NAU Pottery Ramada
Final Report

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1.0 Project Introduction

The purpose of this project is to analyze and design a ramada to cover an existing kiln for Northern Arizona University's ceramics department. The existing kiln is one of the only kilns on-site without roof coverage and therefore is exposed to rain, snow, and sun year-round which can cause weathering and erosion to occur to the structure as well as becomes an inconvenience to those using the kiln to fire their ceramic art pieces. This report details the design for a ramada that will prevent the structure and users from being directly exposed to inclement weather.

1.1 Project Location

The project site is located on Northern Arizona University's south campus, specifically adjacent to the ceramics department building. The address is 1919 S Lone Tree Rd, Flagstaff, AZ 86001. Figure 1-1 shows a vicinity map of NAU's south campus and the surrounding infrastructure near the Ceramics complex. Figure 1-2 shows an aerial view of the kiln in which the ramada will cover.

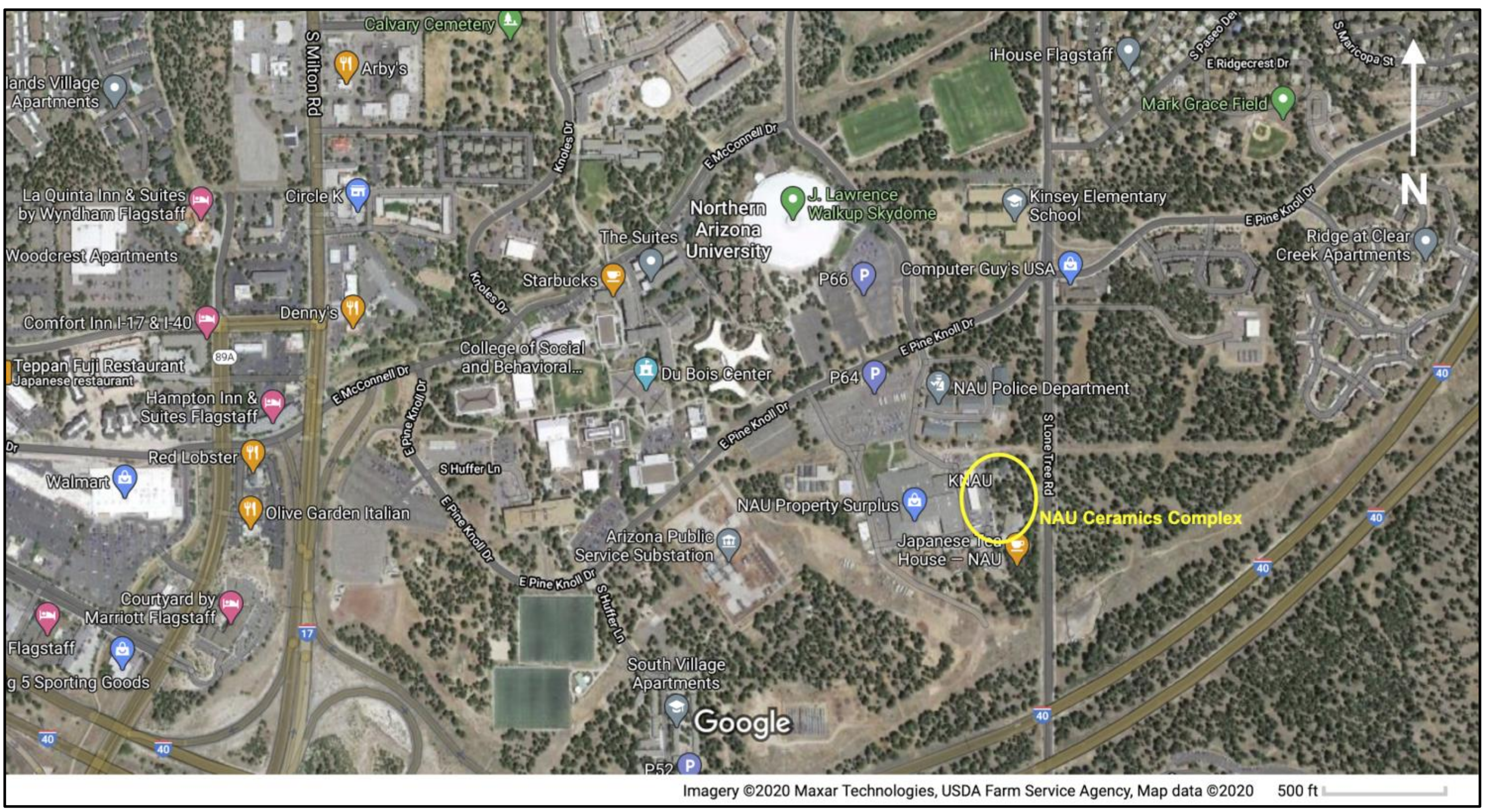

Figure 1- 1: Vicinity Map Detailing NAU's South Campus in Relation to the Ceramics Complex

Figure 1- 2: Project Location Shown in Yellow Over the Existing Kiln

1.2 Project Constraints and Limitations

The constraints for the project are as follows: 1) limited space, 2) ability to obtain permits, 3) budget, 4) resources, and 5) keeping a similar aesthetic of existing ramadas. The limitations for the project are as follows: 1) designing drainage structures, 3) analysis of the watershed, 4) limited geotechnical testing due to the inoperable direct shear machine, and 5) COVID-19's limited access to campus facilities.

1.3 Project Objectives

The objective of this project is to design a ramada that allows students to have overhead protection from inclement weather when they are using the kiln to fire their ceramic projects. With no protection over the kiln, the wood that is stockpiled for fuel can become wet. Before the wood is used for fuel it must be dried. The new ramada will be large enough to keep the wood dry when storing it next to the kiln, keep the kiln and students dry, as well as direct water runoff away from the kiln and its surrounding areas.

1.4 Project Deliverables

The project deliverable unique to this project includes a plan set for construction.

2.0 Site Investigation

Numerous site visits were conducted to understand the layout of the site and the needs of the client. Upon inspection at the site visit, it was determined the area in which the kiln is located is densely populated by other kilns/structures that the ceramic department uses frequently. These structures and obstructions in the surrounding area were documented during two survey sessions. Soil samples were also collected on the site to perform geotechnical analysis.

2.1 Survey Work

In order to completely capture the topographic map of the site, the team completed two surveying sessions on-site. Approximately 120 data points were collected in total. These data points were then used to create a topographic and site map. A total station, data collector, and prism rod were used to take each of the shots. Figure 2-1 shows the topographic map of the area, existing structures, and soil sampling locations. It is apparent in Figure 2-1 that the site is on a slope, having a higher elevation on the west-side and decreasing elevation as one travels to the east-side. It is important to fully capture the topography of the site for the hydraulic runoff analysis of the site and ramada design. The client specified that there must be access in which a wheelbarrow can pass through on all sides of the kiln. This will be taken into consideration when designing the geometry of the ramada.

