \left(\frac{F_{cE}}{F^{\circ}_c}\right)}{2 \cdot c} = 1.316$ $\beta := \frac{\frac{F_{cE}}{F^{\circ}_c}}{c} = 1.382$

$C_P := \alpha - \sqrt[2]{\alpha^2 - \beta} = 0.725$

$F'_c := F^{\circ}_c \cdot C_P = 857.156\ psi$ $857.156 > 797.365$ $f_c = 797.365\ psi$

$\lambda := \frac{l_e}{d} = 19.091$ $19.091 < 50$

$f_{cnet} := \frac{P_c}{A} = 797.365\ psi$ $797.365 < 1183$ $F^{\circ}_c = 1183\ psi$

6x8 Timber Southern Pine Dense Select Structural 86

Interior Main Space Columns

$l := 11.25\ in + 2.54075\ ft + 10\ ft = 13.478\ ft$

$T_{w1} := 90\ in$ $T_{l1} := 120\ in$ $A_1 := T_{w1} \cdot T_{l1} = 75\ ft^2$

$T_{w2} := 235.5\ in$ $T_{l2} := 349.1905\ in$ $A_2 := T_{w2} \cdot T_{l2} = 571.072\ ft^2$

$$D_{beam} := \frac{38\ plf \cdot (120\ in)}{A_1}$$

$D_t := D_{truss} + D + D_{rCorrection} + D_{beam} = 27.625\ psf$

$D_1 := D_t \cdot A_1 = 2.072\ kip$

$L_r := 20\ psf$ $L_{rCorrection} = 19.931\ psf$

$L := 10\ psf$

$S_{maxCorrection} = 37.223\ psf$

$W_{maxCorrection} = 26.798\ psf$

$D_{es} := 11.584\ psf$ $W_{gl} = 0.089\ \frac{kip}{ft}$

$D_2 := D_{es} \cdot A_2 + W_{gl} \cdot T_{l2} = 9.209\ kip$ $Dead := D_1 + D_2 = 11.281\ kip$

$L_{rt} := L_{rCorrection} \cdot (A_1 + A_2) = 12.877\ kip$

$L_t := L \cdot (A_1 + A_2) = 6.461\ kip$

$Snow := S_{maxCorrection} \cdot (A_1 + A_2) = 24.049\ kip$

$Wind := W_{maxCorrection} \cdot (A_1 + A_2) = 17.314\ kip$

$P_1 := (1.4 \cdot Dead) = 15.793\ kip$

$P_2 := (1.2 \cdot Dead + 1.6 \cdot L_t + \max(0.5 \cdot L_{rt}, 0.5 \cdot Snow)) = 35.899\ kip$

$P_3 := (1.2 \cdot Dead + \max(1.6 \cdot L_{rt}, Snow) + \max(L_t, 0.5 \cdot Wind)) = 46.243\ kip$

$P_4 := (1.2 \cdot Dead + Wind + L_t + \max(0.5 \cdot L_{rt}, 0.3 \cdot Snow)) = 44.526\ kip$

$P_5 := (0.9 \cdot Dead + Wind) = 27.467\ kip$

$P_c := P_3 = 46.243\ kip$ $c := 0.8$ (fixed, pinned)

$l = 13.478\ ft$ $K_e := 1$ $l_e := l \cdot K_e = 13.478\ ft$

$F_c := 525\ psi$ $E := 12000\ ksi$ $E_{min} := 440\ ksi$

Column Section Selected: 12x12 Timber Southern Pine No. 2

(Selected to fit in wall system, can make larger to reduce pricing)

$b := 11.5\ in$ $\quad d := 11.5\ in$ $\quad A := b \cdot d = 0.918\ ft^2$

Factors

$C_D := 1$ $\quad C_t := 1$ $\quad C_i := 1$ $\quad C_F := 1$ $\quad C_M := 1$ $\quad C_P := 1$ $\quad C_T := 1$

$F°_c := F_c \cdot C_D \cdot C_t \cdot C_M \cdot C_T = 525\ psi$

$E'_{min} := E_{min} \cdot C_t \cdot C_i \cdot C_T = 440000\ psi$

$f_c := \frac{P_c}{A} = 349.661\ psi$

$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\frac{l_e}{d}\right)^2} = 1828.482\ psi$

$\alpha := \frac{1 + \left(\frac{F_{cE}}{F°_c}\right)}{2 \cdot c} = 2.802$ $\quad \beta := \frac{\frac{F_{cE}}{F°_c}}{c} = 4.354$

$C_P := \alpha - \sqrt[2]{\alpha^2 - \beta} = 0.932$

$F'_c := F°_c \cdot C_P = 489.253\ psi$ $\quad 489.253 > 349.661$ $\quad f_c = 349.661\ psi$

$\lambda := \frac{l_e}{d} = 14.064$ $\quad 14.064 < 50$

$f_{cnet} := \frac{P_c}{A} = 349.661\ psi$ $\quad 349.661 < 525$ $\quad F°_c = 525\ psi$

12x12 Timber Southern Pine No. 2

Exterior Column Event Space

$L_r := 20\ psf$ $\quad L_{rCorrection} = 19.931\ psf$ $\quad L_{rCorrection} := 19.931\ psf$

$L := 10\ psf$ $\quad S_{maxCorrection} := 37.223\ psf$

$S_{maxCorrection} = 37.223\ psf$ $\quad W_{maxCorrection} := 26.798\ psf$

$W_{maxCorrection} = 26.798\ psf$

$D_{beam} := 45\ plf \cdot (120\ in) = 450\ lbf$ $\quad W_{gl} = 0.089\ \frac{kip}{ft}$

$D_{truss} := W_{gl} \cdot 29.1\ ft = 2.594\ kip$

$A_t := 58.419\ ft^2 + 383.673\ ft^2 + 229.897\ ft^2 = 671.989\ ft^2$

$D_{es} := 11.584\ psf$

$Dead := A_t \cdot D_{es} + D_{beam} + D_{truss} = 10.828\ kip$

$L_{rt} := L_{rCorrection} \cdot A_t = 13.393\ kip$

$L_t := L \cdot A_t = 6.72\ kip$

$Snow := S_{maxCorrection} \cdot A_t = 25.013\ kip$

$Wind := W_{maxCorrection} \cdot A_t = 18.008\ kip$

$P_1 := (1.4 \cdot Dead) = 15.159\ kip$

$P_2 := (1.2 \cdot Dead + 1.6 \cdot L_t + \max(0.5 \cdot L_{rt}, 0.5 \cdot Snow)) = 36.252\ kip$

$P_3 := (1.2 \cdot Dead + \max(1.6 \cdot L_{rt}, Snow) + \max(L_t, 0.5 \cdot Wind)) = 47.011\ kip$

$P_4 := (1.2 \cdot Dead + Wind + L_t + \max(0.5 \cdot L_{rt}, 0.3 \cdot Snow)) = 45.226\ kip$

$P_5 := (0.9 \cdot Dead + Wind) = 27.753\ kip$

$P_c := P_3 = 47.011\ kip$ $\quad c := 0.8$

$l := 17.434\ ft$ $\quad K_e := 1$ $\quad l_e := l \cdot K_e = 17.434\ ft$

$F_c := 525\ psi$ $\quad E := 12000\ ksi$ $\quad E_{min} := 440\ ksi$

Column Section Selected: 12x12 Timber Southern Pine No. 2

$b := 11.5\ in$ $\quad d := 11.5\ in$ $\quad A := b \cdot d = 0.918\ ft^2$

Factors

$C_D := 1$ $C_t := 1$ $C_i := 1$ $C_F := 1$ $C_M := 0.91$ $C_P := 1$ $C_T := 1$

$F^{\circ}_c := F_c \cdot C_D \cdot C_t \cdot C_M \cdot C_T = 477.75\ psi$

$E'_{min} := E_{min} \cdot C_t \cdot C_i \cdot C_T = 440000\ psi$

$f_c := \dfrac{P_c}{A} = 355.473\ psi$

$F_{cE} := \dfrac{0.822 \cdot E'_{min}}{\left(\dfrac{l_e}{d}\right)^2} = 1092.858\ psi$

$\alpha := \dfrac{1 + \left(\dfrac{F_{cE}}{F^{\circ}_c}\right)}{2 \cdot c} = 2.055$ $\beta := \dfrac{\dfrac{F_{cE}}{F^{\circ}_c}}{c} = 2.859$

$C_P := \alpha - \sqrt[2]{\alpha^2 - \beta} = 0.887$

$F'_c := F^{\circ}_c \cdot C_P = 423.995\ psi$ $423.995 > 355.473$ $f_c = 355.473\ psi$

$\lambda := \dfrac{l_e}{d} = 18.192$ $18.192 < 50$

$f_{cnet} := \dfrac{P_c}{A} = 355.473\ psi$ $355.473 < 477.75$ $F^{\circ}_c = 477.75\ psi$

12x12 Timber Southern Pine No. 2

Lower Event Space Interior

$A_t := 544.44\ ft^2$

$D_{hardwood} := 4\ psf$

$D_{gypsum} := 2.2\ psf$

$D_{sub} := 2.5\ psf$

$L := 100\ psf$

$D_{joist} := \dfrac{3.5\ in \cdot 7.25\ in \cdot G \cdot \gamma}{24\ in} = 3.024\ psf$

$L_t := L \cdot A_t = 54.444\ kip$

$D_{beam} := 50\ plf \cdot 22.685\ ft = 1.134\ kip$

$D_t := \left(D_{hardwood} + D_{gypsum} + D_{sub} + D_{joist}\right) \cdot A_t + D_{beam} = 7.517\ kip$

$P_c := 1.2 \cdot D_t + 1.6 \cdot L_t = 96.131\ kip$

$l := 9.104\ ft$ $K_e := 1$ $l_e := l \cdot K_e = 9.104\ ft$

$F_c := 525\ psi$ $E := 12000\ ksi$ $E_{min} := 440\ ksi$ $c := 0.8$

Column Section Selected: 16x16 Timber Southern Pine No. 2

$b := 15.5\ in$ $d := 15.5\ in$ $A := b \cdot d = 1.668\ ft^2$

Factors

$C_D := 1$ $C_t := 1$ $C_i := 1$ $C_F := 1$ $C_M := 1$ $C_P := 1$ $C_T := 1$

$$F^{\circ}_c := F_c \cdot C_D \cdot C_t \cdot C_M \cdot C_T = 525\ psi$$

$$E'_{min} := E_{min} \cdot C_t \cdot C_i \cdot C_T = 440000\ psi$$

$$f_c := \frac{P_c}{A} = 400.129\ psi$$

$$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\frac{l_e}{d}\right)^2} = 7280.495\ psi$$

$$\alpha := \frac{1 + \left(\frac{F_{cE}}{F^{\circ}_c}\right)}{2 \cdot c} = 9.292 \qquad \beta := \frac{\frac{F_{cE}}{F^{\circ}_c}}{c} = 17.335$$

$$C_P := \alpha - \sqrt[2]{\alpha^2 - \beta} = 0.985$$

$F'_c := F^{\circ}_c \cdot C_P = 517.093\ psi$ $423.995 > 355.473$ $f_c = 400.129\ psi$

$\lambda := \frac{l_e}{d} = 7.048$ $18.192 < 50$

$f_{cnet} := \frac{P_c}{A} = 400.129\ psi$ $355.473 < 477.75$ $F^{\circ}_c = 525\ psi$

16x16 Timber Southern Pine No. 2

Exterior Lower Event Space Column

$A_1 := 11.1886\ ft \cdot 13.185\ ft = 147.522\ ft^2$

$A_2 := 11.1886\ ft \cdot 8\ ft = 89.509\ ft^2$

$D_f := D_{hardwood} + D_{gypsum} + D_{sub}$ $\quad D_{wj} := \frac{4\ plf}{16\ in}$ $\quad D_{sj} := \frac{25\ plf}{32\ in}$ $\quad D_b := 22\ plf \cdot 11.1886\ ft$

$L := 100\ psf$

$L_t := L \cdot (A_1 + A_2) = 23.703\ kip$

$D_t := D_f \cdot (A_1 + A_2) + D_b + D_{wj} \cdot A_2 + D_{sj} \cdot A_1 = 3.96\ kip$

$P_c := 1.2 \cdot D_t + 1.6 \cdot L_t = 42.677\ kip$

$l := 9.104\ ft$ $\quad K_e := 1$ $\quad l_e := l \cdot K_e = 9.104\ ft$

$F_c := 525\ psi$ $\quad E := 12000\ ksi$ $\quad E_{min} := 440\ ksi$ $\quad c := 0.8$

Column Section Selected: 12x12 Timber Southern Pine No. 2

$b := 11.5\ in$ $\quad d := 11.5\ in$ $\quad A := b \cdot d = 0.918\ ft^2$

Factors

$C_D := 1$ $\quad C_t := 1$ $\quad C_i := 1$ $\quad C_F := 1$ $\quad C_M := 0.91$ $\quad C_P := 1$ $\quad C_T := 1$

$F^\circ_c := F_c \cdot C_D \cdot C_t \cdot C_M \cdot C_T = 477.75\ psi$

$E'_{min} := E_{min} \cdot C_t \cdot C_i \cdot C_T = 440000\ psi$

$f_c := \frac{P_c}{A} = 322.697\ psi$

$$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\frac{l_e}{d}\right)^2} = 4007.681\ psi$$

$$\alpha := \frac{1 + \left(\frac{F_{cE}}{F^\circ_c}\right)}{2 \cdot c} = 5.868 \qquad \beta := \frac{\frac{F_{cE}}{F^\circ_c}}{c} = 10.486$$

$$C_P := \alpha - \sqrt[2]{\alpha^2 - \beta} = 0.974$$

$F'_c := F°_c \cdot C_P = 465.514\ psi$ $\quad 465.514 > 322.697 \quad f_c = 322.697\ psi$

$\lambda := \frac{l_e}{d} = 9.5$ $\quad 18.192 < 50$

$f_{cnet} := \frac{P_c}{A} = 322.697\ psi$ $\quad 322.697 < 477.75 \quad F°_c = 477.75\ psi$

12x12 Timber Southern Pine No. 2

Lower Wall Column

$$P_{c1} := \frac{1.2 \cdot D_t + 1.6 \cdot L_t}{2} = 21.338\ kip$$

$A_t := 8.561\ ft \cdot 16\ ft = 136.976\ ft^2$

$D_f := D_{hardwood} + D_{gypsum} + D_{sub}$ $\quad D_{wj} := \frac{4\ plf}{16\ in}$ $\quad D_b := 22\ plf \cdot 8.561\ ft$

$L := 100\ psf$

$L_t := L \cdot A_t = 13.698\ kip$

$D_t := (D_f + D_{wj}) \cdot A_t + D_b = 1.791\ kip$

$P_{c2} := 1.2 \cdot D_t + 1.6 \cdot L_t = 24.065\ kip$

$P_c := P_2$

$l := 9.104\ ft$ $\quad K_e := 1$ $\quad l_e := l \cdot K_e = 9.104\ ft$

$F_c := 525\ psi$ $\quad E := 12000\ ksi$ $\quad E_{min} := 440\ ksi$ $\quad c := 0.8$

Column Section Selected: 10x10 Timber Southern Pine No. 2

$b := 9.5\ in$ $\quad d := 9.5\ in$ $\quad A := b \cdot d = 0.627\ ft^2$

Factors

$C_D := 1$ $\quad C_t := 1$ $\quad C_i := 1$ $\quad C_F := 1$ $\quad C_M := 0.91$ $\quad C_P := 1$ $\quad C_T := 1$

$F°_c := F_c \cdot C_D \cdot C_t \cdot C_M \cdot C_T = 477.75\ psi$

$E'_{min} := E_{min} \cdot C_t \cdot C_i \cdot C_T = 440000\ psi$

$f_c := \frac{P_c}{A} = 401.688\ psi$

$$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\frac{l_e}{d}\right)^2} = 2734.921 \ psi$$

$$\alpha := \frac{1 + \left(\frac{F_{cE}}{F°_c}\right)}{2 \cdot c} = 4.203 \qquad \beta := \frac{\frac{F_{cE}}{F°_c}}{c} = 7.156$$

$$C_P := \alpha - \sqrt[2]{\alpha^2 - \beta} = 0.961$$

$F'_c := F°_c \cdot C_P = 459.217 \ psi$ $\qquad$ $459.217 > 401.688$ $\qquad$ $f_c = 401.688 \ psi$

$\lambda := \frac{l_e}{d} = 11.5$ $\qquad$ $11.5 < 50$

$f_{cnet} := \frac{P_c}{A} = 401.688 \ psi$ $\qquad$ $401.688 < 477.75$ $\qquad$ $F°_c = 477.75 \ psi$

10x10 Timber Southern Pine No. 2

Wind

Snow

$w_{snow} := 46\ psf$

Rain

$m15 := 7.87\ \frac{in}{hr}$

$m60 := 3.73\ \frac{in}{hr}$

Risk Category III Site Soil Class DE

$V := 116\ mph$ (ASCE Hazard Tool)

$V := 116$

$z := 28\ ft + 7.25\ in = 28.604\ ft$

$K_{zt} := 1.0$ Under 1000ft

$K_d := 0.85$

Exposure C

$K_z := 0.94 + (z - 25\ ft) \cdot \frac{(0.98 - 0.94)}{(30\ ft - 25\ ft)} = 0.969$

$q_z := 0.002568\ K_z \cdot K_{zt} \cdot K_d \cdot V^2\ psf = 28.456\ psf$

$C_{pw} := 0.8$

$C_{ps} := -0.7$

$G := 0.85$

$GC_{pi} := 0.18$ +or- (enclosed)

$\theta := \operatorname{atan}\left(\frac{1}{12}\right) \cdot \frac{180}{\pi} = 4.764$

<u>North Wind</u>

Roof Pressures

$\theta = 4.764$

$C_{pwr} := 0.3 + (\theta - 35) \cdot \frac{(0.4 - 0.3)}{(45 - 35)} = -0.002$

$h := z = 28.604\ ft$

$L := 93\ ft + 10.75\ in = 93.896\ ft$ $C_{plr} := -0.3$ $B := 91\ ft + 2\ in + \frac{3}{8}\ in = 91.198\ ft$

$\frac{h}{L} = 0.305$ $\frac{L}{B} = 1.03$ $C_{pl} := -0.5$

Leeward

$p_{leewardroof} := q_z \cdot K_d \cdot C_{plr} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -2.134 \\ -12.379 \end{bmatrix} psf$

Overhang

$C_{po} := 0.8$

$p_{overhang} := q_z \cdot K_d \cdot C_{po} = 19.35\ psf$

Wall Pressures

$$p_{windward} := q_z \cdot K_d \cdot C_{pw} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} 24.472 \\ 14.228 \end{bmatrix} \; psf$$

$$p_{side} := q_z \cdot K_d \cdot C_{ps} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -11.809 \\ -22.054 \end{bmatrix} \; psf$$

$$p_{leeward} := q_z \cdot K_d \cdot C_{pl} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -6.972 \\ -17.216 \end{bmatrix} \; psf$$

South Wind

Roof Pressures

Windward

$$p_{windwardroof} := q_z \cdot K_d \cdot C_{pwr} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} 5.065 \\ -5.179 \end{bmatrix} \; psf$$

East Wind

Roof Pressures

$C_{psr1} := -0.9$ $\quad C_{psr2} := -0.9$ $\quad C_{psr3} := -0.5$ $\quad C_{psr4} := -0.3$

Side

0 to $\dfrac{h}{2} = 14.302 \; ft$

$$p_{side1roof} := q_z \cdot K_d \cdot C_{psr1} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -16.647 \\ -26.891 \end{bmatrix} \; psf$$

$\dfrac{h}{2}$ *to* $h = 28.604 \; ft$

$$p_{side2roof} := q_z \cdot K_d \cdot C_{psr2} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -16.647 \\ -26.891 \end{bmatrix} \; psf$$

h *to* $2\ h = 57.208 \; ft$

$$p_{side3roof} := q_z \cdot K_d \cdot C_{psr3} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -6.972 \\ -17.216 \end{bmatrix} \; psf$$

$2\ h$ *to* *end*

$$p_{side4roof} := q_z \cdot K_d \cdot C_{psr4} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -2.134 \\ -12.379 \end{bmatrix} \; psf$$

$$F_{wMax} := p_{side1roof}(1) = -26.891 \; psf$$

$\dfrac{B}{L} = 0.971$ $\qquad C_{pl} := -0.5$

Wall Pressures

$$p_{windward} := q_z \cdot K_d \cdot C_{pw} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} 24.472 \\ 14.228 \end{bmatrix} \ psf$$

$$p_{side} := q_z \cdot K_d \cdot C_{ps} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -11.809 \\ -22.054 \end{bmatrix} \ psf$$

$$p_{leeward} := q_z \cdot K_d \cdot C_{pl} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -6.972 \\ -17.216 \end{bmatrix} \ psf$$

$$p_{wWallMax} := p_{windward}(0)$$

Rain Load

$\rho := 62.4 \ pcf$

$d_s :=$ $\quad d_h :=$ $\quad d_p :=$

$$R := 5.2 \cdot (d_s + d_h + d_p) = ?$$

$$f_r := \begin{bmatrix} 0.08 \\ 0.25 \end{bmatrix}$$

$b := 92 \ ft + 2.5 \ in$

$L_r := 95 \ ft + 2 \ in + \frac{3}{8} \ in$ $\quad A := L_r \cdot b = 8778.041 \ ft^2$ $\quad i := 7.85 \ \frac{in}{hr}$

$A := \frac{L_r \cdot b}{ft^2} = 8778.041$ $\quad i := 7.85$ $\quad \frac{A \cdot i}{400} = 172.269$

Length of free flow off edge is $\geq \frac{A \cdot i}{400}$ therefor d_h is negligible.

$$Q := 0.0104 \cdot A \cdot i \cdot \frac{gal}{min} = 716.639 \ gpm$$

Snow Load

<u>Balanced Load</u> Risk Category III

Ground Load

$P_g := 46 \ psf$ (ASCE Hazard Tool)

Surface Roughness

Category C (Rural)

$C_e := 1.0$

Exposure

Partially Exposed (Forested Rural Location)

Thermal Factor

$C_t := 1.16$

Slope Factor

$C_s := 1$ $\theta = 4.764$ Unobstructed slippery surface

$P_s := 0.7 \cdot C_s \cdot C_e \cdot C_t \cdot P_g = 37.352\ psf$

Unbalanced Snow Load

$l_{rs} := b$

$l_u := l_{rs} = 92.208\ ft$ $L_u := 92.208$ $P_g = 46\ psf$ $p_g := 46$

$h_D := 0.43 \cdot \sqrt[3]{L_u} \cdot \sqrt[4]{p_g + 10} - 1.5 = 3.814$

$\gamma := 0.13 \cdot p_g + 14 = 19.98$ $\gamma := 19.98\ pcf$

Assuming standard residential area for other cabin, S=12ft

$P_{un} := h_D \cdot ft \cdot \frac{\gamma}{\sqrt[2]{12}} = 21.999\ psf$ $W_{un} := \frac{8}{3} \cdot h_D \cdot ft \cdot \sqrt[2]{12} = 35.234\ ft$

$S_{max} := P_s = 37.352\ psf$

Walls

South Event Space Wall $G := 0.55$ $\gamma := 62.4\ pcf$

Vertical Loads — From Boise Cascade

Suspended wood channel system $D_1 := 2.5\ psf$

18 gauge metal decking roofing $D_2 := 3\ psf$

1" Rigid Insulation (3") $D_3 := 1.5\ psf \cdot 3$

Purlin $D_4 := \frac{3.5\ in \cdot 11.25\ in \cdot G \cdot \gamma}{40\ in} = 2.815\ psf$

Standard Roof $L_1 := 20\ psf$ (ASCE)

$S_{max} = 37.352\ psf$

$F_{wMax} = -26.891\ psf$ $T_w := 19.72\ ft^2$ $W_l := 12\ in$

Horizontal Loads

$W_l := 12\ in$ $H := 17.434\ ft + 44\ in = 21.101\ ft$

$p_{wWallMax} = 24.472\ psf$ To account for truss height if it were to be raised

$l := 12\ ft$ $l_{upper} := \sqrt{(1\ ft)^2 + l^2} = 12.042\ ft$

$D := D_1 + D_2 + D_3 + D_4 = 12.815\ psf$

$D_r := D \cdot \frac{l}{l_{upper}} = 12.771\ psf$

$L_r := L_1 \cdot \frac{l}{l_{upper}} = 19.931\ psf$ $\qquad L_l := 0\ psf$

$Snow := S_{max} \cdot \frac{l}{l_{upper}} = 37.223\ psf$

$Wind := \left| F_{wMax} \cdot \frac{l}{l_{upper}} \right| = 26.798\ psf$

$Wall := p_{wWallMax} = 24.472\ psf$

$P_1 := (1.4 \cdot D_r \cdot T_w) = 0.353\ kip$

$P_2 := T_w \cdot (1.2 \cdot D_r + 1.6 \cdot L_l + \max(0.5 \cdot L_r, 0.5 \cdot Snow)) = 0.669\ kip$

$P_3 := T_w \cdot (1.2 \cdot D_r + \max(1.6 \cdot L_r, Snow) + \max(L_l, 0.5 \cdot Wind)) = 1.3\ kip$

$P_4 := T_w \cdot (1.2 \cdot D_r + Wind + L_l + \max(0.5 \cdot L_r, 0.3 \cdot Snow)) = 1.051\ kip$

$P_5 := T_w \cdot (0.9 \cdot D_r + Wind) = 0.755\ kip$ $\qquad P_c := \max(P_1, P_2, P_3, P_4, P_5)$

$w_{wall} := Wall \cdot W_l$ $\qquad l := H = 21.101\ ft$

Check 2 3x8

$Spacing := W_l = 12\ in$

$b := 2 \cdot 2.5\ in$ $\quad d := 7.25\ in$ $\quad A := b \cdot d = 36.25\ in^2$ $\quad I := \frac{b \cdot d^3}{12} = 158.783\ in^4$

$S := \frac{I}{d} = 21.901\ in^3$ $\quad F_b := 925\ psi$ $\quad F_c := 1350\ psi$

$M := \frac{w_{wall} \cdot l^2}{8} = 1.362\ kip \cdot ft$ $\quad E := 1400\ ksi$ $\quad E_{min} := 510\ ksi$

Factors

$C_D := 1$ $\quad C_t := 1$ $\quad C_i := 1$ $\quad C_F := 1$ $\quad C_M := 1$ $\quad C_P := 1$ $\quad C_T := 1$

$C_r := 1.15$ $\quad c := 0.8$ $\quad C_L := 1$ (sheathing)

$f_b := \frac{M}{S} = 746.271\ psi$ $f_c := \frac{P_c}{A} = 35.875\ psi$

$E'_{min} := E_{min} \cdot C_t \cdot C_i \cdot C_T = 510000\ psi$

$F°_b := F_b \cdot C_D \cdot C_t \cdot C_i \cdot C_F \cdot C_M \cdot C_r = 1063.75\ psi$

$F'_b := F°_b \cdot C_L = 1063.75\ psi$ $1063.75 > 746.271$ $f_b = 746.271\ psi$

$l_u := 15\ ft$ $\frac{l_u}{d} = 24.828 > 7$

$l_e := 1.63 \cdot l_u + 3 \cdot d = 26.263\ ft$

$R_B := \sqrt[2]{\frac{l_e \cdot d}{b^2}} = 9.56$

$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\frac{l_e}{d}\right)^2} = 221.862\ psi$ $>$ $f_c := \frac{P_c}{A} = 35.875\ psi$

$F°_c := F_c \cdot C_D \cdot C_t \cdot C_M \cdot C_T = 1350\ psi$ $1350 > 35.875$ $f_c = 35.875\ psi$

$F'_c := F°_c \cdot C_T = 1350\ psi$ $1350 > 35.875$

South facing event space wall designed as 2 3x8 Southern Pine No. 2 at 12" o.c.

Event space East and West wall

$T_w := 7.9675\ ft$

$D_r = 12.771\ psf$

$L_r = 19.931\ psf$ $L_l := 0\ psf$

$Snow = 37.223\ psf$

$Wind = 26.798\ psf$ $W_l := 12\ in$

$Wall := p_{wWallMax} = 24.472\ psf$

$$w_1 := (1.4 \cdot D_r \cdot T_w) = 0.142\ \frac{1}{ft} \cdot kip$$

$$w_2 := T_w \cdot (1.2 \cdot D_r + 1.6 \cdot L_l + \max(0.5 \cdot L_r, 0.5 \cdot Snow)) = 0.27\ \frac{1}{ft} \cdot kip$$

$$w_3 := T_w \cdot (1.2 \cdot D_r + \max(1.6 \cdot L_r, Snow) + \max(L_l, 0.5 \cdot Wind)) = 0.525\ \frac{1}{ft} \cdot kip$$

$$w_4 := T_w \cdot (1.2 \cdot D_r + Wind + L_l + \max(0.5 \cdot L_r, 0.3 \cdot Snow)) = 0.425\ \frac{1}{ft} \cdot kip$$

$$w_5 := T_w \cdot (0.9 \cdot D_r + Wind) = 0.305\ \frac{1}{ft} \cdot kip \qquad w_c := \max(w_1, w_2, w_3, w_4, w_5)$$

$$P_c := w_c \cdot W_l = 0.525\ kip$$

Check 2 2x8

$$Spacing := W_l = 12\ in$$

$$b := 2 \cdot 1.5\ in \qquad d := 7.25\ in \qquad A := b \cdot d = 21.75\ in^2 \qquad I := \frac{b \cdot d^3}{12} = 95.27\ in^4$$

$$S := \frac{I}{d} = 13.141\ in^3 \qquad F_b := 1250\ psi \qquad F_c := 1500\ psi$$

$$M := \frac{w_{wall} \cdot l^2}{8} = 1.362\ kip \cdot ft \qquad E := 1400\ ksi \qquad E_{min} := 510\ ksi$$

Factors

$$C_D := 1 \qquad C_t := 1 \qquad C_i := 1 \qquad C_F := 1 \qquad C_M := 1 \qquad C_P := 1 \qquad C_T := 1$$

$$C_r := 1.15 \qquad c := 0.8 \qquad C_L := 1$$ (sheathing)

$$f_b := \frac{M}{S} = 1243.784\ psi \qquad f_c := \frac{P_c}{A} = 24.158\ psi$$

$$E'_{min} := E_{min} \cdot C_t \cdot C_i \cdot C_T = 510000\ psi$$

$$F^{\circ}_b := F_b \cdot C_D \cdot C_t \cdot C_i \cdot C_F \cdot C_M \cdot C_r = 1437.5\ psi$$

$$F'_b := F^{\circ}_b \cdot C_L = 1437.5\ psi \qquad 1063.75 > 1243.784 \qquad f_b = 1243.784\ psi$$

$$l_u := 15\ ft + 44\ in \qquad \frac{l_u}{d} = 30.897 > 7$$

$$l_e := 1.63 \cdot l_u + 3 \cdot d = 32.239\ ft$$

$$R_B := \sqrt[2]{\frac{l_e \cdot d}{b^2}} = 17.653$$

$$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\frac{l_e}{d}\right)^2} = 147.227\ psi \quad > \quad f_c := \frac{P_c}{A} = 24.158\ psi$$

$$F^{*}_c := F_c \cdot C_D \cdot C_t \cdot C_M \cdot C_T = 1500\ psi \qquad 1500 > 42.479 \qquad f_c = 24.158\ psi$$

$$F'_c := F^{*}_c \cdot C_T = 1500\ psi \qquad 1500 > 42.479$$

South facing event space wall designed as 2 2x8 Southern Pine No. 1 at 12" o.c.

North event space wall

$$T_w := 7.496\ ft$$

$$D_{truss} := \frac{G \cdot \gamma \cdot 3.5\ in \cdot 7.25\ in}{29.156\ in} \cdot \frac{\left(\frac{1\ ft + 3.541\ ft}{2}\right) + \sqrt{\left(\frac{1\ ft + 3.541\ ft}{2}\right)^2 + (40\ in)^2} + 40\ in}{40\ in} = 7.196\ psf$$

$$D_n := D_r + D_{truss} = 19.967\ psf$$

$$L_r = 19.931\ psf \qquad L_l := 0\ psf$$

$$Snow = 37.223\ psf$$

$$Wind = 26.798\ psf \qquad W_l := 16\ in$$

$$Wall := p_{wWallMax} = 24.472\ psf$$

$$w_1 := \left(1.4 \cdot D_r \cdot T_w\right) = 0.134\ \frac{1}{ft} \cdot kip$$

$$w_2 := T_w \cdot \left(1.2 \cdot D_r + 1.6 \cdot L_l + \max\left(0.5 \cdot L_r, 0.5 \cdot Snow\right)\right) = 0.254\ \frac{1}{ft} \cdot kip$$

$$w_3 := T_w \cdot \left(1.2 \cdot D_r + \max\left(1.6 \cdot L_r, Snow\right) + \max\left(L_l, 0.5 \cdot Wind\right)\right) = 0.494\ \frac{1}{ft} \cdot kip$$

$$w_4 := T_w \cdot \left(1.2 \cdot D_r + Wind + L_l + \max\left(0.5 \cdot L_r, 0.3 \cdot Snow\right)\right) = 0.399\ \frac{1}{ft} \cdot kip$$

$$w_5 := T_w \cdot \left(0.9 \cdot D_r + Wind\right) = 0.287\ \frac{1}{ft} \cdot kip \qquad w_c := \max\left(w_1, w_2, w_3, w_4, w_5\right)$$

$$P_c := w_c \cdot W_l = 659.122\ lbf$$

Check 2x6

$Spacing := W_l = 16\ in$

$b := 1.5\ in$ $d := 5.5\ in$ $A := b \cdot d = 8.25\ in^2$ $I := \frac{b \cdot d^3}{12} = 20.797\ in^4$

$S := \frac{I}{d} = 3.781\ in^3$ $F_b := 1250\ psi$ $F_c := 1500\ psi$

Factors

$C_D := 1$ $C_t := 1$ $C_i := 1$ $C_F := 1$ $C_M := 1$ $C_P := 1$ $C_T := 1$

$C_r := 1.15$ $c := 0.8$ $C_L := 1$ (sheathing)

$f_c := \frac{P_c}{A} = 79.894\ psi$

$l_u := 16.252\ ft$ $\frac{l_u}{d} = 35.459 > 7$

$l_e := 1.63 \cdot l_u + 3 \cdot d = 27.866\ ft$

$R_B := \sqrt[2]{\frac{l_e \cdot d}{b^2}} = 28.59$

$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\frac{l_e}{d}\right)^2} = 113.413\ psi$ $>$ $f_c := \frac{P_c}{A} = 79.894\ psi$

$F^{\circ}_c := F_c \cdot C_D \cdot C_t \cdot C_M \cdot C_T = 1500\ psi$ $1500 > 42.479$ $f_c = 79.894\ psi$

$F'_c := F^{\circ}_c \cdot C_T = 1500\ psi$ $1500 > 42.479$

South facing event space wall designed as 2x6 Southern Pine No. 2 at 16" o.c.

Remaining exterior main floor walls

$T_w := 7.75\ ft$

$D_n := D_r + D_{truss} = 19.967\ psf$

$L_r = 19.931\ psf$ $L_l := 0\ psf$

$Snow = 37.223\ psf$

$Wind = 26.798\ psf$

$W_l := 12\ in$

$Wall := p_{wWallMax} = 24.472\ psf$

$l := 21.101\ ft - 3.8403\ ft = 17.261\ ft$

$w_1 := (1.4 \cdot D_r \cdot T_w) = 0.139 \ \frac{1}{ft} \cdot kip$

$w_2 := T_w \cdot (1.2 \cdot D_r + 1.6 \cdot L_l + \max(0.5 \cdot L_r, 0.5 \cdot Snow)) = 0.263 \ \frac{1}{ft} \cdot kip$

$w_3 := T_w \cdot (1.2 \cdot D_r + \max(1.6 \cdot L_r, Snow) + \max(L_l, 0.5 \cdot Wind)) = 0.511 \ \frac{1}{ft} \cdot kip$

$w_4 := T_w \cdot (1.2 \cdot D_r + Wind + L_l + \max(0.5 \cdot L_r, 0.3 \cdot Snow)) = 0.413 \ \frac{1}{ft} \cdot kip$

$w_5 := T_w \cdot (0.9 \cdot D_r + Wind) = 0.297 \ \frac{1}{ft} \cdot kip$ $\quad w_c := \max(w_1, w_2, w_3, w_4, w_5)$

$P_c := w_c \cdot W_l = 511.092 \ lbf$ $\quad M := \frac{Wall \cdot W_l \cdot l^2}{8} = 0.911 \ kip \cdot ft$

Check 2 2x6

$Spacing := W_l = 12 \ in$

$b := 2 \cdot 1.5 \ in$ $\quad d := 5.5 \ in$ $\quad A := b \cdot d = 16.5 \ in^2$ $\quad I := \frac{b \cdot d^3}{12} = 41.594 \ in^4$

$S := \frac{I}{d} = 7.563 \ in^3$ $\quad F_b := 1350 \ psi$ $\quad F_c := 1550 \ psi$

$E := 1600 \ ksi$ $\quad E_{min} := 580 \ ksi$

Factors

$C_D := 1$ $\quad C_t := 1$ $\quad C_i := 1$ $\quad C_F := 1$ $\quad C_M := 1$ $\quad C_P := 1$ $\quad C_T := 1$

$C_r := 1.15$ $\quad c := 0.8$ $\quad C_L := 1$ (sheathing)

$f_b := \frac{M}{S} = 1446.173 \ psi$ $\quad f_c := \frac{P_c}{A} = 30.975 \ psi$

$E'_{min} := E_{min} \cdot C_t \cdot C_i \cdot C_T = 580000 \ psi$

$F^*_b := F_b \cdot C_D \cdot C_t \cdot C_i \cdot C_F \cdot C_M \cdot C_r = 1552.5 \ psi$

$F'_b := F^*_b \cdot C_L = 1552.5 \ psi$ $\quad 1552.5 > 1446.173$ $\quad f_b = 1446.173 \ psi$

$l_u := 15 \ ft + 44 \ in$ $\quad \frac{l_u}{d} = 40.727 > 7$

$l_e := 1.63 \cdot l_u + 3 \cdot d = 31.802 \ ft$

$R_B := \sqrt[2]{\frac{l_e \cdot d}{b^2}} = 15.271$

$$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\frac{l_e}{d}\right)^2} = 99.029\ psi \quad > \quad f_c := \frac{P_c}{A} = 30.975\ psi$$

$$F^{\circ}_c := F_c \cdot C_D \cdot C_t \cdot C_M \cdot C_T = 1550\ psi \qquad 1550 > 30.975 \qquad f_c = 30.975\ psi$$

$$F'_c := F^{\circ}_c \cdot C_T = 1550\ psi \qquad 1550 > 30.975$$

Remaining exterior main floor walls designed as 2 2x6 Southern Pine No. 1 at 12" o.c.

Lower event space southern wall

$$T_w := 12\ ft$$

$$D_{hardwood} := 4\ psf$$
$$D_{gypsum} := 2.2\ psf$$
$$D_{sub} := 2.5\ psf$$

$$D_{joist} := \frac{3.5\ in \cdot 7.25\ in \cdot G \cdot \gamma}{24\ in} = 3.024\ psf$$

$$D_n := D_{joist} + D_{hardwood} + D_{gypsum} + D_{sub} = 11.724\ psf$$
$$L_r := 0\ psf \qquad W_l := 12\ in$$
$$L_n := 100\ psf$$
$$Snow = 37.223\ psf \qquad l := 9\ ft$$
$$Wind = 26.798\ psf \qquad Wall = 24.472\ psf$$

$$w_2 := T_w \cdot (1.2 \cdot D_r + 1.6 \cdot L_l + \max(0.5 \cdot L_r, 0.5 \cdot Snow)) = 0.407\ \frac{1}{ft} \cdot kip$$

$$w_3 := T_w \cdot (1.2 \cdot D_r + \max(1.6 \cdot L_r, Snow) + \max(L_l, 0.5 \cdot Wind)) = 0.791\ \frac{1}{ft} \cdot kip$$

$$w_4 := T_w \cdot (1.2 \cdot D_r + Wind + L_l + \max(0.5 \cdot L_r, 0.3 \cdot Snow)) = 0.639\ \frac{1}{ft} \cdot kip$$

$$w_5 := T_w \cdot (0.9 \cdot D_r + Wind) = 0.46\ \frac{1}{ft} \cdot kip \qquad w_c := \max(w_1, w_2, w_3, w_4, w_5)$$

$$P_c := w_c \cdot W_l = 791.369\ lbf \qquad M := \frac{Wall \cdot W_l \cdot l^2}{8} = 0.248\ kip \cdot ft$$

Check 2x8

$$Spacing := W_l = 12\ in$$

$$b := 1.5\ in \qquad d := 7.75\ in \qquad A := b \cdot d = 11.625\ in^2 \qquad I := \frac{b \cdot d^3}{12} = 58.186\ in^4$$

$$S := \frac{I}{d} = 7.508\ in^3 \qquad F_b := 925\ psi \qquad F_c := 1350\ psi$$

$$M := \frac{w_{wall} \cdot l^2}{8} = 0.248\ kip \cdot ft \qquad E := 1400\ ksi \qquad E_{min} := 510\ ksi$$

Factors

$C_D := 1$ $C_t := 1$ $C_i := 1$ $C_F := 1$ $C_M := 1$ $C_P := 1$ $C_T := 1$

$C_r := 1.15$ $c := 0.8$ $C_L := 1$ (sheathing)

$f_b := \frac{M}{S} = 396.041\ psi$ $f_c := \frac{P_c}{A} = 68.075\ psi$

$E'_{min} := E_{min} \cdot C_t \cdot C_i \cdot C_T = 510000\ psi$

$F°_b := F_b \cdot C_D \cdot C_t \cdot C_i \cdot C_F \cdot C_M \cdot C_r = 1063.75\ psi$

$F'_b := F°_b \cdot C_L = 1063.75\ psi$ $1063.75 > 396.041$ $f_b = 396.041\ psi$

$l_u := 15\ ft + 44\ in$ $\frac{l_u}{d} = 28.903 > 7$

$l_e := 1.63 \cdot l_u + 3 \cdot d = 32.364\ ft$

$R_B := \sqrt[2]{\frac{l_e \cdot d}{b^2}} = 36.575$

$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\frac{l_e}{d}\right)^2} = 166.938\ psi$ $>$ $f_c := \frac{P_c}{A} = 68.075\ psi$

$F°_c := F_c \cdot C_D \cdot C_t \cdot C_M \cdot C_T = 1350\ psi$ $1350 > 68.075$ $f_c = 68.075\ psi$

$F'_c := F°_c \cdot C_T = 1350\ psi$ $1350 > 68.075$

Lower event space southern wall designed as 2x8 Southern Pine No. 2 at 12" o.c. studs

Overhang wall supporting beam

$A_{t1} := 20.78\ ft^2$ $A_{t2} := 100.93\ ft^2$ $A_{t3} := 158.22\ ft^2$ $A_{t4} := 35.16\ ft^2$

$T_{w1} := \frac{A_{t1}}{16.82154\ ft}$ $T_{w23} := 6\ ft$ $T_{w4} := 16\ in$ $l_1 := 16.8215\ ft$ $l_2 := l_1$

$l_{34} := 26.37\ ft$

$D_{joist1} := \frac{4\ plf}{16\ in}$ $D_{joist2} := \frac{1.5\ in \cdot 5.5\ in \cdot G \cdot \gamma}{1\ ft} = 1.966\ psf$ $D_{board} := 0.75\ in \cdot G \cdot \gamma = 2.145\ psf$

$D_{beam} := \frac{3 \cdot 2.5\ in \cdot 7.25\ in \cdot G \cdot \gamma}{5\ ft} = 2.592\ psf$ $D_{railing} := \frac{4\ plf}{6\ ft}$

$D_1 := D_{joist1} + D_{hardwood} + D_{gypsum} + D_{sub}$ $\quad D_{23} := D_{joist2} + D_{beam} + D_{board} + D_{railing}$

$L_n := 100\ psf$ $\quad D_4 := D_{hardwood} + D_{gypsum} + D_{sub}$

$T_{nr} := 0\ ft$

$D_{r12} := 12.771\ psf$ $\quad T_{r12} := 19.72\ ft^2$ $\quad W_l := 12\ in$

$$D_{wall} := \frac{(2 \cdot 2.5\ in \cdot 7.75\ in) \cdot G \cdot \gamma \cdot 21.101\ ft}{W_l} + 2.5\ in \cdot 7.75\ in \cdot G \cdot \gamma = 199.494\ plf$$

$Snow = 37.223\ psf$

$Wind = 26.798\ psf$ $\quad$ Estimated beam weight $\quad W := 65\ plf$

$L_r := 20\ psf$

$$w_1 := 1.2 \cdot (T_{w1} \cdot (D_1 + D_{r12}) + T_{w23} \cdot D_{23} + W + D_{wall}) + 1.6 \cdot (T_{w1} + T_{w23}) \cdot L_n + \max(0.5 \cdot T_{w1} \cdot L_r, 0.5 \cdot T_{w1} \cdot Snow) = 1.587\ \frac{kip}{ft}$$

$$w_2 := 1.2 \cdot (T_{w1} \cdot (D_1 + D_{r12}) + T_{w23} \cdot D_{23} + W + D_{wall}) + \max(1.6 \cdot T_{w1} \cdot L_r, T_{w1} \cdot Snow) + \max((T_{w1} + T_{w23}) \cdot L_n, 0.5 \cdot T_{w1} \cdot Wind) = 1.176\ \frac{kip}{ft}$$

$$w_3 := 1.2 \cdot (T_{w1} \cdot (D_1 + D_{r12}) + T_{w23} \cdot D_{23} + W + D_{wall}) + \max(1.6 \cdot T_{w1} \cdot L_r, T_{w1} \cdot Snow) + \max((T_{w1} + T_{w23}) \cdot L_n, 0.5 \cdot T_{w1} \cdot Wind) = 1.176\ \frac{kip}{ft}$$

$$w_4 := 1.2 \cdot (T_{w1} \cdot (D_1 + D_{r12}) + T_{w23} \cdot D_{23} + W + D_{wall}) + T_{w1} \cdot Wind + (T_{w1} + T_{w23}) \cdot L_n + \max(0.5 \cdot T_{w1} \cdot L_r, 0.3 \cdot T_{w1} \cdot Snow) = 1.177\ \frac{1}{ft} \cdot kip$$

$$w_5 := 0.9 \cdot (T_{w1} \cdot (D_1 + D_{r12}) + T_{w23} \cdot D_{23} + W + D_{wall}) + T_{w1} \cdot Wind = 0.338\ \frac{kip}{ft}$$

$$w_{c1} := \max(w_1, w_2, w_3, w_4, w_5)$$

$$M_{max1} := \max\left(\frac{w_{c1} \cdot l_1^2}{8}, \frac{9 \cdot w_{c1} \cdot l_1^2}{128}, \frac{49 \cdot w_{c1} \cdot l_1^2}{512}\right) = 56.146\ kip \cdot ft$$

$$V_{max1} := \max\left(\frac{9 \cdot w_{c1} \cdot l_1}{16}, \frac{5 \cdot w_{c1} \cdot l_1}{8}\right) = 16.689\ kip$$

$$w_1 := 1.2 \cdot (T_{w4} \cdot D_4 + T_{w23} \cdot D_{23} + W + D_{wall}) + 1.6 \cdot (T_{w4} + T_{w23}) \cdot L_n + \max(0.5 \cdot T_{nr} \cdot L_r, 0.5 \cdot T_{nr} \cdot Snow) = 1.558\ \frac{kip}{ft}$$

$$w_2 := 1.2 \cdot (T_{w4} \cdot D_4 + T_{w23} \cdot D_{23} + W + D_{wall}) + \max(1.6 \cdot T_{nr} \cdot L_r, T_{nr} \cdot Snow) + \max((T_{w4} + T_{w23}) \cdot L_n, 0.5 \cdot T_{nr} \cdot Wind) = 1.118\ \frac{kip}{ft}$$

$$w_3 := 1.2 \cdot (T_{w4} \cdot D_4 + T_{w23} \cdot D_{23} + W + D_{wall}) + \max(1.6 \cdot T_{nr} \cdot L_r, T_{nr} \cdot Snow) + \max((T_{w4} + T_{w23}) \cdot L_n, 0.5 \cdot T_{nr} \cdot Wind) = 1.118\ \frac{kip}{ft}$$

$$w_4 := 1.2 \cdot (T_{w4} \cdot D_4 + T_{w23} \cdot D_{23} + W + D_{wall}) + T_{nr} \cdot Wind + (T_{w4} + T_{w23}) \cdot L_n + \max(0.5 \cdot T_{nr} \cdot L_r, 0.3 \cdot T_{nr} \cdot Snow) = 1.118\ \frac{kip}{ft}$$

$$w_5 := 0.9 \cdot (T_{w4} \cdot D_4 + T_{w23} \cdot D_{23} + W + D_{wall}) + T_{nr} \cdot Wind = 0.288\ \frac{kip}{ft}$$

$$w_{c2} := \max(w_1, w_2, w_3, w_4, w_5)$$

$M_{max2} := \frac{w_{c2} \cdot l_{34}^{2}}{8} = 135.399\ kip \cdot ft$ $\qquad V_{max2} := \frac{w_{c2} \cdot l_{34}}{2} = 20.538\ kip$

Test W12x65 A992

$W = 65\ plf$ $\quad A := 19.1\ in^2$ $\quad d := 12.1\ in$ $\quad b_f := 12\ in$ $\quad t_w := 0.39\ in$

$t_f := 0.605\ in$ $\quad h_{tw} := 24.9$ $\quad I_x := 533\ in^4$ $\quad Z_x := 96.8\ in^3$ $\quad S_x := 87.9\ in^3$

$r_x := 5.28\ in$ $\quad I_y := 174\ in^4$ $\quad r_y := 3.02\ in$ $\quad J := 2.18\ in^4$ $\quad C_w := 5780\ in^6$

$r_{ts} := 3.38\ in$ $\quad h_0 := 11.5\ in$ $\quad h := h_{tw} \cdot t_w$

$F_y := 50\ ksi$ $\quad F_u := 65\ ksi$ $\quad E := 29000\ ksi$

Front beam controls design

$w_l := 1.6 \cdot (T_{w4} + T_{w23}) \cdot L_n$ $\qquad w_d := 1.2 \cdot (T_{w4} \cdot D_4 + T_{w23} \cdot D_{23} + W + D_{wall})$

$w_{st} := w_l \cdot 0.5 = 586.667\ \frac{lbf}{ft}$

$w_{lt} := w_d + w_l \cdot 0.5 = 971.042\ \frac{lbf}{ft}$

$\delta_{st} := \frac{5 \cdot w_{st} \cdot l_{34}^{4}}{384 \cdot E \cdot I_x} = 0.413\ in$ $\qquad \Delta_{st} := \frac{l_{34}}{360} = 0.879\ in$

$\delta_{lt} := \frac{5 \cdot w_{lt} \cdot l_{34}^{4}}{384 \cdot E \cdot I_x} = 0.683\ in$ $\qquad \Delta_{LT} := \frac{(l_{34})}{360} = 0.879\ in$

$\delta_{tot} := \delta_{st} + \delta_{lt} = 1.096\ in$ $\qquad \Delta_{tot} := \frac{l_{34}}{240} = 1.319\ in$

Flexure

$M_{xYield} := F_y \cdot S_x = 366.25\ kip \cdot ft$ $\qquad > 135.399\ (kip \cdot ft)$

FLB

$\lambda_f := 0.5 \cdot \frac{b_f}{t_f} = 9.917$ $\qquad \lambda_w := \frac{h}{t_w} = 24.9 \quad \le \quad 3.76 \cdot \sqrt{\frac{E}{F_y}} = 90.553$

$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 9.152$ $\qquad \lambda_{rf} := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 24.083$

$M_p := Z_x \cdot F_y = 403.333\ kip \cdot ft$

$\lambda_{pf} \le \lambda_f \le \lambda_{rf}$

$$\phi M_{nFLB} := 0.9 \cdot \left(M_p - \left(M_p - 0.7 \cdot F_y \cdot S_x\right) \cdot \left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right) = 356.217 \; kip \cdot ft$$

$> 135.399 \; (kip \cdot ft)$

LTB

$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 10.667 \; ft$ $\qquad c := 1$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_0} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_0}\right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E}\right)^2}} = 35.146 \; ft$$

$r_{ts} := \sqrt{\frac{\sqrt{I_y \cdot C_w}}{S_x}} = 3.378 \; in$ $\qquad L_b := l_{34} = 26.37 \; ft$

$L_p < L_b \le L_r$

$M(x) := V_{max2} \cdot x - \frac{1}{2} \cdot w_{c2} \cdot x^2$ $\qquad M_{max} := M_{max2}$

$M_A := M\left(\frac{1}{4} \cdot l_{34}\right)$ $\qquad M_B := M\left(\frac{2}{4} \cdot l_{34}\right)$ $\qquad M_C := M\left(\frac{3}{4} \cdot l_{34}\right)$

$M_A := M_{max}$ $\qquad M_B := M_{max}$ $\qquad M_C := M_{max}$

$$C_b := min\left(3, \frac{12.5 \cdot M_{max}}{2.5 \cdot M_{max} + 3 \cdot M_A + 4 \cdot M_B + 3 \cdot M_C}\right) = 1$$

$$\phi M_{nLTB} := 0.9 \cdot min\left(M_p, C_b \cdot \left(M_p - \left(M_p - 0.7 \cdot F_y \cdot S_x\right) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right)\right) = 278.155 \; kip \cdot ft \; > 135.399 \; kip \cdot ft$$

Shear Check

$A_w := d \cdot t_w = 4.719 \; in^2$ $\qquad \phi := 0.9$

$\phi V_n := \phi \cdot A_w \cdot 0.6 \cdot F_y = 127.413 \; kip$ $\qquad > 20.538 \; kip$

Beam over the center window of main event space designed as W16X45 A36 steel.

Southern Most Columns

$L_r := 20\ psf$

$L := 10\ psf$

$S_{maxCorrection} = 37.223\ psf$

$W_{maxCorrection} = 26.798\ psf$

$L_{rCorrection} := 19.931\ psf$

$S_{maxCorrection} := 37.223\ psf$

$W_{maxCorrection} := 26.798\ psf$

$W_{gl} := 0.083\ \frac{kip}{ft}$

$D_{beam} := 45\ plf \cdot (120\ in) = 450\ lbf$

$D_{truss} := W_{gl} \cdot 29.1\ ft = 2.415\ kip$

$A_t := 58.419\ ft^2 + 383.673\ ft^2 + 229.897\ ft^2 = 671.989\ ft^2$

$D_{es} := 11.584\ psf$

$Dead := A_t \cdot D_{es} + D_{beam} + D_{truss} = 10.65\ kip$

$L_{rt} := L_{rCorrection} \cdot A_t = 13.393\ kip$

$L_t := L \cdot A_t = 6.72\ kip$

$Snow := S_{maxCorrection} \cdot A_t = 25.013\ kip$

$Wind := W_{maxCorrection} \cdot A_t = 18.008\ kip$

$D_{column} := 11.25\ in \cdot 11.25\ in \cdot G \cdot \gamma \cdot 17.434\ ft = 0.526\ kip$

$l_{34} := \frac{26.37\ ft}{2}$

$D_{joist2} := \frac{1.5\ in \cdot 5.5\ in \cdot G \cdot \gamma}{1\ ft} = 1.966\ psf$

$D_{board} := 0.75\ in \cdot G \cdot \gamma = 2.145\ psf$

$D_{beam2} := \frac{3 \cdot 2.5\ in \cdot 7.25\ in \cdot G \cdot \gamma}{5\ ft} = 2.592\ psf$

$D_{railing} := \frac{4\ plf}{6\ ft}$

$D_{23} := D_{joist2} + D_{beam2} + D_{board} + D_{railing} = 7.37\ psf$

$D_4 := D_{hardwood} + D_{gypsum} + D_{sub}$

$D_i := \frac{0.424\ \frac{kip}{ft}}{1.2} \cdot 5\ ft$

$L_i := \frac{3.36\ \frac{kip}{ft}}{1.6} \cdot 5\ ft$

From upper column

$D_{column} := 11.25\ in \cdot 11.25\ in \cdot G \cdot \gamma \cdot 17.434\ ft = 0.526\ kip$

$Dead := A_t \cdot D_{es} + D_{beam} + D_{truss} = 10.65\ kip$

$L_{rt} := L_{rCorrection} \cdot A_t = 13.393\ kip$

$L_t := L \cdot A_t = 6.72\ kip$

$Snow := S_{maxCorrection} \cdot A_t = 25.013\ kip$

$Wind := W_{maxCorrection} \cdot A_t = 18.008\ kip$

From interior

$D_i = 1.767\ kip$

$L_i = 10.5\ kip$

From exterior

$D_{ex} := D_{23} \cdot 6\ ft \cdot (l_{34} + 8.411\ ft) = 0.955\ kip$

$L_{ex} := 100\ psf \cdot 6\ ft \cdot (l_{34} + 8.411\ ft) = 12.958\ kip$

Total

$DeadAll := D_{column} + Dead + D_i + D_{ex} = 13.897\ kip$

$LiveAll := L_t + L_i + L_{ex} = 30.177\ kip$

$LiveRoof := L_{rt} = 13.393\ kip$

$Snow = 25.013\ kip$

$Wind = 18.008\ kip$

$w_1 := 1.2 \cdot DeadAll + 1.6 \cdot LiveAll + \max(LiveRoof, 0.5 \cdot Snow) = 78.354\ kip$

$w_2 := 1.2 \cdot DeadAll + \max(1.6 \cdot LiveRoof, Snow) + \max(LiveAll, 0.5 \cdot Wind) = 71.867\ kip$

$w_3 := 1.2 \cdot DeadAll + \max(1.6 \cdot LiveRoof, Snow) + \max(LiveAll, 0.5 \cdot Wind) = 71.867\ kip$

$w_4 := 1.2 \cdot DeadAll + Wind + LiveAll + \max(0.5 \cdot LiveRoof, 0.3 \cdot Snow) = 72.366\ kip$

$w_5 := 0.9 \cdot DeadAll + Wind = 30.515\ kip$

$P_c := w_1 = 78.354\ kip$

$l := 9.162\ ft$ $K_e := 1$ $l_e := l \cdot K_e = 9.162\ ft$

$F_c := 525\ psi$ $E := 12000\ ksi$ $E_{min} := 440\ ksi$

Column Section Selected: 16x16 Timber Southern Pine No. 2

$b := 15.5\ in$ $d := 15.5\ in$ $A := b \cdot d = 1.668\ ft^2$

Factors

$C_D := 1$ $C_t := 1$ $C_i := 1$ $C_F := 1$ $C_M := 0.91$ $C_P := 1$ $C_T := 1$

$F^{\circ}_c := F_c \cdot C_D \cdot C_t \cdot C_M \cdot C_T = 477.75\ psi$

$E'_{min} := E_{min} \cdot C_t \cdot C_i \cdot C_T = 440000\ psi$

$f_c := \frac{P_c}{A} = 326.135\ psi$

$$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\frac{l_e}{d}\right)^2} = 7188.608\ psi \qquad > \qquad f_c = 326.135\ psi$$

$$\alpha := \frac{1 + \left(\frac{F_{cE}}{F^{\circ}_c}\right)}{2 \cdot c} = 8.023 \qquad \beta := \frac{\frac{F_{cE}}{F^{\circ}_c}}{c} = 15.047$$

$$C_P := \alpha - \sqrt[2]{\alpha^2 - \beta} = 1$$

$$F'_c := F^{\circ}_c \cdot C_P = 477.75\ psi \qquad 477.75 > 326.135 \qquad f_c = 326.135\ psi$$

$$\lambda := \frac{l_e}{d} = 7.093 \qquad 7.093 < 50$$

$$f_{cnet} := \frac{P_c}{A} = 326.135\ psi \qquad 326.135 < 477.75 \qquad F^{\circ}_c = 477.75\ psi$$

16x16 Timber Southern Pine No. 2

Primary Footing

$$D_{newColumn} := 15.5\ in \cdot 15.5\ in \cdot G \cdot \gamma \cdot 9.1\ ft = 521.062\ lbf$$

$$DeadAll := D_{column} + Dead + D_i + D_{ex} + D_{newColumn} = 14.418\ kip$$

$$LiveAll := L_t + L_i + L_{ex} = 30.177\ kip$$

$$LiveRoof := L_{rt} = 13.393\ kip$$

$$Snow = 25.013\ kip$$

$$Wind = 18.008\ kip$$

$$w_1 := 1.2 \cdot DeadAll + 1.6 \cdot LiveAll + \max(LiveRoof, 0.5 \cdot Snow) = 78.979\ kip$$

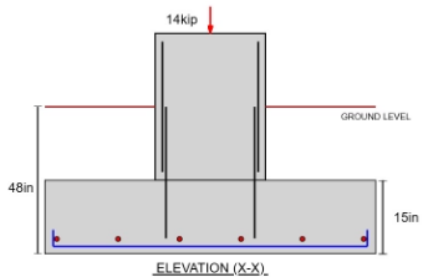
$$w_2 := 1.2 \cdot DeadAll + \max(1.6 \cdot LiveRoof, Snow) + \max(LiveAll, 0.5 \cdot Wind) = 72.493\ kip$$

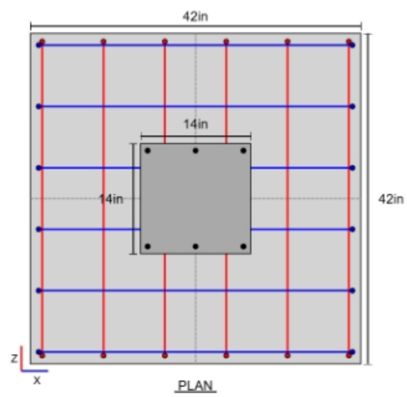
$$w_3 := 1.2 \cdot DeadAll + \max(1.6 \cdot LiveRoof, Snow) + \max(LiveAll, 0.5 \cdot Wind) = 72.493\ kip$$

$$w_4 := 1.2 \cdot DeadAll + Wind + LiveAll + \max(0.5 \cdot LiveRoof, 0.3 \cdot Snow) = 72.991\ kip$$

$$w_5 := 0.9 \cdot DeadAll + Wind = 30.984\ kip$$

From loadings; dead, live, and roof live will be used for foundation tabulation.





NAME	LENGTH	WIDTH	HEIGHT	DEPTH	REINF X	REINF Z	COL
FOOTING 1	42	42	15	48	6#4	6#4	6#4

Lodge Joint Footing

6x8 Timber Southern Pine Dense Select Structural 86

$$DeadColumn := 24.97\ psf \cdot 408.15\ ft^2 + 5.5\ in \cdot 7.5\ in \cdot G \cdot \gamma \cdot 11\ ft = 10.3\ kip$$
$$LiveColumn := 10\ psf \cdot 408.15\ ft^2 = 4.082\ kip$$
$$LiveRoofColumn := 19.931\ psf \cdot 408.15\ ft^2 = 8.135\ kip$$
$$SnowColumn := 37.223\ psf \cdot 408.15\ ft^2 = 15.193\ kip$$
$$WindColumn := 26.798\ psf \cdot 408.15\ ft^2 = 10.938\ kip$$

W14X22 A36 Steel Beam

$$DeadBeam := 1.525\ \frac{kip}{ft} \cdot 13\ ft = 19.825\ kip$$
$$LiveBeam := 1.525\ \frac{kip}{ft} \cdot 13\ ft = 19.825\ kip$$

All

$$DeadAll := DeadColumn + DeadColumn = 20.599\ kip$$
$$LiveAll := LiveColumn + LiveBeam = 23.907\ kip$$
$$LiveRoof := LiveRoofColumn = 8.135\ kip$$
$$Snow := SnowColumn = 15.193\ kip$$
$$Wind := WindColumn = 10.938\ kip$$

$$w_1 := 1.2 \cdot DeadAll + 1.6 \cdot LiveAll + \max(LiveRoof, 0.5 \cdot Snow) = 71.104\ kip$$

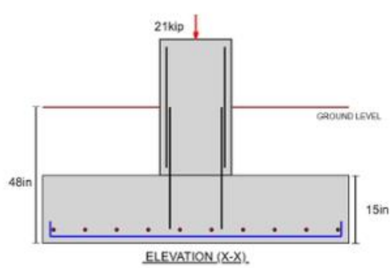
$$w_2 := 1.2 \cdot DeadAll + \max(1.6 \cdot LiveRoof, Snow) + \max(LiveAll, 0.5 \cdot Wind) = 63.818\ kip$$

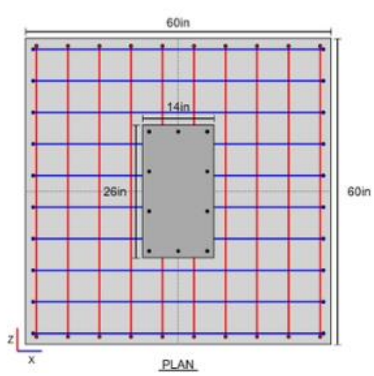
$$w_3 := 1.2 \cdot DeadAll + \max(1.6 \cdot LiveRoof, Snow) + \max(LiveAll, 0.5 \cdot Wind) = 63.818\ kip$$

$$w_4 := 1.2 \cdot DeadAll + Wind + LiveAll + \max(0.5 \cdot LiveRoof, 0.3 \cdot Snow) = 64.121\ kip$$

$$w_5 := 0.9 \cdot DeadAll + Wind = 29.477\ kip$$

From loadings; dead, live, and roof live will be used for foundation tabulation.





NAME	LENGTH	WIDTH	HEIGHT	DEPTH	REINF X	REINF Z	COL
FOOTING 1	60	60	15	48	10#4	10#4	10#4

First Floor Perimeter Strip Footing

Worst Case Design

$7.75\ ft$

$DeadWall := 19.967\ psf \cdot 7.75\ ft + \frac{(2 \cdot 1.5\ in \cdot 5.5\ in \cdot G \cdot \gamma \cdot 17\ ft)}{12\ in} = 0.222\ \frac{kip}{ft}$

$LiveRoofWall := 19.931\ psf \cdot 7.75\ ft = 0.154\ \frac{kip}{ft}$

$SnowWall := 37.223\ psf \cdot 7.75\ ft = 0.288\ \frac{kip}{ft}$

$WindWall := 26.798\ psf \cdot 7.75\ ft = 0.208\ \frac{kip}{ft}$

$DeadFloor := 10.4\ psf \cdot 7.25\ ft = 0.075\ \frac{kip}{ft}$

$LiveFloor := 100\ psf \cdot 7.25\ ft = 0.725\ \frac{kip}{ft}$

$DeadAll := DeadWall + DeadFloor = 0.297\ \frac{kip}{ft}$

$LiveAll := LiveFloor = 0.725\ \frac{kip}{ft}$

$LiveRoof := LiveRoofWall = 0.154\ \frac{kip}{ft}$

$Snow := SnowWall = 0.288\ \frac{kip}{ft}$

$Wind := WindWall = 0.208\ \frac{kip}{ft}$

$w_1 := 1.2 \cdot DeadAll + 1.6 \cdot LiveAll + \max(LiveRoof, 0.5 \cdot Snow) = 1.671\ \frac{kip}{ft}$

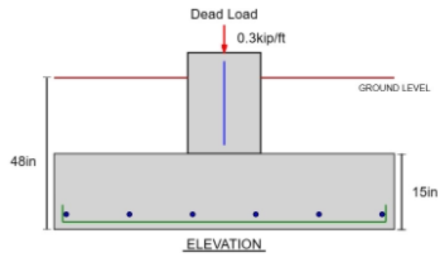
$w_2 := 1.2 \cdot DeadAll + \max(1.6 \cdot LiveRoof, Snow) + \max(LiveAll, 0.5 \cdot Wind) = 1.37\ \frac{kip}{ft}$

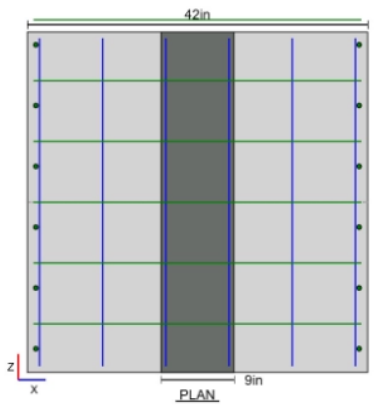
$w_3 := 1.2 \cdot DeadAll + \max(1.6 \cdot LiveRoof, Snow) + \max(LiveAll, 0.5 \cdot Wind) = 1.37\ \frac{kip}{ft}$

$w_4 := 1.2 \cdot DeadAll + Wind + LiveAll + \max(0.5 \cdot LiveRoof, 0.3 \cdot Snow) = 1.376\ \frac{kip}{ft}$

$w_5 := 0.9 \cdot DeadAll + Wind = 0.475\ \frac{kip}{ft}$

From loadings; dead, live, and roof live will be used for foundation tabulation.
Roof live load listed as Wind/Seismic for program.





NAME	WIDTH	HEIGTH	DEPTH	MAIN R	SEC. R	WALL	DOWEL
SF1	42	15	48	#5@7.5	6#5	#4@10	-

Strip Footing Retaining Wall

$$DeadWall := 19.967\ psf \cdot 7.5\ ft + \frac{1.5\ in \cdot 5.5\ in \cdot G \cdot \gamma \cdot 17.25\ ft}{16\ in} = 0.175\ \frac{kip}{ft}$$

$$RoofLive := 19.931\ psf \cdot 7.5\ ft = 0.149\ \frac{kip}{ft}$$

$$Snow := 37.223\ psf \cdot 7.5\ ft = 0.279\ \frac{kip}{ft}$$

$$Wind := 26.798\ psf \cdot 7.5\ ft = 0.201\ \frac{kip}{ft}$$

$$DeadFloor := 27.844\ plf + 10.4\ psf \cdot \frac{342\ in}{2} + 20.878\ plf = 0.197\ \frac{kip}{ft}$$

$$LiveFloor := 100\ psf \cdot \frac{342\ in}{2} = 1.425\ \frac{kip}{ft}$$

$$DeadAll := DeadWall + DeadFloor + 150\ pcf \cdot (8\ ft \cdot 2\ ft + 11.3\ ft \cdot 2\ ft) = 6.162\ \frac{kip}{ft}$$

$$LiveAll := LiveFloor = 1.425\ \frac{kip}{ft}$$

$$w_1 := 1.2 \cdot DeadAll + 1.6 \cdot LiveAll + \max(LiveRoof, 0.5 \cdot Snow) = 9.829\ \frac{kip}{ft}$$

$$w_2 := 1.2 \cdot DeadAll + \max(1.6 \cdot LiveRoof, Snow) + \max(LiveAll, 0.5 \cdot Wind) = 9.099\ \frac{kip}{ft}$$

$$w_3 := 1.2 \cdot DeadAll + \max(1.6 \cdot LiveRoof, Snow) + \max(LiveAll, 0.5 \cdot Wind) = 9.099\ \frac{kip}{ft}$$

$$w_4 := 1.2 \cdot DeadAll + Wind + LiveAll + \max(0.5 \cdot LiveRoof, 0.3 \cdot Snow) = 9.104\ \frac{kip}{ft}$$

$$w_5 := 0.9 \cdot DeadAll + Wind = 5.747\ \frac{kip}{ft}$$

$$w_{max} := \max(w_1, w_2, w_3, w_4, w_5) = 9.829\ \frac{kip}{ft}$$

$\phi' := 25$ $c' := 1.45\ psi$ (estimated values used) $\gamma' := 55.5\ \frac{lbf}{ft^3}$

Wall height + soil to base height $Depth := 9.1\ ft + 4.6\ ft$ $B := 7\ ft + 4\ in$

$N_c := 25.1$ $N_q := 12.7$ $N_\gamma := 9.2$ $\sigma'_{z0} := \gamma' \cdot Depth$

$$q_n := c' \cdot N_c + \sigma'_{z0} \cdot N_q + 0.5 \cdot \gamma' \cdot B \cdot N_\gamma = 116.455\ psi$$

$$q_{maxLoad} := q_n \cdot B = 122.977\ \frac{kip}{ft}$$

$$w_{max} = 9.829\ \frac{kip}{ft} \quad < \quad q_{maxLoad} = 122.977\ \frac{kip}{ft}$$

Significant factor of safety used to account for only one side having full soil depth and limited soil data. Estimations of rebar reinforcements will be made based off of the tested strip footing.

Adequate bearing capacity to hold 79 kip loading of spread footing, joint spread and strip footings not needed. Spread footings that would line up with strip footings will be made into consistent strip footing.

Concrete Flooring

5" concrete slab on grade will be used for both interior and exterior locations.

Figure 23: Multi-Use Lodge Structural Loading Calculations

REFERENCES	CALCULATIONS	RESULTS

ACI 318-19 Spread Footing Design

SkyCiv

Dimensions

Symbol	Description	Value
-	Footing Name	FOOTING 1
-	Material	Reinforced Concrete
B	Footing Width	3.5
L	Footing Length	3.5
Df	Foundation Base Depth	4
H	Base Height	15
c_1	Column Length	14
c_2	Column Width	14
col_{height}	Column Height	4
x_{offset}	Column X Offset	0
z_{Offset}	Column Z Offset	0
pof_{col}	Punching Overhang Factor	1

Concrete Properties

Symbol	Description	Value
f'_c	Concrete Compressive Strength	4
w_c	Concrete Unit Weight	150
d_{agg}	Max. Size of Aggregate	1

Reinforcement

Symbol	Description	Value
C_c	Reinforcement Clear Cover	3
f_y	Reinforcement Yield Strength	60
-	Reinforcement Type	Normal
$d_{b,x}$	Footing Bar Size (X Direction)	#4
n_x	Footing Bars in X Direction (n_x)	6
-	X Bars with Hooks?	True
$d_{b,z}$	Footing Bar Size (Z Direction)	#4
n_z	Footing Bars in Z Direction (n_z)	6
-	Z Bars with Hooks?	True
$d_{b,col}$	Dowel Bar Size	#4
$n_{x,col}$	Dowel Bars in X Direction	3
$n_{z,col}$	Dowel Bars in Z Direction	2
-	Dowels with Hooks?	False

Geotechnical Parameters

Symbol	Description	Value
q_a	Allowable Bearing Capacity	2
φ	Foundation-Soil Friction Angle	25
$SF_{overturning}$	Min. Overturning Safety Factor	1.5
$SF_{sliding}$	Min. Sliding Safety Factor	1.5

Load Cases

Name	Case	N (kip)	Mx (kip-ft)	Mz (kip-ft)	Vx (kip)	Vz (kip)
Total DL	DL	15.2	0	0	0	0
Total LL	LL	20.2	0	0	0	0
Total Lr	Lr	13.4	0	0	0	0
Total W/S	W/S	0	0	0	0	0

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Name	Case	N (kip)	Mx (kip-ft)	Mz (kip-ft)	Vx (kip)	Vz (kip)
Dead	DL	14.4	0	0	0	0
Live	LL	20.2	0	0	0	0
RoofLive	Lr	13.4	0	0	0	0

Load Combinations

Name	Dead Load Factor	Live Load Factor	Roof Live Load Factor	Wind / Seismic Factor
S1	1.0	0.0	0.0	0.0
U1	1.2	1.6	1.0	0.0

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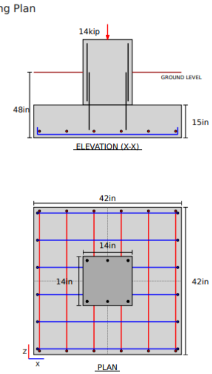
REFERENCES	CALCULATIONS	RESULTS

Spread Footing Plan



NAME	LENGTH	WIDTH	HEIGHT	DEPTH	REINF X	REINF Z	COL
FOOTING 1	42	42	15	48	6#4	6#4	6#4

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REFERENCES	CALCULATIONS	RESULTS

Footing Stability

Maximum Eccentricity Check

Critical Load Combination for Maximum Eccentricity on the Base: S1=1DL + 0LL + 0Lr + 0W/S

Eccentricity

$$\Sigma M_z = -(P \times offset_z) + M_z - (V_z \times D_f) = -(15.235 \times 0) + 0 - (0 \times 48) = 0 \text{ kip*in}$$

$$\Sigma M_x = -(-P \times offset_x) + M_x + (V_x \times D_f) = -(-15.235 \times 0) + 0 + (0 \times 48) = 0 \text{ kip*in}$$

$$\Sigma P = P + \Delta q \times A = 15.2347 + 0 \times 1764 = 15.2347 \text{ kip}$$

$$ratio_e = \frac{\left|\frac{\Sigma M_x}{\Sigma P}\right|}{\left(\frac{L}{6}\right)} + \frac{\left|\frac{\Sigma M_z}{\Sigma P}\right|}{\left(\frac{B}{6}\right)} = \frac{\left|\frac{0}{15.23}\right|}{\left(\frac{42}{6}\right)} + \frac{\left|\frac{0}{15.23}\right|}{\left(\frac{42}{6}\right)} = 0$$

Eccentricity Check

ratio_e <= 1.0
The resultant is within the middle third portion of the base.

PASS

Soil Bearing Pressure Check

Critical Load Combination for Maximum Soil Stress: S1=1DL + 0LL + 0Lr + 0W/S

$$qs_{max} = \frac{\Sigma P}{(B \times L)} + \left(\frac{6 \times \Sigma M_x}{\left((B)^2 \times L\right)}\right) + \left(\frac{6 \times \Sigma M_z}{\left((L)^2 \times B\right)}\right) = \frac{15.2347}{(42 \times 42)} + \left(\frac{6 \times 0}{\left((42)^2 \times 42\right)}\right) + \left(\frac{6 \times 0}{\left((42)^2 \times 42\right)}\right)$$

$$qs_{max} = 0.0086 \text{ ksi}$$

Pedestal wWight $G_{pedestal} =$ 0.82kip

Soil Pressures and Eccentricities Summary

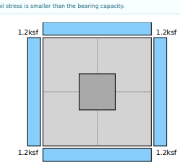
Name	ΣP (kip)	ΣM_x (kip-ft)	e_z (in)	ΣM_z (kip-ft)	e_x (in)	$ratio_e$	qs_{max} (ksf)	qs_{mean} (ksf)	$\frac{qs_{max}}{qs_{mean}}$	qs_{min} (ksf)
S1	15.23	0	0	0	0	0	1.24	1.24	1	1.24

Maximum Soil Stress $qs_{max} =$ 1.24ksf
Mean Soil Stress $qs_{mean} =$ 1.24ksf
Allowable Soil Bearing Capacity $qa =$ 2.00ksf

Soil Bearing Capacity Check

qa >= q_{max}
The maximum soil stress is smaller than the bearing capacity.

PASS



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REFERENCES	CALCULATIONS	RESULTS

Overturning Check

Overturning about X-Axis

Critical load combination for overturning: S1=1DL + 0LL + 0Lr + 0W/S

No moments about X-Axis were considered.

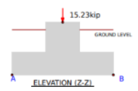
Overturning about Z-Axis

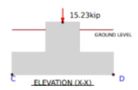
Critical load combination for overturning: S1=1DL + 0LL + 0Lr + 0W/S

No moments about Z-Axis were considered.

The footing weight was not considered.

The overburden soil weight was not considered.





Overturning Checks Summary

LC	Point	Mres_x (kip-ft)	Mover_x (kip-ft)	SF_x	Point	Mres_z (kip-ft)	Mover_z (kip-ft)	SF_z
S1	A	26.66	0	>1000	C	26.66	0	>1000

Overturning Safety Factor $SF_{overturning} =$ >1000.0
Mimimum Safety Factor $SF_{min} =$ 1.5

SF >= 1.5
The foundation is safe against overturning.

PASS

Sliding Check

Critical load combination for sliding: S1=1DL + 0LL + 0Lr + 0W/S

No shear forces were considered.

Sliding Safety Factor $SF_{sliding} =$ >1000.0
Mimimum Safety Factor $SF_{min} =$ 1.5

SF >= 1.5

PASS

The foundation is safe against sliding.