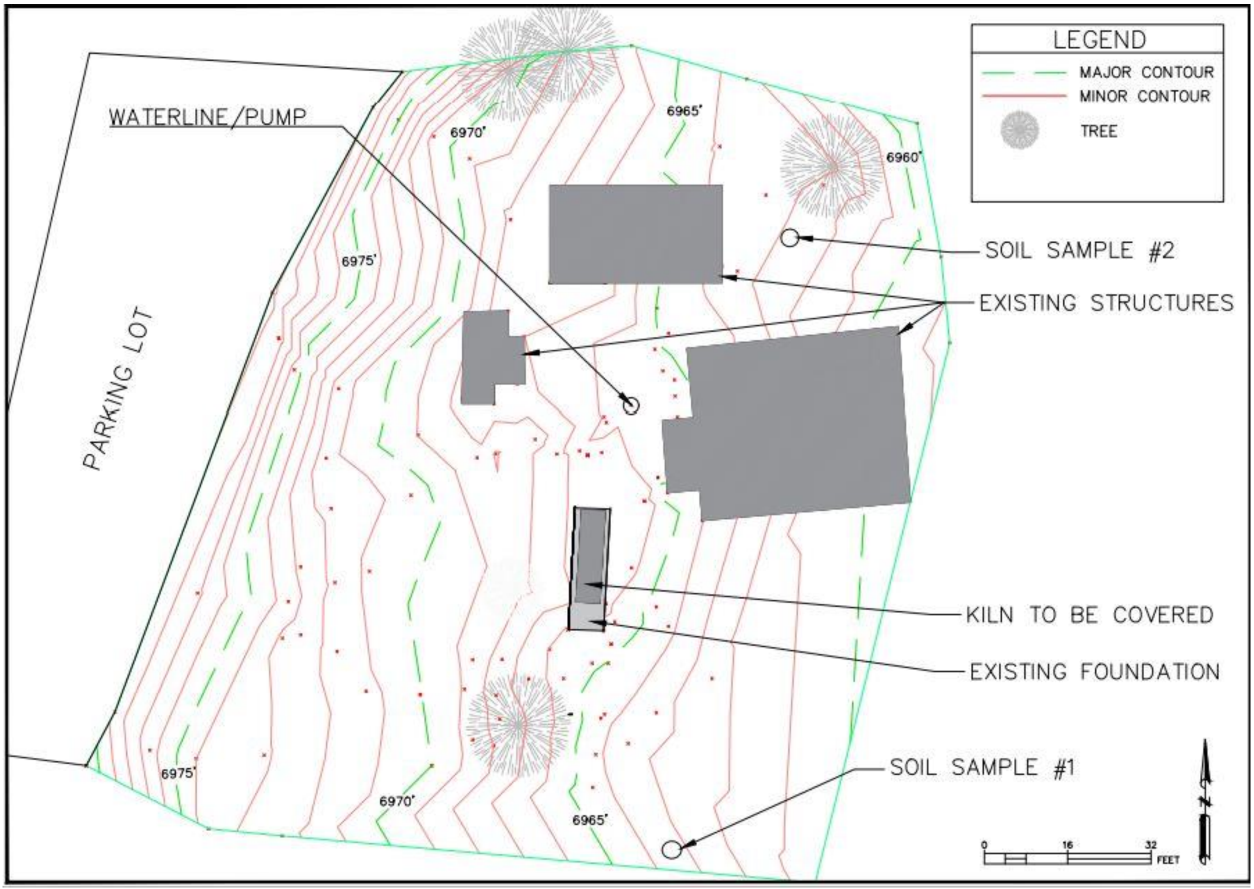

Figure 2- 1: Topographic Map of the Project Site and Surrounding Area

2.2 Geotechnical Sampling

Soil samples were taken from the Ceramics Complex at Northern Arizona University (1919 S Lone Tree Rd, Flagstaff, AZ 86011). Two samples in total were taken near the uncovered kiln. One sample was located to the south of the kiln, and the other sample was located to the north of the kiln at the edge of the ceramics complex. This was done to ensure there was no engineered fill from the kiln construction collected in the samples. Refer to Figure 2-1 above to see the location of samples. The team collected soil samples using a hand auger, a shovel, and an electric hammer drill. The volume of soil taken from each hole was approximately 5 quarts. The samples were placed in two sterile 6 Quart clear plastic stacking storage container totes. The samples were stored in a room temperature drawer in the geotechnical lab. The samples were used for testing in our geotechnical analysis to determine the soil classification. Two soil samples were collected North and South of the proposed structure at a depth of approximately 2.5 ft, shown in Figure 2-1. When attempting to collect soil samples at the site, we found that it was difficult to dig deeper than 2.5 feet due to the large amount of rock found just below the surface. The team believes the rock found was limestone. Further details of the geotechnical analysis of the site investigation will be discussed in a later section of this report.

3.0 Hydraulic Drainage Analysis

The team completed a minor hydraulic analysis of the site to determine the water runoff path.

3.1 Flow Analysis

Currently, rainfall is discharged from the adjacent west parking lot above the ceramic complex. The path of runoff flowing perpendicular to the contour lines, trends directly towards the west side of the proposed structure's location as shown in Figure 3-1. To prevent water runoff from running through the project site, the team proposes constructing a berm just west of the kiln's foundation that will divert the water flow further south and more importantly away from the kiln.

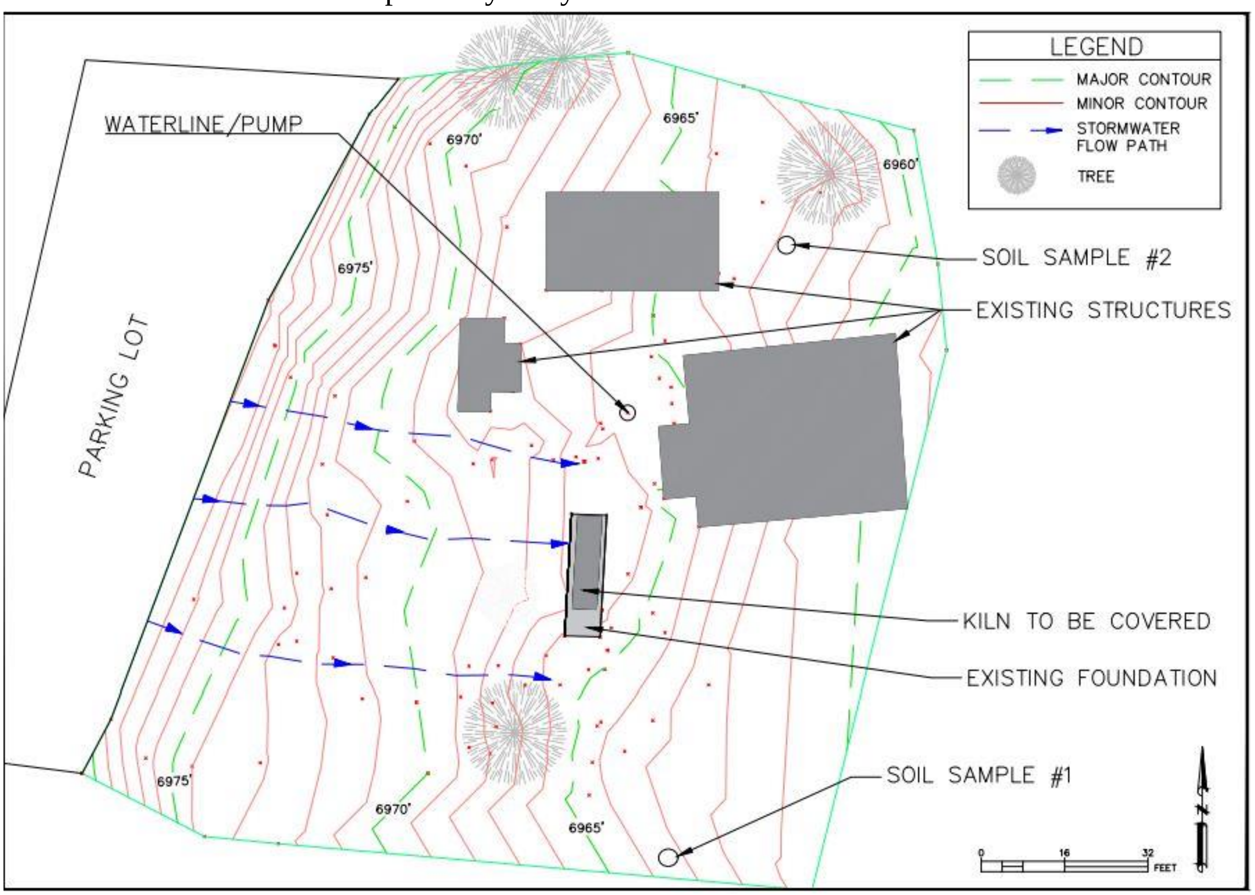

Figure 3- 1: Existing Topographic Map with Existing Flow Paths

3.2 Berm Design

The proposed berm will be 2 feet tall on the west side and gradually sloped towards the east side. It will be 30 feet long by 3 feet wide and its eastern most edge will be located approximately 15’ from the west edge of the existing kiln foundation. This is shown in Figure 3-2.