REFERENCES	CALCULATIONS	RESULTS

Footing Strength

$$d_x = H - c - \frac{db_x}{2} = 15 - 3 - \frac{0.5}{2} = 11.75\ \text{in}$$

$$d_z = H - c - db_x - \frac{db_z}{2} = 15 - 3 - 0.5 - \frac{0.5}{2} = 11.25\ \text{in}$$

$$\rho_x = \frac{n_x \times \left(\frac{\pi \times (db_x)^2}{4}\right)}{(B \times d_x)} \times 1000 = \frac{6 \times \left(\frac{3.14 \times (0.5)^2}{4}\right)}{(42 \times 11.75)} \times 1000 = 2.39\ /1000$$

$$\rho_z = \frac{n_z \times \left(\frac{\pi \times (db_z)^2}{4}\right)}{(L \times d_z)} \times 1000 = \frac{6 \times \left(\frac{3.14 \times (0.5)^2}{4}\right)}{(42 \times 11.25)} \times 1000 = 2.49\ /1000$$

One Way Shear Check in X

22.5.5.1

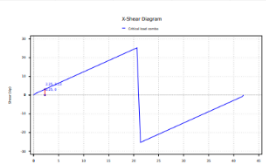
$$V_{cx} = 8 \times 0.1 \times (\rho_x)^{\frac{1}{3}} \times rf \times B \times d_x = 8 \times 0.1 \times (2.39)^{\frac{1}{3}} \times 63.25 \times 42 \times 11.75 = 33371.2\ \text{lb}$$

$$l_{l1} = \frac{(L - c_1)}{2} - d_x + offset_x = \frac{(42 - 14)}{2} - 11.75 + 0 = 2.25\ \text{in}$$

$$l_{l2} = \frac{(L - c_1)}{2} - d_x - offset_x = \frac{(42 - 14)}{2} - 11.75 - 0 = 2.25\ \text{in}$$

Maximum Shear X-Axis Summary

LC	dist 1 (in)	Shear 1 (kip)	dist 2 (in)	Shear 2 (kip)
U1	2.25	3.13	39.75	-3.13



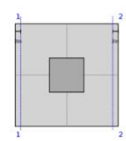
Critical Load Combination: U1=1.2DL + 1.6LL + 1Lr + 0W/S

Maximum Shear that can be Resisted at the Section $V_{cx} =$ 33.37kip
Shear Strength Reduction Factor $\phi =$ 0.75
Maximum Factored Shear Force at the Section $V_u =$ 3.13kip

$$Shear_{XRatio} = \frac{V_u}{\phi \times V_c} = \frac{3.13}{0.75 \times 33.37}$$

$$Shear_{XRatio} = 0.13$$

UTILITY: 0.13



One Way Shear Check in Z

22.5.5.1

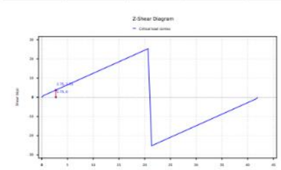
$$V_{cz} = 8 \times 0.1 \times (\rho_z)^{\frac{1}{3}} \times rf \times L \times d_z = 8 \times 0.1 \times (2.49)^{\frac{1}{3}} \times 63.25 \times 42 \times 11.25 = 32417.65\ \text{lb}$$

$$b_{l1} = \frac{(B - c_2)}{2} - d_z + offset_z = \frac{(42 - 14)}{2} - 11.25 + 0 = 2.75\ \text{in}$$

$$b_{l2} = \frac{(B - c_2)}{2} - d_z - offset_z = \frac{(42 - 14)}{2} - 11.25 - 0 = 2.75\ \text{in}$$

Max Shear Z-Axis Summary

LC	dist 1 (in)	Shear 1 (kip)	dist 2 (in)	Shear 2 (kip)
U1	2.75	3.73	39.25	-3.73



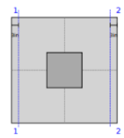
Critical Load Combination: U1=1.2DL + 1.6LL + 1Lr + 0W/S

Maximum Shear that can be Resisted at the Section $V_{cz} =$ 32.42kip
Shear Strength Reduction Factor $\phi =$ 0.75
Maximum Factored Shear Force at the Section $V_u =$ 3.73kip

$$Shear_{ZRatio} = \frac{V_u}{\phi \times V_c} = \frac{3.73}{0.75 \times 32.42}$$

$$Shear_{ZRatio} = 0.15$$

UTILITY: 0.15



Punching Shear Check

Ultimate Soil Pressure Summary

Name	ΣP (kip)	ΣM_x (kip-ft)	e_z (in)	ΣM_z (kip-ft)	e_x (in)	$ratio_e$	qs_{max} (ksf)	qs_{mean} (ksf)	$\frac{qs_{max}}{qs_{mean}}$	qs_{min} (ksf)
U1	50.56	0	0	0	0	0	4.13	4.13	1	4.13

Critical Load Combination: U1=1.2DL + 1.6LL + 1Lr + 0W/S

Maximum Soil Stress	$qu_{max} =$	4.13ksf
Mean Soil Stress	$qu_{mean} =$	4.13ksf

$$d_p = \frac{(d_x + d_z)}{2} = \frac{(11.75 + 11.25)}{2} = 11.5\ \text{in}$$

Ratio of Long to Short Dimensions (Column)	$\beta =$	1.00
Modification Factor for Lightweight Concrete	$\lambda =$	1.00
Perimeter of Critical Section	$bo =$	102.00in
X-Axis Distance	$b1 =$	25.50in
Z-Axis Distance	$b2 =$	25.50in
Equivalent Polar Moment of Inertia	$Jc_x =$	133588in⁴
Equivalent Polar Moment of Inertia	$Jc_z =$	133588in⁴
Factor for the Moment Fraction	$\gamma_{vx} =$	0.40
Factor for the Moment Fraction	$\gamma_{vz} =$	0.40
X-axis Largest Distance to the Extreme Fiber	$c_x =$	12.75in
Z-axis Largest Distance to the Extreme Fiber	$c_z =$	12.75in

$$v_c = min(4, 2 + \frac{4}{\beta}, 2 + \frac{\alpha_s \times d_p}{bo}) \times \lambda \times rf$$

$$v_c = min(4, 2 + \frac{4}{1}, 2 + \frac{40 \times 11.5}{102.00}) \times 1.00 \times 63.25$$

$$v_c = 252.98psi$$

22.6.5.2

$$v_{un} = \frac{qu_m \times (B \times L - (d_1 \times d_2))}{(bo \times d_p)} = \frac{28.66 \times (42 \times 42 - (25.5 \times 25.5))}{(102 \times 11.5)} = 27.22\ \text{psi}$$

22.6.5.2

$$v_{umx} = \frac{\gamma_{vx} \times |Mu_z| \times cx}{Jc_x} = \frac{0.4 \times |0| \times 12.75}{133587.59} = 0\ \text{psi}$$

22.6.5.2

$$v_{umz} = \frac{\gamma_{vz} \times |Mu_x| \times cz}{Jc_z} = \frac{0.4 \times |0| \times 12.75}{133587.59} = 0\ \text{psi}$$

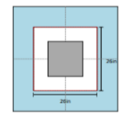
22.6.5.2

$$v_u = v_{un} + v_{umx} + v_{umz} = 27.22 + 0 + 0 = 27.22\ \text{psi}$$

$$Punching_{Ratio} = \frac{v_u}{\phi \times v_c} = \frac{27.22}{0.75 \times 252.98}$$

$$Punching_{Ratio} = 0.14$$

UTILITY: 0.14



Bending Check (X)

13.2.7.1

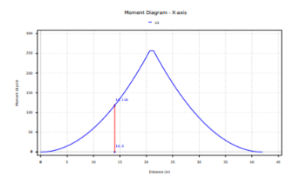
$$l_{21} = \frac{(L - c_1)}{2} + offset_x = \frac{(42 - 14)}{2} + 0 = 14\ \text{in}$$

13.2.7.1

$$l_{22} = \frac{(L - c_1)}{2} - offset_x = \frac{(42 - 14)}{2} - 0 = 14\ \text{in}$$

Max Moment X-Axis Summary

LC	dist 1 (in)	Moment 1 (kip-ft)	dist 2 (in)	Moment 2 (kip-ft)
U1	14	9.83	28	9.83



Critical Load Combination: U1=1.2DL + 1.6LL + 1Lr + 0W/S

Maximum Moment at the Critical Section	$M_{ux} =$	9.83kip-ft
Depth of Equivalent Rectangular Compressive Stress Block	$\beta_1 =$	0.85

Modulus of Elasticity of Reinforcement	$E_s =$	29000.00ksi
Bending Strength Reduction Factor (Tension Controlled)	$\phi =$	0.90

Calculation of the Strength Coefficient

Eq. 4.10 CRSI Manual

$$R_{ux} = \frac{M_{ux}}{\left(\varphi \times B \times (d_x)^2\right)} = \frac{117984.533}{\left(0.9 \times 42 \times (11.75)^2\right)} = 22.608\ \text{psi}$$

Reinforcement Area Check (X)

Eq. 4.9 CRSI Manual

$$As_{reqx} = \frac{0.85 \times f_c \times B \times d_x}{f_y} \times \left(1 - \sqrt{1 - \frac{2 \times R_{ux}}{(0.85 \times f_c)}}\right) = \frac{0.85 \times 4000 \times 42 \times 11.75}{60000} \times \left(1 - \sqrt{1 - \frac{2 \times 22.61}{(0.85 \times 4000)}}\right)$$

$$As_{reqx} = 0.19\ \text{in}^2$$

7.6.1.1

$$As_{minx} = 0.0018 \times H \times B = 0.0018 \times 15 \times 42$$

$$As_{minx} = 1.13\ \text{in}^2$$

Eq. 4.9 CRSI Manual

$$As_{maxx} = \frac{0.85 \times \beta_1 \times f_c \times B \times d_x}{f_y} \times \left(\frac{0.003}{\left(\left(\frac{f_y}{E_s}\right) + 0.006\right)}\right) = \frac{0.85 \times 0.85 \times 4000 \times 42 \times 11.75}{60000} \times \left(\frac{0.003}{\left(\left(\frac{60000}{29000000}\right) + 0.006\right)}\right)$$

$$As_{maxx} = 8.84\ \text{in}^2$$

$$As_{provx} = \frac{n_x \times \pi \times (db_x)^2}{4} = \frac{6 \times 3.14 \times (0.5)^2}{4}$$

$$As_{provx} = 1.18\ \text{in}^2$$

Reinforcement Area (X)

As_{provx} = 1.178 > As_{reqx} = 0.187

Provided reinforcement area > required reinforcement area.

PASS

Minimum Reinforcement Area (X)

As_{provx} = 1.178 > As_{minx} = 1.134

Provided reinforcement area > minimum reinforcement area.

PASS

Maximum Reinforcement Area (X)

As_{provx} = 1.178 < As_{maxx} = 8.838

Provided reinforcement area < maximum reinforcement area due to ductility.

PASS

Calculation of the Maximum Resisting Moment (X)

Eq. 4.3 CRSI Manual

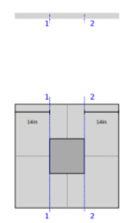
$$M_{nx} = As_{provx} \times f_y \times \left(d_x - \frac{(As_{provx} \times f_y)}{(1.7 \times B \times f_c)}\right) = 1.18 \times 60000 \times \left(11.75 - \frac{(1.18 \times 60000)}{(1.7 \times 42 \times 4000)}\right) = 813063.85\ \text{lb-in}$$

Nominal Flexural Strength	$M_{nx} =$	67.76kip-ft

$$Bending_{XRatio} = \frac{M_{ux}}{\phi \times M_{nx}} = \frac{117.98}{0.9 \times 813.06}$$

$$Bending_{XRatio} = 0.16$$

UTILITY: 0.16



REFERENCES	CALCULATIONS	RESULTS

Bending Check Z-Axis

13.2.7.1

$$b_{l1} = \frac{(B - c_1)}{2} + offset_x = \frac{(42 - 14)}{2} + 0 = 14\,\text{in}$$

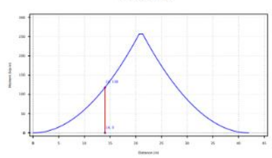
13.2.7.1

$$b_{l2} = \frac{(B - c_1)}{2} - offset_x = \frac{(42 - 14)}{2} - 0 = 14\,\text{in}$$

Max Moment Z-Axis Summary

LC	dist 1 (in)	Moment 1 (kip-ft)	dist 2 (in)	Moment 2 (kip-ft)
U1	14	9.83	28	9.83

Moment Diagram - (Z)



Critical Load Combination: U1=1.2DL + 1.6LL + 1Lr + 0W/S

Maximum Moment at the Critical Section	$M_{uz} =$	9.83kip-ft
Depth of Equivalent Rectangular Compressive Stress Block	$\beta_1 =$	0.85
Modulus of Elasticity of Reinforcement	$E_s =$	29000.00ksi
Bending Strength Reduction Factor (Tension Controlled)	$\phi =$	0.90
Ratio of Long to Short Footing Dimensions	$\beta =$	1.00

Calculation of the Strength Coefficient

Eq. 4.10 CRSI Manual

$$R_{nz} = \frac{M_{uz}}{\left(\varphi \times L \times (d_z)^2\right)} = \frac{117984.533}{\left(0.9 \times 42 \times (11.25)^2\right)} = 24.662\,\text{psi}$$

Reinforcement Area Check (Z)

Eq. 4.9 CRSI Manual

$$As_{reqz} = \frac{0.85 \times f_c \times L \times d_z}{f_y} \times \left(1 - \sqrt{1 - \frac{2 \times R_{nz}}{(0.85 \times f_c)}}\right) = \frac{0.85 \times 4000 \times 42 \times 11.25}{60000} \times \left(1 - \sqrt{1 - \frac{2 \times 24.66}{(0.85 \times 4000)}}\right)$$

$$As_{reqz} = 0.19\,\text{in}^2$$

Reinforcement Distribution Factor

R13.3.3.3

$$\gamma_s = \frac{2 \times \beta}{(\beta + 1)} = \frac{2 \times 1}{(1 + 1)}$$

$$\gamma_s = 1$$

R13.3.3.3

$$As_{reqzs} = As_{reqz} \times \gamma_s = 0.19 \times 1$$

$$As_{reqzs} = 0.19\,\text{in}^2$$

7.6.1.1

$$As_{minz} = 0.0018 \times H \times L = 0.0018 \times 15 \times 42$$

$$As_{minz} = 1.13\,\text{in}^2$$

Eq. 4.9 CRSI Manual

$$As_{maxz} = \frac{0.85 \times \beta_1 \times f_c \times L \times d_z}{f_y} \times \left(\frac{0.003}{\left(\left(\frac{f_y}{E_s}\right) + 0.006\right)}\right) = \frac{0.85 \times 0.85 \times 4000 \times 42 \times 11.25}{60000} \times \left(\frac{0.003}{\left(\left(\frac{60000}{29000000}\right) + 0.006\right)}\right)$$

$$As_{maxz} = 8.46\,\text{in}^2$$

$$As_{provz} = \frac{n_z \times \pi \times (db_z)^2}{4} = \frac{6 \times 3.14 \times (0.5)^2}{4}$$

$$As_{provz} = 1.18\,\text{in}^2$$

Reinforcement Area (Z)

As_{provz} = 1.178 > As_{reqx} = 0.195
Provided reinforcement area > required reinforcement area. — PASS

Minimum Reinforcement Area (Z)

As_{provz} = 1.178 > As_{minz} = 1.134
Provided reinforcement area > minimum reinforcement area. — PASS

Maximum Reinforcement Area (Z)

As_{provz} = 1.178 < As_{maxx} = 8.462
Provided reinforcement area < maximum reinforcement area due to ductility. — PASS

Calculation of the Maximum Resisting Moment (Z). It considers the Reinforcement Distribution Factor.

Eq. 4.3 CRSI Manual

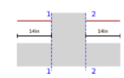
$$M_{nz} = \frac{As_{provz} \times f_y \times \left(d_z - \frac{(As_{provz} \times f_y)}{(1.7 \times L \times f_c \times \gamma_s)}\right)}{\gamma_s} = \frac{1.18 \times 60000 \times \left(11.25 - \frac{(1.18 \times 60000)}{(1.7 \times 42 \times 4000 \times 1)}\right)}{1} = 777720.94\,\text{lb-in}$$

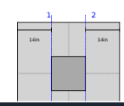
Nominal Flexural Strength	$M_{nz} =$	64.81kip-ft

$$Bending_{ZRatio} = \frac{M_{uz}}{\phi \times M_{nz}} = \frac{117.98}{0.9 \times 777.72}$$

$$Bending_{ZRatio} = 0.17$$

UTILITY: 0.17





REFERENCES	CALCULATIONS	RESULTS

Transfer of Forces

Vertical Transfer - Compression Check

22.8.3.1

Bearing Strength Calculation

$$A_1 = c_1 \times c_2 = 14 \times 14 = 196\,\text{in2}$$

$$A_2 = (min(l_{21}, 2 \times H) + c_1 + min(l_{22}, 2 \times H)) \times (min(b_{21}, 2 \times H) + (c_2 + min(b_{22}, 2 \times H))$$
$$A_2 = (min(14.00, 2 \times 15.00) + (14.00) + min(14.00, 2 \times 15.00)) \times (min(14.00, 2 \times 15.00) + (14.00) + min(14.00, 2 \times 15.00))$$
$$A_2 = 1764.00 in2$$

Table 22.8.3.2

$$B_n = min\left(\sqrt{\frac{A_2}{A_1}}, 2\right) \times 0.85 \times f_c \times A_1$$
$$B_n = min\left(\sqrt{\frac{1764.00}{196.00}}, 2.0\right) \times 0.85 \times 4000 \times 196.00$$
$$B_n = 1332800\,\text{lb}$$

Critical Load Combination for Bearing: U1=1.2DL + 1.6LL + 1Lr + 0W/S

Factored Bearing Load	$B_u =$	50.56kip
Bearing Strength Reduction Factor (Table 21.2.1.2)	$\phi =$	0.65
Reduced Bearing Strength	$\phi Bn =$	866.32kip

$$Bearing_{Ratio} = \frac{B_u}{\phi \times B_n} = \frac{50.56}{0.65 \times 1332.80}$$

$$Bearing_{Ratio} = 0.06$$

UTILITY: 0.06

16.3.4.1

$$As_{mindowels} = 0.005 \times A_1 = 0.005 \times 196 = 0.98\,\text{in2}$$

$$As_{dowels} = \frac{\pi \times (db_{col})^2}{4} \times n = \frac{3.14 \times (0.5)^2}{4} \times 6 = 1.18\,\text{in2}$$

Asd_{prov} = 1.18 > $Asd_{min/req}$ = 0.98
Provided dowels reinforcement area > minimum/required reinforcement area. — PASS

Footing Details

Reinforcement Spacing Check

7.7.2.3

$$s_{max1} = min(3 * H, 18.00)$$

$$s_{max1} = min(3 * 15, 18.00)$$

$$s_{max1} = 18\,\text{in}$$

24.3

$$s_{max2} = min(15.0 * 40000/f_s - (2.5 * c_c), 12 * 40000/f_s)$$

$$s_{max2} = min(15.0 * 40000/40000 - (2.5 * 3), 12 * 40000/40000)$$

$$s_{max2} = 7.5\,\text{in}$$

Spacing for the Reinforcement along X-Axis

25.2

$$s_{minx} = max\,(1.0, db_x, (4 \times d_{agg})/3.0)) + db_x$$
$$s_{min} = max\,(1.0, 0.5, 4 \times 1.00/3.0)) + 0.5$$
$$s_{min} = 1.83 in$$

$$s_x = \frac{(B - 2 \times c - db_z)}{(n_x - 1)} = \frac{(42 - 2 \times 3 - 0.5)}{(6 - 1)}$$

$$s_x = 7.1 \text{ in}$$

s_{max} = 7.5 > s_x = 7.1 > s_{min} = 1.833
Provided spacing is O.K.

PASS

Spacing for the Reinforcement along Z-Zxis

25.2

$$s_{minz} = max(1.0, db_z, (4 \times d_{agg})/3.0)) + db_z$$
$$s_{min} = max(1.0, 0.5, 4 \times 1.00/3.0)) + 0.5$$
$$s_{min} = 1.83 in$$

$$s_z = \frac{(L - 2 \times c - db_z)}{(n_z - 1)} = \frac{(42 - 2 \times 3 - 0.5)}{(6 - 1)}$$

$$s_z = 7.1 \text{ in}$$

s_{max} = 7.5 > s_z = 7.1 > s_{min} = 1.833
Provided spacing is O.K.

PASS

REFERENCES	CALCULATIONS	RESULTS

Development Length Check (X)

Distance to Nearest Concrete Surface or Half Spacing	$cb_x =$	3.25
Root of Concrete Strength Value min(√f'c, 100psi)	$rf =$	63.25
Epoxi Modification Factor	$\psi_e h =$	1.00
Confining Reinforcement Factor	$\psi_r =$	1.60
Location Factor	$\psi_o =$	1.00
Concrete Strength Factor	$\psi_c =$	0.87
Lightweight Concrete Factor	$\lambda =$	1.00

25.4.3.1

$$ldhx = \frac{(f_y \times \psi_e \times \psi_r \times \psi_o \times \psi_c)}{(55 \times \lambda \times rf)} \times (db_x)^{1.5} = \frac{(60000 \times 1 \times 1.6 \times 1 \times 0.87)}{(55 \times 1 \times 63.25)} \times (0.5)^{1.5}$$

$$ldhx = 8.46 \text{ in}$$

25.4.3.1

$$ld_{minx} = max(8 * db_x, 6.0)$$

$$ld_{minx} = max(8 * 0.5, 6.0)$$

$$ld_{minx} = 6 \text{ in}$$

10.21 CRSI Manual

$$Distance_{avail} = min(l_{21}, l_{22}) - c$$

$$Distance_{avail} = min(14, 14) - 3$$

$$Distance_{avail} = 11 \text{ in}$$

Development Length (X) $ld_{calc} =$ 8.46

ld_{calcx} = 8.46 < $Distance_{avail}$ = 11
Available length > minimum development length.

PASS

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SkyCiv

REFERENCES	CALCULATIONS	RESULTS

Development Length Check (Z)

Distance to Nearest Concrete Surface or Half Spacing	$cb_z =$	3.25
Root of Concrete Strength Value min(√f'c, 100psi)	$rf =$	63.25
Epoxi Modification Factor	$\psi_e h =$	1.00
Confining Reinforcement Factor	$\psi_r =$	1.60
Location Factor	$\psi_o =$	1.00
Concrete Strength Factor	$\psi_c =$	0.87
Lightweight Concrete Factor	$\lambda =$	1.00

25.4.3.1

$$ldhz = \frac{(f_y \times \psi_e \times \psi_r \times \psi_o \times \psi_c)}{(55 \times \lambda \times rf)} \times (db_z)^{1.5} = \frac{(60000 \times 1 \times 1.6 \times 1 \times 0.87)}{(55 \times 1 \times 63.25)} \times (0.5)^{1.5}$$

$$ldhz = 8.46 \text{ in}$$

25.4.3.1

$$ld_{minz} = max(8 * db_z, 6.0)$$

$$ld_{minz} = max(8 * 0.5, 6.0)$$

$$ld_{minz} = 6 \text{ in}$$

10.21 CRSI Manual

$$Distance_{avail} = min(b_{21}, b_{22}) - c$$

$$Distance_{avail} = min(14, 14) - 3$$

$$Distance_{avail} = 11 \text{ in}$$

Development Length (Z) $ld_{calc} =$ 8.46

ld_{calcz} = 8.46 < $Distance_{avail}$ = 11
Available length > minimum development length.

PASS

Minimum Footing Height Check

13.3.1.2

$$H_{min} = 6 + db_z + \frac{db_x}{2} + c = 6 + 0.5 + \frac{0.5}{2} + 3 = 9.75 \text{ in}$$

Column Dowels Compression Development Length

25.4.9.2

$$l_d = max\left(0.0003 \times f_y \times \psi_r \times db_{col}, \frac{\psi_r \times db_{col} \times f_y}{50 \times \lambda \times rf}\right)$$
$$l_d = max\left(0.0003 \times 60000 \times 1 \times 0.5, \frac{1 \times 0.5 \times 60000}{50 \times 1 \times 63.25}\right)$$
$$l_d = 9.49 \text{ in}$$

Min. Dowels Development Length (25.4.9.1) $l_{dmin} =$ 9.75in

25.4.9.2

$$H_{dowels} = l_d + \frac{hook_{dim}}{2} + db_{col} + c = 9.49 + \frac{0}{2} + 0.5 + 3 = 12.99 \text{ in}$$

Footing Base Height $H =$ 15.00in

H = 15 > H_{min} = 12.99
Footing height is OK.

PASS

Results Summary

Result Name	Results
STABILITY CHECKS	
Eccentricity Ratio	PASS
Bearing Capacity	PASS
Overturning	PASS
Sliding	PASS
STRENGTH UTILIZATION	

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Result Name	Results
Shear (X)	0.13
Shear (Z)	0.15
Punching Shear	0.14
Bending Capacity (X)	0.16
Bending Capacity (Z)	0.17
Bearing Capacity	0.06
DETAILING	
Min/Max Reinforcement (X)	PASS
Mix/Max Reinforcement (Z)	PASS
Dowels Reinforcement	PASS
Bar Spacing (X)	PASS
Bar Spacing (Z)	PASS
Development Length (X)	PASS
Development Length (Z)	PASS
Footing Height Check	PASS

About this Calculator

Calculator Name: ACI 318-19 Spread Footing Design
Description: This calculator is in imperial units and it is used for the design of reinforced concrete spread footings. It also considers column eccentricities on both axes.

Powered by SkyCiv

URL: https://platform.skyciv.com/quick-design?uid=8002-aci-pad-footing-design-calculator&footing_name=FOOTING%201&footing_material=rcfooting&footing_width=3.5&footing_length=3.5&footing_depth=4&footing_height=15&col_c1=14&col_c2=14&col_height=4&col_x=0&col_z=0&punching_overhang_factor=1&fc=4&unit_weight=150&max_aggregate_size=1&cover=3&f_y=60&reinforcement_type=normal&bar_size_x=4&n_x=6&x_bars_with_hooks=true&bar_size_z=4&n_z=6&z_bars_with_hooks=true&col_bar_size=4&col_nx_bar=3&col_nz_bar=2&dowels_with_hooks=false&q_a=2&phi_footing_soil=25&min_overturning_SF=1.5&min_sliding_SF=1.5&consider_soil_above_foundation=false&consider_foundation_weight=false&elevation=X&footing_dims=true&show_loading=1&show_reinforcement=true&show_ground_level=true

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REFERENCES	CALCULATIONS	RESULTS

ACI 318-19 Spread Footing Design

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Dimensions

Symbol	Description	Value
-	Footing Name	FOOTING 1
-	Material	Reinforced Concrete
B	Footing Width	3.5
L	Footing Length	3.5
Df	Foundation Base Depth	4
H	Base Height	15
c_1	Column Length	16
c_2	Column Width	16
col_{height}	Column Height	4
x_{offset}	Column X Offset	0
z_{offset}	Column Z Offset	0
pof_{col}	Punching Overhang Factor	1

Concrete Properties

Symbol	Description	Value
f'_c	Concrete Compressive Strength	4
w_c	Concrete Unit Weight	150
d_{agg}	Max. Size of Aggregate	1

Reinforcement

Symbol	Description	Value
C_c	Reinforcement Clear Cover	3
f_y	Reinforcement Yield Strength	60
-	Reinforcement Type	Normal
$d_{b,x}$	Footing Bar Size (X Direction)	#4
n_x	Footing Bars in X Direction (n_x)	6
-	X Bars with Hooks?	True
$d_{b,z}$	Footing Bar Size (Z Direction)	#4
n_z	Footing Bars in Z Direction (n_z)	6
-	Z Bars with Hooks?	True
$d_{b,col}$	Dowel Bar Size	#4
$n_{x,col}$	Dowel Bars in X Direction	3
$n_{z,col}$	Dowel Bars in Z Direction	3
-	Dowels with Hooks?	False

Geotechnical Parameters

Symbol	Description	Value
q_a	Allowable Bearing Capacity	2
φ	Foundation-Soil Friction Angle	25
$SF_{overturning}$	Min. Overturning Safety Factor	1.5
$SF_{sliding}$	Min. Sliding Safety Factor	1.5

Load Cases

Name	Case	N (kip)	Mx (kip-ft)	Mz (kip-ft)	Vx (kip)	Vz (kip)
Total DL	DL	16.3	0	0	0	0
Total LL	LL	20.2	0	0	0	0
Total Lr	Lr	13.4	0	0	0	0
Total W/S	W/S	0	0	0	0	0

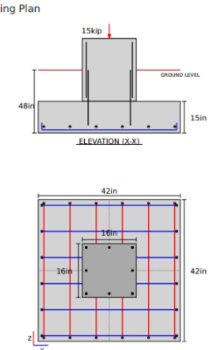
Name	Case	N (kip)	Mx (kip-ft)	Mz (kip-ft)	Vx (kip)	Vz (kip)
Dead	DL	15.2	0	0	0	0
Live	LL	20.2	0	0	0	0
RoofLive	Lr	13.4	0	0	0	0

Load Combinations

Name	Dead Load Factor	Live Load Factor	Roof Live Load Factor	Wind / Seismic Factor
S1	1.0	0.0	0.0	0.0
U1	1.2	1.6	1.0	0.0

REFERENCES	CALCULATIONS	RESULTS

Spread Footing Plan



NAME	LENGTH	WIDTH	HEIGHT	DEPTH	REINF X	REINF Z	COL
FOOTING 1	42	42	15	48	6#4	6#4	8#4

REFERENCES	CALCULATIONS	RESULTS

Footing Stability

Maximum Eccentricity Check

Critical Load Combination for Maximum Eccentricity on the Base: S1=1DL + 0LL + 0Lr + 0W/S

Eccentricity

$$\Sigma M_x = -(P \times offset_z) + M_x - (V_z \times D_f) = -(16.267 \times 0) + 0 - (0 \times 48) = 0 \text{ kip*in}$$

$$\Sigma M_z = -(-P \times offset_x) + M_z + (V_x \times D_f) = -(-16.267 \times 0) + 0 + (0 \times 48) = 0 \text{ kip*in}$$

$$\Sigma P = P + \Delta q \times A = 16.2667 + 0 \times 1764 = 16.2667 \text{ kip}$$

$$ratio_e = \frac{\left|\frac{\Sigma M_x}{\Sigma P}\right|}{\left(\frac{L}{6}\right)} + \frac{\left|\frac{\Sigma M_z}{\Sigma P}\right|}{\left(\frac{B}{6}\right)} = \frac{\left|\frac{0}{16.27}\right|}{\left(\frac{42}{6}\right)} + \frac{\left|\frac{0}{16.27}\right|}{\left(\frac{42}{6}\right)} = 0$$

Eccentricity Check

ratio_e <= 1.0
The resultant is within the middle third portion of the base. **PASS**

Soil Bearing Pressure Check

Critical Load Combination for Maximum Soil Stress: S1=1DL + 0LL + 0Lr + 0W/S

$$qs_{max} = \frac{\Sigma P}{(B \times L)} + \left(\frac{6 \times \Sigma M_z}{((B)^2 \times L)}\right) + \left(\frac{6 \times \Sigma M_x}{((L)^2 \times B)}\right) = \frac{16.2667}{(42 \times 42)} + \left(\frac{6 \times 0}{((42)^2 \times 42)}\right) + \left(\frac{6 \times 0}{((42)^2 \times 42)}\right)$$

$$qs_{max} = 0.0092 \text{ ksi}$$

Pedestal wWight $G_{pedestal} =$ 1.07kip

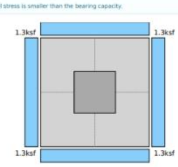
Soil Pressures and Eccentricities Summary

Name	ΣP (kip)	ΣM_x (kip-ft)	e_z (in)	ΣM_z (kip-ft)	e_x (in)	$ratio_e$	qs_{max} (ksf)	qs_{mean} (ksf)	$\frac{qs_{max}}{qs_{mean}}$	qs_{min} (ksf)
S1	16.27	0	0	0	0	0	1.33	1.33	1	1.33

Maximum Soil Stress	$qs_{max} =$	1.33ksf
Mean Soil Stress	$qs_{mean} =$	1.33ksf
Allowable Soil Bearing Capacity	$qa =$	2.00ksf

Soil Bearing Capacity Check

qa >= q_{max}
The maximum soil stress is smaller than the bearing capacity. **PASS**



REFERENCES	CALCULATIONS	RESULTS

Overturning Check

Overturning about X-Axis

Critical load combination for overturning: S1=1DL + 0LL + 0Lr + 0W/S

No moments about X-Axis were considered.

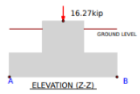
Overturning about Z-Axis

Critical load combination for overturning: S1=1DL + 0LL + 0Lr + 0W/S

No moments about Z-Axis were considered.

The footing weight was not considered.

The overburden soil weight was not considered.



Overturning Checks Summary

LC	Point	Mres_x (kip-ft)	Mover_x (kip-ft)	SF_x	Point	Mres_z (kip-ft)	Mover_z (kip-ft)	SF_z
S1	A	28.47	0	>1000	C	28.47	0	>1000

Overturning Safety Factor $SF_{overturning} =$ >1000.0

Mimimum Safety Factor $SF_{min} =$ 1.5

SF >= 1.5

The foundation is safe against overturning.

PASS

Sliding Check

Critical load combination for sliding: S1=1DL + 0LL + 0Lr + 0W/S

No shear forces were considered.

Sliding Safety Factor $SF_{sliding} =$ >1000.0

Mimimum Safety Factor $SF_{min} =$ 1.5

SF >= 1.5

PASS

The foundation is safe against sliding.

REFERENCES	CALCULATIONS	RESULTS

Footing Strength

$$d_x = H - c - \frac{db_x}{2} = 15 - 3 - \frac{0.5}{2} = 11.75 \text{ in}$$

$$d_z = H - c - db_x - \frac{db_z}{2} = 15 - 3 - 0.5 - \frac{0.5}{2} = 11.25 \text{ in}$$

$$\rho_x = \frac{n_x \times \left(\frac{\pi \times (db_x)^2}{4}\right)}{(B \times d_x)} \times 1000 = \frac{6 \times \left(\frac{3.14 \times (0.5)^2}{4}\right)}{(42 \times 11.75)} \times 1000 = 2.39 \ /1000$$

$$\rho_z = \frac{n_z \times \left(\frac{\pi \times (db_z)^2}{4}\right)}{(L \times d_z)} \times 1000 = \frac{6 \times \left(\frac{3.14 \times (0.5)^2}{4}\right)}{(42 \times 11.25)} \times 1000 = 2.49 \ /1000$$

One Way Shear Check in X

22.5.5.1

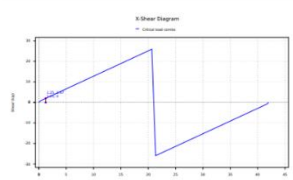
$$V_{cx} = 8 \times 0.1 \times (\rho_x)^{\frac{1}{3}} \times rf \times B \times d_x = 8 \times 0.1 \times (2.39)^{\frac{1}{3}} \times 63.25 \times 42 \times 11.75 = 33371.2 \text{ lb}$$

$$l_{l1} = \frac{(L - c_1)}{2} - d_x + offset_x = \frac{(42 - 16)}{2} - 11.75 + 0 = 1.25 \text{ in}$$

$$l_{l2} = \frac{(L - c_1)}{2} - d_x - offset_x = \frac{(42 - 16)}{2} - 11.75 - 0 = 1.25 \text{ in}$$

Maximum Shear X-Axis Summary

LC	dist 1 (in)	Shear 1 (kip)	dist 2 (in)	Shear 2 (kip)
U1	1.25	1.97	40.75	-1.97



Critical Load Combination: U1=1.2DL + 1.6LL + 1Lr + 0W/S

Maximum Shear that can be Resisted at the Section $V_{cx} =$ 33.37kip

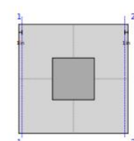
Shear Strength Reduction Factor $\phi =$ 0.75

Maximum Factored Shear Force at the Section $V_u =$ 1.97kip

$$Shear_{XRatio} = \frac{V_u}{\phi \times V_c} = \frac{1.97}{0.75 \times 33.37}$$

$$Shear_{XRatio} = 0.08$$

UTILITY: 0.08



One Way Shear Check in Z

22.5.5.1

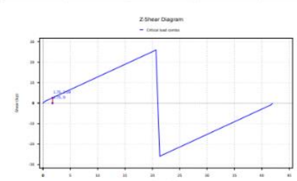
$$V_{cz} = 8 \times 0.1 \times (\rho_z)^{\frac{1}{3}} \times rf \times L \times d_z = 8 \times 0.1 \times (2.49)^{\frac{1}{3}} \times 63.25 \times 42 \times 11.25 = 32417.65 \text{ lb}$$

$$b_{l1} = \frac{(B - c_2)}{2} - d_z + offset_z = \frac{(42 - 16)}{2} - 11.25 + 0 = 1.75 \text{ in}$$

$$b_{l2} = \frac{(B - c_2)}{2} - d_z - offset_z = \frac{(42 - 16)}{2} - 11.25 - 0 = 1.75 \text{ in}$$

Max Shear Z-Axis Summary

LC	dist 1 (in)	Shear 1 (kip)	dist 2 (in)	Shear 2 (kip)
U1	1.75	2.59	40.25	-2.59



Critical Load Combination: U1=1.2DL + 1.6LL + 1Lr + 0W/S

Maximum Shear that can be Resisted at the Section $V_{cz} =$ 32.42kip

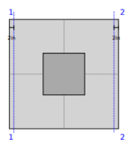
Shear Strength Reduction Factor $\phi =$ 0.75

Maximum Factored Shear Force at the Section $V_u =$ 2.59kip

$$Shear_{ZRatio} = \frac{V_u}{\phi \times V_c} = \frac{2.59}{0.75 \times 32.42}$$

$$Shear_{ZRatio} = 0.11$$

UTILITY: 0.11



Punching Shear Check

Ultimate Soil Pressure Summary

Name	ΣP (kip)	$ΣM_x$ (kip-ft)	e_z (in)	$ΣM_z$ (kip-ft)	e_x (in)	$ratio_e$	qs_{max} (ksf)	qs_{mean} (ksf)	$\frac{qs_{max}}{qs_{mean}}$	qs_{min} (ksf)
U1	51.84	0	0	0	0	0	4.23	4.23	1	4.23

Critical Load Combination: U1=1.2DL + 1.6LL + 1Lr + 0W/S

Maximum Soil Stress	$qu_{max} =$	4.23ksf
Mean Soil Stress	$qu_{mean} =$	4.23ksf

$$d_p = \frac{(d_x + d_z)}{2} = \frac{(11.75 + 11.25)}{2} = 11.5 \text{ in}$$

Ratio of Long to Short Dimensions (Column)	$\beta =$	1.00
Modification Factor for Lightweight Concrete	$\lambda =$	1.00
Perimeter of Critical Section	$bo =$	110.00in
X-Axis Distance	$b1 =$	27.50in
Z-Axis Distance	$b2 =$	27.50in
Equivalent Polar Moment of Inertia	$Jc_x =$	166413in^4
Equivalent Polar Moment of Inertia	$Jc_z =$	166413in^4
Factor for the Moment Fraction	$\gamma_{vz} =$	0.40
Factor for the Moment Fraction	$\gamma_{vx} =$	0.40
X-axis Largest Distance to the Extreme Fiber	$c_x =$	13.75in
Z-axis Largest Distance to the Extreme Fiber	$c_z =$	13.75in

$$v_c = min(4, 2 + \frac{4}{\beta}, 2 + \frac{\alpha_s \times d_p}{bo}) \times \lambda \times rf$$

$$v_c = min(4, 2 + \frac{4}{1}, 2 + \frac{40 \times 11.5}{110.00}) \times 1.00 \times 63.25$$

$$v_c = 252.98psi$$

22.6.5.2

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$$v_{un} = \frac{qu_m \times (B \times L - (d_1 \times d_2))}{(bo \times d_p)} = \frac{29.39 \times (42 \times 42 - (27.5 \times 27.5))}{(110 \times 11.5)} = 23.41 \text{ psi}$$

22.6.5.2 $$v_{umx} = \frac{\gamma_{vx} \times |Mu_z| \times cx}{Jc_x} = \frac{0.4 \times |0| \times 13.75}{166413.39} = 0 \text{ psi}$$

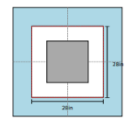
22.6.5.2 $$v_{umz} = \frac{\gamma_{vz} \times |Mu_x| \times cz}{Jc_z} = \frac{0.4 \times |0| \times 13.75}{166413.39} = 0 \text{ psi}$$

22.6.5.2 $$v_u = v_{un} + v_{umx} + v_{umz} = 23.41 + 0 + 0 = 23.41 \text{ psi}$$

$$Punching_{Ratio} = \frac{v_u}{\phi \times v_c} = \frac{23.41}{0.75 \times 252.98}$$

$$Punching_{Ratio} = 0.12$$

UTILITY: 0.12



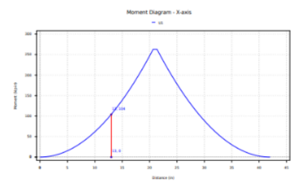
Bending Check (X)

13.2.7.1 $$l_{21} = \frac{(L - c_1)}{2} + offset_x = \frac{(42 - 16)}{2} + 0 = 13 \text{ in}$$

13.2.7.1 $$l_{22} = \frac{(L - c_1)}{2} - offset_x = \frac{(42 - 16)}{2} - 0 = 13 \text{ in}$$

Max Moment X-Axis Summary

LC	dist 1 (in)	Moment 1 (kip-ft)	dist 2 (in)	Moment 2 (kip-ft)
U1	13	8.69	29	8.69



Critical Load Combination: U1=1.2DL + 1.6LL + 1Lr + 0W/S

Maximum Moment at the Critical Section	$M_{ux} =$	8.69kip-ft
Depth of Equivalent Rectangular Compressive Stress Block	$\beta_1 =$	0.85

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Modulus of Elasticity of Reinforcement	$E_s =$	29000.00ksi
Bending Strength Reduction Factor (Tension Controlled)	$\phi =$	0.90

Calculation of the Strength Coefficient

Eq. 4.10 CRSI Manual $$R_{nx} = \frac{M_{ux}}{\left(\varphi \times B \times (d_x)^2\right)} = \frac{104241.6}{\left(0.9 \times 42 \times (11.75)^2\right)} = 19.974 \text{ psi}$$

Reinforcement Area Check (X)

Eq. 4.9 CRSI Manual $$As_{reqx} = \frac{0.85 \times f_c \times B \times d_x}{f_y} \times \left(1 - \sqrt{1 - \frac{2 \times R_{nx}}{(0.85 \times f_c)}}\right) = \frac{0.85 \times 4000 \times 42 \times 11.75}{60000} \times \left(1 - \sqrt{1 - \frac{2 \times 19.97}{(0.85 \times 4000)}}\right)$$

$$As_{reqx} = 0.16 \text{ in}^2$$

7.6.1.1 $$As_{minx} = 0.0018 \times H \times B = 0.0018 \times 15 \times 42$$

$$As_{minx} = 1.13 \text{ in}^2$$

Eq. 4.9 CRSI Manual $$As_{maxx} = \frac{0.85 \times \beta_1 \times f_c \times B \times d_x}{f_y} \times \left(\frac{0.003}{\left(\left(\frac{f_y}{E_s}\right) + 0.006\right)}\right) = \frac{0.85 \times 0.85 \times 4000 \times 42 \times 11.75}{60000} \times \left(\frac{0.003}{\left(\left(\frac{60000}{29000000}\right) + 0.006\right)}\right)$$

$$As_{maxx} = 8.84 \text{ in}^2$$

$$As_{provx} = \frac{n_x \times \pi \times (db_x)^2}{4} = \frac{6 \times 3.14 \times (0.5)^2}{4}$$

$$As_{provx} = 1.18 \text{ in}^2$$

Reinforcement Area (X)

As_{provx} = 1.178 > As_{reqx} = 0.165
Provided reinforcement area > required reinforcement area.