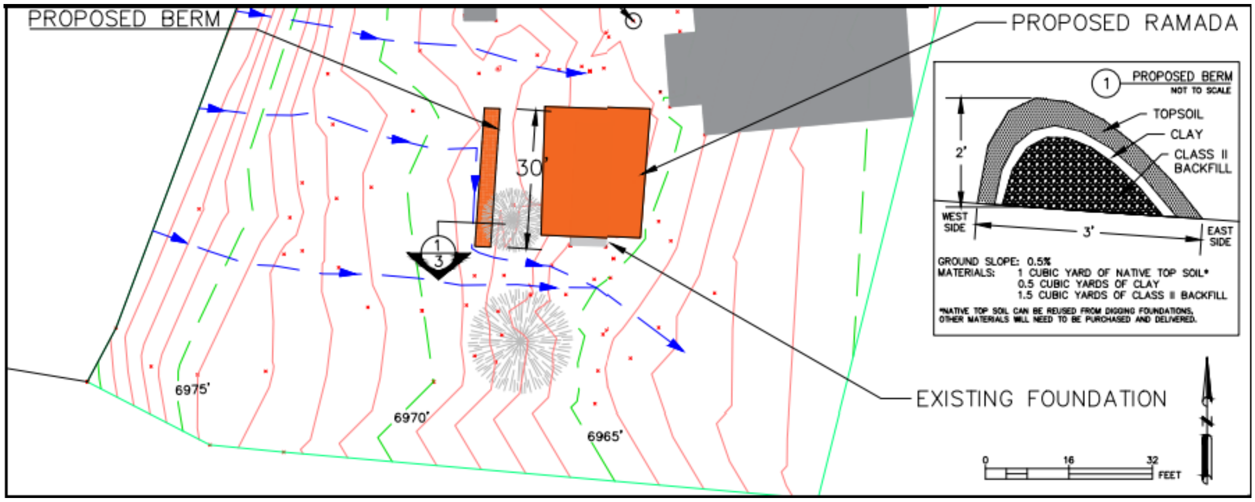

Figure 3- 2: Proposed Topographic Map with Water Flow Path and Berm

The typical height of berms range from 18 to 24 inches [13]. The length of the berm was based on the length of the kiln foundation. The berm needed to be longer than the kiln foundation to ensure water did not come in contact with the kiln foundation. The berm will have a slope of approximately 45 degrees. The berm was determined to be 2 feet tall by 3 feet wide based on the relatively small area draining to the existing kiln. Without doing a drainage analysis of the watershed, the team decided that the berm did not need to be excessively large to reroute the water. The berm will require approximately 3 cubic yards of soil (for the top soil, clay, and class II backfill). Approximately 7 cubic yards of soil will be taken and used from the excavated holes for the ramada footings.

4.0 Geotechnical Analysis

Following the collection of soils during the site investigation, the team processed, analyzed, and classified the soil in the lab using the following tests to classify the soil: *Atterberg Limits (ASTM D4318),* and *Particle Size Distribution Using the Sedimentations (ASTM D7928-17)*. It was planned to also complete the *Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions (ASTM D3080/D3080M-11),* but due to laboratory time constraints and required maintenance on the machine, the team was unable to complete the direct shear testing. Instead, the geotechnical report produced during the construction of the nearby ceramics building was used to fill in missing data the team could not produce. The geotechnical testing was done by Speedie and Associates, dated August 9th, 2012.

4.1 Atterberg Limits

The Atterberg Limits testing produced a Plastic Limit value and Liquid Limit value, which allowed the team to determine a plasticity index, which was used in the USCS soil classification. The results from sample hole 1 and 2 were averaged to get a plastic limit of 32.93, and a liquid limit of 24.19. This data can be seen in Appendix A, Table 3-1. The plastic limit of soil is defined as the water content at which the soil begins to crumble when rolled into a ⅛ inch thread. The plastic limit is used to classify the fine-grained soil, indicating the toughness index of a soil, give an idea of the consultancy of the soil, predict the consolidation properties of soil while computing the settlement and bearing capacity of the soil, and used in determining the plasticity index of soil. The Atterberg Limits allowed the team to classify the soil using AASHTO. The soil classification was A-1-b-Stone fragments; gravel and sand.

4.2 Liquid Limit

The liquid limit (LL) is defined as the minimum water content at which a part of soil cut by a groove of standard dimension will flow together for a distance of 12 mm (1/2 inch) under an impact of 25 blows in the device. LL of soil is a very important property of fine grained soil (or cohesive soil), the value of liquid limit is used to classify fine grained soil, it gives us information regarding the state of consistency of soil on site, the LL of soil can be used to predict the consolidation properties of soil while calculating allowable bearing capacity and settlement of foundation, and the LL value of soil is also used to calculate the activity of clays and toughness index of soil [11].

4.3 Plasticity Index

The plasticity index (PI) is a measure of the plasticity of a soil. The plasticity index is the size of the range of water contents where the soil exhibits plastic properties. Soils with a high PI tend to be clay, those with a lower PI tend to be silt, and those with a PI of 0 (non-plastic) tend to have little or no silt or clay. The PI ranges can be seen below [10].

(0) – Non-plastic

(<7) – Slightly plastic

(7-17) – Medium plastic

(>17) – Highly plastic

4.4 Sieve Analysis

The team also conducted a sieve analysis to determine the particle size distribution of the soil and USCS classification of the soil. Hole 1 produced results found below.

Table 4- 1: Hole 1 Particle Size Distribution Results

Hole 1 Particle Distribution	
% Sand	79.21
% Gravel	11.29
% Fines	9.49
Coefficients	
D_{10}	0.15
D_{30}	0.25
D_{60}	0.60
C_C	0.69
C_U	4.00

The following figure is a visual representation of how the soil passed through the sieves.

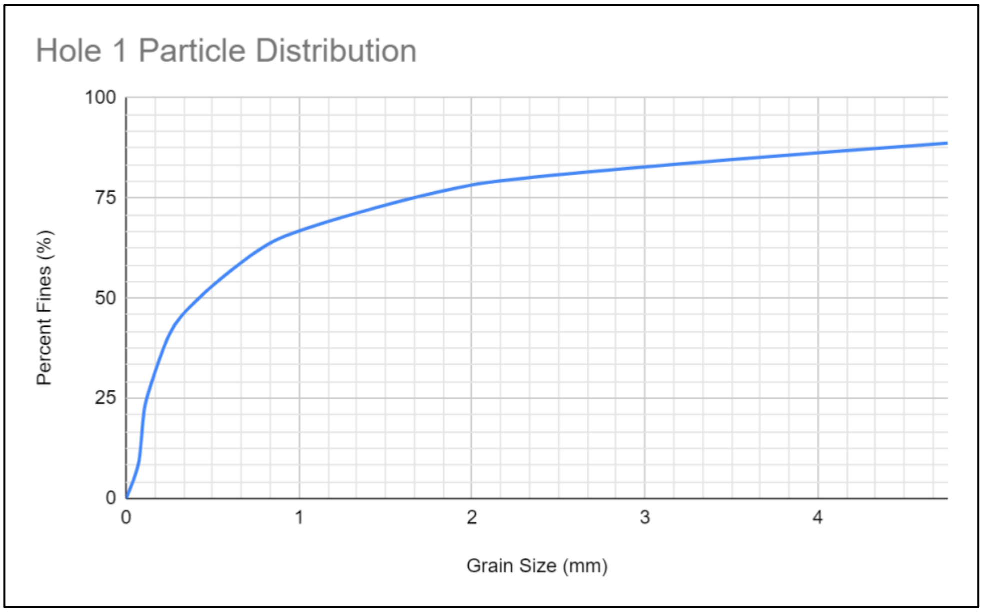

Figure 4- 1: Hole 1 Particle Size Distribution

The test results from hole 2 can be seen below in, Table 4-2, and Figure 4-2.

Table 4- 2: Hole 2 Particle Size Distribution Results

Hole 2 Particle Distribution	
% Sand	78.43
% Gravel	17.69
% Fines	3.87
Coefficients	
D_{10}	0.15
D_{30}	0.45
D_{60}	1.55
C_C	0.87
C_U	10.33

The following figure is a visual representation of how the soil passed through the sieves.

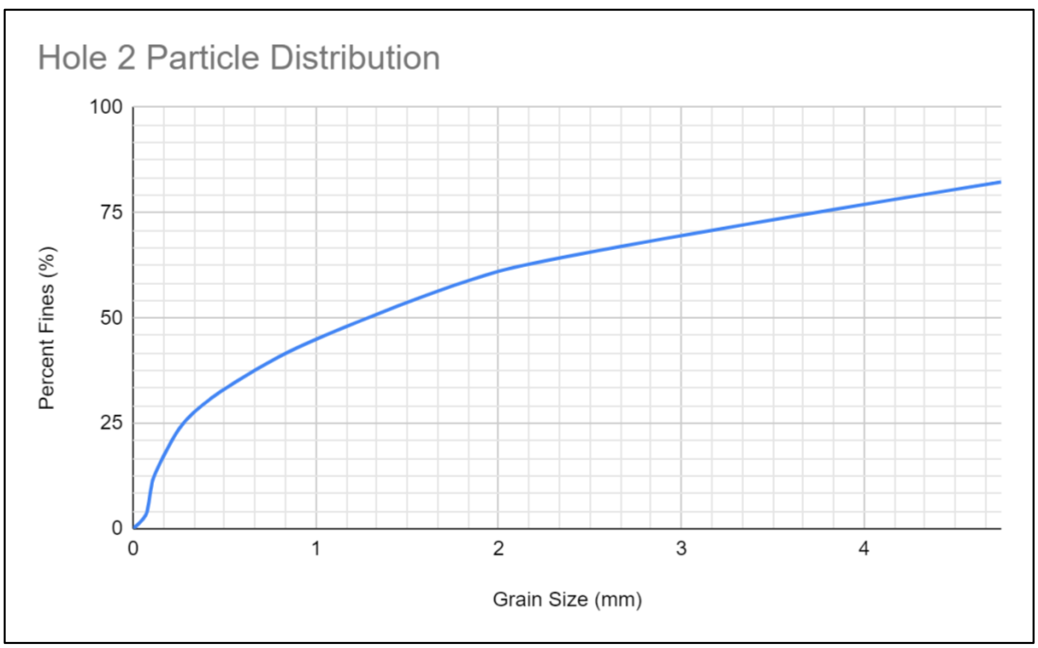

Figure 4- 2: Hole 2 Particle Size Distribution

The results from hole 1 and 2 were averaged to calculate an average Coefficient of Curvature, Coefficient of Uniformity, D10, D30, and D60. The coefficient of curvature

(Cc) is the parameter estimated using the gradation curve through sieve analysis. This parameter is used to classify the soil as well graded or poorly graded and is given by the relation as below.

The coefficient of uniformity (Cu) is defined as the ratio of D60 to D10. A value of Cu greater than 4-6 classifies the soil as well graded. When Cu is less than 4 it is classified as poorly graded or uniformly graded soil. D60 is the size of the sieve hole in which 60% of soil will pass through it. D30 is the size of the sieve hole in which 30% of soil will pass through it. D10 is the size of the sieve hole in which 10% of soil will pass through it. The averaged Cc, and Cu can be seen in Table 4-3.

Table 4- 3: Averaged Cc and Cu

Average Cc	0.783
Average Cu	7.17

These results were used to determine the USCS soil classification. The soil was classified as SP-SM poorly graded sand with silt from 0” - 2.5’. Limestone bedrock was reached at 2-2.5’.

4.5 Direct Shear

The Project Manual for the nearby Ceramics building was provided by Gregory Mace, Associate Director of Engineering and Inspection Services at NAU. The project manual was stamped by Johnson Waltzer Associates, LLC. The project manual includes a complete geotechnical report conducted by a local geotechnical firm called Speedie and Associates. A summary of the data pertaining to the needs of this project are as follows: 1) the native upper soils typically consist of silty clayey gravel with subordinate amounts of sand, 2) underlying these upper soils at depths ranging from 2 to 2.25 feet is limestone bedrock, 3) liquid limits on the order of 32 to 42 percent with plasticity indices from 4 to 10 percent, 4) a recommended safe allowable bearing capacity of 6,000 psf can be utilized for design, and 5) lateral pressures seen below in Table 4-4 [7].

Table 4- 4: Speedie and Associates Lateral Pressures

Active pressure (unrestrained walls)	35 pcf
Active pressure (restrained walls)	60 pcf
Passive pressure (continuous footings)	300 pcf
Passive pressure (spread footings or drilled piers)	350 pcf
COF (with passive pressure)	0.35
COF (without passive pressure)	0.45

5.0 Structural Analysis

5.1 Ramada Geometry

The basic geometry of the ramada was determined based on the surrounding structures, materials, and aesthetics within the ceramics complex. After taking measurements of the existing foundation, kiln, and walkways, the team designed the width and length of the ramada so that there is enough space to easily transport ceramic materials between walkways and gave enough distance between our proposed columns and existing structures. There were two design options for the roof of the ramada: a pitched roof or a monosloped roof, seen below in Figure 5-1.