PASS

Minimum Reinforcement Area (X)

As_{provx} = 1.178 > As_{minx} = 1.134
Provided reinforcement area > minimum reinforcement area.

PASS

Maximum Reinforcement Area (X)

As_{provx} = 1.178 < As_{maxx} = 8.838
Provided reinforcement area < maximum reinforcement area due to ductility.

PASS

Calculation of the Maximum Resisting Moment (X)

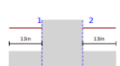
Eq. 4.3 CRSI Manual $$M_{nx} = As_{provx} \times f_y \times \left(d_x - \frac{(As_{provx} \times f_y)}{(1.7 \times B \times f_c)}\right) = 1.18 \times 60000 \times \left(11.75 - \frac{(1.18 \times 60000)}{(1.7 \times 42 \times 4000)}\right) = 813063.85 \text{ lb-in}$$

Nominal Flexural Strength	$M_{nx} =$	67.76kip-ft

$$Bending_{XRatio} = \frac{M_{ux}}{\phi \times M_{nx}} = \frac{104.24}{0.9 \times 813.06}$$

$$Bending_{XRatio} = 0.14$$

UTILITY: 0.14



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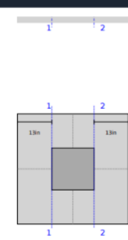

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REFERENCES	CALCULATIONS	RESULTS

Bending Check Z-Axis

13.2.7.1

$$b_{21} = \frac{(B - c_2)}{2} + offset_z = \frac{(42 - 16)}{2} + 0 = 13\,\text{in}$$

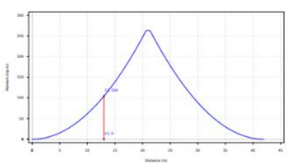
13.2.7.1

$$b_{22} = \frac{(B - c_2)}{2} - offset_z = \frac{(42 - 16)}{2} - 0 = 13\,\text{in}$$

Max Moment Z-Axis Summary

LC	dist 1 (in)	Moment 1 (kip-ft)	dist 2 (in)	Moment 2 (kip-ft)
U1	13	8.69	29	8.69

Moment Diagram - (Z)



Critical Load Combination: U1=1.2DL + 1.6LL + 1Lr + 0W/S

Maximum Moment at the Critical Section	$M_{uz} =$	8.69kip-ft
Depth of Equivalent Rectangular Compressive Stress Block	$\beta_1 =$	0.85
Modulus of Elasticity of Reinforcement	$E_s =$	29000.00ksi
Bending Strength Reduction Factor (Tension Controlled)	$\phi =$	0.90
Ratio of Long to Short Footing Dimensions	$\beta =$	1.00

Calculation of the Strength Coefficient

Eq. 4.10 CRSI Manual

$$R_{nz} = \frac{M_{uz}}{\left(\varphi \times L \times (d_z)^2\right)} = \frac{104241.6}{\left(0.9 \times 42 \times (11.25)^2\right)} = 21.789\ \text{psi}$$

Reinforcement Area Check (Z)

Eq. 4.9 CRSI Manual

$$As_{reqz} = \frac{0.85 \times f_c \times L \times d_z}{f_y} \times \left(1 - \sqrt{1 - \frac{2 \times R_{nz}}{(0.85 \times f_c)}}\right) = \frac{0.85 \times 4000 \times 42 \times 11.25}{60000} \times \left(1 - \sqrt{1 - \frac{2 \times 21.79}{(0.85 \times 4000)}}\right)$$

$$As_{reqz} = 0.17\ \text{in}^2$$

Reinforcement Distribution Factor

R13.3.3.3

$$\gamma_s = \frac{2 \times \beta}{(\beta + 1)} = \frac{2 \times 1}{(1 + 1)}$$

$$\gamma_s = 1$$

R13.3.3.3

$$As_{reqzs} = As_{reqz} \times \gamma_s = 0.17 \times 1$$

$$As_{reqzs} = 0.17\ \text{in}^2$$

7.6.1.1

$$As_{minz} = 0.0018 \times H \times L = 0.0018 \times 15 \times 42$$

$$As_{minz} = 1.13\ \text{in}^2$$

Eq. 4.9 CRSI Manual

$$As_{maxz} = \frac{0.85 \times \beta_1 \times f_c \times L \times d_z}{f_y} \times \left(\frac{0.003}{\left(\left(\frac{f_y}{E_s}\right) + 0.006\right)}\right) = \frac{0.85 \times 0.85 \times 4000 \times 42 \times 11.25}{60000} \times \left(\frac{0.003}{\left(\left(\frac{60000}{29000000}\right) + 0.006\right)}\right)$$

$$As_{maxz} = 8.46\ \text{in}^2$$

$$As_{provz} = \frac{n_z \times \pi \times (db_z)^2}{4} = \frac{6 \times 3.14 \times (0.5)^2}{4}$$

$$As_{provz} = 1.18\ \text{in}^2$$

Reinforcement Area (Z)

As_{provz} = 1.178 > As_{reqx} = 0.172
Provided reinforcement area > required reinforcement area. — **PASS**

Minimum Reinforcement Area (Z)

As_{provz} = 1.178 > As_{minz} = 1.134
Provided reinforcement area > minimum reinforcement area. — **PASS**

Maximum Reinforcement Area (Z)

As_{provz} = 1.178 < As_{maxx} = 8.462
Provided reinforcement area < maximum reinforcement area due to ductility. — **PASS**

Calculation of the Maximum Resisting Moment (Z). It considers the Reinforcement Distribution Factor.

Eq. 4.3 CRSI Manual

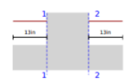
$$M_{nz} = \frac{As_{provz} \times f_y \times \left(d_z - \frac{(As_{provz} \times f_y)}{(1.7 \times L \times f_c \times \gamma_s)}\right)}{\gamma_s} = \frac{1.18 \times 60000 \times \left(11.25 - \frac{(1.18 \times 60000)}{(1.7 \times 42 \times 4000 \times 1)}\right)}{1} = 777720.94\ \text{lb-in}$$

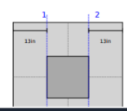
Nominal Flexural Strength	$M_{nz} =$	64.81kip-ft

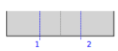
$$Bending_{ZRatio} = \frac{M_{uz}}{\phi \times M_{nz}} = \frac{104.24}{0.9 \times 777.72}$$

$$Bending_{ZRatio} = 0.15$$

UTILITY: 0.15







REFERENCES	CALCULATIONS	RESULTS

Transfer of Forces

Vertical Transfer - Compression Check

22.8.3.1 Bearing Strength Calculation

$$A_1 = c_1 \times c_2 = 16 \times 16 = 256\ \text{in2}$$

$$A_2 = (min(l_{21}, 2 \times H) + c_1 + min(l_{22}, 2 \times H)) \times (min(b_{21}, 2 \times H) + (c_2 + min(b_{22}, 2 \times H))$$

$$A_2 = (min(13.00, 2 \times 15.00) + (16.00) + min(13.00, 2 \times 15.00)) \times (min(13.00, 2 \times 15.00) + (16.00) + min(13.00, 2 \times 15.00))$$

$$A_2 = 1764.00 in2$$

Table 22.8.3.2

$$B_n = min\left(\sqrt{\frac{A_2}{A_1}}, 2\right) \times 0.85 \times f_c \times A_1$$

$$B_n = min\left(\sqrt{\frac{1764.00}{256.00}}, 2.0\right) \times 0.85 \times 4000 \times 256.00$$

$$B_n = 1740800\ \text{lb}$$

Critical Load Combination for Bearing: U1=1.2DL + 1.6LL + 1Lr + 0W/S

Factored Bearing Load	$B_u =$	51.84kip
Bearing Strength Reduction Factor (Table 21.2.1.2)	$\phi =$	0.65
Reduced Bearing Strength	$\phi Bn =$	1131.52kip

$$Bearing_{Ratio} = \frac{B_u}{\phi \times B_n} = \frac{51.84}{0.65 \times 1740.80}$$

$$Bearing_{Ratio} = 0.05$$

UTILITY: 0.05

16.3.4.1

$$As_{mindowels} = 0.005 \times A_1 = 0.005 \times 256 = 1.28\ \text{in2}$$

$$As_{dowels} = \frac{\pi \times (db_{col})^2}{4} \times n = \frac{3.14 \times (0.5)^2}{4} \times 8 = 1.57\ \text{in2}$$

Asd_{prov} = 1.57 > $Asd_{min/req}$ = 1.28
Provided dowels reinforcement area > minimum/required reinforcement area. — **PASS**

Footing Details

Reinforcement Spacing Check

7.7.2.3

$$s_{max1} = min(3 * H, 18.00)$$

$$s_{max1} = min(3 * 15, 18.00)$$

$$s_{max1} = 18\ \text{in}$$

24.3

$$s_{max2} = min(15.0 * 40000/f_s - (2.5 * c_c), 12 * 40000/f_s)$$

$$s_{max2} = min(15.0 * 40000/40000 - (2.5 * 3), 12 * 40000/40000)$$

$$s_{max2} = 7.5\ \text{in}$$

Spacing for the Reinforcement along X-Axis

25.2

$$s_{minx} = max\,(1.0, db_x, (4 \times d_{agg})/3.0)) + db_x$$

$$s_{min} = max\,(1.0, 0.5, 4 \times 1.00/3.0)) + 0.5$$

$$s_{min} = 1.83 in$$

$$s_x = \frac{(B - 2 \times c - db_x)}{(n_x - 1)} = \frac{(42 - 2 \times 3 - 0.5)}{(6 - 1)}$$

$$s_x = 7.1 \text{ in}$$

s_{max} = 7.5 > s_x = 7.1 > s_{min} = 1.833
Provided spacing is O.K.

PASS

Spacing for the Reinforcement along Z-Zxis

25.2

$$s_{minz} = max\left(1.0, db_z, (4 \times d_{agg})/3.0\right) + db_z$$
$$s_{min} = max\left(1.0, 0.5, 4 \times 1.00/3.0\right) + 0.5$$
$$s_{min} = 1.83 in$$

$$s_z = \frac{(L - 2 \times c - db_z)}{(n_z - 1)} = \frac{(42 - 2 \times 3 - 0.5)}{(6 - 1)}$$

$$s_z = 7.1 \text{ in}$$

s_{max} = 7.5 > s_z = 7.1 > s_{min} = 1.833
Provided spacing is O.K.

PASS

REFERENCES	CALCULATIONS	RESULTS

Development Length Check (X)

Distance to Nearest Concrete Surface or Half Spacing	$cb_x =$	3.25
Root of Concrete Strength Value min(√f'c, 100psi)	$rf =$	63.25
Epoxi Modification Factor	$\psi_e h =$	1.00
Confining Reinforcement Factor	$\psi_r =$	1.60
Location Factor	$\psi_o =$	1.00
Concrete Strength Factor	$\psi_c =$	0.87
Lightweight Concrete Factor	$\lambda =$	1.00

25.4.3.1

$$ldhx = \frac{(f_y \times \psi_e \times \psi_r \times \psi_o \times \psi_c)}{(55 \times \lambda \times rf)} \times (db_x)^{1.5} = \frac{(60000 \times 1 \times 1.6 \times 1 \times 0.87)}{(55 \times 1 \times 63.25)} \times (0.5)^{1.5}$$

$$ldhx = 8.46 \text{ in}$$

25.4.3.1

$$ld_{minx} = max(8 * db_x, 6.0)$$

$$ld_{minx} = max(8 * 0.5, 6.0)$$

$$ld_{minx} = 6 \text{ in}$$

10.21 CRSI Manual

$$Distance_{avail} = min(l_{21}, l_{22}) - c$$

$$Distance_{avail} = min(13, 13) - 3$$

$$Distance_{avail} = 10 \text{ in}$$

Development Length (X) $ld_{calc} =$ 8.46

ld_{calcx} = 8.46 < $Distance_{avail}$ = 10
Available length > minimum development length.

PASS

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REFERENCES	CALCULATIONS	RESULTS

Development Length Check (Z)

Distance to Nearest Concrete Surface or Half Spacing	$cb_z =$	3.25
Root of Concrete Strength Value min(√f'c, 100psi)	$rf =$	63.25
Epoxi Modification Factor	$\psi_e h =$	1.00
Confining Reinforcement Factor	$\psi_r =$	1.60
Location Factor	$\psi_o =$	1.00
Concrete Strength Factor	$\psi_c =$	0.87
Lightweight Concrete Factor	$\lambda =$	1.00

25.4.3.1

$$ldhz = \frac{(f_y \times \psi_e \times \psi_r \times \psi_o \times \psi_c)}{(55 \times \lambda \times rf)} \times (db_z)^{1.5} = \frac{(60000 \times 1 \times 1.6 \times 1 \times 0.87)}{(55 \times 1 \times 63.25)} \times (0.5)^{1.5}$$

$$ldhz = 8.46 \text{ in}$$

25.4.3.1

$$ld_{minz} = max(8 * db_z, 6.0)$$

$$ld_{minz} = max(8 * 0.5, 6.0)$$

$$ld_{minz} = 6 \text{ in}$$

10.21 CRSI Manual

$$Distance_{avail} = min(b_{21}, b_{22}) - c$$

$$Distance_{avail} = min(13, 13) - 3$$

$$Distance_{avail} = 10 \text{ in}$$

Development Length (Z) $ld_{calc} =$ 8.46

ld_{calcz} = 8.46 < $Distance_{avail}$ = 10
Available length > minimum development length.

PASS

Minimum Footing Height Check

13.3.1.2

$$H_{min} = 6 + db_i + \frac{db_z}{2} + c = 6 + 0.5 + \frac{0.5}{2} + 3 = 9.75 \text{ in}$$

Column Dowels Compression Development Length

25.4.9.2

$$l_d = max\left(0.0003 \times f_y \times \psi_r \times db_{col}, \frac{\psi_r \times db_{col} \times f_y}{50 \times \lambda \times rf}\right)$$
$$l_d = max\left(0.0003 \times 60000 \times 1 \times 0.5, \frac{1 \times 0.5 \times 60000}{50 \times 1 \times 63.25}\right)$$
$$l_d = 9.49 \text{ in}$$

Min. Dowels Development Length (25.4.9.1) $l_{dmin} =$ 9.75in

25.4.9.2

$$H_{dowels} = l_d + \frac{hook_{dim}}{2} + db_{col} + c = 9.49 + \frac{0}{2} + 0.5 + 3 = 12.99 \text{ in}$$

Footing Base Height $H =$ 15.00in

H = 15 > H_{min} = 12.99
Footing height is OK.

PASS

Results Summary

Result Name	Results
STABILITY CHECKS	
Eccentricity Ratio	PASS
Bearing Capacity	PASS
Overturning	PASS
Sliding	PASS
STRENGTH UTILIZATION	

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Result Name	Results
Shear (X)	0.08
Shear (Z)	0.11
Punching Shear	0.12
Bending Capacity (X)	0.14
Bending Capacity (Z)	0.15
Bearing Capacity	0.05
DETAILING	
Min/Max Reinforcement (X)	PASS
Mix/Max Reinforcement (Z)	PASS
Dowels Reinforcement	PASS
Bar Spacing (X)	PASS
Bar Spacing (Z)	PASS
Development Length (X)	PASS
Development Length (Z)	PASS
Footing Height Check	PASS

About this Calculator

Calculator Name: ACI 318-19 Spread Footing Design
Description: This calculator is in imperial units and it is used for the design of reinforced concrete spread footings. It also considers column eccentricities on both axes.

Powered by SkyCiv

URL: https://platform.skyciv.com/quick-design?uid=8002-aci-pad-footing-design-calculator&footing_name=FOOTING%201&footing_material=rcfooting&footing_width=3.5&footing_length=3.5&footing_depth=4&footing_height=15&col_c1=16&col_c2=16&col_height=4&col_x=0&col_z=0&punching_overhang_factor=1&fc=4&unit_weight=150&max_aggregate_size=1&cover=3&f_y=60&reinforcement_type=normal&bar_size_x=4&n_x=6&x_bars_with_hooks=true&bar_size_z=4&n_z=6&z_bars_with_hooks=true&col_bar_size=4&col_nx_bar=3&col_nz_bar=3&dowels_with_hooks=false&q_a=2&phi_footing_soil=25&min_overturning_SF=1.5&min_sliding_SF=1.5&consider_soil_above_foundation=false&consider_foundation_weight=false&elevation=X&footing_dims=true&show_loading=1&show_reinforcement=true&show_ground_level=true

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REFERENCES | CALCULATIONS | RESULTS

ACI 318-19 Spread Footing Design

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Dimensions

Symbol	Description	Value
-	Footing Name	FOOTING 1
-	Material	Reinforced Concrete
B	Footing Width	5
L	Footing Length	5
Df	Foundation Base Depth	4
H	Base Height	15
c_1	Column Length	14
c_2	Column Width	26
col_{height}	Column Height	4
x_{offset}	Column X Offset	0
z_{Offset}	Column Z Offset	0
pof_{ol}	Punching Overhang Factor	1

Concrete Properties

Symbol	Description	Value
f'_c	Concrete Compressive Strength	4
w_c	Concrete Unit Weight	150
d_{agg}	Max. Size of Aggregate	1

Reinforcement

Symbol	Description	Value
C_r	Reinforcement Clear Cover	3
f_y	Reinforcement Yield Strength	60
-	Reinforcement Type	Normal
$d_{b,x}$	Footing Bar Size (X Direction)	#4
n_x	Footing Bars in X Direction (n_x)	10
-	X Bars with Hooks?	True
$d_{b,z}$	Footing Bar Size (Z Direction)	#4
n_z	Footing Bars in Z Direction (n_z)	10
-	Z Bars with Hooks?	True
$d_{b,col}$	Dowel Bar Size	#4
$n_{x,col}$	Dowel Bars in X Direction	3
$n_{z,col}$	Dowel Bars in Z Direction	4
-	Dowels with Hooks?	False

Geotechnical Parameters

Symbol	Description	Value
q_a	Allowable Bearing Capacity	2
φ	Foundation-Soil Friction Angle	25
$SF_{overturning}$	Min. Overturning Safety Factor	1.5
$SF_{sliding}$	Min. Sliding Safety Factor	1.5

Load Cases

Name	Case	N (kip)	Mx (kip-ft)	Mz (kip-ft)	Vx (kip)	Vz (kip)
Total DL	DL	22.1	0	0	0	0
Total LL	LL	23.9	0	0	0	0
Total Lr	Lr	8.1	0	0	0	0
Total W/S	W/S	0	0	0	0	0

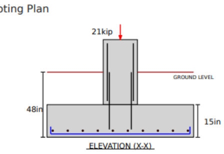
Name	Case	N (kip)	Mx (kip-ft)	Mz (kip-ft)	Vx (kip)	Vz (kip)
Dead	DL	20.6	0	0	0	0
Live	LL	23.9	0	0	0	0
RoofLive	Lr	8.1	0	0	0	0

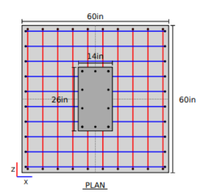
Load Combinations

Name	Dead Load Factor	Live Load Factor	Roof Live Load Factor	Wind / Seismic Factor
S1	1.0	0.0	0.0	0.0
U1	1.2	1.6	1.0	0.0

REFERENCES | CALCULATIONS | RESULTS

Spread Footing Plan





NAME	LENGTH	WIDTH	HEIGHT	DEPTH	REINF X	REINF Z	COL
FOOTING 1	60	60	15	48	10#4	10#4	10#4

REFERENCES | CALCULATIONS | RESULTS

Footing Stability

Maximum Eccentricity Check

Critical Load Combination for Maximum Eccentricity on the Base: S1=1DL + 0LL + 0Lr + 0W/S

Eccentricity

$$\Sigma M_x = -(P \times offset_z) + M_x - (V_z \times D_f) = -(22.116 \times 0) + 0 - (0 \times 48) = 0\,\text{kip*in}$$

$$\Sigma M_z = -(-P \times offset_x) + M_z + (V_x \times D_f) = -(-22.116 \times 0) + 0 + (0 \times 48) = 0\,\text{kip*in}$$

$$\Sigma P = P + \Delta q \times A = 22.1157 + 0 \times 3600 = 22.1157\,\text{kip}$$

$$ratio_e = \frac{\left|\frac{\Sigma M_x}{\Sigma P}\right|}{\left(\frac{L}{6}\right)} + \frac{\left|\frac{\Sigma M_z}{\Sigma P}\right|}{\left(\frac{B}{6}\right)} = \frac{\left|\frac{0}{22.12}\right|}{\left(\frac{60}{6}\right)} + \frac{\left|\frac{0}{22.12}\right|}{\left(\frac{60}{6}\right)} = 0$$

Eccentricity Check

ratio_e <= 1.0
The resultant is within the middle third portion of the base.

PASS

Soil Bearing Pressure Check

Critical Load Combination for Maximum Soil Stress: S1=1DL + 0LL + 0Lr + 0W/S

$$qs_{max} = \frac{\Sigma P}{(B \times L)} + \left(\frac{6 \times \Sigma M_x}{((B)^2 \times L)}\right) + \left(\frac{6 \times \Sigma M_z}{((L)^2 \times B)}\right) = \frac{22.1157}{(60 \times 60)} + \left(\frac{6 \times 0}{((60)^2 \times 60)}\right) + \left(\frac{6 \times 0}{((60)^2 \times 60)}\right)$$

$$qs_{max} = 0.0061\,\text{ksi}$$

Pedestal wWight $G_{pedestal} =$ 1.52kip

Soil Pressures and Eccentricities Summary

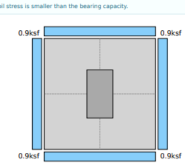
Name	ΣP (kip)	ΣM_x (kip-ft)	e_z (in)	ΣM_z (kip-ft)	e_x (in)	$ratio_e$	qs_{max} (ksf)	qs_{mean} (ksf)	$\frac{qs_{max}}{qs_{mean}}$	qs_{min} (ksf)
S1	22.12	0	0	0	0	0	0.88	0.88	1	0.88

Maximum Soil Stress $qs_{max} =$ 0.88ksf
Mean Soil Stress $qs_{mean} =$ 0.88ksf
Allowable Soil Bearing Capacity $qa =$ 2.00ksf

Soil Bearing Capacity Check

qa >= q_{max}
The maximum soil stress is smaller than the bearing capacity.

PASS



REFERENCES	CALCULATIONS	RESULTS

Overturning Check

Overturning about X-Axis

Critical load combination for overturning: S1=1DL + 0LL + 0Lr + 0W/S

No moments about X-Axis were considered.

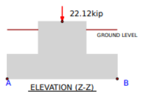
Overturning about Z-Axis

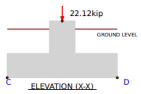
Critical load combination for overturning: S1=1DL + 0LL + 0Lr + 0W/S

No moments about Z-Axis were considered.

The footing weight was not considered.

The overburden soil weight was not considered.





Overturning Checks Summary

LC	Point	Mres_x (kip-ft)	Mover_x (kip-ft)	SF_x	Point	Mres_z (kip-ft)	Mover_z (kip-ft)	SF_z
S1	A	55.29	0	>1000	C	55.29	0	>1000

Overturning Safety Factor $SF_{overturning} =$ >1000.0

Mimimum Safety Factor $SF_{min} =$ 1.5

SF >= 1.5

The foundation is safe against overturning.

PASS

Sliding Check

Critical load combination for sliding: S1=1DL + 0LL + 0Lr + 0W/S

No shear forces were considered.

Sliding Safety Factor $SF_{sliding} =$ >1000.0

Mimimum Safety Factor $SF_{min} =$ 1.5

SF >= 1.5

PASS

The foundation is safe against sliding.

REFERENCES	CALCULATIONS	RESULTS

Footing Strength

$$d_x = H - c - \frac{db_x}{2} = 15 - 3 - \frac{0.5}{2} = 11.75 \text{ in}$$

$$d_z = H - c - db_x - \frac{db_z}{2} = 15 - 3 - 0.5 - \frac{0.5}{2} = 11.25 \text{ in}$$

$$\rho_x = \frac{n_x \times \left(\frac{\pi \times (db_x)^2}{4}\right)}{(B \times d_x)} \times 1000 = \frac{10 \times \left(\frac{3.14 \times (0.5)^2}{4}\right)}{(60 \times 11.75)} \times 1000 = 2.79\ /1000$$

$$\rho_z = \frac{n_z \times \left(\frac{\pi \times (db_z)^2}{4}\right)}{(L \times d_z)} \times 1000 = \frac{10 \times \left(\frac{3.14 \times (0.5)^2}{4}\right)}{(60 \times 11.25)} \times 1000 = 2.91\ /1000$$

One Way Shear Check in X

22.5.5.1

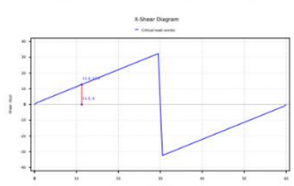
$$V_{cx} = 8 \times 0.1 \times (\rho_x)^{\frac{1}{3}} \times rf \times B \times d_x = 8 \times 0.1 \times (2.79)^{\frac{1}{3}} \times 63.25 \times 60 \times 11.75 = 50186.78 \text{ lb}$$

$$l_{11} = \frac{(L - c_1)}{2} - d_x + offset_x = \frac{(60 - 14)}{2} - 11.75 + 0 = 11.25 \text{ in}$$

$$l_{12} = \frac{(L - c_1)}{2} - d_x - offset_x = \frac{(60 - 14)}{2} - 11.75 - 0 = 11.25 \text{ in}$$

Maximum Shear X-Axis Summary

LC	dist 1 (in)	Shear 1 (kip)	dist 2 (in)	Shear 2 (kip)
U1	11.25	12.69	48.75	-12.69



Critical Load Combination: U1=1.2DL + 1.6LL + 1Lr + 0W/S

Maximum Shear that can be Resisted at the Section $V_{cx} =$ 50.19kip

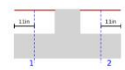
Shear Strength Reduction Factor $\phi =$ 0.75

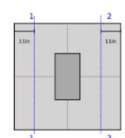
Maximum Factored Shear Force at the Section $V_u =$ 12.69kip

$$Shear_{XRatio} = \frac{V_u}{\phi \times V_c} = \frac{12.69}{0.75 \times 50.19}$$

$$Shear_{XRatio} = 0.34$$

UTILITY: 0.34





One Way Shear Check in Z

22.5.5.1

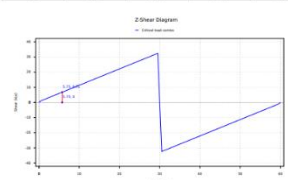
$$V_{cz} = 8 \times 0.1 \times (\rho_z)^{\frac{1}{3}} \times rf \times L \times d_z = 8 \times 0.1 \times (2.91)^{\frac{1}{3}} \times 63.25 \times 60 \times 11.25 = 48752.75 \text{ lb}$$

$$b_{11} = \frac{(B - c_2)}{2} - d_z + offset_z = \frac{(60 - 26)}{2} - 11.25 + 0 = 5.75 \text{ in}$$

$$b_{12} = \frac{(B - c_2)}{2} - d_z - offset_z = \frac{(60 - 26)}{2} - 11.25 - 0 = 5.75 \text{ in}$$

Max Shear Z-Axis Summary

LC	dist 1 (in)	Shear 1 (kip)	dist 2 (in)	Shear 2 (kip)
U1	5.75	6.75	54.25	-6.75



Critical Load Combination: U1=1.2DL + 1.6LL + 1Lr + 0W/S

Maximum Shear that can be Resisted at the Section $V_{cz} =$ 48.75kip

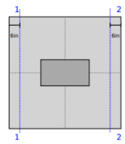
Shear Strength Reduction Factor $\phi =$ 0.75

Maximum Factored Shear Force at the Section $V_u =$ 6.75kip

$$Shear_{ZRatio} = \frac{V_u}{\phi \times V_c} = \frac{6.75}{0.75 \times 48.75}$$

$$Shear_{ZRatio} = 0.18$$

UTILITY: 0.18



Punching Shear Check

Ultimate Soil Pressure Summary

Name	ΣP (kip)	$ΣM_x$ (kip-ft)	e_z (in)	$ΣM_z$ (kip-ft)	e_x (in)	$ratio_e$	qs_{max} (ksf)	qs_{mean} (ksf)	qs_{max}/qs_{mean}	qs_{min} (ksf)
U1	64.79	0	0	0	0	0	2.59	2.59	1	2.59

Critical Load Combination: U1=1.2DL + 1.6LL + 1Lr + 0W/S

Maximum Soil Stress	$qu_{max} =$	2.59ksf
Mean Soil Stress	$qu_{mean} =$	2.59ksf

$$d_p = \frac{(d_x + d_z)}{2} = \frac{(11.75 + 11.25)}{2} = 11.5 \text{ in}$$

Ratio of Long to Short Dimensions (Column)	$\beta =$	1.00
Modification Factor for Lightweight Concrete	$\lambda =$	1.00
Perimeter of Critical Section	$bo =$	126.00in
X-Axis Distance	$b1 =$	25.50in
Z-Axis Distance	$b2 =$	37.50in
Equivalent Polar Moment of Inertia	$Jc_x =$	178455in^4
Equivalent Polar Moment of Inertia	$Jc_z =$	316771in^4
Factor for the Moment Fraction	$\gamma_{vz} =$	0.35
Factor for the Moment Fraction	$\gamma_{vx} =$	0.45
X-axis Largest Distance to the Extreme Fiber	$c_x =$	12.75in
Z-axis Largest Distance to the Extreme Fiber	$c_z =$	18.75in

$$v_c = min(4, 2 + \frac{4}{\beta}, 2 + \frac{\alpha_s \times d_p}{bo}) \times \lambda \times rf$$

$$v_c = min(4, 2 + \frac{4}{1}, 2 + \frac{40 \times 11.5}{126.00}) \times 1.00 \times 63.25$$

$$v_c = 252.98 psi$$

22.6.5.2

$$v_{un} = \frac{qu_m \times (B \times L - (d_1 \times d_2))}{(bo \times d_p)} = \frac{18 \times (60 \times 60 - (25.5 \times 37.5))}{(126 \times 11.5)} = 32.84 \text{ psi}$$

22.6.5.2

$$v_{umx} = \frac{\gamma_{vz} \times |Mu_x| \times cx}{Jc_x} = \frac{0.35 \times |0| \times 12.75}{178454.84} = 0 \text{ psi}$$

22.6.5.2

$$v_{umz} = \frac{\gamma_{vx} \times |Mu_z| \times cz}{Jc_z} = \frac{0.45 \times |0| \times 18.75}{316771.09} = 0 \text{ psi}$$

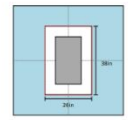
22.6.5.2

$$v_u = v_{un} + v_{umx} + v_{umz} = 32.84 + 0 + 0 = 32.84 \text{ psi}$$

$$Punching_{Ratio} = \frac{v_u}{\phi \times v_c} = \frac{32.84}{0.75 \times 252.98}$$

$$Punching_{Ratio} = 0.17$$

UTILITY: 0.17



Bending Check (X)

13.2.7.1

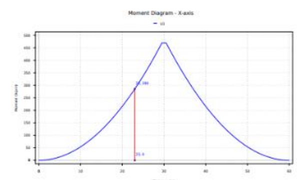
$$l_{x1} = \frac{(L - c_1)}{2} + offset_x = \frac{(60 - 14)}{2} + 0 = 23 \text{ in}$$

13.2.7.1

$$l_{x2} = \frac{(L - c_1)}{2} - offset_x = \frac{(60 - 14)}{2} - 0 = 23 \text{ in}$$

Max Moment X-Axis Summary

LC	dist 1 (in)	Moment 1 (kip-ft)	dist 2 (in)	Moment 2 (kip-ft)
U1	23	23.8	37	23.8



Critical Load Combination: U1=1.2DL + 1.6LL + 1Lr + 0W/S

Maximum Moment at the Critical Section	$M_{ux} =$	23.80kip-ft

Depth of Equivalent Rectangular Compressive Stress Block	$\beta_1 =$	0.85
Modulus of Elasticity of Reinforcement	$E_s =$	29000.00ksi
Bending Strength Reduction Factor (Tension Controlled)	$\phi =$	0.90

Calculation of the Strength Coefficient

Eq. 4.10 CRSI Manual

$$R_{nx} = \frac{M_{ux}}{\left(\varphi \times B \times (d_x)^2\right)} = \frac{285615.917}{\left(0.9 \times 60 \times (11.75)^2\right)} = 38.31 \text{ psi}$$

Reinforcement Area Check (X)

Eq. 4.9 CRSI Manual

$$As_{reqx} = \frac{0.85 \times f_c \times B \times d_x}{f_y} \times \left(1 - \sqrt{1 - \frac{2 \times R_{nx}}{(0.85 \times f_c)}}\right) = \frac{0.85 \times 4000 \times 60 \times 11.75}{60000} \times \left(1 - \sqrt{1 - \frac{2 \times 38.31}{(0.85 \times 4000)}}\right)$$

$$As_{reqx} = 0.45 \text{ in}^2$$

7.6.1.1

$$As_{minx} = 0.0018 \times H \times B = 0.0018 \times 15 \times 60$$

$$As_{minx} = 1.62 \text{ in}^2$$

Eq. 4.9 CRSI Manual

$$As_{maxx} = \frac{0.85 \times \beta_1 \times f_c \times B \times d_x}{f_y} \times \left(\frac{0.003}{\left(\left(\frac{f_y}{E_s}\right) + 0.006\right)}\right) = \frac{0.85 \times 0.85 \times 4000 \times 60 \times 11.75}{60000} \times \left(\frac{0.003}{\left(\left(\frac{60000}{29000000}\right) + 0.006\right)}\right)$$

$$As_{maxx} = 12.63 \text{ in}^2$$

$$As_{provx} = \frac{n_x \times \pi \times (db_x)^2}{4} = \frac{10 \times 3.14 \times (0.5)^2}{4}$$

$$As_{provx} = 1.96 \text{ in}^2$$

Reinforcement Area (X)

As_{provx} = 1.963 > As_{reqx} = 0.453
Provided reinforcement area > required reinforcement area.

PASS

Minimum Reinforcement Area (X)

As_{provx} = 1.963 > As_{minx} = 1.62
Provided reinforcement area > minimum reinforcement area.

PASS

Maximum Reinforcement Area (X)

As_{provx} = 1.963 < As_{maxx} = 12.625
Provided reinforcement area < maximum reinforcement area due to ductility.

PASS

Calculation of the Maximum Resisting Moment (X)

Eq. 4.3 CRSI Manual

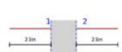
$$M_{nx} = As_{provx} \times f_y \times \left(d_x - \frac{(As_{provx} \times f_y)}{(1.7 \times B \times f_c)}\right) = 1.96 \times 60000 \times \left(11.75 - \frac{(1.96 \times 60000)}{(1.7 \times 60 \times 4000)}\right) = 1350246.78 \text{ lb-in}$$

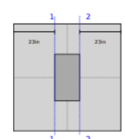
Nominal Flexural Strength	$M_{nx} =$	112.52kip-ft

$$Bending_{XRatio} = \frac{M_{ux}}{\phi \times M_{nx}} = \frac{285.62}{0.9 \times 1350.25}$$

$$Bending_{XRatio} = 0.24$$

UTILITY: 0.24





REFERENCES	CALCULATIONS	RESULTS

Bending Check Z-Axis

13.2.7.1

$$b_{z1} = \frac{(B - c_2)}{2} + offset_z = \frac{(60 - 26)}{2} + 0 = 17 \text{ in}$$

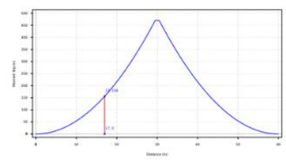
13.2.7.1

$$b_{z2} = \frac{(B - c_2)}{2} - offset_z = \frac{(60 - 26)}{2} - 0 = 17 \text{ in}$$

Max Moment Z-Axis Summary

LC	dist 1 (in)	Moment 1 (kip-ft)	dist 2 (in)	Moment 2 (kip-ft)
U1	17	13	43	13

Moment Diagram - (Z)



Critical Load Combination: U1=1.2DL + 1.6LL + 1Lr + 0W/S

Maximum Moment at the Critical Section	$M_{uz} =$	13.00kip-ft
Depth of Equivalent Rectangular Compressive Stress Block	$\beta_1 =$	0.85
Modulus of Elasticity of Reinforcement	$E_s =$	29000.00ksi
Bending Strength Reduction Factor (Tension Controlled)	$\phi =$	0.90
Ratio of Long to Short Footing Dimensions	$\beta =$	1.00

Calculation of the Strength Coefficient

Eq. 4.10 CRSI Manual

$$R_{nz} = \frac{M_{uz}}{\left(\varphi \times L \times (d_z)^2\right)} = \frac{156035.917}{\left(0.9 \times 60 \times (11.25)^2\right)} = 22.831 \text{ psi}$$

Reinforcement Area Check (Z)

Eq. 4.9 CRSI Manual

$$As_{reqz} = \frac{0.85 \times f_c \times L \times d_z}{f_y} \times \left(1 - \sqrt{1 - \frac{2 \times R_{nz}}{(0.85 \times f_c)}}\right) = \frac{0.85 \times 4000 \times 60 \times 11.25}{60000} \times \left(1 - \sqrt{1 - \frac{2 \times 22.83}{(0.85 \times 4000)}}\right)$$

$$As_{reqz} = 0.26 \text{ in}^2$$

Reinforcement Distribution Factor

R13.3.3.3

$$\gamma_s = \frac{2 \times \beta}{(\beta + 1)} = \frac{2 \times 1}{(1 + 1)}$$

$$\gamma_s = 1$$

R13.3.3.3

$$As_{reqzs} = As_{reqz} \times \gamma_s = 0.26 \times 1$$

$$As_{reqzs} = 0.26 \text{ in}^2$$

7.6.1.1

$$As_{minz} = 0.0018 \times H \times L = 0.0018 \times 15 \times 60$$

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$$As_{minz} = 1.62 \text{ in}^2$$

Eq. 4.9 CRSI Manual

$$As_{maxz} = \frac{0.85 \times \beta_1 \times f_c \times L \times d_z}{f_y} \times \left(\frac{0.003}{\left(\left(\frac{f_y}{E_s}\right) + 0.006\right)}\right) = \frac{0.85 \times 0.85 \times 4000 \times 60 \times 11.25}{60000} \times \left(\frac{0.003}{\left(\left(\frac{60000}{29000000}\right) + 0.006\right)}\right)$$

$$As_{maxz} = 12.09 \text{ in}^2$$

$$As_{provz} = \frac{n_z \times \pi \times (db_z)^2}{4} = \frac{10 \times 3.14 \times (0.5)^2}{4}$$

$$As_{provz} = 1.96 \text{ in}^2$$

Reinforcement Area (Z)

As_{provz} = 1.963 > As_{reqx} = 0.258
Provided reinforcement area > required reinforcement area. — **PASS**

Minimum Reinforcement Area (Z)

As_{provz} = 1.963 > As_{minz} = 1.62
Provided reinforcement area > minimum reinforcement area. — **PASS**

Maximum Reinforcement Area (Z)

As_{provz} = 1.963 < As_{maxx} = 12.088
Provided reinforcement area < maximum reinforcement area due to ductility. — **PASS**

Calculation of the Maximum Resisting Moment (Z). It considers the Reinforcement Distribution Factor.

Eq. 4.3 CRSI Manual

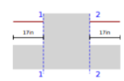
$$M_{nz} = \frac{As_{provz} \times f_y \times \left(d_z - \frac{(As_{provz} \times f_y)}{(1.7 \times L \times f_c \times \gamma_s)}\right)}{\gamma_s} = \frac{1.96 \times 60000 \times \left(11.25 - \frac{(1.96 \times 60000)}{(1.7 \times 60 \times 4000 \times 1)}\right)}{1} = 1291341.92 \text{ lb-in}$$

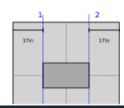
Nominal Flexural Strength	$M_{nz} =$	107.61kip-ft

$$Bending_{ZRatio} = \frac{M_{uz}}{\phi \times M_{nz}} = \frac{156.04}{0.9 \times 1291.34}$$

$$Bending_{ZRatio} = 0.13$$

UTILITY: 0.13





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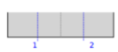

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REFERENCES	CALCULATIONS	RESULTS

Transfer of Forces

Vertical Transfer - Compression Check

22.8.3.1 Bearing Strength Calculation

$$A_1 = c_1 \times c_2 = 14 \times 26 = 364 \text{ in2}$$

$$A_2 = (min(l_{21}, 2 \times H) + c_1 + min(l_{22}, 2 \times H)) \times (min(b_{21}, 2 \times H) + (c_2 + min(b_{22}, 2 \times H))$$
$$A_2 = (min(23.00, 2 \times 15.00) + (14.00) + min(23.00, 2 \times 15.00)) \times (min(17.00, 2 \times 15.00) + (26.00) + min(17.00, 2 \times 15.00))$$
$$A_2 = 3600.00 in2$$

Table 22.8.3.2

$$B_n = min\left(\sqrt{\frac{A_2}{A_1}}, 2\right) \times 0.85 \times f_c \times A_1$$
$$B_n = min\left(\sqrt{\frac{3600.00}{364.00}}, 2.0\right) \times 0.85 \times 4000 \times 364.00$$
$$B_n = 2475200 \text{ lb}$$

Critical Load Combination for Bearing: U1=1.2DL + 1.6LL + 1Lr + 0W/S

Factored Bearing Load	$B_u =$	64.79kip
Bearing Strength Reduction Factor (Table 21.2.1.2)	$\phi =$	0.65
Reduced Bearing Strength	$\phi Bn =$	1608.88kip

$$Bearing_{Ratio} = \frac{B_u}{\phi \times B_n} = \frac{64.79}{0.65 \times 2475.20}$$

$$Bearing_{Ratio} = 0.04$$

UTILITY: 0.04

16.3.4.1

$$As_{mindowels} = 0.005 \times A_1 = 0.005 \times 364 = 1.82 \text{ in2}$$

$$As_{dowels} = \frac{\pi \times (db_{col})^2}{4} \times n = \frac{3.14 \times (0.5)^2}{4} \times 10 = 1.96 \text{ in2}$$

Asd_{prov} = 1.96 > $Asd_{min/req}$ = 1.82
Provided dowels reinforcement area > minimum/required reinforcement area. — **PASS**

Footing Details

Reinforcement Spacing Check

7.7.2.3

$$s_{max1} = min(3 * H, 18.00)$$

$$s_{max1} = min(3 * 15, 18.00)$$

$$s_{max1} = 18 \text{ in}$$

24.3

$$s_{max2} = min(15.0 * 40000/f_s - (2.5 * c_c), 12 * 40000/f_s)$$

$$s_{max2} = min(15.0 * 40000/40000 - (2.5 * 3), 12 * 40000/40000)$$

$$s_{max2} = 7.5 \text{ in}$$

Spacing for the Reinforcement along X-Axis

25.2

$$s_{minz} = max\left(1.0, db_z, (4 \times d_{agg})/3.0\right) + db_z$$
$$s_{min} = max\left(1.0, 0.5, 4 \times 1.00/3.0\right) + 0.5$$
$$s_{min} = 1.83 in$$

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$$s_x = \frac{(B - 2 \times c - db_x)}{(n_x - 1)} = \frac{(60 - 2 \times 3 - 0.5)}{(10 - 1)}$$

$$s_x = 5.94 \text{ in}$$

s_{max} = 7.5 > s_x = 5.94 > s_{min} = 1.833
Provided spacing is O.K.

PASS

Spacing for the Reinforcement along Z-Zxis

25.2

$$s_{minz} = max\,(1.0, db_z, (4 \times d_{agg})/3.0)) + db_z$$
$$s_{min} = max\,(1.0, 0.5, 4 \times 1.00/3.0)) + 0.5$$
$$s_{min} = 1.83 in$$

$$s_z = \frac{(L - 2 \times c - db_z)}{(n_z - 1)} = \frac{(60 - 2 \times 3 - 0.5)}{(10 - 1)}$$

$$s_z = 5.94 \text{ in}$$

s_{max} = 7.5 > s_z = 5.94 > s_{min} = 1.833
Provided spacing is O.K.

PASS

REFERENCES	CALCULATIONS	RESULTS

Development Length Check (X)

Distance to Nearest Concrete Surface or Half Spacing	$cb_x =$	3.25
Root of Concrete Strength Value min(√f'c, 100psi)	$rf =$	63.25
Epoxi Modification Factor	$\psi_e h =$	1.00
Confining Reinforcement Factor	$\psi_r =$	1.60
Location Factor	$\psi_o =$	1.00
Concrete Strength Factor	$\psi_c =$	0.87
Lightweight Concrete Factor	$\lambda =$	1.00

25.4.3.1

$$ldhx = \frac{(f_y \times \psi_e \times \psi_r \times \psi_o \times \psi_c)}{(55 \times \lambda \times rf)} \times (db_x)^{1.5} = \frac{(60000 \times 1 \times 1.6 \times 1 \times 0.87)}{(55 \times 1 \times 63.25)} \times (0.5)^{1.5}$$

$$ldhx = 8.46 \text{ in}$$

25.4.3.1

$$ld_{minx} = max(8 * db_x, 6.0)$$

$$ld_{minx} = max(8 * 0.5, 6.0)$$

$$ld_{minx} = 6 \text{ in}$$

10.21 CRSI Manual

$$Distance_{avail} = min(l_{21}, l_{22}) - c$$

$$Distance_{avail} = min(23, 23) - 3$$

$$Distance_{avail} = 20 \text{ in}$$

Development Length (X) $ld_{calc} =$ 8.46

ld_{calcx} = 8.46 < $Distance_{avail}$ = 20
Available length > minimum development length.

PASS

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REFERENCES	CALCULATIONS	RESULTS

Development Length Check (Z)

Distance to Nearest Concrete Surface or Half Spacing	$cb_z =$	3.25
Root of Concrete Strength Value min(√f'c, 100psi)	$rf =$	63.25
Epoxi Modification Factor	$\psi_e h =$	1.00
Confining Reinforcement Factor	$\psi_r =$	1.60
Location Factor	$\psi_o =$	1.00
Concrete Strength Factor	$\psi_c =$	0.87
Lightweight Concrete Factor	$\lambda =$	1.00

25.4.3.1

$$ldhz = \frac{(f_y \times \psi_e \times \psi_r \times \psi_o \times \psi_c)}{(55 \times \lambda \times rf)} \times (db_z)^{1.5} = \frac{(60000 \times 1 \times 1.6 \times 1 \times 0.87)}{(55 \times 1 \times 63.25)} \times (0.5)^{1.5}$$

$$ldhz = 8.46 \text{ in}$$

25.4.3.1

$$ld_{minz} = max(8 * db_z, 6.0)$$

$$ld_{minz} = max(8 * 0.5, 6.0)$$

$$ld_{minz} = 6 \text{ in}$$

10.21 CRSI Manual

$$Distance_{avail} = min(b_{21}, b_{22}) - c$$

$$Distance_{avail} = min(17, 17) - 3$$

$$Distance_{avail} = 14 \text{ in}$$

Development Length (Z) $ld_{calc} =$ 8.46

ld_{calcz} = 8.46 < $Distance_{avail}$ = 14
Available length > minimum development length.

PASS

Minimum Footing Height Check

13.3.1.2

$$H_{min} = 6 + db_z + \frac{db_x}{2} + c = 6 + 0.5 + \frac{0.5}{2} + 3 = 9.75 \text{ in}$$

Column Dowels Compression Development Length

25.4.9.2

$$l_d = max\left(0.0003 \times f_y \times \psi_r \times db_{col}, \frac{\psi_r \times db_{col} \times f_y}{50 \times \lambda \times rf}\right)$$
$$l_d = max\left(0.0003 \times 60000 \times 1 \times 0.5, \frac{1 \times 0.5 \times 60000}{50 \times 1 \times 63.25}\right)$$
$$l_d = 9.49 \text{ in}$$

Min. Dowels Development Length (25.4.9.1) $l_{dmin} =$ 9.75in

25.4.9.2

$$H_{dowels} = l_d + \frac{hook_{diam}}{2} + db_{col} + c = 9.49 + \frac{0}{2} + 0.5 + 3 = 12.99 \text{ in}$$

Footing Base Height $H =$ 15.00in

H = 15 > H_{min} = 12.99
Footing height is OK.

PASS

Results Summary

Result Name	Results
STABILITY CHECKS	
Eccentricity Ratio	PASS
Bearing Capacity	PASS
Overturning	PASS
Sliding	PASS
STRENGTH UTILIZATION	

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Result Name	Results
Shear (X)	0.34
Shear (Z)	0.18
Punching Shear	0.17
Bending Capacity (X)	0.24
Bending Capacity (Z)	0.13
Bearing Capacity	0.04
DETAILING	
Min/Max Reinforcement (X)	PASS
Mix/Max Reinforcement (Z)	PASS
Dowels Reinforcement	PASS
Bar Spacing (X)	PASS
Bar Spacing (Z)	PASS
Development Length (X)	PASS
Development Length (Z)	PASS
Footing Height Check	PASS

About this Calculator

Calculator Name: ACI 318-19 Spread Footing Design
Description: This calculator is in imperial units and it is used for the design of reinforced concrete spread footings. It also considers column eccentricities on both axes.

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URL: https://platform.skyciv.com/quick-design?uid=8002-aci-pad-footing-design-calculator&footing_name=FOOTING%201&footing_material=rcfooting&footing_width=5&footing_length=5&footing_depth=4&footing_height=15&col_c1=14&col_c2=26&col_height=4&col_x=0&col_z=0&punching_overhang_factor=1&fc=4&unit_weight=150&max_aggregate_size=1&cover=3&f_y=60&reinforcement_type=normal&bar_size_x=4&n_x=10&x_bars_with_hooks=true&bar_size_z=4&n_z=10&z_bars_with_hooks=true&col_bar_size=4&col_nx_bar=3&col_nz_bar=4&dowels_with_hooks=false&q_a=2&phi_footing_soil=25&min_overturning_SF=1.5&min_sliding_SF=1.5&consider_soil_above_foundation=false&consider_foundation_weight=false&elevation=X&footing_dims=true&show_loading=1&show_reinforcement=true&show_ground_level=true

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REFERENCES | CALCULATIONS | RESULTS

ACI 318-19 Strip Footing Design

SkyCiv

Dimensions

Symbol	Description	Value
$Name$	Footing Designation	SF1
$Mat_{footing}$	Material	Reinforced Concrete
B	Width	3.5 ft
Df	Foundation Base Depth	4 ft
H	Base Height	15 in
Mat_{wall}	Material	Reinforced Concrete
b_{wall}	Width	9 in
e_{wall}	Wall Eccentricity	0 in

Concrete Properties

Symbol	Description	Value
f'_c	Specified Compressive Strength	3 ksi
w_c	Unit Weight	150 lb/ft^3
d_{agg}	Max. Size Of Aggregate	1 in

Reinforcement

Symbol	Description	Value
C_c	Clear Cover	3 in
f_y	Specified Yield Strength	60 ksi
$Type$	Type	Normal
$d_{b,mainbar}$	Main Bar Size	#5
$s_{mainbar}$	Main Bar Spacing (center-to-center)	7.5 in
-	Use Main Bars with Hooks?	True
$d_{b,secondarybar}$	Transverse Bar Size	#5
$No.$	Number Of Transverse Bars	6 No.
$d_{b,wall}$	Wall Bar Size	#4
$s_{wall,bars}$	Wall Bar Spacing	10 in
-	Double Layer of Wall Reinforcement?	False
-	Use Dowels for Wall Reinforcement?	False

Geotechnical Parameters

Symbol	Description	Value
q_a	Allowable Bearing Capacity	7 ksf
φ	Foundation-Soil Friction Angle	20 °
$SF_{overturning}$	Min. Overturning SF	1.5
$SF_{sliding}$	Min. Sliding SF	1.5

Loads

Name	Axial Load (kip/ft)	Moment (kip*ft/ft)	Shear (kip/ft)
DL	0.3	0	0
LL	0.725	0	0
WS	0.16	0	0

Load Combinations

Name	Dead Load Factor	Live Load Factor	Wind / Seismic Factor
S1	1.0	1.0	1.0
U1	1.2	1.6	1.0

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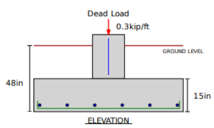
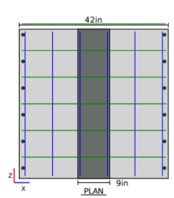
REFERENCES | CALCULATIONS | RESULTS

Strip Footing Plan





NAME	WIDTH	HEIGTH	DEPTH	MAIN R	SEC. R	WALL	DOWEL
SF1	42	15	48	#5@7.5	6#5	#4@10	-

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REFERENCES | CALCULATIONS | RESULTS

Step 1

Footing Stability

Maximum Eccentricity Check

Critical load combination for maximum soil stress: S1

Eccentricity

$$M_{full} = (P \times e_{wall}) + M + (V \times D_f) = (1.185 \times 0) + 0 + (0 \times 48) = 0 \text{ kip*in}$$

$$P_{full} = P + \Delta q \times A = 1.185 + 0 \times 504 = 1.185 \text{ kip}$$

$$e = \left|\frac{M_{full}}{P_{full}}\right| = \left|\frac{0}{1.18}\right| = 0 \text{ in}$$

Maximum Allowed Eccentricity

$$e_{max} = \frac{B}{6} = \frac{42}{6} = 7 \text{ in}$$

Maximum eccentricity $e =$ 0.00in
Maximum allowed eccentricity $e_{max} =$ 7.00in

Check for Eccentricity

$e <= e_{max}$
The resultant is within the middle third portion of the base.

PASS

Soil Bearing Pressure Check

Critical load combination for maximum soil stress: S1

$$q_{max} = \frac{P_{full}}{A} + \left(\frac{6 \times M_{full}}{\left((B)^2 \times L\right)}\right) = \frac{1.185}{504} + \left(\frac{6 \times 0}{\left((42)^2 \times 12\right)}\right) = 0.002 \text{ ksi}$$

Soil Pressures and Eccentricities Summary

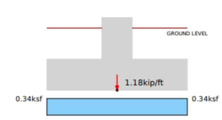
Name	ΣP kip/ft	ΣM kip*ft/ft	e in	$ratio_e$	qs_{max} ksf	$\frac{qs_{max}}{qs_{mean}}$	qs_{min} ksf
S1	1.18	0	0	0	0.34	1	0.34

Allowable soil bearing capacity $qa =$ 7.00ksf

Check for Soil Bearing Capacity

$qa >= q_{max}$
The maximum soil stress is smaller than the bearing capacity.

PASS



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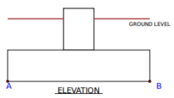
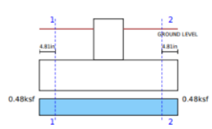
REFERENCES	CALCULATIONS	RESULTS
	Overturning Check	
	Critical load combination for overturning: S1	
	No moments were considered.	
	Overturning safety factor $SF_{overturning} =$ >1000.0	
	Mimimum safety factor $SF_{min} =$ 1.5	
		
	Check for Overturning	
	SF >= 1.5 The foundation is safe against overturning.	PASS
	Sliding Check	
	Critical load combination for sliding: S1	
	No shear forces were considered.	
	Sliding safety factor $SF_{sliding} =$ >1000.0	
	Mimimum safety factor $SF_{min} =$ 1.5	
	Check for Sliding	
	SF >= 1.5 The foundation is safe against sliding.	PASS

REFERENCES	CALCULATIONS	RESULTS
Step 2	**Footing Strength**	
	Critical load combination for ultimate soil stress: U1	
	Ultimate Soil Pressures and Eccentricities Summary	

Name	ΣP kip/ft	ΣM kip*ft/ft	e in	$ratio_e$	qs_{max} ksf	$\frac{qs_{max}}{qs_{mean}}$	qs_{min} ksf
U1	1.68	0	0	0	0.48	1	0.48

$$d = H - c - \frac{db}{2} = 15 - 3 - \frac{0.625}{2} = 11.688\ \text{in}$$

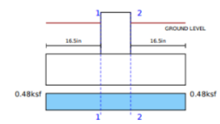
$$\rho = \frac{\frac{L}{s} \times \left(\frac{\pi \times (db)^2}{4}\right)}{(L \times d)} = \frac{\frac{12}{7.5} \times \left(\frac{3.14159 \times (0.625)^2}{4}\right)}{(12 \times 11.6875)} = 0.0035$$

REFERENCES	CALCULATIONS	RESULTS
	One-way Shear Check	
22.5.5.1	$V_c = 8 \times \lambda \times (\rho)^{\frac{1}{3}} \times rf \times L \times d = 8 \times 1 \times (0)^{\frac{1}{3}} \times 54.77 \times 12 \times 11.69$	
	$V_c = 9330.59\ \text{lb}$	
	$b_{11} = \frac{(B - b_w)}{2} - d + e_{wall} = \frac{(42 - 9)}{2} - 11.69 + 0 = 4.81\ \text{in}$	
	$b_{12} = \frac{(B - b_w)}{2} - d - e_{wall} = \frac{(42 - 9)}{2} - 11.69 - 0 = 4.81\ \text{in}$	
	$V_{u1} = \left((qu_{min} \times b_{11}) + \frac{(qu_{max} - qu_{min}) \times (b_{11})^2}{(2 \times B)}\right) \times L = \left((3.33 \times 4.81) + \frac{(3.33 - 3.33) \times (4.81)^2}{(2 \times 42)}\right) \times 12$	
	$V_{u1} = 192.5\ \text{lb}$	
	$V_{u2} = \left((qu_{max} \times b_{12}) - \frac{(qu_{max} - qu_{min}) \times (b_{12})^2}{(2 \times B)}\right) \times L = \left((3.33 \times 4.81) - \frac{(3.33 - 3.33) \times (4.81)^2}{(2 \times 42)}\right) \times 12$	
	$V_{u2} = 192.5\ \text{lb}$	
	Maximum shear that can be resisted at the section $V_c =$ 9.33kip	
	Shear strength reduction factor $\phi =$ 0.75	
	Maximum factored shear force at the section $V_u =$ 0.19kip	
	$Shear_{Ratio} = \frac{V_u}{\phi \times V_c} = \frac{192.50}{0.75 \times 9330.59}$	
	$Shear_{Ratio} = 0.03$	UTILITY: 0.03
		

REFERENCES	CALCULATIONS	RESULTS
	Bending Check	
13.2.7.1	$b_{21} = \frac{(B - b_w)}{2} + e_{wall} = \frac{(42 - 9)}{2} + 0 = 16.5\ \text{in}$	
13.2.7.1	$b_{22} = \frac{(B - b_w)}{2} - e_{wall} = \frac{(42 - 9)}{2} - 0 = 16.5\ \text{in}$	
	$M_{u1} = \left(\left(\frac{qu_{min} \times (b_{21})^2}{2}\right) + \frac{(qu_{max} - qu_{min}) \times (b_{21})^3}{(6 \times B)}\right) \times L = \left(\left(\frac{3.33 \times (16.5)^2}{2}\right) + \frac{(3.33 - 3.33) \times (16.5)^3}{(6 \times 42)}\right) \times 12$	
	$M_{u1} = 5445\ \text{lb-in}$	
	$M_{u2} = \left(\left(\frac{qu_{max} \times (b_{22})^2}{2}\right) - \frac{(qu_{max} - qu_{min}) \times (b_{22})^3}{(B \times 6)}\right) \times L = \left(\left(\frac{3.33 \times (16.5)^2}{2}\right) - \frac{(3.33 - 3.33) \times (16.5)^3}{(42 \times 6)}\right) \times 12$	
	$M_{u2} = 5445\ \text{lb-in}$	
	Depth of equivalent rectangular compressive stress block $\beta_1 =$ 0.85	
	Modulus of elasticity of reinforcement $E_s =$ 29000.00ksi	
	Bending strength reduction factor (tension controlled) $\phi =$ 0.90	
	Calculation of the Strength Coefficient	
Eq. 4.10 CRSI Manual	$R_n = \frac{M_u}{\left(\varphi \times L \times (d)^2\right)} = \frac{5445}{\left(0.9 \times 12 \times (11.69)^2\right)} = 3.69\ \text{psi}$	

REFERENCES	CALCULATIONS	RESULTS
	Main Reinforcement Area Check	
Eq. 4.9 CRSI Manual	$As_{req} = \frac{0.85 \times f_c \times L \times d}{f_y} \times \left(1 - \sqrt{1 - \frac{2 \times R_n}{(0.85 \times f_c)}}\right) = \frac{0.85 \times 3000 \times 12 \times 11.69}{60000} \times \left(1 - \sqrt{1 - \frac{2 \times 3.69}{(0.85 \times 3000)}}\right)$	
	$As_{req} = 0.01\ in^2$	
7.6.1.1	$As_{min} = 0.0018 \times H \times L = 0.0018 \times 15 \times 12$	
	$As_{min} = 0.32\ in^2$	
Eq. 4.9 CRSI Manual	$As_{max} = \frac{0.85 \times \beta_1 \times f_c \times L \times d}{f_y} \times \left(\frac{0.003}{\left(\left(\frac{f_y}{E_s}\right) + 0.006\right)}\right) = \frac{0.85 \times 0.85 \times 3000 \times 12 \times 11.69}{60000} \times \left(\frac{0.003}{\left(\left(\frac{60000}{29000000}\right) + 0.006\right)}\right)$	
	$As_{max} = 1.88\ in^2$	
	$As_{prov} = \frac{\pi \times (db)^2}{4} \times \left(\frac{L}{(s)}\right) = \frac{3.14 \times (0.63)^2}{4} \times \left(\frac{12}{(7.5)}\right)$	
	$As_{prov} = 0.49\ in^2$	
	As_{prov} = 0.491 > As_{req} = 0.009 Provided reinforcement area > required reinforcement area.	PASS
	As_{prov} = 0.491 > As_{min} = 0.324 Provided reinforcement area > minimum reinforcement area.	PASS
	As_{prov} = 0.491 < As_{max} = 1.884 Provided reinforcement area < maximum reinforcement area due to ductility.	PASS

REFERENCES	CALCULATIONS	RESULTS
	Secondary Reinforcement Area Check	
24.4.3.2	$Ast_{req} = 0.0018 \times B \times H = 0.0018 \times 42 \times 15$	
	$Ast_{req} = 1.13\ in^2$	
	$Ast_{prov} = \frac{\pi \times (db)^2}{4} \times n = \frac{3.14 \times (0.63)^2}{4} \times 6$	
	$Ast_{prov} = 1.84\ in^2$	
	Ast_{prov} = 1.841 > Ast_{req} = 1.134 Provided reinforcement area > required reinforcement area.	PASS
	Calculation of the maximum resisting moment	
Eq. 4.3 CRSI Manual	$M_n = As_{prov} \times f_y \times \left(d - \frac{(As_{prov} \times f_y)}{(1.7 \times L \times f_c)}\right) = 0.49 \times 60000 \times \left(11.69 - \frac{(0.49 \times 60000)}{(1.7 \times 12 \times 3000)}\right) = 330051.34$ lb-in	
	$Bending_{Ratio} = \frac{M_u}{\phi \times M_n} = \frac{5445.00}{0.9 \times 330051.34}$	
	$Bending_{Ratio} = 0.02$	UTILITY: 0.02



REFERENCES	CALCULATIONS	RESULTS
	Transfer of Forces	
	Vertical Transfer - Compression Check	
22.8.3.1	Bearing Strength Calculation	
	$A_1 = L \times b_{wall} = 12 \times 9 = 108$ in2	
	$A_2 = (min(b_{21}, 2 \times H) + (factor * b_{wall} + min(b_{22}, 2 \times H)) \times L$ $A_2 = (min(16.50, 2 \times 15.00) + (1 \times 9.00) + min(16.50, 2 \times 15.00)) \times 12.00$ $A_2 = 504.00\ in^2$	
Table 22.8.3.2	$B_n = \min\left(\sqrt{\frac{A_2}{A_1}}, 2\right) \times 0.85 \times f_c \times A_1$ $B_n = \min\left(\sqrt{\frac{504.00}{108.00}}, 2.0\right) \times 0.85 \times 3000 \times 108.00$ $B_n = 550800$ lb	
	Critical load combination for bearing: U1 Factored bearing load $B_u =$ 1.68kip Bearing strength reduction factor (Table 21.2.1.2) $\phi =$ 0.65 Reduced bearing strength $\phi Bn =$ 358.02kip	
	$Bearing_{Ratio} = \frac{B_u}{\phi \times B_n} = \frac{1.68}{0.65 \times 550.80}$	
	$Bearing_{Ratio} = 0.00$	UTILITY: 0.00

REFERENCES	CALCULATIONS	RESULTS
Step 3	**Footing Details**	
	Main Reinforcement Spacing Check Tensile stress in reinforcement for service loads $fs = 2/3 * fy =$ 40000ksi	
7.7.2.3	$s_{max1} = min(3 * H, 18.00)$	
	$s_{max1} = min(3 * 15, 18.00)$	
	$s_{max1} = 18$ in	
24.3	$s_{max2} = min(15.0 * 40000/f_s - (2.5 * c_c), 12 * 40000/f_s)$	
	$s_{max2} = min(15.0 * 40000/40000 - (2.5 * 3), 12 * 40000/40000)$	
	$s_{max2} = 7.5$ in	
25.2	$s_{min} = max\,(1.0, db, (4 \times d_{agg})/3.0)) + db$ $s_{min} = max\,(1.0, 0.625, 4 \times 1.00/3.0)) + 0.625$ $s_{min} = 1.96in$	
	s_{max} = 7.5 > s = 7.5 > s_{min} = 1.958 Provided spacing is O.K.	PASS
	Transverse Reinforcement Spacing Check	
25.2	$s_{min} = max\,(1.0, dbt, (4 \times d_{agg})/3.0)) + dbt$ $s_{min} = max\,(1.0, 0.625, 4 \times 1.00/3.0)) + 0.625$ $s_{min} = 1.96in$	
	s_{max} = 7.5 > s_t = 6.83 > s_{min} = 1.96 Transverse spacing is O.K.	PASS

REFERENCES	CALCULATIONS	RESULTS
	Main Reinforcement Development Length Check	
	Distance to nearest concrete surface or half spacing $c_b =$ 3.31in	
	Root of concrete strength value min(√f'c,100psi) $rf =$ 54.77psi	
	Epoxi modification factor $\psi_e h =$ 1.00	
	Confining reinforcement factor $\psi_r =$ 1.60	
	Location factor $\psi_o =$ 1.00	
	Concrete strength factor $\psi_c =$ 0.80	
	Lightweight concrete factor $\lambda =$ 1.00	
25.4.3.1	$ldh = \frac{(f_y \times \psi_e \times \psi_r \times \psi_o \times \psi_c)}{(55 \times \lambda \times rf)} \times (db)^{1.5} = \frac{(60000 \times 1 \times 1.6 \times 1 \times 0.8)}{(55 \times 1 \times 54.77)} \times (0.63)^{1.5}$	
	$ldh = 12.6 \text{ in}$	
25.4.3.1	$ld_{min} = max(8 * db, 6.0)$	
	$ld_{min} = max(8 * 0.63, 6.0)$	
	$ld_{min} = 6 \text{ in}$	
10.21 CRSI Manual	$Distance_{avail} = min(b_{21}, b_{22}) - c$	
	$Distance_{avail} = min(16.5, 16.5) - 3$	
	$Distance_{avail} = 13.5 \text{ in}$	
	Main bar development length $ld_{calc} =$ 12.60 in	
	ld_{calc} = 12.6 < $Distance_{avail}$ = 13.5 Available length > minimum development length.	PASS
	Minimum Footing Height Check	
13.3.1.2	$H_{min} = 6 + \frac{db}{2} + c = 6 + \frac{0.63}{2} + 3 = 9.31 \text{ in}$	
	Footing base height $H =$ 15.00in	
	H = 15 > H_{min} = 9.31 Footing height is OK.	PASS

REFERENCES	CALCULATIONS	RESULTS
	Results Summary	

Result Name	Results
ACI 318-19 Compliance	PASS
STABILITY CHECKS	
Soil Bearing Capacity	PASS
Maximum Eccentricity	PASS
Overturning Safety	PASS
Sliding Safety	PASS
STRENGTH CHECKS	
Check for Shear Capacity	0.03
Main Reinforcement Area	PASS
Minimum Reinforcement Area	PASS
Maximum Reinforcement Area	PASS
Transverse Reinforcement Area	PASS
Check for Bending Capacity	0.02
Check for Bearing Capacity	0.00
DETAILS	
Main Reinforcement Spacing	PASS
Transverse Bars Spacing	PASS
Development Length Main Reinforcement	PASS
Check Footing Height	PASS

About this Calculator

Calculator Name: ACI 318-19 Strip Footing Design
Description: The ACI 318-19 Strip Footing calculator is used for the design of reinforced concrete and plain concrete strip footings to ACI 318-19. The calculator can deal with either reinforced concrete or masonry walls.

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URL: https://platform.skyciv.com/quick-design?uid=8001-aci318-strip-footing-design&footing_name=SF1&footing_width=3.5&footing_depth=4&footing_height=15&footing_material=rcfooting&f_c=3&unit_weight=150&max_aggregate_size=1&cover=3&f_y=60&reinforcement_type=normal&main_bar_size=5&s_main_bar=7.5&main_bars_with_hooks=true&secondary_bar_size=5&transverse_bar_no=6&wall_material=rcwall&wall_width=9&wall_eccentricity=0&wall_bar_size=4&s_wall_bar=10&double_layer=false&use_dowels=false&q_a=7&phi_footing_soil=20&consider_soil_above_foundation=false&consider_foundation_weight=false&min_overturning_SF=1.5&min_sliding_SF=1.5&wall_dead_axial=0.3&wall_dead_moment=0&wall_dead_shear=0&wall_live_axial=0.725&wall_live_moment=0&wall_live_shear=0&wall_ws_axial=0.16&wall_ws_moment=0&wall_ws_shear=0&show_material=true&footing_dims=true&show_loading=1&show_reinforcement=true

Contact: fernando.lopez@skyciv.com

Figure 24: Multi-Use Lodge Structural Footing Analysis

Appendix D: Year-Round Cabins

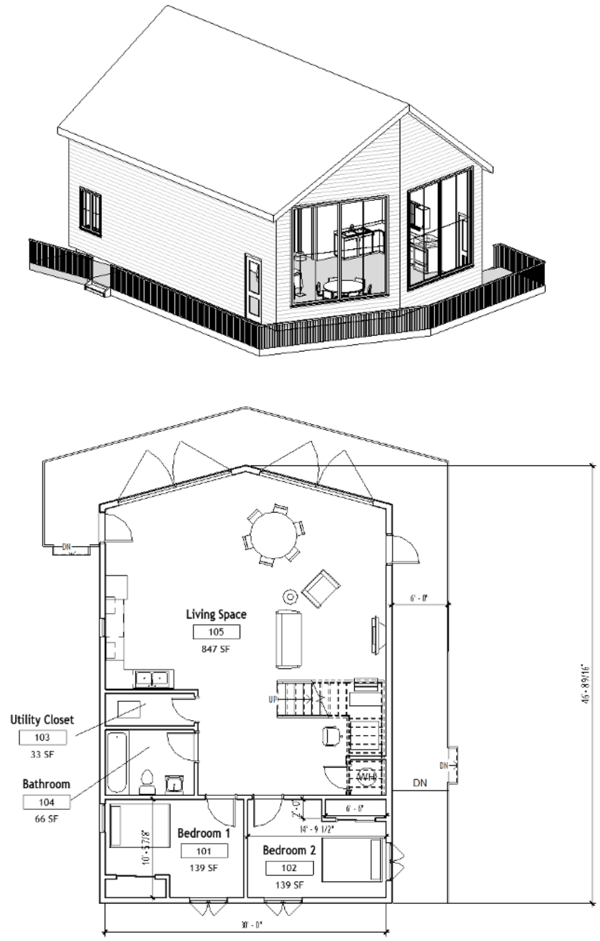


Figure 7: Cabin Alternative Design 1

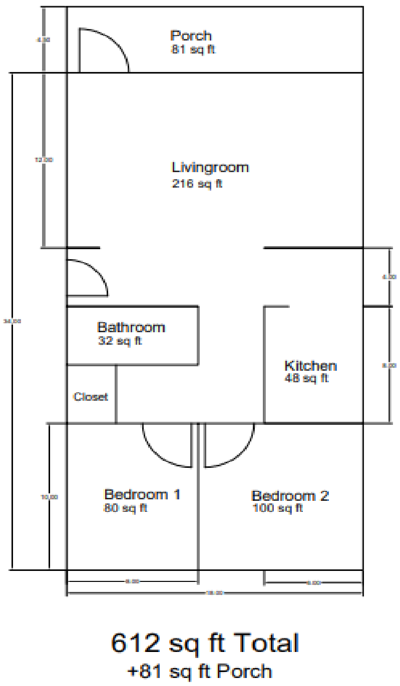


Figure 8: Cabin Alternative Design 2

Figure 11: Cabin Final Design

East Cabin - Wind Loads

General Information

Eave height

$h_e := 12.5\ ft$

Building width

$l := 374.5\ in$

Roof angle

$\theta := \operatorname{atan}\left(\dfrac{h_e}{\left(\dfrac{l}{2}\right)}\right) = 0.675$ $\quad \theta \cdot \dfrac{180}{\pi} = 38.697$ $\quad z := 330.125\ in$ $\quad z = 27.51\ ft$

Risk Category II Site Soil Class DE

$V := 109\ mph$ (ASCE Hazard Tool)

$V := 109$

$K_{zt} := 1.0$ Under 1000ft

$K_d := 0.85$

Exposure C

$C_{pw} := 0.8$ $\quad K_z := 0.98$

$C_{ps} := -0.7$

$q_z := 0.002568\ K_z \cdot K_{zt} \cdot K_d \cdot V^2\ psf = 25.415\ psf$

$G := 0.85$

$GC_{pi} := 0.18$ +or- (enclosed)

East Wind

Wall Pressures

$h := \dfrac{(36\ ft + 8.78\ in) + (24\ ft)}{2} = 30.366\ ft$

$L := 49\ ft + 4\ in = 49.333\ ft$ $\quad B := 31\ ft + 3.25\ in = 31.271\ ft$

$\dfrac{h}{L} = 0.616$ $\quad temp := \dfrac{L}{B} = 1.578$ $\quad C_{pl} := -0.5 + (temp - 1) \cdot \dfrac{(-0.3 + 0.5)}{(2 - 1)} = -0.384$

Slope (Assumed Values) $\quad H := 15\ ft$ $\quad x := 0\ ft$ $\quad L_h := 15\ ft$ $\quad z := 740\ ft$

$\dfrac{H}{L_h} = 1$ $\quad \dfrac{x}{L_h} = 0$ $\quad \dfrac{z}{L_h} = 49.333$

Assume slope negligeable

$$p_{windward} := q_z \cdot K_d \cdot C_{pw} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} 21.857 \\ 12.708 \end{bmatrix} \; psf$$

$$p_{side} := q_z \cdot K_d \cdot C_{ps} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -10.547 \\ -19.697 \end{bmatrix} \; psf$$

$$p_{leeward} := q_z \cdot K_d \cdot C_{pl} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -3.731 \\ -12.881 \end{bmatrix} \; psf$$

Roof Pressures

$$\theta := \left(\operatorname{atan}\left(\frac{h_e}{\left(\frac{l}{2}\right)} \right) \right) \cdot \frac{180}{\pi} = 38.697$$

$$\frac{h}{L} = 0.616$$

Side

0 to $\frac{h}{2} = 15.183 \; ft$ $\quad C_{psr1} := -0.9 \quad C_{psr2} := -0.9 \quad C_{psr3} := -0.5$

$$p_{side1roof} := q_z \cdot K_d \cdot C_{psr1} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -14.868 \\ -24.017 \end{bmatrix} \; psf \qquad C_{psr4} := -0.3$$

$\frac{h}{2}$ *to* $h = 30.366 \; ft$

$$p_{side2roof} := q_z \cdot K_d \cdot C_{psr2} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -14.868 \\ -24.017 \end{bmatrix} \; psf$$

h *to* $2\ h = 60.732 \; ft$

$$p_{side3roof} := q_z \cdot K_d \cdot C_{psr3} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -6.227 \\ -15.376 \end{bmatrix} \; psf$$

$2\ h$ *to* *end*

$$p_{side4roof} := q_z \cdot K_d \cdot C_{psr4} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -1.906 \\ -11.056 \end{bmatrix} \; psf$$

Overhang

$$C_{po} := 0.8$$

$$p_{overhang} := q_z \cdot K_d \cdot C_{po} = 17.282 \; psf$$

West Wind

Wall Pressures

$z := 27\ ft + 6\ in$ $h_e := 15\ ft$

$h := \frac{h_e + z}{2} = 21.25\ ft$

$\left(\operatorname{atan}\left(\frac{z - h_e}{\left(\frac{l}{2}\right)}\right)\right) \cdot \frac{180}{\pi} = 38.697$

$\frac{h}{L} = 0.431$ $\frac{L}{B} = 1.578$ $C_{pl} := -0.5$

$p_{windward} := q_z \cdot K_d \cdot C_{pw} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} 21.857 \\ 12.708 \end{bmatrix}\ psf$

$p_{side} := q_z \cdot K_d \cdot C_{ps} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -10.547 \\ -19.697 \end{bmatrix}\ psf$

$p_{leeward} := q_z \cdot K_d \cdot C_{pl} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -6.227 \\ -15.376 \end{bmatrix}\ psf$

Roof Pressures

Side

0 to $\frac{h}{2} = 10.625\ ft$ $C_{psr1} := -0.9$ $C_{psr2} := -0.9$ $C_{psr3} := -0.5$

$p_{side1roof} := q_z \cdot K_d \cdot C_{psr1} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -14.868 \\ -24.017 \end{bmatrix}\ psf$ $C_{psr4} := -0.3$

$\frac{h}{2}$ *to* $h = 21.25\ ft$

$p_{side2roof} := q_z \cdot K_d \cdot C_{psr2} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -14.868 \\ -24.017 \end{bmatrix}\ psf$

h *to* $2\ h = 42.5\ ft$

$p_{side3roof} := q_z \cdot K_d \cdot C_{psr3} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -6.227 \\ -15.376 \end{bmatrix}\ psf$

$2\ h$ *to* *end*

$p_{side4roof} := q_z \cdot K_d \cdot C_{psr4} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -1.906 \\ -11.056 \end{bmatrix}\ psf$

Overhang

$C_{po} := 0.8$

$p_{overhang} := q_z \cdot K_d \cdot C_{po} = 17.282\ psf$

$p_{wmax} := p_{side2roof}(1) = -24.017\ psf$

South Wind

Wall Pressures

$$h := \frac{(36\ ft + 8.78\ in) + (24\ ft)}{2} = 30.366\ ft$$

$$B := 49\ ft + 4\ in = 49.333\ ft \qquad L := 31\ ft + 3.25\ in = 31.271\ ft$$

$$\frac{h}{L} = 0.971 \qquad \frac{L}{B} = 0.634$$

$$C_{pl} := -0.5$$

$$p_{windward} := q_z \cdot K_d \cdot C_{pw} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} 21.857 \\ 12.708 \end{bmatrix} psf \qquad p_{wMax} := p_{windward}(0)$$

$$p_{side} := q_z \cdot K_d \cdot C_{ps} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -10.547 \\ -19.697 \end{bmatrix} psf$$

$$p_{leeward} := q_z \cdot K_d \cdot C_{pl} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -6.227 \\ -15.376 \end{bmatrix} psf$$