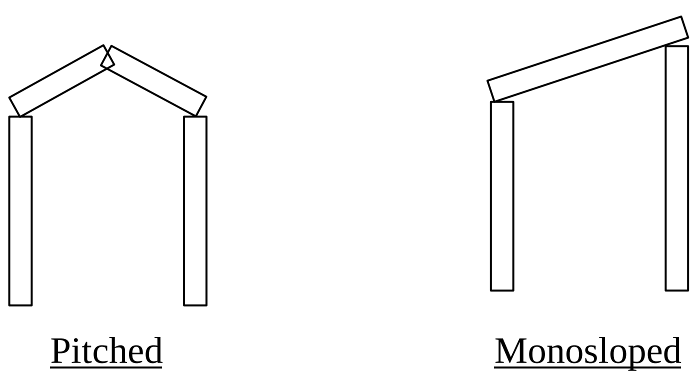

Figure 5- 1: Roof Alternatives

A decision matrix was created to weigh out the pros and cons of each structure design. The decision matrix can be seen in Table 5-1. The criteria chosen were: ability to shed water, design difficulty, construction difficulty, construction feasibility, client preference, cost of materials, and allowable design height to fit the kiln’s chimney. The criteria were weighted subjectively to what the team thought would best fit the area. The client preferred the monoslope roof but wasn't opposed to the other design option since existing nearby ramadas have sloped roofs. Construction feasibility was a criteria the team kept in mind due to the possibility of construction management students building this ramada in the future. The "allowable design height to fit chimney" criteria was a design criteria because of the hazards it could pose if the chimney was covered. Based on the decision matrix results, it was determined that the favorable design would be the monoslope roof. Both the pitched and monoslope roof would have to be paired with an additional water routing system to account for the new discharge amount. The monoslope was favorable because of cost of materials and constructability. The monoslope roof design would only require a simply supported beam, ridge beam, with joists. The pitched roof would have required a ridge beam and end posts.

Table 5- 1: Decision Matrix for Roof Geometry

		Monoslope		**Pitched**	
Criteria	**Weight**	**Score***	**Weighted Score**	**Score***	**Weighted Score**
Shed Water Away from Other Kilns	0.2	9	1.8	5	1
Design Difficulty	0.1	6	0.6	6	0.6
Construction Feasibility	0.3	7	2.1	4	1.2
Client Preference	0.1	9	0.9	6	0.6
Cost of Materials	0.25	10	2.5	7	1.75
Aesthetics	0.05	5	0.25	10	0.5
Allowable Design Height to Fit Chimney	0.1	7.5	0.75	4	0.4
Total	**1**	**N/A**	**8.9**	**N/A**	**6.05**

*Based on a scale of 1-10 (1 being the lowest score, 10 being the highest score

It was determined that the ramada would be rectangular in shape with a width of 18 feet, a length of 24 feet, and a roof slope of 1:18. The slope of the roof depended heavily on ensuring that the chimney protruded enough through the roof to prevent any possible cases of the roof catching on fire. Therefore, the height of the columns governed the slope of the roof. The height of the east columns will be 10 feet tall and the height of the west columns will be 9 feet tall to ensure appropriate clearance. The ramada will have 8 columns in total, 4 columns located at each corner and 4 additional columns located midlength of the longer edge. The number of columns was determined by the beam analysis. When the design initially had 4 columns, the beam spanning between the columns was 24' long and needed to be a very large and expensive member. Therefore, more columns were added to reduce the length of the beams. Six columns were attempted but the beam size was still too large and expensive. Eight columns resulted in standard sized beams. A visual representation of these measurements can also be seen in Figures 5-2 and 5-3 located below.