Roof Pressures

$$\theta = 38.697$$

$$C_{pwr} := 0.3 + (\theta - 35) \cdot \frac{(0.4 - 0.3)}{(45 - 35)} = 0.337$$

$$C_{plr} := -0.6$$

Windward

$$p_{windwardroof} := q_z \cdot K_d \cdot C_{pwr} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} 11.854 \\ 2.705 \end{bmatrix} psf$$

Leeward

$$p_{leewardroof} := q_z \cdot K_d \cdot C_{plr} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -8.387 \\ -17.536 \end{bmatrix} psf$$

Overhang

$$C_{po} := 0.8$$

$$p_{overhang} := q_z \cdot K_d \cdot C_{po} = 17.282\ psf$$

North Wind

Wall Pressures

$$h := \frac{(36\ ft + 8.78\ in) + (24\ ft)}{2} = 30.366\ ft$$

$$B := 49\ ft + 4\ in = 49.333\ ft \qquad L := 31\ ft + 3.25\ in = 31.271\ ft$$

$$\frac{h}{L} = 0.971 \qquad \frac{L}{B} = 0.634$$

$$C_{pl} := -0.5$$

$$p_{windward} := q_z \cdot K_d \cdot C_{pw} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} 21.857 \\ 12.708 \end{bmatrix}\ psf$$

$$p_{side} := q_z \cdot K_d \cdot C_{ps} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -10.547 \\ -19.697 \end{bmatrix}\ psf$$

$$p_{leeward} := q_z \cdot K_d \cdot C_{pl} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -6.227 \\ -15.376 \end{bmatrix}\ psf$$

Roof Pressures

$$\theta = 38.697$$

$$C_{pwr} := 0.3 + (\theta - 35) \cdot \frac{(0.4 - 0.3)}{(45 - 35)} = 0.337$$

$$C_{plr} := -0.6$$

Windward

$$p_{windwardroof} := q_z \cdot K_d \cdot C_{pwr} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} 11.854 \\ 2.705 \end{bmatrix}\ psf$$

Leeward

$$p_{leewardroof} := q_z \cdot K_d \cdot C_{plr} + q_z \cdot \begin{bmatrix} GC_{pi} \\ -GC_{pi} \end{bmatrix} = \begin{bmatrix} -8.387 \\ -17.536 \end{bmatrix}\ psf$$

Overhang

$$C_{po} := 0.8$$

$$p_{overhang} := q_z \cdot K_d \cdot C_{po} = 17.282\ psf$$

Snow Loads

Balanced Load   Risk Category II

Ground Load

$P_g := 40\ psf$ (ASCE Hazard Tool)

Surface Roughness

Category C (Rural)

$C_e := 1.0$

Exposure

Partially Exposed (Forested Rural Location)

Thermal Factor

(Assumed based off of Vaulted Roof spacing)

$C_t := 1.17$

Slope Factor

$C_s := 0.51$ $\theta = 38.697$ Unobstructed slippery surface

$P_s := 0.7 \cdot C_s \cdot C_e \cdot C_t \cdot P_g = 16.708\ psf$

Unbalanced Snow Load

$l_{rs} := 33\ ft + 2.5\ in$

$l_u := \frac{l_{rs}}{2} = 16.604\ ft$ $L_u := 16.604$ $P_g = 40\ psf$ $p_g := 40$

$h_D := 0.43 \cdot \sqrt[3]{L_u} \cdot \sqrt[4]{p_g + 10} - 1.5 = 1.417$

$\gamma := 0.13 \cdot p_g + 14 = 19.2$ $\gamma := 19.2\ pcf$

Assuming standard residential area for other cabin, S=12ft

$P_{un} := h_D \cdot ft \cdot \frac{\gamma}{\sqrt[2]{12}} = 7.854\ psf$ $W_{un} := \frac{8}{3} \cdot h_D \cdot ft \cdot \sqrt[2]{12} = 13.09\ ft$

East Cabin - Truss System

asce7-11 pg 570

Truss Design

Kingspost Design Used

Equations from NDS 2018 Ed

Wood Properties from NDS Supplement 2018 Ed

Load Calculation — From Boise Cascade

Suspended wood channel system	$D_1 := 2.5\ psf$
18 gauge metal decking roofing	$D_2 := 3\ psf$
Membrane	$D_3 := 0.35\ psf$
2x8 rafters @16in w/ fiberglass	$D_4 := 8\ psf$
1/2" osb plywood sheathing	$D_5 := 1.7\ psf$

Standard Roof $L_1 := 20\ psf$ (ASCE)

$D_{upper} := D_1 + D_2 + D_3 + D_4 + D_5 = 15.55\ psf$ $D_{lower} := 0\ psf$

$L_{rupper} := L_1 = 20\ psf$ $L_{lower} := 0\ psf$

$l := 31\ ft + 3.25\ in = 31.271\ ft$ $h_{ridge} := 36\ ft + 8.78\ in$ $h_{eave} := 24\ ft$

$h_{truss} := h_{ridge} - h_{eave} = 12.732\ ft$

$l_{upper} := 2 \cdot \sqrt{\left(\frac{l}{2}\right)^2 + h_{truss}^{\ 2}} = 40.327\ ft$

$D_{upperCorrection} := D_{upper} \cdot \frac{l}{l_{upper}} = 12.058\ psf$

$L_{rupperCorrection} := L_{rupper} \cdot \frac{l}{l_{upper}} = 15.509\ psf$

$span_{total} := 42\ ft + 6\ in$

$\frac{span_{total}}{5} = 8.5\ ft$

$\frac{span_{total}}{6} = 7.083\ ft$

$P_s = 16.708\ psf$

$P_{sCor} := P_s \cdot \frac{l}{l_{upper}} = 12.956\ psf$

$p_{wmax} = -24.017\ psf$

$p_{wmaxCor} := |p_{wmax}| \cdot \frac{l}{l_{upper}} = 18.624\ psf$

$T_w := 126.5\ in$ $Snow := 16.708\ psf \cdot \left(\frac{l}{l_{upper}}\right)$ $Wind := 24.017\ psf \cdot \left(\frac{l}{l_{upper}}\right)$

Load Combinations

$$w_{upper1} := T_w \cdot \left(1.4 \cdot D_{upperCorrection}\right) = 177.957\ plf$$

$$w_{upper2} := T_w \cdot \left(1.2 \cdot D_{upperCorrection} + 1.6 \cdot 0\ psf + \max\left(0.5 \cdot L_{rupperCorrection}, 0.5 \cdot P_{sCor}\right)\right) = 234.278\ plf$$

$$w_{upper3} := T_w \cdot \left(1.2 \cdot D_{upperCorrection} + \max\left(1.6 \cdot L_{rupperCorrection}, P_{sCor}\right) + \max\left(0\ psf, 0.5 \cdot p_{wmaxCor}\right)\right) = 512.278\ plf$$

$$w_{upper4} := T_w \cdot \left(1.2 \cdot D_{upperCorrection} + p_{wmaxCor} + 0\ psf + \max\left(0.5 \cdot L_{rupperCorrection}, 0.3 \cdot P_{sCor}\right)\right) = 430.605\ plf$$

$$w_{upper5} := T_w \cdot \left(0.9 \cdot D_{upperCorrection} + p_{wmaxCor}\right) = 310.728\ plf$$

$w_{upper} := w_{upper3}$ $\quad 1.6 \cdot L_{rupperCorrection} = 24.814\ psf$ $\quad P_{sCor} = 12.956\ psf$

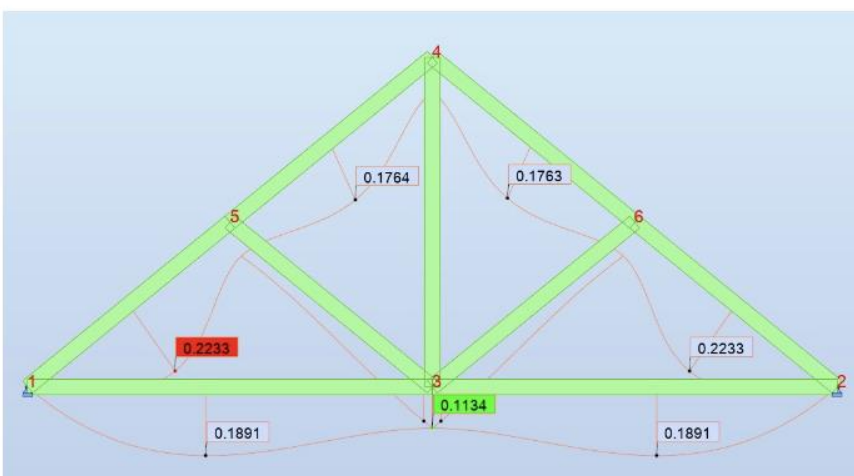
Self weight to be added, no added weight to lower beam (exposed beam)

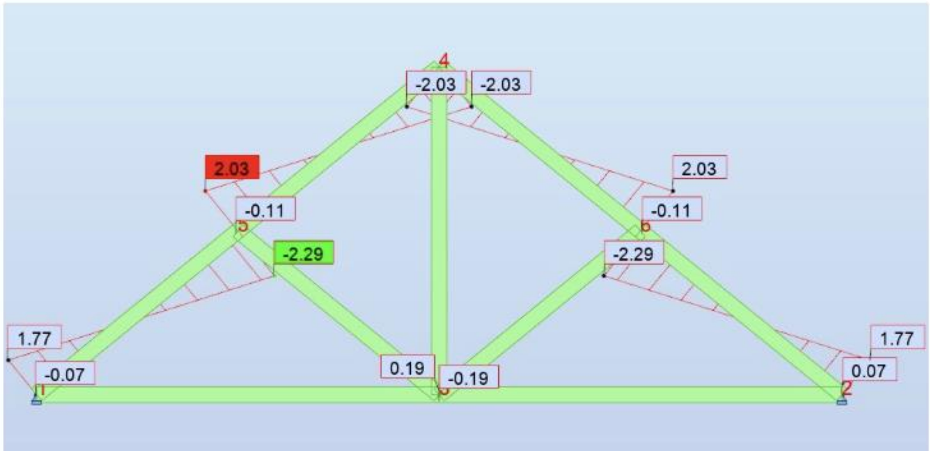
Wood design based off of use of Southern Pine Timber

Southern Pine Select Structural 4x8

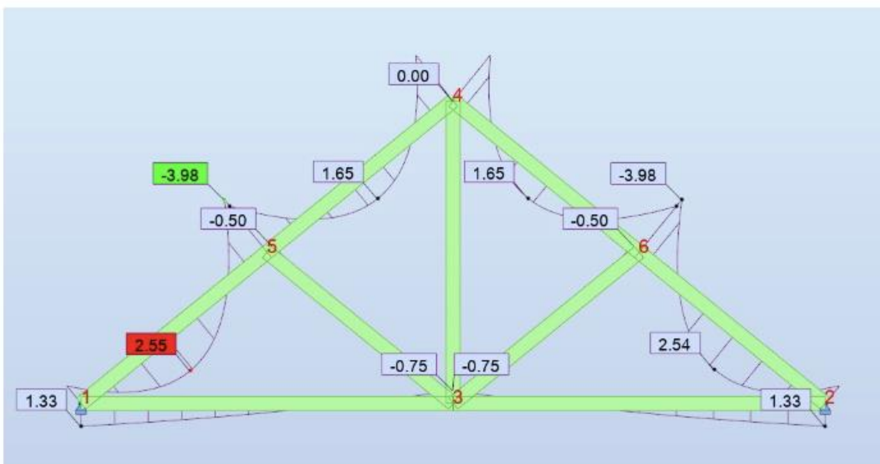
$T_w \cdot D_{upperCorrection} = 0.127\ \frac{kip}{ft}$ $\quad T_w \cdot L_{rupperCorrection} = 0.163\ \frac{kip}{ft}$

$T_w \cdot p_{wmaxCor} = 0.196\ \frac{kip}{ft}$ $\quad$ For use in Robot





$P_i := 2.29\ kip$ $P_l := 2.03\ kip$ $P_u := 2.03\ kip$



$M_l := 3.98\ kip \cdot ft$ $M_u := 2.55\ kip \cdot ft$

Bottom Chord Design

4x8 Section $F_t := 1350\ psi$ $F_c := 1700\ psi$ $F_b := 1950\ psi$

$b := 3.5\ in$ $d := 7.25\ in$ $A := b \cdot d = 25.375\ in^2$ $I := \frac{b \cdot d^3}{12} = 111.148\ in^4$

$S := \frac{I}{\frac{d}{2}} = 30.661\ in^3$ $E := 1800000\ psi$ $E_{min} := 660000\ psi$

Adjustment Factors

$C_D := 1.0$ $C_{fu} := 1.00$ $C_i := 1.0$ $C_M := 1.0$ $C_r := 1.00$

$C_t := 1.0$ $C_F := 1.0$ (Implemented with Southern Pine Table)

$F°_b := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 1950\ psi$

$l_e > 96 \qquad l_{eT} := 96 \qquad E_T := 1800000$

$COV_E := 0.25 \quad K_M := 1200 \qquad K_T := 1 - 1.645 \cdot (COV_E) \quad C_T := 1 + \frac{K_M \cdot l_{eT}}{K_T \cdot E_T} = 1.109$

(Table 4B)

$l_u := L \qquad \frac{L}{d} = 51.759 \qquad \geq 7 \qquad l_e := 0.9 \cdot l_u + 3 \cdot d = 29.956\ ft$

$E'_{min} := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = 731745.223\ psi$

$R_B := \sqrt{\frac{l_e \cdot d}{b^2}} = 14.586 \qquad F_{bE} := \frac{1.2 \cdot E_{min}}{{R_B}^2} = 3722.67\ psi$

$$C_L := \frac{1 + \left(\frac{F_{bE}}{F°_b}\right)}{1.9} - \sqrt{\left(\frac{1 + \left(\frac{F_{bE}}{F°_b}\right)}{1.9}\right)^2 - \frac{\left(\frac{F_{bE}}{F°_b}\right)}{0.95}} = 0.953$$

$f_t := \frac{P_l}{A} = 80\ psi \qquad f_b := \frac{M_l}{S} = 1557.656\ psi$

$F'_t := F_t \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i = 1350\ psi$

$F'_b := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 1857.506\ psi$

Check

$F'_t = 1350\ psi \quad > \quad f_t = 80\ psi$

$F'_b = 1857.506\ psi \quad > \quad f_b = 1557.656\ psi$

Design is adequate

$w_{ST} := 0.5 \cdot L_{rupperCorrection} \cdot T_w = 0.082\ \frac{kip}{ft}$

$w_{LT} := (D_{upperCorrection} + 0.5 \cdot L_{rupperCorrection}) \cdot T_w = 0.209\ \frac{kip}{ft}$

$\delta_{ST} := \frac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot \left(\frac{L}{3}\right)^4}{E \cdot I} = 0.058\ in \quad < \quad \Delta_{ST} := \frac{\left(\frac{L}{3}\right)}{360} = 0.347\ in$

$\delta_{LT} := \frac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot \left(\frac{L}{3}\right)^4}{E \cdot I} = 0.147\ in \quad < \quad \Delta_{LT} := \frac{\left(\frac{L}{3}\right)}{360} = 0.347\ in$

$\delta_{tot} := (1.5 \cdot \delta_{LT}) + \delta_{ST} = 0.278\ in \quad < \quad \Delta_{tot} := \frac{\left(\frac{L}{3}\right)}{240} = 0.521\ in$

Design is adequate

4x8s of Southern Pine Select
Structural used for Bottom Chord

Top Chord Design

SP Select Structural

4x8 Section

$F_t := 1350\ psi$ $\quad F_c := 1700\ psi$ $\quad F_b := 1950\ psi$

$b := 3.5\ in$ $\quad d := 7.25\ in$ $\quad A := b \cdot d = 25.375\ in^2$ $\quad I := \frac{b \cdot d^3}{12} = 111.148\ in^4$

$S := \frac{I}{\frac{d}{2}} = 30.661\ in^3$ $\quad E := 1800000\ psi$ $\quad E_{min} := 660000\ psi$

Adjustment Factors

$C_D := 1.15$ $\quad C_L := 1.0$ $\quad C_i := 1.0$ $\quad C_M := 1.0$ $\quad C_F := 1.0$ $\quad C_r := 1.0$

$C_t := 1.0$ $\quad C_{fu} := 1.0$ $\quad C_P := 1.0$ $\quad C_T := 1.0$

$f_c := \frac{P_u}{A} = 80\ psi$ $\quad f_b := \frac{M_u}{S} = 997.996\ psi$

$F'_c := F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i = 1955\ psi$

$F'_b := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 2242.5\ psi$

Check

$F'_c = 1955\ psi \quad > \quad f_c = 80\ psi$

$F'_b = 2242.5\ psi \quad > \quad f_b = 997.996\ psi$

Design is adequate

$R_B := \sqrt{\frac{\left(\frac{L}{3}\right) \cdot d}{b^2}} = 8.604$ $\quad F_{cStar} := \frac{F'_c}{C_P} = 1955\ psi$

$E'_{min} := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = 660000\ psi$

$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\frac{\frac{L}{3}}{d}\right)^2} = 1822.606\ psi$ $\quad F_{bE} := \frac{1.2 \cdot E'_{min}}{{R_B}^2} = 10698.523\ psi$

$c := 0.8$

$\alpha := \frac{1 + \frac{F_{cE}}{F_{cStar}}}{2 \cdot c} = 1.208$ $\quad \beta := \frac{\frac{F_{cE}}{F_{cStar}}}{c} = 1.165$

$$C_P := \alpha - \sqrt{\alpha^2 - \beta} = 0.666$$

$$F'_c := F_{cStar} \cdot C_P = 1302.54\ psi \quad > \quad f_c = 80\ psi$$

$$\frac{f_c}{F_{cE}} + \left(\frac{f_b}{F_{bE}}\right)^2 = 0.053 \quad < \quad 1$$

Design is adequate

$$w_{ST} := 0.5 \cdot L_{rupperCorrection} \cdot T_w = 0.082\ \frac{kip}{ft}$$

$$w_{LT} := \left(D_{upperCorrection} + 0.5 \cdot L_{rupperCorrection}\right) \cdot T_w = 0.209\ \frac{kip}{ft}$$

$$\delta_{ST} := \frac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot \left(\frac{L}{3}\right)^4}{E \cdot I} = 0.058\ in \quad < \quad \Delta_{ST} := \frac{\left(\frac{L}{3}\right)}{360} = 0.347\ in$$

$$\delta_{LT} := \frac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot \left(\frac{L}{3}\right)^4}{E \cdot I} = 0.147\ in \quad < \quad \Delta_{LT} := \frac{\left(\frac{L}{3}\right)}{360} = 0.347\ in$$

$$\delta_{tot} := \left(1.5 \cdot \delta_{LT}\right) + \delta_{ST} = 0.278\ in \quad < \quad \Delta_{tot} := \frac{\left(\frac{L}{3}\right)}{240} = 0.521\ in$$

4x8s of Southern Pine Select Structural used for Top Chord Design is adequate

Web Design

4x8 Section $F_t := 1350\ psi$ $F_c := 1700\ psi$ $F_b := 1950\ psi$

$b := 3.5\ in$ $d := 7.25\ in$ $A := b \cdot d = 25.375\ in^2$ $I := \frac{b \cdot d^3}{12} = 111.148\ in^4$

$S := \frac{I}{\frac{d}{2}} = 30.661\ in^3$ $E := 1800000\ psi$ $E_{min} := 660000\ psi$

Adjustment Factors

$C_D := 1.15$ $C_i := 1.0$ $C_M := 1.0$ $C_F := 1.0$ $C_t := 1.0$

$$f_t := \frac{P_i}{A} = 90.246\ psi$$

$$F'_t := F_t \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i = 1552.5\ psi$$

Check

$F'_t = 1552.5\ psi$ > $f_t = 90.246\ psi$

$\frac{f_t}{F'_t} = 0.058$ < 1 Design is adequate

$w_{ST} := 0.5 \cdot L_{rupperCorrection} \cdot T_w = 0.082\ \frac{kip}{ft}$

$w_{LT} := (D_{upperCorrection} + 0.5 \cdot L_{rupperCorrection}) \cdot T_w = 0.209\ \frac{kip}{ft}$

$\delta_{ST} := \frac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot \left(\frac{L}{3}\right)^4}{E \cdot I} = 0.058\ in$ < $\Delta_{ST} := \frac{\left(\frac{L}{3}\right)}{360} = 0.347\ in$

$\delta_{LT} := \frac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot \left(\frac{L}{3}\right)^4}{E \cdot I} = 0.147\ in$ < $\Delta_{LT} := \frac{\left(\frac{L}{3}\right)}{360} = 0.347\ in$

$\delta_{tot} := (1.5 \cdot \delta_{LT}) + \delta_{ST} = 0.278\ in$ < $\Delta_{tot} := \frac{\left(\frac{L}{3}\right)}{240} = 0.521\ in$

Design is adequate

Purlin Design

3x8 Southern Pine No. 2

$T_w := 40\ in$ $span := 126.5\ in$

Load Calculation	From Boise Cascade	
Suspended wood channel system	$D_1 := 2.5\ psf$	
18 gauge metal decking roofing	$D_2 := 3\ psf$	
Membrane	$D_3 := 0.35\ psf$	
1/2" osb plywood sheathing	$D_4 := 1.7\ psf$	
1" Rigid Insulation (3")	$D_5 := 1.5\ psf \cdot 3$	(West Roofing Systems Inc)
		(First American Roofing & Sidin

Standard Roof $L_1 := 20\ psf$ (ASCE)

$D_{upper} := D_1 + D_2 + D_3 + D_4 + D_5 = 12.05\ psf$ $S_{max} := P_s$

$L_{rupper} := L_1 = 20\ psf$ $L_{lower} := 0\ psf$ $p_{wmax} = -24.017\ psf$

$F_{wMax} := p_{wmax}$

$l_{upper} := \sqrt{(1\ ft)^2 + l^2} = 31.287\ ft$

$$p_{wmaxCor} := |p_{wmax}| \cdot \frac{l}{l_{upper}} = 24.005\ psf$$

$$D_{upperCorrection} := D_{upper} \cdot \frac{l}{l_{upper}} = 12.044\ psf \qquad L_{rupperCorrection} := L_{rupper} \cdot \frac{l}{l_{upper}} = 19.99\ psf$$

$$S_{maxCorrection} := S_{max} \cdot \frac{l}{l_{upper}} = 16.699\ psf \qquad W_{maxCorrection} := |F_{wMax}| \cdot \frac{l}{l_{upper}} = 24.005\ psf$$

$$w_{upper1} := T_w \cdot (1.4 \cdot D_{upperCorrection}) = 56.205\ plf$$

$$w_{upper2} := T_w \cdot (1.2 \cdot D_{upperCorrection} + 1.6 \cdot 0\ psf + \max(0.5 \cdot L_{rupperCorrection}, 0.5 \cdot S_{maxCorrection})) = 81.492\ plf$$

$$w_{upper3} := T_w \cdot (1.2 \cdot D_{upperCorrection} + \max(1.6 \cdot L_{rupperCorrection}, S_{maxCorrection}) + \max(L_{lower}, 0.5 \cdot W_{maxCorrection})) = 194.796\ plf$$

$$w_{upper4} := T_w \cdot (1.2 \cdot D_{upperCorrection} + W_{maxCorrection} + L_{lower} + \max(0.5 \cdot L_{rupperCorrection}, 0.3 \cdot S_{maxCorrection})) = 161.509\ plf$$

$$w_{upper5} := T_w \cdot (0.9 \cdot D_{upperCorrection} + W_{maxCorrection}) = 116.148\ plf$$

$$D_{upperCorrection} \cdot T_w = 0.04\ \frac{kip}{ft} \qquad S_{maxCorrection} \cdot T_w = 0.056\ \frac{kip}{ft}$$

$$W_{maxCorrection} \cdot T_w = 0.08\ \frac{kip}{ft}$$

3x8 Section $F_v := 175\ psi$ $F_t := 2350\ psi$ $F_b := 1950\ psi$

$$b := 2.5\ in \qquad d := 7.25\ in \qquad A := b \cdot d = 18.125\ in^2 \qquad I := \frac{b \cdot d^3}{12} = 79.391\ in^4 \qquad A_p := A$$

$$S := \frac{I}{\frac{d}{2}} = 21.901\ in^3 \qquad E := 1800000\ psi \qquad E_{min} := 660000\ psi$$

$$M_l := 2.04\ kip \cdot ft$$

$$w := T_w \cdot \left(1.2 \cdot \left(D_{upperCorrection} + \frac{A \cdot G \cdot 62.4\ pcf}{40\ in}\right) + \max(1.6 \cdot L_{rupperCorrection}, S_{maxCorrection}) + \max(L_{lower}, 0.5 \cdot W_{maxCorrection})\right) = 202.8$$

$$M_l := \frac{w \cdot span^2}{8} = 2.817\ kip \cdot ft \qquad V := \frac{w \cdot span}{2} = 1.069\ kip$$

Adjustment Factors

$C_D := 1.0$ $C_{fu} := 1.00$ $C_i := 1.0$ $C_M := 1.0$ $C_r := 1.00$

$C_t := 1.0$ $C_F := 1.0$ (Implemented with Southern Pine Table)

$$F'_v := F_v \cdot C_D \cdot C_M \cdot C_t \cdot C_i = 175\ psi$$

$$f_v := \frac{3 \cdot V}{2 \cdot A} = 88.466\ psi$$

$$f_v < F'_v$$

$$F^{\circ}_b := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 1950\ psi$$

$l_u := span$ $\frac{l_u}{d} = 17.448 \geq 7$ $l_e := 1.63 \cdot l_u + 3 \cdot d = 18.995\ ft$

$$E'_{min} := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = 660000\ psi$$

$R_B := \sqrt{\frac{l_e \cdot d}{b^2}} = 16.261$ $F_{bE} := \frac{1.2 \cdot E_{min}}{{R_B}^2} = 2995.278\ psi$

$$C_L := \frac{1 + \left(\frac{F_{bE}}{F^{\circ}_b}\right)}{1.9} - \sqrt{\left(\frac{1 + \left(\frac{F_{bE}}{F^{\circ}_b}\right)}{1.9}\right)^2 - \frac{\left(\frac{F_{bE}}{F^{\circ}_b}\right)}{0.95}} = 0.929$$

$f_t := \frac{P_l}{A} = 112\ psi$ $f_b := \frac{M_l}{S} = 1543.578\ psi$

$$F'_t := F_t \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i = 2350\ psi$$

$$F'_b := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 1811.418\ psi$$

Check

$F'_t = 2350\ psi$ > $f_t = 112\ psi$

$F'_b = 1811.418\ psi$ > $f_b = 1543.578\ psi$

Design is adequate

$$w_{ST} := 0.5 \cdot L_{rupperCorrection} \cdot T_w = 0.033\ \frac{kip}{ft}$$

$$w_{LT} := \left(D_{upperCorrection} + \frac{A \cdot G \cdot 62.4\ pcf}{40\ in} + 0.5 \cdot L_{rupperCorrection}\right) \cdot T_w = 0.08\ \frac{kip}{ft}$$

$\delta_{ST} := \frac{6.9 \cdot 10^{-3} \cdot w_{ST} \cdot (span)^4}{E \cdot I} = 0.034\ in$ < $\Delta_{ST} := \frac{(span)}{360} = 0.351\ in$

$\delta_{LT} := \frac{6.9 \cdot 10^{-3} \cdot w_{LT} \cdot (span)^4}{E \cdot I} = 0.083\ in$ < $\Delta_{LT} := \frac{(span)}{360} = 0.351\ in$

$\delta_{tot} := (1.5 \cdot \delta_{LT}) + \delta_{ST} = 0.158\ in$ < $\Delta_{tot} := \frac{(span)}{240} = 0.527\ in$

Design is adequate

Cabin Floor Plan - 1st Floor

1 beam through middle, then 1 joist design

Dead Loads

Hardwood floor (everywhere except Bathroom)	$D_1 := 4\ psf$
3/4" ceramic tile on 1/2" mortar (bathroom)	$D_2 := 16\ psf$
Subfloor 3/4"	$D_3 := 2.5\ psf$
1/2" gypsum board (hung)	$D_4 := 2.2\ psf$
Joists and Beam	TBD
2x4 16"o.c. framing (used to account for room dividers and ceilings above)	$D_5 := 1.1\ psf$

$Dead := D_1 + D_3 + D_4 + D_5 = 9.8\ psf$

$w_d := Dead$

Live Loads

Residential, all other areas $L_1 := 40\ psf$ $w_l := L_1$

$Live := L_1 = 40\ psf$

Beam

$L := 45\ ft + 6\ in$

$Span1 := 13\ ft + 1\ in$

$Span2 := 7\ ft + 4\ in$

$Span3 := \frac{L - (Span1 + Span2)}{2} = 12.542\ ft$

$Span4 := Span3 = 12.542\ ft$ $Span_{max} := Span3$

Joist

Dead Loads

Hardwood floor (everywhere except Bathroom) $D_1 := 4\ psf$

3/4" ceramic tile on 1/2" mortar (bathroom) $D_2 := 16\ psf$

$L := 15\ ft$

Subfloor 3/4" $D_3 := 2.5\ psf$

1/2" gypsum board (hung) $D_4 := 2.2\ psf$

Joists and Beam TBD

2x4 16"o.c. framing $D_5 := 1.1\ psf$
(used to account for room dividers and ceilings above)

$Dead := D_1 + D_3 + D_4 + D_5 = 9.8\ psf$ $\quad Dead_f := Dead$

$w_d := Dead$

Live Loads

Residential, all other areas $L_1 := 40\ psf$

$Live := L_1 = 40\ psf$

$Span := 15\ ft$ $\quad Spacing := 24\ in$ $\quad l := 15\ ft$

$T_w := 24\ in$

3x10 Select Structural

$b := 2.5\ in$ $\quad d := 9.25\ in$ $\quad I := \frac{b \cdot d^3}{12} = 164.886\ in^4$

$A := b \cdot d = 23.125\ in^2$ $\quad \gamma := 62.4\ \frac{lbf}{ft^3}$ $\quad G := 0.55$ $\quad E := 1800\ ksi$

$E_{min} := 660\ ksi$

$S := \frac{I}{\left(\frac{d}{2}\right)} = 35.651\ in^3$ $\quad A_j := A$ $\quad F_b := 1700\ psi$

$F_v := 750\ psi$

$C_D := 1$ $\quad C_m := 1$ $\quad C_F := 1$ $\quad C_r := 1.15$

$C_t := 1$ $\quad C_{fu} := 1$ $\quad C_i := 1$ $\quad C_L := 1$ $\quad Joist := \frac{\gamma \cdot G \cdot A}{T_w}$

$w := (1.2 \cdot (Dead + Joist) + 1.6 \cdot Live) \cdot T_w = 158.134\ plf$

$M := \frac{w \cdot L^2}{8} = 4.448\ kip \cdot ft$ $\quad V := \frac{w \cdot L}{2} = 1.186\ kip$

$F'_b := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 1955\ psi$

$l_u := span$ $\quad \frac{l_u}{d} = 13.676 \geq 7$ $\quad l_e := 1.63 \cdot l_u + 3 \cdot d = 19.495\ ft$

$E'_{min} := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = 660000\ psi$

$$R_B := \sqrt{\frac{l_e \cdot d}{b^2}} = 18.607 \qquad F_{bE} := \frac{1.2 \cdot E'_{min}}{{R_B}^2} = 2287.44 \ psi$$

$$C_L := \frac{1 + \left(\frac{F_{bE}}{F°_b}\right)}{1.9} - \sqrt{\left(\frac{1 + \left(\frac{F_{bE}}{F°_b}\right)}{1.9}\right)^2 - \frac{\left(\frac{F_{bE}}{F°_b}\right)}{0.95}} = 0.872$$

Design for Bending Stress

$$F'_b := F_b \cdot C_D \cdot C_m \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 1705.258 \ psi$$

$$f_b := \frac{M}{S} = 1497.015 \ psi \qquad f_b < F'_b$$

Design for Shear Stress

$$F'_v := F_v \cdot C_D \cdot C_m \cdot C_t = 750 \ psi$$

$$f_v := \frac{3 \cdot V}{2 \cdot A} = 76.93 \ psi \qquad f_v < F'_v$$

$$w_{lj} := 1.6 \cdot Live \cdot T_w \qquad w_{dj} := 1.2 \cdot (Dead + Joist) \cdot T_w$$

Design for Deflection

$$w_{stj} := w_{lj} \cdot 0.5 = 64 \ \frac{lbf}{ft}$$

$$w_{ltj} := w_{dj} + w_{lj} \cdot 0.5 = 94.134 \ \frac{lbf}{ft}$$

$$\delta_{stj} := \frac{5 \cdot w_{stj} \cdot L^4}{384 \cdot E \cdot I} = 0.246 \ in \qquad \Delta_{st} := \frac{L}{360} = 0.5 \ in$$

$$\delta_{ltj} := \frac{5 \cdot w_{ltj} \cdot L^4}{384 \cdot E \cdot I} = 0.361 \ in \qquad \Delta_{LT} := \frac{(L)}{360} = 0.5 \ in$$

$$\delta_{totj} := \delta_{stj} + \delta_{ltj} = 0.607 \ in \qquad \Delta_{tot} := \frac{L}{240} = 0.75 \ in$$

$$\delta_{stj} < \Delta_{st}$$

Adaquate

$$\delta_{totj} < \Delta_{tot}$$

3x10 joists of Southern Pine Select Structural spaced at 24" o.c. recomended to be used for the cabins' floor joists

Floor Beam
Beam Calculations
Beam

$L := 45\ ft + 6\ in$

$Span1 := 13\ ft + 1\ in$

$Span2 := 7\ ft + 4\ in$

$Span3 := \frac{L - (Span1 + Span2)}{2} = 12.542\ ft$

$Span4 := Span3 = 12.542\ ft$

$Span_{max} := Span1$

Max -Moment from all spans loading
Max +Moment from pattern loading (1 and 3)
Max Shear from adjacent loading (span 1 and 2)

$D_j := \frac{A \cdot G \cdot \gamma}{24\ in} = 2.756\ psf$

$Dead := D_1 + D_3 + D_4 + D_5 + D_j = 12.556\ psf$

$w_d := Dead$

Built up section of 3 3x12 Southern Pine Dense Select Structural

$G := 0.55$ $\rho_w := \frac{62.4\ lbf}{ft^3}$ $b := 2.5\ in$ $d := 11.25\ in$

$D_{self} := G \cdot \rho_w \cdot (3 \cdot b) \cdot d = 20.109\ \frac{lbf}{ft}$

$B := 30\ ft$

Tributary Width

$T_w := \frac{B}{2} = 15\ ft$

$w_{dro} := (Dead \cdot T_w + D_{self} + A_j \cdot G \cdot \gamma) = 0.214\ \frac{kip}{ft}$

$A := b \cdot d \cdot 3 = 84.375\ in^2$

$w_{lro} := (Live \cdot T_w) = 0.6\ \frac{kip}{ft}$

$w_{dead} := 1.2 \cdot (Dead \cdot T_w + D_{self} + A_j \cdot G \cdot \gamma) = 256.748\ \frac{lbf}{ft}$

$w_{live} := 1.6 \cdot Live \cdot T_w = 960\ \frac{lbf}{ft}$

$I := \frac{1}{12} \cdot 3 \cdot b \cdot d^3 = 889.893\ in^4$

$w := w_{dead} + w_{live} = 1216.748\ \frac{lbf}{ft}$

Design for Bending Stress

Moment

$M_n := 21.97\ kip \cdot ft$ $M_p := 18.84\ kip \cdot ft$ $V := 9.44\ kip$

$S := \frac{I}{\left(\frac{d}{2}\right)}$ $M := \max(M_n, M_p)$

$F_b := 1800\ psi$

$F_v := 175\ psi$

$E := 1800\ ksi$

$E_{min} := 660\ ksi$

$f_b := \frac{M}{S} = 1666.465\ psi$

$C_D := 1$ $C_m := 1$ $C_F := 1$ $C_r := 1$

$C_t := 1$ $C_{fu} := 1$ $C_i := 1$ $C_L := 1$

$F°_b := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 1800\ psi$

$l_u := Span_{max}$ $\frac{l_u}{d} = 13.956 \ge 7$ $l_e := 1.63 \cdot l_u + 3 \cdot d = 24.138\ ft$

$E'_{min} := E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T = 660000\ psi$

$R_B := \sqrt{\frac{l_e \cdot d}{b^2}} = 22.834$ $F_{bE} := \frac{1.2 \cdot E'_{min}}{{R_B}^2} = 1519.022\ psi$

$C_L := \frac{1 + \left(\frac{F_{bE}}{F°_b}\right)}{1.9} - \sqrt{\left(\frac{1 + \left(\frac{F_{bE}}{F°_b}\right)}{1.9}\right)^2 - \frac{\left(\frac{F_{bE}}{F°_b}\right)}{0.95}} = 0.739$ $C_L := 1.0$ from joists

$F'_b := F_b \cdot C_D \cdot C_m \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r = 1800\ psi$

$f_b < F'_b$

$\frac{F'_b}{f_b} = 1.08$

Design for Shear Stress

$$F'_v := F_v \cdot C_D \cdot C_m \cdot C_t \cdot C_i = 175\ psi$$

$$f_v := \frac{3 \cdot V}{2 \cdot A} = 167.822\ psi$$

$f_v < F'_v$ Adaquate

Design for Deflection

$$w_{st} := w_{live} \cdot 0.5 = 480\ \frac{lbf}{ft}$$

$$w_{lt} := w_{dead} + w_{live} \cdot 0.5 = 736.748\ \frac{lbf}{ft}$$

$$\delta_{st} := \frac{5 \cdot w_{st} \cdot Span_{max}^{\ 4}}{384 \cdot E \cdot I} = 0.198\ in \qquad \Delta_{st} := \frac{Span_{max}}{360} = 0.436\ in$$

$$\delta_{lt} := \frac{5 \cdot w_{lt} \cdot Span_{max}^{\ 4}}{384 \cdot E \cdot I} = 0.303\ in \qquad \Delta_{LT} := \frac{\left(Span_{max}\right)}{360} = 0.436\ in$$

$$\delta_{tot} := \delta_{st} + \delta_{lt} = 0.501\ in \qquad \Delta_{tot} := \frac{Span_{max}}{240} = 0.654\ in$$

$$\delta_{stj} < \Delta_{st}$$

$$\delta_{totj} < \Delta_{tot}$$

A built-up beam consisting of 3 3x12's of Southern Pine Dense Select Structural to be used

Column

$$A_t := \left(162.5\ in + 91.45\ in\right) \cdot 187.25\ in$$

$$D_{hardwood} := 4\ psf$$
$$D_{gypsum} := 2.2\ psf$$
$$D_{sub} := 2.5\ psf$$

$$L := 40\ psf$$

$$D_{joist} := \frac{2.5\ in \cdot 9.25\ in \cdot G \cdot \gamma}{24\ in} = 2.756\ psf$$

$$L_t := L \cdot A_t = 13.209\ kip$$

$$D_{beam} := \left(3 \cdot 2.5\ in \cdot 11.25\ in \cdot G \cdot \gamma\right) \cdot \left(162.5\ in + 91.45\ in\right) = 0.426\ kip$$

$$D_t := \left(D_{hardwood} + D_{gypsum} + D_{sub} + D_{joist}\right) \cdot A_t + D_{beam} = 4.209\ kip$$

$$P_c := 1.2 \cdot D_t + 1.6 \cdot L_t = 26.184\ kip$$

$l := 9.104\ ft$ $\quad K_e := 1$ $\quad l_e := l \cdot K_e = 9.104\ ft$

$F_c := 525\ psi$ $\quad E := 12000\ ksi$ $\quad E_{min} := 440\ ksi$ $\quad c := 0.8$

Column Section Selected: 6x6 Timber Southern Pine No. 2

$b := 5.5\ in$ $\quad d := 5.5\ in$ $\quad A := b \cdot d = 0.21\ ft^2$

Factors

$C_D := 1$ $\quad C_t := 1$ $\quad C_i := 1$ $\quad C_F := 1$ $\quad C_M := 1$ $\quad C_P := 1$ $\quad C_T := 1$

$F°_c := F_c \cdot C_D \cdot C_t \cdot C_M \cdot C_T = 525\ psi$

$E'_{min} := E_{min} \cdot C_t \cdot C_i \cdot C_T = 440000\ psi$

$f_c := \frac{P_c}{A} = 865.603\ psi$

$$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\frac{l_e}{d}\right)^2} = 916.691\ psi$$

$$\alpha := \frac{1 + \left(\frac{F_{cE}}{F°_c}\right)}{2 \cdot c} = 1.716 \qquad \beta := \frac{\frac{F_{cE}}{F°_c}}{c} = 2.183$$

$$C_P := \alpha - \sqrt[2]{\alpha^2 - \beta} = 0.843$$

$F'_c := F°_c \cdot C_P = 442.445\ psi$ $\quad 423.995 > 108.989$ $\quad f_c = 865.603\ psi$

$\lambda := \frac{l_e}{d} = 19.863$ $\quad 18.192 < 50$

$f_{cnet} := \frac{P_c}{A} = 865.603\ psi$ $\quad 355.473 < 477.75$ $\quad F°_c = 525\ psi$

6x6 Timber Southern Pine No. 2

$M := \frac{w_{pw} \cdot l^2}{8} = 0.302\ kip \cdot ft$ $\quad E := 1400\ ksi$ $\quad E_{min} := 510\ ksi$

Factors

$C_D := 1$ $\quad C_t := 1$ $\quad C_i := 1$ $\quad C_F := 1$ $\quad C_M := 1$ $\quad C_P := 1$ $\quad C_T := 1$

$C_r := 1.15$ $\quad c := 0.8$

$f_b := \frac{M}{S} = 958.188\ psi$ $\quad f_c := \frac{P_c}{A} = 120.698\ psi$

$E'_{min} := E_{min} \cdot C_t \cdot C_i \cdot C_T = 510000\ psi$

$F°_b := F_b \cdot C_D \cdot C_t \cdot C_i \cdot C_F \cdot C_M \cdot C_{fu} \cdot C_r = 1150\ psi$

$F'_b := F°_b \cdot C_L = 1150\ psi$ $\quad 1150 > 958.188$ $\quad f_b = 958.188\ psi$

$l_u := 15\ ft$ $\quad \frac{l_u}{d} = 32.727 > 7$

$l_e := 1.63 \cdot l_u + 3 \cdot d = 25.825\ ft$

$R_B := \sqrt[2]{\frac{l_e \cdot d}{b^2}} = 27.523$

$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\frac{l_e}{d}\right)^2} = 132.046\ psi$ $\quad > $ $\quad f_c := \frac{P_c}{A} = 120.698\ psi$

$F°_c := F_c \cdot C_D \cdot C_t \cdot C_M \cdot C_T = 1400\ psi$ $\quad 1400 > 120.698$ $\quad f_c = 120.698\ psi$

$F'_c := F°_c \cdot C_T = 1400\ psi$ $\quad 1400 > 120.698$

Same design for upper walls.

Interior Non-Load Bearing

Vertical load

2x8 ceiling joists @16" o.c., R-49 insulation, 1/2"gypsum board $\quad D_c := 7\ psf$

1/2"gypsum board $\quad D_g := 2.2\ psf$

$D_{ceiling} := D_c + D_g = 9.2\ psf$

$L_{ceiling} := 10\ psf$

$w := 1.2 \cdot D_{ceiling} + 1.6 \cdot L_{ceiling} = 27.04\ psf$ $\quad T_w := 9.2\ ft$ $\quad Spacing := 24\ in$

$P_c := w \cdot T_w \cdot Spacing$

Check 2x4 at 24"

$b := 1.5\ in \quad d := 5.5\ in \qquad A := b \cdot d = 8.25\ in^2 \qquad I := \frac{b \cdot d^3}{12} = 20.797\ in^4$

$S := \frac{I}{d} = 3.781\ in^3 \qquad F_b := 1100\ psi \qquad F_c := 1450\ psi \qquad E_{min} := 510\ ksi$

$l_u := 12\ ft \qquad \frac{l_u}{d} = 26.182 > 7 \qquad E'_{min} := E_{min} \cdot C_t \cdot C_i \cdot C_T$

$l_e := 1.63 \cdot l_u + 3 \cdot d = 20.935\ ft$

$R_B := \sqrt[2]{\frac{l_e \cdot d}{b^2}} = 24.781$

$F_{cE} := \frac{0.822 \cdot E'_{min}}{\left(\frac{l_e}{d}\right)^2} = 200.937\ psi \qquad > \qquad f_c := \frac{P_c}{A} = 60.307\ psi$

$F^*_c := F_c \cdot C_D \cdot C_t \cdot C_M \cdot C_T = 1450\ psi \qquad 1400 > 120.698 \qquad f_c = 60.307\ psi$

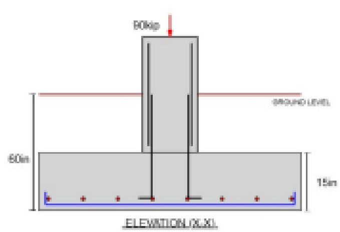
$F'_c := F^*_c \cdot C_T = 1450\ psi \qquad 1400 > 120.698$

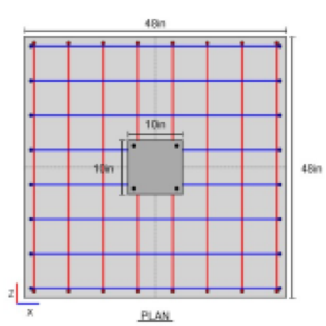
Studs for non load-bearing walls designed as 2x4 Southern Pine No. 2 at 24" o.c.
Studs for lower floor to be 2x6 in order to hide column

Spread Footing

SkyCiv used for preliminary design

$P_c := 26.184\ kip + 5.5\ in \cdot 5.5\ in \cdot G \cdot \gamma \cdot 8.17\ ft \cdot 1.2 = 26.255\ kip$





NAME	LENGTH	WIDTH	HEIGHT	DEPTH	REINF X	REINF Z	COL
FOOTING 1	48	48	15	60	8#4	8#4	4#4

Upper Strip Footing

$$D := 292.817\ plf + \frac{4.21\ kip}{2 \cdot 12\ in} = 2.398\ \frac{kip}{ft}$$

$$L := 80\ plf + \frac{13.21\ kip}{2 \cdot 12\ in} = 6.685\ \frac{kip}{ft}$$

$$L_r := 311.924\ plf \qquad L_r = 0.312\ \frac{kip}{ft}$$

$$Sn := 202.167\ plf \qquad Sn = 0.202\ \frac{kip}{ft}$$

$$W := 290.607\ plf \qquad W = 0.291\ \frac{kip}{ft}$$

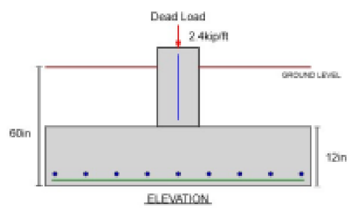
$$w_{upper1} := (1.4 \cdot D) = 3.357\ \frac{kip}{ft}$$

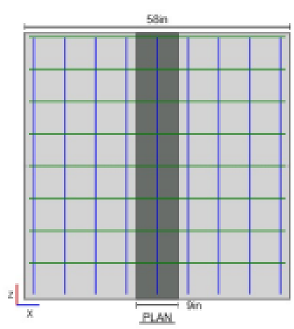
$$w_{upper2} := (1.2 \cdot D + 1.6 \cdot L + \max(0.5 \cdot L_r, 0.5 \cdot Sn)) = 13.729\ \frac{kip}{ft}$$ controls design

$$w_{upper3} := (1.2 \cdot D + \max(1.6 \cdot L_r, Sn) + \max(L, 0.5 \cdot W)) = 10.061\ \frac{kip}{ft}$$

$$w_{upper4} := (1.2 \cdot D + W + L + \max(0.5 \cdot L_r, 0.3 \cdot Sn)) = 10.009\ \frac{kip}{ft}$$

$$w_{upper5} := (0.9 \cdot D + W) = 2.449\ \frac{kip}{ft}$$





NAME	WIDTH	HEIGTH	DEPTH	MAIN R	SEC. R	WALL	DOWEL
SF1	57.99999999999994	12	60	#5@7	9#5	#4@14	-

Strip Footing Retaining Wall

$$D := 292.817\ plf + \frac{4.21\ kip}{2 \cdot 12\ in} + 150\ pcf \cdot (4.5\ ft \cdot 1.5\ ft + 11.3\ ft \cdot 1\ ft) = 5.105\ \frac{kip}{ft}$$

$$L := 80\ plf + \frac{13.21\ kip}{2 \cdot 12\ in} = 6.685\ \frac{kip}{ft}$$

$$L_r = 0.312\ \frac{kip}{ft}$$

$$Sn = 0.202\ \frac{kip}{ft}$$

$$W = 0.291\ \frac{kip}{ft}$$

$\phi' := 25$ $\quad c' := 1.45\ psi$ (estimated values used) $\quad \gamma' := 55.5\ \frac{lbf}{ft^3}$

Wall height + soil to base height $\quad Depth := 9.1\ ft + 4.6\ ft \quad B := 4.5\ ft \quad 2\ ft + 1\ ft + 1.5\ ft = 4.5\ ft$

$N_c := 25.1 \quad N_q := 12.7 \quad N_\gamma := 9.2 \quad \sigma'_{z0} := \gamma' \cdot Depth$

$$q_n := c' \cdot N_c + \sigma'_{z0} \cdot N_q + 0.5 \cdot \gamma' \cdot B \cdot N_\gamma = 111.432\ psi$$

$$q_{maxLoad} := q_n \cdot B = 72.208\ \frac{kip}{ft}$$

$$w_{upper1} := (1.4 \cdot D) = 7147.444\ plf$$

$$w_{upper2} := (1.2 \cdot D + 1.6 \cdot L + \max(0.5 \cdot L_r, 0.5 \cdot Sn)) = 16978.342\ plf$$

$$w_{upper3} := (1.2 \cdot D + \max(1.6 \cdot L_r, Sn) + \max(L, 0.5 \cdot W)) = 13310.459\ plf$$

$$w_{upper4} := (1.2 \cdot D + W + L + \max(0.5 \cdot L_r, 0.3 \cdot Sn)) = 13257.949\ plf$$

$$w_{upper5} := (0.9 \cdot D + W) = 4885.392\ plf$$

$$w_{max} := \max(w_{upper1}, w_{upper2}, w_{upper3}, w_{upper4}, w_{upper5}) = 16.978\ \frac{kip}{ft}$$

$$w_{max} = 16.978\ \frac{kip}{ft} \quad < \quad q_{maxLoad} = 72.208\ \frac{kip}{ft}$$

Significant factor of safety used to account for only one side having full soil depth and limited soil data. Estimations of rebar reinforcements will be made based off of the tested strip footing.

Concrete Flooring

5" concrete slab on grade will be used for both interior and exterior locations.

Figure 25: Cabin Loading Calculations

REFERENCES	CALCULATIONS	RESULTS

ACI 318-19 Spread Footing Design

SkyCiv

Dimensions

Symbol	Description	Value
-	Footing Name	FOOTING 1
-	Material	Reinforced Concrete
B	Footing Width	4
L	Footing Length	4
Df	Foundation Base Depth	5
H	Base Height	15
c_1	Column Length	10
c_2	Column Width	10
col_{height}	Column Height	4
x_{offset}	Column X Offset	0
z_{offset}	Column Z Offset	0
pof_{col}	Punching Overhang Factor	1

Concrete Properties

Symbol	Description	Value
f'_c	Concrete Compressive Strength	4
w_c	Concrete Unit Weight	150
d_{agg}	Max. Size of Aggregate	1

Reinforcement

Symbol	Description	Value
C_c	Reinforcement Clear Cover	3
f_y	Reinforcement Yield Strength	60
-	Reinforcement Type	Normal
$d_{b,x}$	Footing Bar Size (X Direction)	#4
n_x	Footing Bars in X Direction (n_x)	8
-	X Bars with Hooks?	True
$d_{b,z}$	Footing Bar Size (Z Direction)	#4
n_z	Footing Bars in Z Direction (n_z)	8
-	Z Bars with Hooks?	True
$d_{b,col}$	Dowel Bar Size	#4
$n_{x,col}$	Dowel Bars in X Direction	2
$n_{z,col}$	Dowel Bars in Z Direction	2
-	Dowels with Hooks?	True

Geotechnical Parameters

Symbol	Description	Value
q_a	Allowable Bearing Capacity	2
φ	Foundation-Soil Friction Angle	25
$SF_{overturning}$	Min. Overturning Safety Factor	1.5
$SF_{sliding}$	Min. Sliding Safety Factor	1.5

Load Cases

Name	Case	N (kip)	Mx (kip-ft)	Mz (kip-ft)	Vx (kip)	Vz (kip)
Total DL	DL	26.7	0	0	0	0
Total LL	LL	0	0	0	0	0
Total Lr	Lr	0	0	0	0	0
Total W/S	W/S	0	0	0	0	0

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Name	Case	N (kip)	Mx (kip-ft)	Mz (kip-ft)	Vx (kip)	Vz (kip)
Dead	DL	26.3	0	0	0	0

Load Combinations

Name	Dead Load Factor	Live Load Factor	Roof Live Load Factor	Wind / Seismic Factor
S1	1.0	0.0	0.0	0.0
U1	1.2	1.6	0.5	0.0

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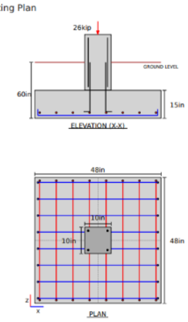
REFERENCES	CALCULATIONS	RESULTS

Spread Footing Plan



NAME	LENGTH	WIDTH	HEIGHT	DEPTH	REINF X	REINF Z	COL
FOOTING 1	48	48	15	60	8#4	8#4	4#4

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REFERENCES	CALCULATIONS	RESULTS

Footing Stability

Maximum Eccentricity Check

Critical Load Combination for Maximum Eccentricity on the Base: S1=1DL + 0LL + 0Lr + 0W/S

Eccentricity

$$\Sigma M_x = -(P \times offset_z) + M_x - (V_z \times D_f) = -(26.717 \times 0) + 0 - (0 \times 60) = 0 \text{ kip*in}$$

$$\Sigma M_z = -(-P \times offset_x) + M_z + (V_x \times D_f) = -(-26.717 \times 0) + 0 + (0 \times 60) = 0 \text{ kip*in}$$

$$\Sigma P = P + \Delta q \times A = 26.7167 + 0 \times 2304 = 26.7167 \text{ kip}$$

$$ratio_e = \frac{\left|\frac{\Sigma M_x}{\Sigma P}\right|}{\left(\frac{L}{6}\right)} + \frac{\left|\frac{\Sigma M_z}{\Sigma P}\right|}{\left(\frac{B}{6}\right)} = \frac{\left|\frac{0}{26.72}\right|}{\left(\frac{48}{6}\right)} + \frac{\left|\frac{0}{26.72}\right|}{\left(\frac{48}{6}\right)} = 0$$

Eccentricity Check

ratio_e <= 1.0
The resultant is within the middle third portion of the base.

PASS

Soil Bearing Pressure Check

Critical Load Combination for Maximum Soil Stress: S1=1DL + 0LL + 0Lr + 0W/S

$$q_{s_{max}} = \frac{\Sigma P}{(B \times L)} + \left(\frac{6 \times \Sigma M_x}{((B)^2 \times L)}\right) + \left(\frac{6 \times \Sigma M_z}{((L)^2 \times B)}\right) = \frac{26.7167}{(48 \times 48)} + \left(\frac{6 \times 0}{((48)^2 \times 48)}\right) + \left(\frac{6 \times 0}{((48)^2 \times 48)}\right)$$

$$q_{s_{max}} = 0.0116 \text{ ksi}$$

Pedestal weight $G_{pedestal} =$ 0.42kip

Soil Pressures and Eccentricities Summary

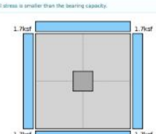
Name	ΣP (kip)	ΣMx (kip-ft)	ez (in)	ΣMz (kip-ft)	ex (in)	ratio_e	qs_max (ksf)	qs_mean (ksf)	qs_max / qs_mean	qs_min (ksf)
S1	26.72	0	0	0	0	0	1.67	1.67	1	1.67

Maximum Soil Stress $q_{s_{max}} =$ 1.67ksf
Mean Soil Stress $q_{s_{mean}} =$ 1.67ksf
Allowable Soil Bearing Capacity $q_a =$ 2.00ksf

Soil Bearing Capacity Check

qa >= q_{max}
The maximum soil stress is smaller than the bearing capacity.

PASS



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REFERENCES	CALCULATIONS	RESULTS

Overturning Check

Overturning about X-Axis

Critical load combination for overturning: S1=1DL + 0LL + 0Lr + 0W/S

No moments about X-Axis were considered.

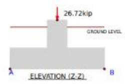
Overturning about Z-Axis

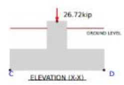
Critical load combination for overturning: S1=1DL + 0LL + 0Lr + 0W/S

No moments about Z-Axis were considered.

The footing weight was not considered.

The overburden soil weight was not considered.





Overturning Checks Summary

LC	Point	Mres_x (kip-ft)	Mover_x (kip-ft)	SF_x	Point	Mres_z (kip-ft)	Mover_z (kip-ft)	SF_z
S1	A	53.43	0	>1000	C	53.43	0	>1000

Overturning Safety Factor $SF_{overturning} =$ >1000.0
Minimum Safety Factor $SF_{min} =$ 1.5

SF >= 1.5
The foundation is safe against overturning.

PASS

Sliding Check

Critical load combination for sliding: S1=1DL + 0LL + 0Lr + 0W/S

No shear forces were considered.

Sliding Safety Factor $SF_{sliding} =$ >1000.0
Minimum Safety Factor $SF_{min} =$ 1.5

SF >= 1.5

PASS

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The foundation is safe against sliding.

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REFERENCES	CALCULATIONS	RESULTS

Footing Strength

$$d_x = H - c - \frac{db_x}{2} = 15 - 3 - \frac{0.5}{2} = 11.75 \text{ in}$$

$$d_z = H - c - db_x - \frac{db_z}{2} = 15 - 3 - 0.5 - \frac{0.5}{2} = 11.25 \text{ in}$$

$$\rho_x = \frac{n_x \times \left(\frac{\pi \times (db_x)^2}{4}\right)}{(B \times d_x)} \times 1000 = \frac{8 \times \left(\frac{3.14 \times (0.5)^2}{4}\right)}{(48 \times 11.75)} \times 1000 = 2.79 \text{ /1000}$$

$$\rho_z = \frac{n_z \times \left(\frac{\pi \times (db_z)^2}{4}\right)}{(L \times d_z)} \times 1000 = \frac{8 \times \left(\frac{3.14 \times (0.5)^2}{4}\right)}{(48 \times 11.25)} \times 1000 = 2.91 \text{ /1000}$$

One Way Shear Check in X

22.5.5.1

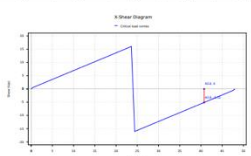
$$V_{cx} = 8 \times 0.1 \times (\rho_x)^{\frac{1}{3}} \times rf \times B \times d_x = 8 \times 0.1 \times (2.79)^{\frac{1}{3}} \times 63.25 \times 48 \times 11.75 = 40149.43 \text{ lb}$$

$$l_{l1} = \frac{(L - c_1)}{2} - d_x + offset_x = \frac{(48 - 10)}{2} - 11.75 + 0 = 7.25 \text{ in}$$

$$l_{l2} = \frac{(L - c_1)}{2} - d_x - offset_x = \frac{(48 - 10)}{2} - 11.75 - 0 = 7.25 \text{ in}$$

Maximum Shear X-Axis Summary

LC	dist 1 (in)	Shear 1 (kip)	dist 2 (in)	Shear 2 (kip)
U1	7.25	5.11	40.75	-5.11



Critical Load Combination: U1=1.2DL + 1.6LL + 0.5Lr + 0W/S

Maximum Shear that can be Resisted at the Section $V_{cx} =$ 40.15kip
Shear Strength Reduction Factor $\phi =$ 0.75
Maximum Factored Shear Force at the Section $V_u =$ -5.11kip

$$Shear_{XRatio} = \frac{V_u}{\phi \times V_c} = \frac{5.11}{0.75 \times 40.15}$$

$$Shear_{XRatio} = 0.17$$

UTILITY: 0.17

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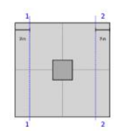


One Way Shear Check in Z

22.5.5.1

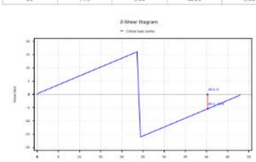
$$V_{cz} = 8 \times 0.1 \times (\rho_z)^{\frac{1}{3}} \times rf \times L \times d_z = 8 \times 0.1 \times (2.91)^{\frac{1}{3}} \times 63.25 \times 48 \times 11.25 = 39002.2 \text{ lb}$$

$$b_{l1} = \frac{(B - c_2)}{2} - d_z + offset_z = \frac{(48 - 10)}{2} - 11.25 + 0 = 7.75 \text{ in}$$

$$b_{l2} = \frac{(B - c_2)}{2} - d_z - offset_z = \frac{(48 - 10)}{2} - 11.25 - 0 = 7.75 \text{ in}$$

Max Shear Z-Axis Summary

LC	dist 1 (in)	Shear 1 (kip)	dist 2 (in)	Shear 2 (kip)
U1	7.75	5.44	40.25	-5.44



Critical Load Combination: U1=1.2DL + 1.6LL + 0.5Lr + 0W/S

Maximum Shear that can be Resisted at the Section $V_{cz} =$ 39.00kip
Shear Strength Reduction Factor $\phi =$ 0.75
Maximum Factored Shear Force at the Section $V_u =$ -5.44kip

$$Shear_{ZRatio} = \frac{V_u}{\phi \times V_c} = \frac{5.44}{0.75 \times 39.00}$$

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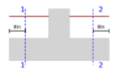
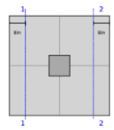
$$Shear_{ZRatio} = 0.19$$

UTILITY: 0.19





Punching Shear Check

Ultimate Soil Pressure Summary

Name	ΣP (kip)	ΣM_x (kip-ft)	e_z (in)	ΣM_z (kip-ft)	e_x (in)	$ratio_e$	qs_{max} (ksf)	qs_{mean} (ksf)	$\frac{qs_{max}}{qs_{mean}}$	qs_{min} (ksf)
U1	32.06	0	0	0	0	0	2	2	1	2

Critical Load Combination: U1=1.2DL + 1.6LL + 0.5Lr + 0W/S

Maximum Soil Stress	$qu_{max} =$	2.00ksf
Mean Soil Stress	$qu_{mean} =$	2.00ksf

$$d_p = \frac{(d_x + d_z)}{2} = \frac{(11.75 + 11.25)}{2} = 11.5 \text{ in}$$

Ratio of Long to Short Dimensions (Column)	$\beta =$	1.00
Modification Factor for Lightweight Concrete	$\lambda =$	1.00
Perimeter of Critical Section	$bo =$	86.00in
X-Axis Distance	$b1 =$	21.50in
Z-Axis Distance	$b2 =$	21.50in
Equivalent Polar Moment of Inertia	$Jc_x =$	81644in⁴
Equivalent Polar Moment of Inertia	$Jc_z =$	81644in⁴
Factor for the Moment Fraction	$\gamma_{vx} =$	0.40
Factor for the Moment Fraction	$\gamma_{vz} =$	0.40
X-axis Largest Distance to the Extreme Fiber	$c_x =$	10.75in
Z-axis Largest Distance to the Extreme Fiber	$c_z =$	10.75in

$$v_c = min(4, 2 + \frac{4}{\beta}, 2 + \frac{\alpha_s \times d_p}{bo}) \times \lambda \times rf$$

$$v_c = min(4, 2 + \frac{4}{1}, 2 + \frac{40 \times 11.5}{86.00}) \times 1.00 \times 63.25$$

$$v_c = 252.98 psi$$

22.6.5.2

$$v_{un} = \frac{qu_{n} \times (B \times L - (d_1 \times d_2))}{(bo \times d_p)} = \frac{13.91 \times (48 \times 48 - (21.5 \times 21.5))}{(86 \times 11.5)} = 25.91 \text{ psi}$$

22.6.5.2 $$v_{umx} = \frac{\gamma_{vx} \times |Mu_x| \times cx}{Jc_x} = \frac{0.4 \times |0| \times 10.75}{81644.01} = 0 \text{ psi}$$

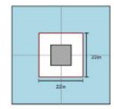
22.6.5.2 $$v_{umz} = \frac{\gamma_{vz} \times |Mu_z| \times cz}{Jc_z} = \frac{0.4 \times |0| \times 10.75}{81644.01} = 0 \text{ psi}$$

22.6.5.2 $$v_u = v_{un} + v_{umx} + v_{umz} = 25.91 + 0 + 0 = 25.91 \text{ psi}$$

$$Punching_{Ratio} = \frac{v_u}{\phi \times v_c} = \frac{25.91}{0.75 \times 252.98}$$

$$Punching_{Ratio} = 0.14$$

UTILITY: 0.14



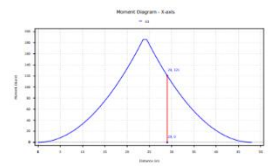
Bending Check (X)

13.2.7.1 $$l_{21} = \frac{(L - c_1)}{2} + offset_x = \frac{(48 - 10)}{2} + 0 = 19 \text{ in}$$

13.2.7.1 $$l_{22} = \frac{(L - c_1)}{2} - offset_x = \frac{(48 - 10)}{2} - 0 = 19 \text{ in}$$

Max Moment X-Axis Summary

LC	dist 1 (in)	Moment 1 (kip-ft)	dist 2 (in)	Moment 2 (kip-ft)
U1	19	10.05	29	10.05



Critical Load Combination: U1=1.2DL + 1.6LL + 0.5Lr + 0W/S

Maximum Moment at the Critical Section	$M_{ux} =$	10.05kip-ft
Depth of Equivalent Rectangular Compressive Stress Block	$\beta_1 =$	0.85

Modulus of Elasticity of Reinforcement	$E_s =$	29000.00ksi
Bending Strength Reduction Factor (Tension Controlled)	$\phi =$	0.90

Calculation of the Strength Coefficient

Eq. 4.10 CRSI Manual $$R_{ux} = \frac{M_{ux}}{\left(\varphi \times B \times (d_x)^2\right)} = \frac{120545.6}{\left(0.9 \times 48 \times (11.75)^2\right)} = 20.211 \text{ psi}$$

Reinforcement Area Check (X)

Eq. 4.9 CRSI Manual $$As_{reqx} = \frac{0.85 \times f_c \times B \times d_x}{f_y} \times \left(1 - \sqrt{1 - \frac{2 \times R_{ux}}{(0.85 \times f_c)}}\right) = \frac{0.85 \times 4000 \times 48 \times 11.75}{60000} \times \left(1 - \sqrt{1 - \frac{2 \times 20.21}{(0.85 \times 4000)}}\right)$$

$$As_{reqx} = 0.19 \text{ in}^2$$

7.6.1.1 $$As_{minx} = 0.0018 \times H \times B = 0.0018 \times 15 \times 48$$

$$As_{minx} = 1.3 \text{ in}^2$$

Eq. 4.9 CRSI Manual $$As_{maxx} = \frac{0.85 \times \beta_1 \times f_c \times B \times d_x}{f_y} \times \left(\frac{0.003}{\left(\left(\frac{f_y}{E_s}\right) + 0.006\right)}\right) = \frac{0.85 \times 0.85 \times 4000 \times 48 \times 11.75}{60000} \times \left(\frac{0.003}{\left(\left(\frac{60000}{29000000}\right) + 0.006\right)}\right)$$

$$As_{maxx} = 10.1 \text{ in}^2$$

$$As_{provx} = \frac{n_x \times \pi \times (db_x)^2}{4} = \frac{8 \times 3.14 \times (0.5)^2}{4}$$

$$As_{provx} = 1.57 \text{ in}^2$$

Reinforcement Area (X)

As_{provx} = 1.571 > As_{reqx} = 0.191
Provided reinforcement area > required reinforcement area.

PASS

Minimum Reinforcement Area (X)

As_{provx} = 1.571 > As_{minx} = 1.296
Provided reinforcement area > minimum reinforcement area.

PASS

Maximum Reinforcement Area (X)

As_{provx} = 1.571 < As_{maxx} = 10.1
Provided reinforcement area < maximum reinforcement area due to ductility.

PASS

Calculation of the Maximum Resisting Moment (X)

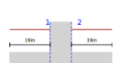
Eq. 4.3 CRSI Manual $$M_{nx} = As_{provx} \times f_y \times \left(d_x - \frac{(As_{provx} \times f_y)}{(1.7 \times B \times f_c)}\right) = 1.57 \times 60000 \times \left(11.75 - \frac{(1.57 \times 60000)}{(1.7 \times 48 \times 4000)}\right) = 1080197.43 \text{ lb-in}$$

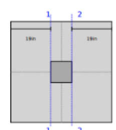
Nominal Flexural Strength	$M_{nx} =$	90.02kip-ft

$$Bending_{XRatio} = \frac{M_{ux}}{\phi \times M_{nx}} = \frac{120.55}{0.9 \times 1080.20}$$

$$Bending_{XRatio} = 0.12$$

UTILITY: 0.12





REFERENCES	CALCULATIONS	RESULTS

Bending Check Z-Axis

13.2.7.1

$$b_{l1} = \frac{(B - c_1)}{2} + offset_z = \frac{(48 - 10)}{2} + 0 = 19\ \text{in}$$

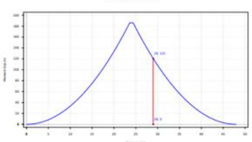
13.2.7.1

$$b_{l2} = \frac{(B - c_1)}{2} - offset_z = \frac{(48 - 10)}{2} - 0 = 19\ \text{in}$$

Max Moment Z-Axis Summary

LC	dist 1 (in)	Moment 1 (kip-ft)	dist 2 (in)	Moment 2 (kip-ft)
U1	19	10.05	29	10.05

Moment Diagram - (Z)



Critical Load Combination: U1=1.2DL + 1.6LL + 0.5Lr + 0W/S

Maximum Moment at the Critical Section	$M_{uz} =$	10.05kip-ft
Depth of Equivalent Rectangular Compressive Stress Block	$\beta_1 =$	0.85
Modulus of Elasticity of Reinforcement	$E_s =$	29000.00ksi
Bending Strength Reduction Factor (Tension Controlled)	$\phi =$	0.90
Ratio of Long to Short Footing Dimensions	$\beta =$	1.00

Calculation of the Strength Coefficient

Eq. 4.10 CRSI Manual

$$R_{nz} = \frac{M_{uz}}{\left(\varphi \times L \times (d_z)^2\right)} = \frac{120545.6}{\left(0.9 \times 48 \times (11.25)^2\right)} = 22.048\ \text{psi}$$

Reinforcement Area Check (Z)

Eq. 4.9 CRSI Manual

$$As_{reqz} = \frac{0.85 \times f_c \times L \times d_z}{f_y} \times \left(1 - \sqrt{1 - \frac{2 \times R_{nz}}{(0.85 \times f_c)}}\right) = \frac{0.85 \times 4000 \times 48 \times 11.25}{60000} \times \left(1 - \sqrt{1 - \frac{2 \times 22.05}{(0.85 \times 4000)}}\right)$$

$$As_{reqz} = 0.2\ \text{in}^2$$

Reinforcement Distribution Factor

R13.3.3.3

$$\gamma_s = \frac{2 \times \beta}{(\beta + 1)} = \frac{2 \times 1}{(1 + 1)}$$

$$\gamma_s = 1$$

R13.3.3.3

$$As_{reqzs} = As_{reqz} \times \gamma_s = 0.2 \times 1$$

$$As_{reqzs} = 0.2\ \text{in}^2$$

7.6.1.1

$$As_{minz} = 0.0018 \times H \times L = 0.0018 \times 15 \times 48$$

$$As_{minz} = 1.3\ \text{in}^2$$

Eq. 4.9 CRSI Manual

$$As_{maxz} = \frac{0.85 \times \beta_1 \times f_c \times L \times d_z}{f_y} \times \left(\frac{0.003}{\left(\left(\frac{f_y}{E_s}\right) + 0.006\right)}\right) = \frac{0.85 \times 0.85 \times 4000 \times 48 \times 11.25}{60000} \times \left(\frac{0.003}{\left(\left(\frac{60000}{29000000}\right) + 0.006\right)}\right)$$

$$As_{maxz} = 9.67\ \text{in}^2$$

$$As_{provz} = \frac{n_z \times \pi \times (db_z)^2}{4} = \frac{8 \times 3.14 \times (0.5)^2}{4}$$

$$As_{provz} = 1.57\ \text{in}^2$$

Reinforcement Area (Z)

As_{provz} = 1.571 > As_{reqx} = 0.199
Provided reinforcement area > required reinforcement area. — PASS

Minimum Reinforcement Area (Z)

As_{provz} = 1.571 > As_{minz} = 1.296
Provided reinforcement area > minimum reinforcement area. — PASS

Maximum Reinforcement Area (Z)

As_{provz} = 1.571 < As_{maxx} = 9.67
Provided reinforcement area < maximum reinforcement area due to ductility. — PASS

Calculation of the Maximum Resisting Moment (Z). It considers the Reinforcement Distribution Factor.

Eq. 4.3 CRSI Manual

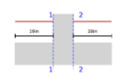
$$M_{nz} = \frac{As_{provz} \times f_y \times \left(d_z - \frac{(As_{provz} \times f_y)}{(1.7 \times L \times f_c \times \gamma_s)}\right)}{\gamma_s} = \frac{1.57 \times 60000 \times \left(11.25 - \frac{(1.57 \times 60000)}{(1.7 \times 48 \times 4000 \times 1)}\right)}{1} = 1033073.54\ \text{lb-in}$$

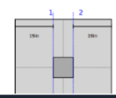
Nominal Flexural Strength	$M_{nz} =$	86.09kip-ft

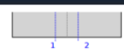
$$Bending_{ZRatio} = \frac{M_{uz}}{\phi \times M_{nz}} = \frac{120.55}{0.9 \times 1033.07}$$

$$Bending_{ZRatio} = 0.13$$

UTILITY: 0.13







REFERENCES	CALCULATIONS	RESULTS

Transfer of Forces

Vertical Transfer - Compression Check

22.8.3.1 Bearing Strength Calculation

$$A_1 = c_1 \times c_2 = 10 \times 10 = 100\ \text{in2}$$

$$A_2 = (min(l_{21}, 2 \times H) + c_1 + min(l_{22}, 2 \times H)) \times (min(b_{21}, 2 \times H) + (c_2 + min(b_{22}, 2 \times H))$$

$$A_2 = (min(19.00, 2 \times 15.00) + (10.00) + min(19.00, 2 \times 15.00)) \times (min(19.00, 2 \times 15.00) + (10.00) + min(19.00, 2 \times 15.00))$$

$$A_2 = 2304.00 in2$$

Table 22.8.3.2

$$B_n = min\left(\sqrt{\frac{A_2}{A_1}}, 2\right) \times 0.85 \times f_c \times A_1$$

$$B_n = min\left(\sqrt{\frac{2304.00}{100.00}}, 2.0\right) \times 0.85 \times 4000 \times 100.00$$

$$B_n = 680000\ \text{lb}$$

Critical Load Combination for Bearing: U1=1.2DL + 1.6LL + 0.5Lr + 0W/S

Factored Bearing Load	$B_u =$	32.06kip
Bearing Strength Reduction Factor (Table 21.2.1.2)	$\phi =$	0.65
Reduced Bearing Strength	$\phi Bn =$	442.00kip

$$Bearing_{Ratio} = \frac{B_u}{\phi \times B_n} = \frac{32.06}{0.65 \times 680.00}$$

$$Bearing_{Ratio} = 0.07$$

UTILITY: 0.07

16.3.4.1

$$As_{mindowels} = 0.005 \times A_1 = 0.005 \times 100 = 0.5\ \text{in2}$$

$$As_{dowels} = \frac{\pi \times (db_{col})^2}{4} \times n = \frac{3.14 \times (0.5)^2}{4} \times 4 = 0.79\ \text{in2}$$

Asd_{prov} = 0.79 > $Asd_{min/req}$ = 0.5
Provided dowels reinforcement area > minimum/required reinforcement area. — PASS

Footing Details

Reinforcement Spacing Check

7.7.2.3

$$s_{max1} = min(3 * H, 18.00)$$

$$s_{max1} = min(3 * 15, 18.00)$$

$$s_{max1} = 18\ \text{in}$$

24.3

$$s_{max2} = min(15.0 * 40000/f_s - (2.5 * c_c), 12 * 40000/f_s)$$

$$s_{max2} = min(15.0 * 40000/40000 - (2.5 * 3), 12 * 40000/40000)$$

$$s_{max2} = 7.5\ \text{in}$$

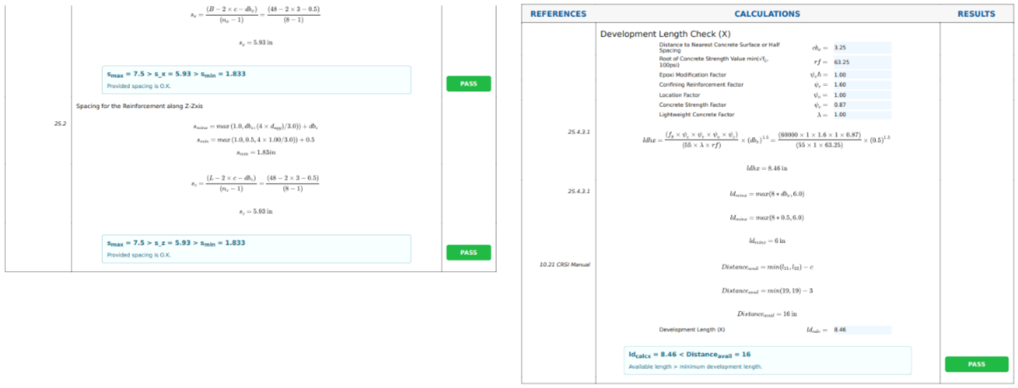
Spacing for the Reinforcement along X-Axis

25.2

$$s_{minx} = max\,(1.0, db_x, (4 \times d_{agg})/3.0)) + db_x$$

$$s_{min} = max\,(1.0, 0.5, 4 \times 1.00/3.0)) + 0.5$$

$$s_{min} = 1.83 in$$



$$s_x = \frac{(B - 2 \times c - db_z)}{(n_x - 1)} = \frac{(48 - 2 \times 3 - 0.5)}{(8 - 1)}$$

$$s_x = 5.93 \text{ in}$$

s_{max} = 7.5 > s_x = 5.93 > s_{min} = 1.833
Provided spacing is O.K. — PASS

Spacing for the Reinforcement along Z-Axis

25.2

$$s_{min} = max(1.0, db_z, (4 \times d_{agg})/3.0)) + db_z$$
$$s_{min} = max(1.0, 0.5, 4 \times 1.00/3.0)) + 0.5$$
$$s_{min} = 1.83 in$$

$$s_z = \frac{(L - 2 \times c - db_x)}{(n_z - 1)} = \frac{(48 - 2 \times 3 - 0.5)}{(8 - 1)}$$

$$s_z = 5.93 \text{ in}$$

s_{max} = 7.5 > s_z = 5.93 > s_{min} = 1.833
Provided spacing is O.K. — PASS

REFERENCES	CALCULATIONS	RESULTS

Development Length Check (X)

Distance to Nearest Concrete Surface or Half Spacing	$cb_x =$	3.25
Root of Concrete Strength Value min(√f'c, 100psi)	$rf =$	63.25
Epoxi Modification Factor	$\psi_e h =$	1.00
Confining Reinforcement Factor	$\psi_c =$	1.60
Location Factor	$\psi_t =$	1.00
Concrete Strength Factor	$\psi_g =$	0.87
Lightweight Concrete Factor	$\lambda =$	1.00

25.4.3.1

$$ldhx = \frac{(f_y \times \psi_e \times \psi_r \times \psi_o \times \psi_c)}{(55 \times \lambda \times rf)} \times (db_x)^{1.5} = \frac{(60000 \times 1 \times 1.6 \times 1 \times 0.87)}{(55 \times 1 \times 63.25)} \times (0.5)^{1.5}$$

$$ldhx = 8.46 \text{ in}$$

25.4.3.1

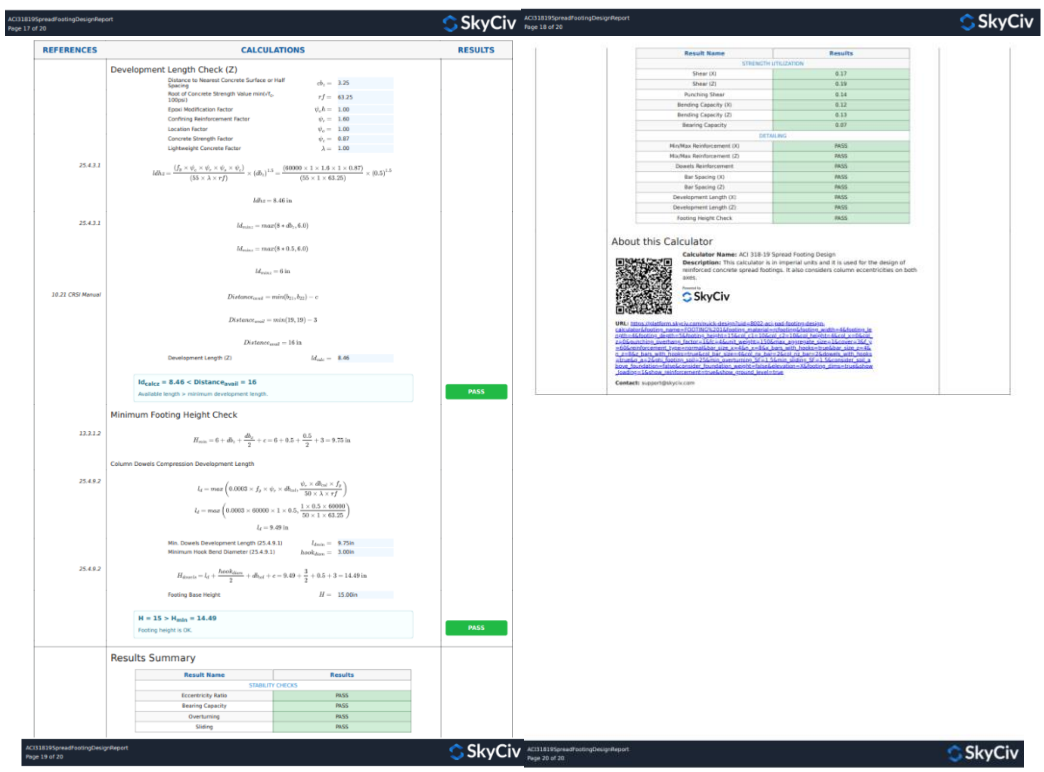
$$ld_{minx} = max(8 * db_x, 6.0)$$
$$ld_{minx} = max(8 * 0.5, 6.0)$$
$$ld_{minx} = 6 \text{ in}$$

10.21 CRSI Manual

$$Distance_{avail} = min(l_{11}, l_{12}) - c$$
$$Distance_{avail} = min(19, 19) - 3$$
$$Distance_{avail} = 16 \text{ in}$$

Development Length (X) $ld_{calc} =$ 8.46

ld_{calcx} = 8.46 < $Distance_{avail}$ = 16
Available length > minimum development length. — PASS



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REFERENCES	CALCULATIONS	RESULTS

Development Length Check (Z)

Distance to Nearest Concrete Surface or Half Spacing	$cb_z =$	3.25
Root of Concrete Strength Value min(√f'c, 100psi)	$rf =$	63.25
Epoxi Modification Factor	$\psi_e h =$	1.00
Confining Reinforcement Factor	$\psi_c =$	1.60
Location Factor	$\psi_t =$	1.00
Concrete Strength Factor	$\psi_g =$	0.87
Lightweight Concrete Factor	$\lambda =$	1.00

25.4.3.1

$$ldhz = \frac{(f_y \times \psi_e \times \psi_r \times \psi_o \times \psi_c)}{(55 \times \lambda \times rf)} \times (db_z)^{1.5} = \frac{(60000 \times 1 \times 1.6 \times 1 \times 0.87)}{(55 \times 1 \times 63.25)} \times (0.5)^{1.5}$$

$$ldhz = 8.46 \text{ in}$$

25.4.3.1

$$ld_{minz} = max(8 * db_z, 6.0)$$
$$ld_{minz} = max(8 * 0.5, 6.0)$$
$$ld_{minz} = 6 \text{ in}$$

10.21 CRSI Manual

$$Distance_{avail} = min(b_{11}, b_{12}) - c$$
$$Distance_{avail} = min(19, 19) - 3$$
$$Distance_{avail} = 16 \text{ in}$$

Development Length (Z) $ld_{calc} =$ 8.46

ld_{calcz} = 8.46 < $Distance_{avail}$ = 16
Available length > minimum development length. — PASS

Minimum Footing Height Check

13.3.1.2

$$H_{min} = 6 + db_z + \frac{db_x}{2} + c = 6 + 0.5 + \frac{0.5}{2} + 3 = 9.75 \text{ in}$$

Column Dowels Compression Development Length

25.4.9.2

$$l_d = max\left(0.0003 \times f_y \times \psi_r \times db_{col}, \frac{\psi_r \times db_{col} \times f_y}{50 \times \lambda \times rf}\right)$$
$$l_d = max\left(0.0003 \times 60000 \times 1 \times 0.5, \frac{1 \times 0.5 \times 60000}{50 \times 1 \times 63.25}\right)$$
$$l_d = 9.49 \text{ in}$$

Min. Dowels Development Length (25.4.9.1)	$l_{dmin} =$	9.75in
Minimum Hook Bend Diameter (25.4.9.1)	$hook_{diam} =$	3.00in

25.4.9.2

$$H_{dowels} = l_d + \frac{hook_{diam}}{2} + db_{col} + c = 9.49 + \frac{3}{2} + 0.5 + 3 = 14.49 \text{ in}$$

Footing Base Height $H =$ 15.00in

H = 15 > H_{min} = 14.49
Footing height is OK. — PASS

Results Summary

Result Name	Results
STABILITY CHECKS	
Eccentricity Ratio	PASS
Bearing Capacity	PASS
Overturning	PASS
Sliding	PASS

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Result Name	Results
STRENGTH UTILIZATION	
Shear (X)	0.17
Shear (Z)	0.19
Punching Shear	0.14
Bending Capacity (X)	0.12
Bending Capacity (Z)	0.13
Bearing Capacity	0.07
DETAILING	
Min/Max Reinforcement (X)	PASS
Min/Max Reinforcement (Z)	PASS
Dowels Reinforcement	PASS
Bar Spacing (X)	PASS
Bar Spacing (Z)	PASS
Development Length (X)	PASS
Development Length (Z)	PASS
Footing Height Check	PASS

About this Calculator

Calculator Name: ACI 318-19 Spread Footing Design
Description: This calculator is in imperial units and it is used for the design of reinforced concrete spread footings. It also considers column eccentricities on both axes.