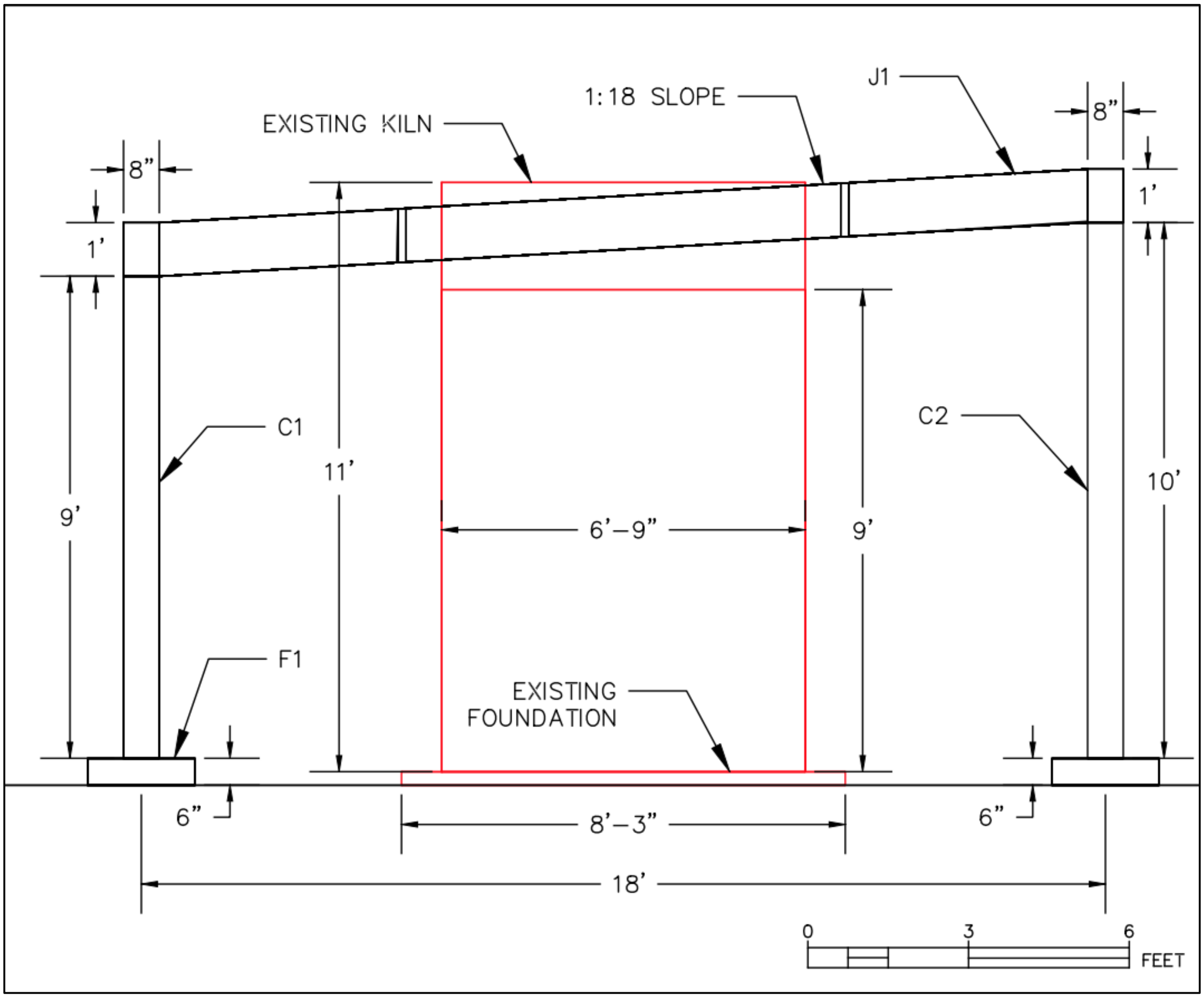

Figure 5- 2: Civil 3D South View of Ramada Geometry

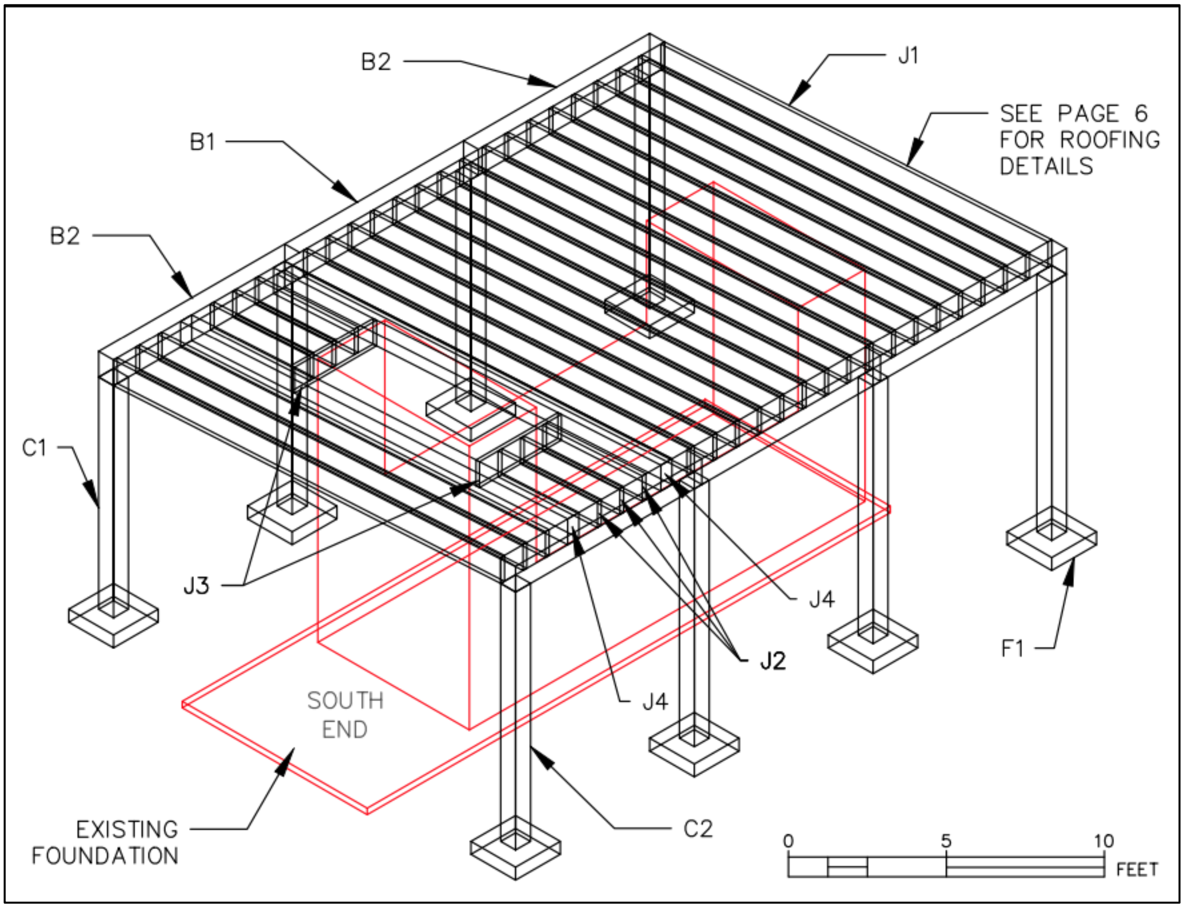

Figure 5- 3: Civil 3D East View of Ramada Geometry

5.2 Design Analysis

The design analysis of the ramada consisted of the creation of design loads, roof decking design, wood elements design including joists, beams, and columns, analysis of lateral force resisting system, connection design, and foundation design. The ASCE 7-16 Minimum Design Loads and Associated Criteria for Building and Other Structures, National Design Specifications for Wood Construction (NDS), and a Simpson Strong-Tie catalog were used in these processes.

5.2.1 Design Loads

Structural analysis was performed by following the guidelines outlined in the International Building Code (IBC), American Society of Civil Engineers (ASCE) 7-16, The City of Flagstaff Standards, and NAU's Technical Standards. The City of Flagstaff mandates that the 2018 IBC be used for structural design. Further, Section 1605 of the IBC specifies that the ASCE 7-16 must be used when determining design loads.

The design loads for the ramada were based on the “Minimum Design Loads and Associated Criteria for Building and Other Structures” (ASCE 7-16) and NAU Technical Standards. The team calculated the dead, live, snow, wind, and seismic loads for the proposed structure.

Table C3.1-1a Minimum Design Dead Loads from ASCE 7-16 was used to determine the dead loads. Within this table, the smallest (thickest) gauge decking was chosen for the roof of the ramada. The total dead load is 6 psf. The live load was based on ASCE 7-16 Table 4.3-1 “Minimum Uniformly Distributed Live Loads” for “Roof areas not intended for occupants” with no live load reduction. Snow load was determined using various tables within ASCE 7-16 and City of Flagstaff standards. The total roof live load is 20 psf. The ramada was classified as Category B for both Surface Roughness and Exposure category due to it being in a wooded area and height of the structure less than 30ft. The new structure is defined as Risk category II due to it not qualifying for other categories. Risk category I includes buildings/structures that pose little-to-no threat to the public in the event of failure. Conversely Risk category III includes buildings/structures that pose high threat to the public in the event of failure. The total snow load is 51 psf.

The design loads for wind are found using Components and Cladding (C&C ASCE 7-16 Chapter 30) and Main Wind Force Resisting System (MWFRS ASCE 7-16 Chapter 27). Using the Applied Technology Council (ATC) Hazards by Location tool, the wind speed in Flagstaff was found to be 101 mph for Risk Category II. Similar to snow load calculation, the wind load was calculated using variables such as exposure category, roof exposure, ground elevation factor, etc. All these factors are found in the ASCE 7-16. The maximum down C&C wind load is 28 psf and the maximum uplift is 35 psf. The maximum MWFRS wind load is 16 psf.

The design load for seismic was found using the ATC Hazards by Location tool. The website provided many variables needed to calculate the seismic load. ASCE 7-16 Chapter 12 was used to calculate the seismic load. Both the vertical and horizontal load caused by seismic activity was calculated. The vertical seismic load is 0.0646*dead load in psf and the horizontal seismic load is 0.2153*seismic weight in psf.

Refer to Appendix B for excel calculations for the dead load, live load, snow load, wind load for a monoslope roof, seismic load, and load combinations. See Table 5-2 for a summary of the design loads.

Table 5- 2: Design Loads

Load Type	Load in psf
Dead	6
Live (roof)	20
Snow	51
Wind (gravity)	30
Wind (uplift)	37
Wind (lateral)	16
Seismic (horizontal)	3.5
Seismic (vertical)	1.4

5.2.2 Decking

The decking to be used is 20-gauge corrugated metal decking. This gauge was determined by looking at a Canam Steel Deck catalog and choosing a gauge that could support the max loading of 57 psf [12]. Canam is the leading manufacturer of steel deck products. The length of the roof is 19’-0” therefore, two lengths of 12’-0” will be used. In order for the ridges to allow for water drainage, the decking shall be laid where the corrugations are running parallel to the joists. The decking will not be a structural member.

5.2.3 Joist Design

The bending capacity, shear capacity, and deflection capacity were calculated for the joists.

5.2.3.1 Calculations

The National Design Specification (NDS) for Wood Construction written by the American Wood Council was used to design the joists. According to the NDS, wood members must be designed to resist bending stress, shear stress, and deflection. More specifically, Chapter 3 Design Provisions and Equations of the NDS was used. For bending stress, the actual bending stress is compared with the allowable bending stress to determine the percent stressed the member is from bending. The actual bending stress was calculated using Equation 5.1.

Equation 5. 1: Actual bending stress

$$f_b = \frac{M}{S}$$

where,

$$f_b = actual\ bending\ stress\ (psi)$$

$$M = bending\ moment\ (lb - in) = \frac{WL^2}{8}$$

$$W = distributed\ load\ (lb/in)$$

$$L = length\ of\ joist\ (in)$$

$$S = section\ modulus\ (in^3) = \frac{bd^2}{6}$$

$$b = breadth\ of\ member (in)$$

$$d = depth\ of\ member\ (in)$$

The allowable bending stress was calculated using Equation 5.2.