Powered by SkyCiv

URL: https://platform.skyciv.com/quick-design?uid=8002-aci-pad-footing-design-calculator&...

Contact: support@skyciv.com

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Figure 26: Cabin Structural Footing Analysis

Appendix E: Nature Playscape

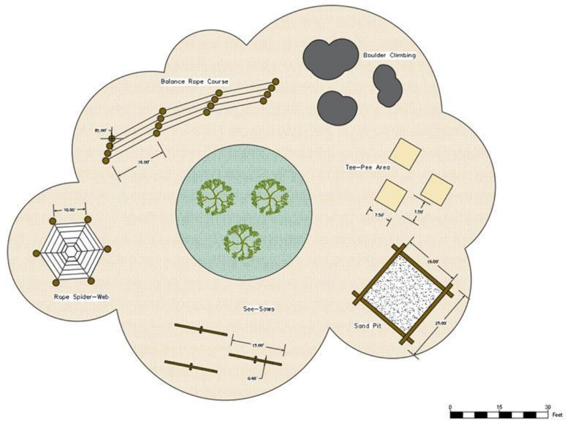

Figure 9: Nature Playscape Alternative Design

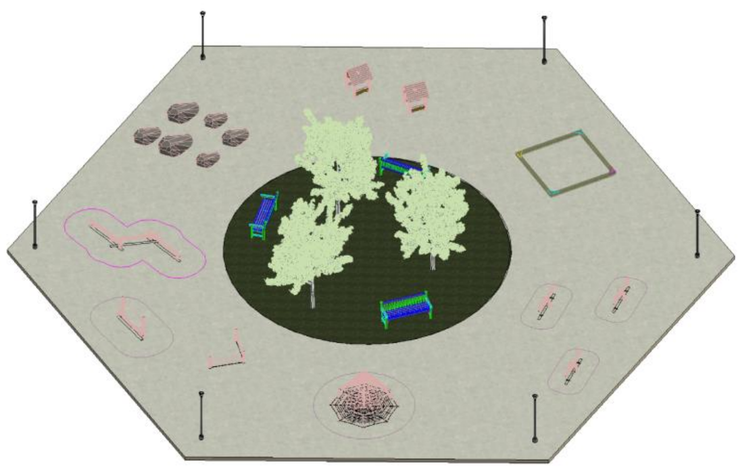

Figure 9: Nature Playscape Design

Appendix F: Road Design

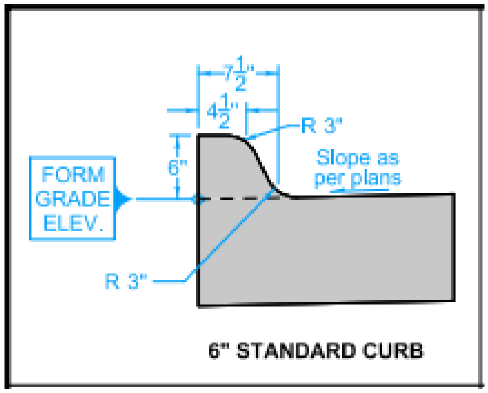

Figure 13: PCC Curb Details

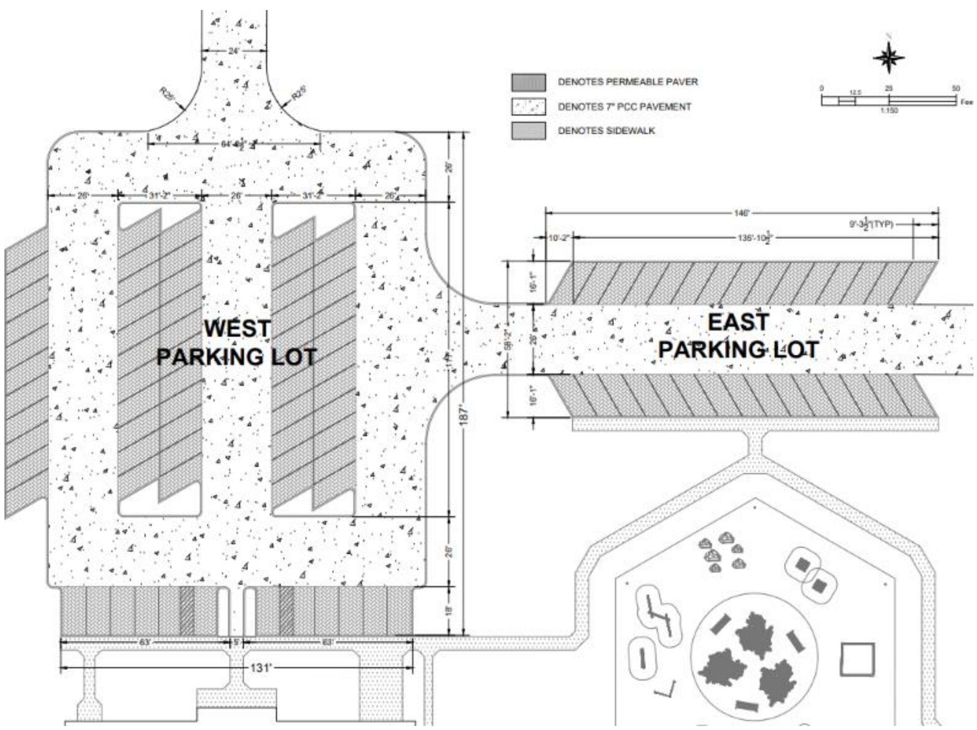

Figure 14: East and West Parking Lots

Appendix G: Grading

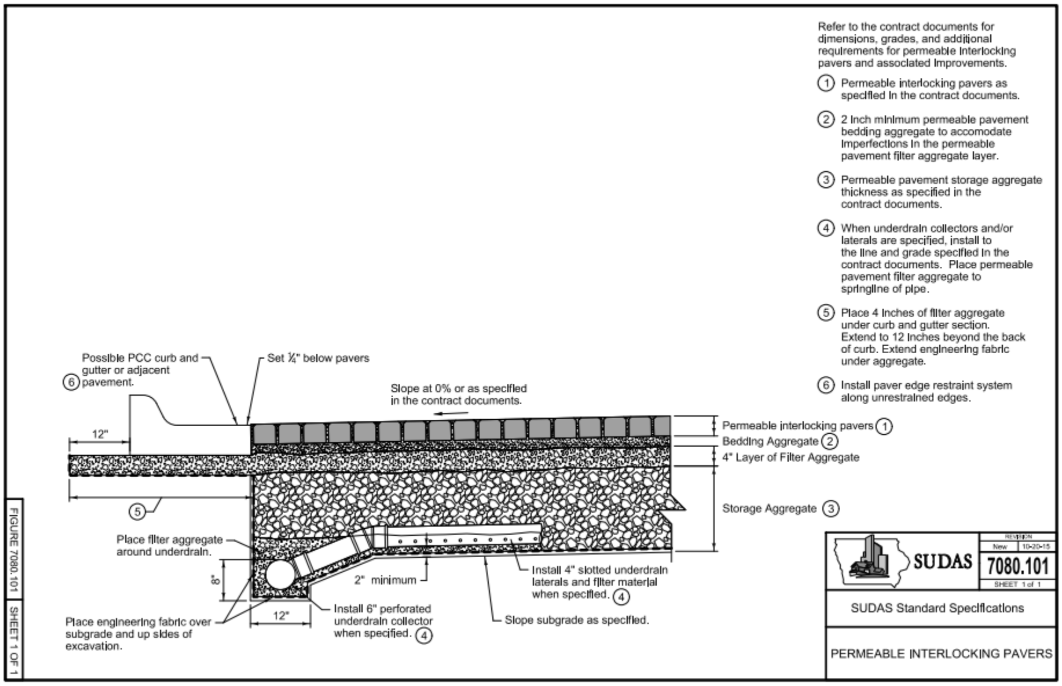

Figure 15: Permeable Paver Typical Design

	PERMEABLE PAVER AREA		PARKING LOT AREA		
West Middle Parking	3092.42	Total	37153.67	230.28	460.56
East Middle Parking	3092.42	Impervious	22014.79	323.09	646.18
West Parking	1564.21				1106.74
South Entrance Parkin	2073.43				
North-East Parking	########				
South-East Parking	########				
	14032.14				

	Concrete	
Park Road	39354.2	39355
Parking	22014.79	22015
Sidewalk	4820.28	4825
Cabin Road	13516.41	13520
Patio	1057.87	1060
SUM	80763.55	80775

Permeable	
3092.42	3095
3092.42	3095
1564.21	1565
2073.43	2075
2104.8300	2105
2104.8300	2105
14032.14	14040

Mulch	
9353.07	9355

Gravel	
4239.26	4240
1249.333	1250
160.3439	165
460.2304	465
2036.637	2040
49.5761	50
7.5607	10
1869.907	1870
10072.85	10090

Point precipitation frequency estimates (inches/hour)
NOAA Atlas 14 Volume 8 Version 2
Data type: Precipitation intensity
Time series type: Partial duration
Project area: Midwestern States
Location n Iowa USA
Station Name: FAIRFIELD
Latitude: 41.0211 Degree
Longitude: -91.9553 Degree
Elevation (USGS): 740 ft

PRECIPITATION FREQUENCY ESTIMATES

by duratior	1	2	5	10	25	50	100	200	500	1000
5-min:	4.52	5.29	6.59	7.7	9.29	10.5	11.9	13.2	15.1	16.5
10-min:	3.31	3.88	4.82	5.63	6.8	7.72	8.68	9.67	11	12.1
15-min:	2.69	3.15	3.92	4.58	5.52	6.28	7.06	7.86	8.97	9.84
30-min:	1.85	2.17	2.73	3.2	3.87	4.4	4.94	5.51	6.28	6.88
60-min:	1.2	1.41	1.78	2.1	2.56	2.94	3.32	3.73	4.3	4.74
2-hr:	0.734	0.868	1.1	1.3	1.6	1.84	2.09	2.36	2.73	3.02
3-hr:	0.55	0.649	0.823	0.978	1.21	1.39	1.59	1.8	2.1	2.34
6-hr:	0.33	0.387	0.489	0.581	0.719	0.835	0.958	1.09	1.28	1.43
12-hr:	0.192	0.224	0.28	0.331	0.408	0.472	0.542	0.617	0.724	0.811
24-hr:	0.112	0.129	0.16	0.187	0.229	0.264	0.301	0.341	0.399	0.445
2-day:	0.065	0.074	0.091	0.105	0.127	0.145	0.164	0.185	0.214	0.237
3-day:	0.048	0.054	0.065	0.075	0.09	0.102	0.115	0.129	0.148	0.163
4-day:	0.038	0.043	0.052	0.059	0.071	0.08	0.089	0.099	0.114	0.125
7-day:	0.026	0.029	0.034	0.039	0.045	0.051	0.056	0.062	0.069	0.075
10-day:	0.021	0.023	0.027	0.031	0.035	0.039	0.043	0.047	0.052	0.056
20-day:	0.014	0.016	0.018	0.021	0.024	0.026	0.028	0.03	0.033	0.036
30-day:	0.012	0.013	0.015	0.017	0.019	0.021	0.023	0.024	0.027	0.028
45-day:	0.01	0.011	0.013	0.014	0.016	0.017	0.019	0.02	0.021	0.023
60-day:	0.008	0.009	0.011	0.012	0.014	0.015	0.016	0.017	0.018	0.019

Date/time (GMT): Sun Apr 6 23:39:19 2025
pyRunTime: 0.023718595504760742

Rational Method

	Runoff Coefficient			
Material	Pervious	Imperviou	Mulch	Gravel
5-Year	0.7	0.95	0.7	0.75
10-Year	0.7	0.96	0.75	0.8

Area (W/ Gravel, SF)			
Pervious	Imperviou	Mulch	Gravel
14040	80775	9355	10090

Area (W/ Gravel, Acre)			
Pervious	Impervious	Mulch	Gravel
0.32231405	1.85433884	0.214761	0.231635

Area (No Gravel, SF)		
Pervious	Imperviou	Mulch
14040	90865	9355

Area (No Gravel, Acre)		
Pervious	Impervious	Mulch
0.32231405	2.08597337	0.214761

12-HR Design Flow (Gravel)

	Parking, Road		Playscape	Driveway	
Material:	Pervious	Imperviou	Mulch	Gravel	SUM
5-Year Flow Rates:	0.063174	0.493254	0.042093	0.048643	0.647164
10-Year Flow Rates	0.07468	0.589235	0.053314	0.061337	0.778566

12-HR Design Flow (No Gravel)

	Parking, Road, Drivew		Playscape	
Material:	Pervious	Imperviou	Mulch	SUM
5-Year Flow Rates:	0.063174	0.554869	0.042093	0.660136
10-Year Flow Rates	0.07468	0.662839	0.053314	0.790834

Figure 16: Rational Method Calculations

Appendix H: Cost

Multi-Use Lodge Cost Estimate					
Item Type	Item Size	Quantity	Unit	Unit Price	Total Price
Windows		1642	SF	25.00	52252.545
14 Risers, Pine treads, box stairs	3'	2	EA	2190.09	5575.53112
Exterior Wood Doors w/ glass	72" x 96"	10	EA	1924.00	24490.596
Exterior Wood Door w/ glass	36" x 84"	1	EA	1388.00	1766.7852
Interior Wood Door w/ glass	36" x 86"	5	EA	991.00	6307.2195
Interior Wood Door	36" x 84"	5	EA	603.00	3837.7935
Vanity top, porcelain enamel on cast iron	20" x 18"	10	EA	422.82	5382.07578
Floor mounted water closet, ADA	1.28 gpf	4	EA	772.67	3934.12657
Floor mounted water closet	1.28 gpf	7	EA	782.32	6970.7059
Water urinal, siphon jet type	1.28 gpf	3	EA	618.65	2362.43876
Benches	60" x 18"	5	EA	750.00	4773.375
Wood tables & Equipment	86" Dia.	24	EA	750.00	22912.2
Wood tables & Equipment	36" Dia.	12	EA	300.00	4582.44
Wood tables & Equipment	48" x 144"	1	EA	1500.00	1909.35
Receptionist L-Desk	72" x 78"	1	EA	1150.00	1463.835
Porch Railings and trim	1" x 4"	232	LF	2.11	623.110008
Treated Pine Decking	2" x 4"	2185	SF	3.12	8677.61388
Paints & coating, varnish 1 coat + sealer		9223	SF	0.98	11505.1576
Softwood clapboard siding		4428	SF	9.24	52080.3471
Polyethylene Membrane, vapor retarder	2 mil	4428	SF	0.15	845.46018
Gypsum Wallboard, on walls, standard	5/8"	18688	SF	1.40	33303.1373
Oak wood flooring, #1 common	2 1/4"	9223	SF	6.01	70557.1398
18-gauge steel roof decking	3" deep	10546	SF	5.54	74368.0621
Wall Framing, studs and plates	2" x 6"	5000	LF	1.64	10437.78
Electrical		12500	SF	8.22	130790.475
Terminal and Package Units		12500	SF	28.00	445515
TJI 560 I-Joists	14"	2255.5	LF	18.00	51678.4671
TJI 560 I-Joists	16"	1032.5	LF	20.00	26285.385
TJI 560 I-Joists	11 7/8"	800	LF	15.70	15987.624
W6X25 A36 Steel Beam		211	LF	50.00	13429.095
W14X22 A36 Steel Beam		171.1	LF	52.62	11460.2777
W12X50 A36 Steel Beam		180.74	LF	76.50	17599.8919
OSB Plywood (Sub-Floor)	3/4"	9223	SF	2.00	23479.9134
Gypsum Board	1/2"	8670	SF	1.40	15450.4602
Wood Panelling (Ceiling)		10546	SF	2.41	32351.5414
Southern Pine No. 2 Dense (Truss)	4" x 8"	6797.8	LF	20.00	173059.156
W16X45 A36 Steel Beam		29.3	LF	96.00	3580.41312
W14X38 A36 Steel Beam		219.53	LF	88.53	24738.3492
2" x 12" Sothern Pine No. 2 Purlin	2" x 12"	1453	LF	6.56	12132.8755
4" x 12" Sothern Pine No. 2 Purlin	4" x 12"	1450	LF	8.56	15799.4527
10" x 10" Southern Pine No. 2 Column	10" x 10"	54.624	LF	24.55	1706.98334
12" x 12" Southern Pine No. 2 Column	12" x 12"	127.46	LF	26.80	4347.9983
16" x 16" Southern Pine No. 2 Column	16" x 16"	111.22	LF	30.00	4247.31089
6" x 8" Southern Pine Dense Select Structural 86 Column	6" x 8"	180	LF	25.00	5728.05
Water Distribution		12500	SF	8.00	127290
Toilet Partitions		13	EA	2.46	40.707342
Cooking Range		1	EA	595.00	757.3755
Excavation Cost		15000	CY	8.00	152748
Refridgerator	18 CF	1	EA	822.00	1046.3238
Two top drawers, Two doors below	30"	3	EA	578.50	2209.11795
Two top drawers, Two doors below	48"	3	EA	699.00	2669.2713
Natural Stone Countertops	24"	30	LF	217.50	8305.6725
2" x 6" Southern Pine No. 2 Wall Framing	2" x 6"	11270	LF	1.64	23526.7561
2" x 8" Southern Pine No. 2 Wall Framing	2" x 8"	3217.8	LF	2.14	8765.30651
W12X65 A992 Steel Beam		117.51	LF	120.3	17994.5973
On Grade Concrete Slab Flooring (5")		306.57	CY	110	42925.936
Footing Concrete		11098	CY	110	1553912.19
Rebar	#4	46654	LF	0.7	41569.882
Rebar	#5	38036	LF	0.9	43574.5365
Mobilization (7%)		1	LS	300000	300000

				Lodge Cost	3767623.22
Contingency				25%	941905.805
				FINAL	**4709529.03**

Figure 17: Cost Estimation for Multi-Use Lodge

Cabin Cost Estimate					
Item Type	Item Size	Quantity	Unit	Unit Price	Total Price
Gas fired, foam lined tank, 10 yr, gas water heater	75 gal	1	EA	3225.88	4106.22265
14 Risers, Pine treads, box stairs	3'	1	EA	2190.09	2787.76556
Casement Windows, two lites	47" x 59"	10	EA	826.92	10525.8647
Water Closet, tank type, vitreous china, floor mounted	1.6 gpf	2	EA	384.02	977.638116
Bath, tub, soaking, acrylic with pop up drain	72" x 42" x 23"	1	EA	2543.16	3237.18836
Birch Face, Flush, Interior, hollow core doors	2' 6" x 6' 8"	2	EA	170.67	434.491686
Birch Face, Flush, Interior, hollow core doors	3' 0" x 6' 8"	3	EA	152.85	583.688295
Birch Face, Flush, Interior, hollow core doors	4' 0" x 6' 8"	1	EA	222.2	282.83838
Doors, glass, sliding, wood, vinyl-clad	6'-0" x 6'8"	1	Opng	1858	2365.0482
Double-Hung, Solid Vinyl Windows	3'-0" x 4'-0"	2	EA	320.73	816.514434
Vanity top, porcelain enamel on cast iron	20" x 18"	2	EA	422.82	1076.41516
Shower, stall, baked enamel, terrazzo receptor	36"	1	EA	1708	2174.1132
Gas-Fired Furnace	100 MBH	1	EA	968.72	1233.08369
Self-Contained single package A.C., air cooled	3-ton	1	EA	6000	7637.4
Ceramic Tiling	6" x 6"	66	SF	2.32	194.906448
Oak wood flooring, #1 common	2 1/4"	1143	SF	6.01	8744.09745
Paints & coating, varnish 1 coat + sealer		1143	SF	0.98	1425.82621
Treated pine decking	2" x 4"	600	SF	3.12	2382.8688
Joists	3" x 10"	660	LF	4.33	3637.69362
Truss System	4" x 8"	511.85	LF	20	13030.6773
Sheathing, plywood on roof, pnuematic nailed	1/2"	1950	SF	1.05	2606.26275
18-gauge steel roof decking	3" deep	1950	SF	5.54	13751.1387
Porch Posts	6" x 6"	48	LF	8.8	537.67296
PVC roofing	60 mils	1950	SF	2.26	5609.6703
Framing Purlins	3" x 8"	576	LF	5	3665.952
Wall Framing, 10' high wall, exterior	2" x 6"	2495	LF	1.64	5208.45222
Wall Framing, 10' high wall, interior	2" x 4"	720	LF	1.3	1191.4344
Cedar Clapboard siding, clear grade,	3/4" x 10"	2302	SF	9.24	27075.194
Polyethylene Membrane, vapor retarder	2 mil	2302	SF	0.15	439.53237
Porch Railings and trim	1" x 4"	101.21	LF	2.11	271.837756
6" thick, 3000 psi, broom finish, cast in-place concrete	6"	674.56	SF	3.83	3288.61963
Custom Wood Windows	12'-6" x 12'-6"	2	EA	8000	20366.4
Exterior Doors	36" x 96"	2	EA	600	1527.48
Corner Base Cabinets	36"	1	EA	871.5	1109.33235
Two top drawers, Two doors below	30"	1	EA	578.5	736.37265
Two top drawers, Two doors below	48"	1	EA	699	889.7571
Natural Stone Countertops	24"	10	LF	217.5	2768.5575
Gypsum Wallboard, on walls, standard	5/8"	4400	SF	1.4	7841.064
Refridgerator	18 CF	1	EA	822	1046.3238
HVAC Ducting		450	LF	25	14320.125
Wall Framing, studs and plates	2" x 6"	5000	LF	1.64	10437.78
BATT Insulation, R13 wool	16" wide	2710	SF	1.36	4691.40024
BATT Insulation, R19 wool	16" wide	1600	SF	1.77	3604.8528
On Grade Concrete Slab Flooring (5")	150pcf, 3ksi	394.2	CF	8.57	4300.23043
Rigid Insulation	Tyvek	2302	SF	1.09	3193.93522
Electrical		1818	SF	15	34711.983
Connections		1818	SF	2	4628.2644
Excavation Cost		3000	CY	6	22912.2
Couch		1	EA	455	579.1695
Cooking Range		1	EA	595	757.3755
Table w/ chairs	86" Dia.	1	EA	750	954.675
Fire Rings		2	EA	250	636.45
Picnic Tables		2	EA	625	1591.125
Outside Furniture	Lawn Chairs	4	EA	20	101.832
Built up wood beam - Southern Pine Dense Select Structural	3" x 12"	9	EA	330	3780.513
Footing Rebar	#4	1543.2	LF	0.7	1375.00185
Footing Rebar	#5	2238.8	LF	0.9	2564.79167
Footing Concrete	150pcf, 3ksi	282.04	CY	125	44876.0895
Mobilization		1	LS	48000	48000

			1 CABIN	**375603.191**
			TOTAL	**751206.382**
Contigency			20%	**150241.276**
			FINAL	**901447.658**

Figure 18: Cost Estimation for the Year-Round Cabins

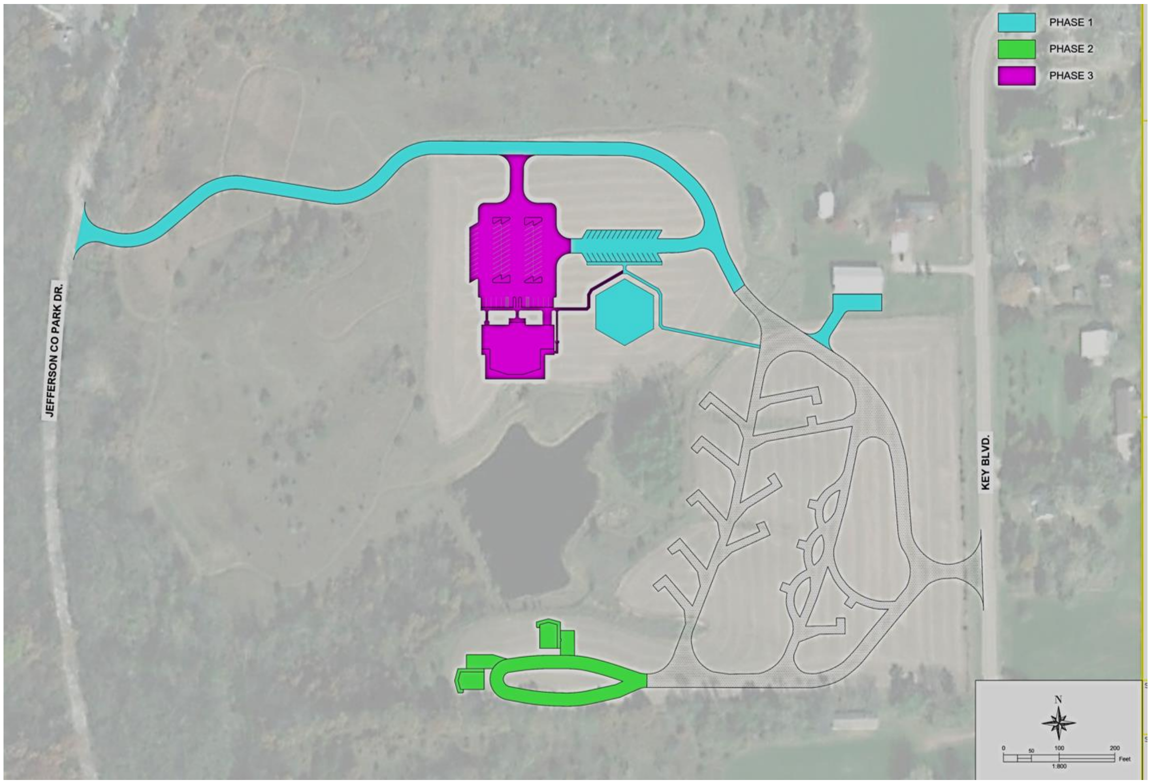

Figure 19: Phasing Plan for Jefferson County Conservation Area Campground Redevelopment

Item No.	SUDAS Code	Item Description	Item Type	Item Size	Quantity	Unit Price (Average)	Unit Price (Median)	Total Price (Average)	Total Price (Median)	Mixed (Higher of Median and Average)
	DIVISION 2									
	2010-B	Clearing and Grubbing (AC)			1.5	$ 7,111.11	$ 11,000.00	$ 10,666.67	$ 16,500.00	$ 16,500.00
	2010-D-1	Topsoil, On-site (CY)			13000	$ 7.81	$ 18.00	$ 101,530.00	$ 234,000.00	$ 234,000.00
	2010-E	Excavation, Class 10, Class 12, or Class 13 (CY)			24000	$ 15.94	$ 16.00	$ 382,560.00	$ 384,000.00	$ 384,000.00
	2010-G	Subgrade Preparation (SY)		6"	11000	$ 3.00	$ 3.15	$ 33,000.00	$ 34,650.00	$ 34,650.00
	2010-J	Subbase (SY)	Modified	6"	11000	$ 9.00	$ 9.74	$ 99,000.00	$ 107,140.00	$ 107,140.00
	DIVISION 3									
	3010-D	Replacement of Unsuitable Backfill Material (CY)			4800	$42.71	$33.38	$ 205,008.00	$ 160,224.00	$ 205,008.00
	DIVISION 4									
	4010-A-1	Sanitary Sewer Gravity Main, Trenched (LF)	PVC	8"	1400	$ 100.03	$ 103.25	$ 140,042.00	$ 144,550.00	$ 144,550.00
	4010-E	Sanitary Sewer Service Stub (LF)	PVC	8"	100	$ 125.00	$ 125.00	$ 12,500.00	$ 12,500.00	$ 12,500.00
	DIVISION 5									
	5010-A-1	Water Main, Trenched (LF)	PVC	6"	1450	$ 48.82	$ 56.00	$ 70,789.00	$ 81,200.00	$ 81,200.00
	5010-C-1	Fitting (EA)	PVC, 22.5d	6"	10	$ 455.00	$ 455.00	$ 4,550.00	$ 4,550.00	$ 4,550.00
	5010-C-1	Fitting (EA)	PVC, 45d	6"	1	$ 475.75	$ 478.68	$ 475.75	$ 478.68	$ 478.68
	5010-D	Water Service Stub (EA)	PVC	6"	3	$ 5,500.00	$ 5,500.00	$ 16,500.00	$ 16,500.00	$ 16,500.00
	5020-A	Valve (EA)		6"	8	$ 1,530.00	$ 1,625.82	$ 12,240.00	$ 13,006.56	$ 13,006.56
	5020-C	Fire Hydrant Assembly (EA)			5	$ 6,286.61	$ 6,118.80	$ 31,433.05	$ 30,594.00	$ 31,433.05
	DIVISION 6									
	6010-A	Manhole (EA)	SW-301	48"	22	$ 1,921.36	$ 5,775.00	$ 42,269.91	$ 127,050.00	$ 127,050.00
	DIVISION 7									
	7010-A	Pavement, PCC (SY)		7"	8500	$ 57.58	$ 72.00	$ 489,430.00	$ 612,000.00	$ 612,000.00
	7010-E	Curb and Gutter (LF)	6"	7"	1600	$ 51.00	$ 51.00	$ 81,600.00	$ 81,600.00	$ 81,600.00
	7030-E	Sidewalk, PCC (SY) *INCLUDES PATIO*		4"	1000	$ 38.41	$ 65.00	$ 38,410.00	$ 65,000.00	$ 65,000.00
	7030-H-2	Driveway, Granular (SY)		6"	1400	$ 20.48	$ 12.00	$ 28,672.00	$ 16,800.00	$ 28,672.00
	7040-B	Subbase Over-excavation (TON)			700	$ 53.50	$ 48.86	$ 37,450.00	$ 34,202.00	$ 37,450.00
	7060-A	Bituminous Seal Coat (SY)			1400	$ 5.67	$ 9.35	$ 7,938.00	$ 13,090.00	$ 13,090.00
	7080-F	Permeable Interlocking Pavers (SY)	Parking		1800	$ 108.74	$ 108.74	$ 195,732.00	$ 195,732.00	$ 195,732.00
	DIVISION 8									
	8020-B	Painted Pavement Markings, Solvent/Waterborne (STA)	Parking		110	$ 32.90	$ 94.00	$ 3,619.00	$ 10,340.00	$ 10,340.00
	8020-G	Painted Symbols and Legends (EA)	Traffic		50	$ 125.00	$ 125.00	$ 6,250.00	$ 6,250.00	$ 6,250.00
	8030-A	Temporary Traffic Control (LS)			1	-	-	$ 30,000.00	$ 30,000.00	$ 30,000.00
	8040-B	Traffic Signs (SF)			120	$ 26.23	$ 27.00	$ 3,147.60	$ 3,240.00	$ 3,240.00
	DIVISION 9									
	9010-A	Conventional Seeding, Seeding, Fertilizing, and Mulching (AC)	Native Grass		3	$ 6,400.00	$ 7,750.00	$ 19,200.00	$ 23,250.00	$ 23,250.00
	9010-A	Conventional Seeding, Seeding, Fertilizing, and Mulching (AC)	Type 1		4.2	$ 2,280.75	$ 2,750.00	$ 9,579.15	$ 11,550.00	$ 11,550.00
	9040-A-1	SWPPP Preparation (LS)			1	-	-	$ 20,000.00	$ 20,000.00	$ 20,000.00
	9040-A-2	SWPPP Management (LS)			1	-	-	$ 20,000.00	$ 20,000.00	$ 20,000.00
	9040-D-1	Filter Sock (LF)			1500	$ 2.49	$ 2.76	$ 3,735.00	$ 4,140.00	$ 4,140.00
	9040-D-2	Filter Socks, Removal (LF)			1500	$ 0.46	$ 0.50	$ 690.00	$ 750.00	$ 750.00
	9040-J	Rip Rap (TON)	Class A		100	$ 82.96	$ 83.50	$ 8,296.00	$ 8,350.00	$ 8,350.00
	9040-N-1	Silt Fence or Silt Fence Ditch Check (LF)			2000	$ 1.71	$ 2.00	$ 3,420.00	$ 4,000.00	$ 4,000.00
	9040-N-2	Silt Fence or Silt Fence Ditch Check, Removal of Sediment (LF)			2000	$ 0.26	$ 0.50	$ 520.00	$ 1,000.00	$ 1,000.00
	9040-N-3	Silt Fence or Silt Fence Ditch Check, Removal of Device (LF)			2000	$ 0.19	$ 0.26	$ 380.00	$ 520.00	$ 520.00
	9060-F	Temporary Fence (LF)	Chain Link	48"	1400	$ 14.84	$ 15.50	$ 20,776.00	$ 21,700.00	$ 21,700.00

Figure 20: Cost Estimation for Division 2 through 9

DIVISION 11														
11020-A	Mobilization (LS)			1	13.5%	13.5%								
11050-A	Concrete Washout (LS)			4	$ 2,926.12	$ 1,500.00	$ 11,704.48	$ 6,000.00	$ 11,704.48					
SP-1	SPECIAL PROVISION 1 - BARK MULCH (CY)		3.00"	600	$ 100.00	$ 100.00	$ 60,000.00	$ 60,000.00	$ 60,000.00					
SP-2	SPECIAL PROVISION 2 - PLAYSCAPE EQUIPMENT/LIGHTS (LS)			1	-	-	$ 60,000.00	$ 60,000.00	$ 60,000.00					
SP-3	SPECIAL PROVISION 3 - UTILITY ALLOWANCE - FIBER, NG, ELECTRICAL (LS)			1	-	-	$ 200,000.00	$ 200,000.00	$ 200,000.00					
SP-4	SPECIAL PROVISION 4 - STORMWATER - SUBDRAINS, INLETS (LS)			1	-	-	$ 150,000.00	$ 150,000.00	$ 138,300.00					
SP-5	SPECIAL PROVISION 5 - WATER METER PITS (EA)			4	-	-	$ 120,000.00	$ 120,000.00	$ 120,000.00					
SP-6	SPECIAL PROVISION 6 - ADDITIONAL UTILITY TRENCHING (EA)			1	-	-	$ 100,000.00	$ 100,000.00	$ 100,000.00					
SP-7	SPECIAL PROVISION 7 - TREE LIBRARY/PLANTING (EA)			50	$ 500.00	$ 500.00	$ 25,000.00	$ 25,000.00	$ 25,000.00					
SP-8	SPECIAL PROVISION 8 - SHRUB PLANTINGS (EA)			45	$ 300.00	$ 300.00	$ 13,500.00	$ 13,500.00	$ 13,500.00					
SP-9	SPECIAL PROVISION 9 - ROADWAY LIGHTING AND TESTING (EA)			20	$ 7,500.00	$ 7,500.00	$ 150,000.00	$ 150,000.00	$ 150,000.00					
SP-10	SPECIAL PROVISION 10 - ADA FEATURES (LS)			1	-	-	$ 40,000.00	$ 40,000.00	$ 40,000.00					
SP-11	SPECIAL PROVISION 11 - SITE FURNISHINGS (LS)			1	-	-	$ 70,000.00	$ 70,000.00	$ 70,000.00					
SP-12	SPECIAL PROVISION 12 - EDUCATIONAL SIGNAGE (LS)			1	-	-	$ 30,000.00	$ 30,000.00	$ 30,000.00					
							$ 3,221,613.60	$ 3,574,957.24	$ 3,629,704.77			$ 490,010.14		**$ 4,119,714.91**

Figure 21: Cost Estimation for Division 11

Appendix I - References

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