Equation 5. 2: Allowable bending stress

$$F'_b = F_b * C_D * C_M * C_t * C_L * C_F * C_{fu} * C_i * C_r$$

where,

$$F'_b: allowable\ bending\ stress\ (psi)$$

$$F_b: bending\ stress\ based\ on\ material\ (psi)$$

$$C_D = load\ distribution\ factor$$

$$C_M = wet\ service\ factor$$

$$C_t = temperature\ factor$$

$$C_L = beam\ stability\ factor$$

$$C_F = size\ factor$$

$$C_{fu} = flat\ use\ factor$$

$$C_i = incising\ factor$$

$$C_r = repetitive\ member\ factor$$

The percentage stressed due to bending was calculated using Equation 5.3.

Equation 5. 3: Percentage of stress due to bending

$$\% = \frac{f_b}{F'_b} * 100$$

where,

$$f_b \ = actual\ bending\ stress\ (psi)$$
$$F'_b : allowable\ bending\ stress\ (psi)$$

For calculating the shear stress in the beam, the same process was used by finding the actual shear stress and comparing it to the allowable shear stress. The actual shear stress was calculated using Equation 5.4.

Equation 5. 4: Actual shear stress

$$f_v \ = \frac{3V}{2bd}$$

where,

$$f_v \ = actual\ shear\ stress\ (psi)$$
$$V \ = shear\ (lb) = \frac{WL}{2}$$
$$W \ = distributed\ load\ (lb/in)$$
$$L \ = length\ of\ joist\ (in)$$
$$b \ = breadth\ of\ member\ (in)$$
$$d \ = depth\ of\ member\ (in)$$

The allowable shear stress was calculated using Equation 5.5.

Equation 5. 5: Allowable shear stress

$$F'_v = F_v * C_D * C_M * C_i$$

where,

$$F'_v: allowable\ shear\ stress\ (psi)$$
$$F_v: shear\ stress\ based\ on\ material\ (psi)$$
$$C_D = load\ distribution\ factor$$
$$C_M = wet\ service\ factor$$
$$C_t = temperature\ factor$$
$$C_i = incising\ factor$$

The percentage stressed due to bending was calculated using Equation 5.6.

Equation 5. 6: Actual bending stress

$$\% = \frac{f_v}{F'_v} * 100$$

where,

$$f_v = actual\ bending\ stress\ (psi)$$
$$F'_v: allowable\ bending\ stress\ (psi)$$

For both the bending check and shear check, the bending stress and shear stress based on the material can be found in the NDS Supplement. More specifically, Tables 4A through 4F provide these values for different types of wood. Hem Fir was chosen for the joists as Hem Fir is the most common type of wood used for 2x members. The NDS specifies in Section 3.5.1 that the deflection of a wood member is to be calculated using standard methods of engineering mechanics. The formula for calculating deflection, in inches, can be seen in Equation 5.7.

Equation 5. 7: Deflection

$$\delta = \frac{5WL^4}{384EI}$$

Calculation	Result
Bending Stress (%)	86.8
Shear Stress (%)	27.9
Deflection (in)	0.72

where,

$$\delta = deflection\ (in)$$
$$W\ = distributed\ load\ (lb/in)$$
$$L\ = length\ of\ joist\ (in)$$
$$E\ = beam\ modulus\ of\ elasticity\ (psi)$$
$$I\ = beam\ moment\ of\ inertia\ (in^4) = \frac{bh^3}{12}$$

This deflection is compared to the allowable beam deflection. The allowable beam deflection was calculated using Equation 5.8.

Equation 5. 8: Allowable beam deflection

$$\delta' = L/240$$

where,

$$\delta' = deflection\ (in)$$
$$L\ = length\ of\ joist\ (in)$$

The allowable deflection is specified in the International Building Code in Table 1604.3. The deflection calculated in Equation 5.7 must not exceed the allowable deflection calculated in Equation 5.8. The calculations for the bending stress, shear stress, and deflection can be found in Appendix B.

5.2.3.2 Results

The selected size for the joists is a 2x12 Hem Fir #2 wood member. A 19 foot 2x12 Hem Fir #2 joist with a tributary width of 1’ was calculated to have a bending stress of 86.8%, a shear stress of 27.9%, and a total deflection of 0.72 inches. A summary of these results can be seen in Table 5-3.

Table 5- 3: Typical Joist Results

Calculation	Result
Bending Stress (%)	86.8
Shear Stress (%)	27.9
Deflection (in)	0.72

Three 19 foot 2x12 Hem Fir #2 joists nailed together with a tributary width of 2.75’ was calculated to have a bending stress of 61.2%, a shear stress of 25.58%, and a deflection of 0.66 inches. These joists have more load applied to them as they are framing the chimney and therefore have a higher tributary width. A summary of these results can be seen in Table 5-4.

Table 5- 4: Typical Chimney Results

Calculation	Result
Bending Stress (%)	61.2%
Shear Stress (%)	25.58%
Deflection (in)	0.66

Figure 5-4 shows the framing around the chimney in more detail.

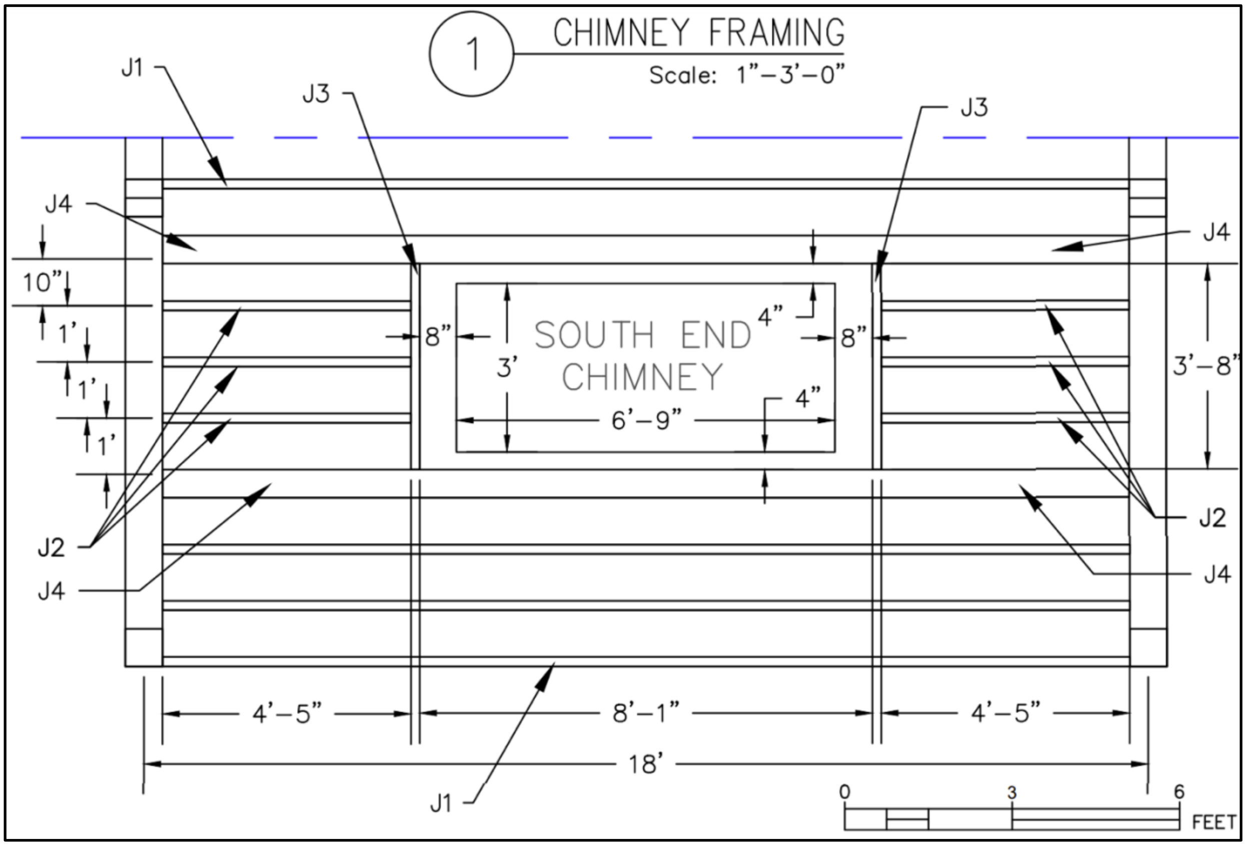

Figure 5- 4: Chimney Framing

5.2.4 Beam Design

5.2.4.1 Calculations

The same methods for calculating the bending stress, shear stress, and deflection for the joist design were used for the beam design. The only difference in the calculations is the loading applied to the beam and the beam material. Members that are larger than a 2x member are most commonly available in Douglas Fir #2 rather than Hem Fir #2. Douglas Fir #2 is a stronger wood type than Hem Fir #2 and therefore has higher bending stress and shear stress capacity. The load applied to the beam was calculated by determining the reaction at the end of a joist and dividing it by the joist spacing to get a load in pounds per linear foot (plf) applied to the beam. The calculations for beams can be found in Appendix D.

5.2.4.2 Results

A 8 foot 8x12 Douglas Fir #2 beam with a tributary width of 9' was calculated to have a bending stress of 59.1%, a shear stress of 15.58%, and a total deflection of 0.12 inches. A summary of these results can be seen in Table 5-5. These can be seen in Figures 5-2 and 5-3.

Table 5- 5: Beam Results

Calculation	Results
Bending Stress (%)	59.1
Shear Stress (%)	15.58
Deflection (in)	0.12

5.2.5 Column Design

5.2.5.1 Calculations

The same concepts such as shear stress and deflection were used to calculate the column capacity. Sections 6, 7, and 10 of Chapter 3 of the NDS specify the calculations needed to design a wood column. Rather than checking the bending stress, columns need to be checked for compression strength. The actual compression stress is compared with the allowable compression stress to determine the percent stressed the member is from compression. The actual compression stress was calculated using Equation 5.9.

Equation 5. 9: Actual compression stress parallel to grain

$$f_c = \frac{P}{A}$$

where,

$$f_c = actual\ compression\ stress\ parallel\ to\ grain\ (psi)$$
$$P = axial\ load\ applied\ to\ column\ (lb)$$
$$A = area\ of\ cross\ section\ (in^2)$$

The allowable compression stress was calculated using Equation 5.10.

Equation 5. 10: Allowable compression stress

$$F'_c = F_c * C_D * C_M * C_t * C_F * C_i * C_P$$

where,

$F'_c: allowable\ compression\ stress\ parallel\ to\ grain\ (psi)$

$F_c: compression\ stress\ parallel\ to\ grain\ based\ on\ material\ (psi)$

$C_D = load\ distribution\ factor$

$C_M = wet\ service\ factor$

$C_t = temperature\ factor$

$C_F = size\ factor$

$C_i = incising\ factor$

$C_P = column\ stability\ factor$

The column stability factor was calculated using Equation 5.11.

Equation 5. 11: Column stability factor

$$C_P = \frac{1 + (F_{CE}/F_c{}^*)}{2c} - ((\frac{1 + (F_{CE}/F_c{}^*)}{2c})^2 - (\frac{F_{CE}/F_c{}^*}{c}))^{1/2}$$

where,

$C_P = column\ stability\ factor$

$$F_{CE} = \frac{0.822 * E_{min}{}'}{(l_e/d)^2}$$

$E_{min}{}' = adjusted\ modulus\ of\ elasticity\ (psi)$

$l_e = effective\ column\ length\ (in) = K_e * l$

$K_e = buckling\ length\ coefficient$

$l = column\ length\ (in)$

$d = depth\ of\ member\ (in)$

$F_c^* = reference\ compression\ design\ value\ parallel\ to$
$grain\ multiplied\ by\ all\ applicable\ adjustment\ factors\ except\ C_P\ (psi)$

$c = 0.8\ for\ sawn\ lumber$

The percentage of stress due to compression was calculated using Equation 5.12.

Equation 5. 12: Percentage of stress due to compression

$$\% = \frac{f_c}{F'_c} * 100$$

where,

$f_c = actual\ compression\ stress\ parallel\ to\ grain\ (psi)$

$F'_c: allowable\ compression\ stress\ parallel\ to\ grain\ (psi)$

The shear stress was calculated using Equation 5.4, 5.5, and 5.6. The deflection was calculated using Equation 5.7, 5.8, and 5.13.

Equation 5. 13: Deflection due to axial load

$$\delta = \frac{PL}{AE}$$

where,

$\delta = deflection\ due\ to\ axial\ load\ (in)$

$P = axial\ load\ applied\ to\ column\ (lb)$

$L = length\ of\ column\ (in)$

$A = area\ of\ cross\ section\ (in^2)$

$E = beam\ modulus\ of\ elasticity\ (psi)$

Due to the beam bearing on the compression member (column), the compressive bearing stress must be calculated for the column. The actual compressive bearing stress was calculated using Equation 5.9. This equation can be found in Section 3.10.1 of the NDS. The actual compressive bearing stress is then compared to the Fc* value found in Equation 5.9 to determine the percent stressed, seen in Equation 5.14.

Equation 5. 14: Actual compression stress parallel to grain

$$\% = \frac{f_c}{F_c^*} * 100$$

where,

$f_c = actual\ compression\ stress\ parallel\ to\ grain\ (psi)$

$F_c^* = reference\ compression\ design\ value\ parallel\ to\ grain$
$multiplied\ by\ all\ applicable\ adjustment\ factors\ except\ C_P\ (psi)$

5.2.5.2 Results

The middle tallest columns, 10 feet tall, were designed and the 9 foot columns will work by inspection. The middle columns are most stressed due to their larger tributary width of the roof. A 8x8 Douglas Fir #2 column with a tributary width of 12' was calculated to have a bending stress of 25.3%, a shear stress of 1.75%, a compression stress of 29.4%, and a total deflection of 0.2 inches. The calculations for columns can be found in Appendix F. A summary of these results can be seen in Table 5-6.

Table 5- 6: Column Results

Calculation	Results
Bending Stress (%)	25.3
Shear Stress (%)	1.75
Compression Stress (%)	29.4
Deflection (in)	0.2

5.2.6 Foundation Design

5.2.6.1 Calculations

The foundation was designed by following Terzaghi's Bearing Capacity Equation for square foundations. From Section 6.3 of *"Principles of Foundation Engineering"* by Braja M. Das and Nagaratnam Sivakugan can be seen below.

Equation 5. 15: Ultimate bearing capacity

$$q_u = 1.3c'N_c + qN_q + 0.4\gamma BN_\gamma \ (square\ foundation)$$

where,

$$c' = soil\ cohesion$$

$$\gamma = unit\ weight\ of\ soil\ (lb/ft^3)$$

$$q = effective\ stress\ at\ the\ bottom\ of\ the\ foundation\ (psi)$$

$$N_c, N_q, N_\gamma = bearing\ capacity\ factors$$

$$B = diameter\ of\ foundation\ (ft)$$

The effective stress is calculated using equation 5.16 from the *"Principles of Foundation Engineering."*

Equation 5. 16: Effective stress

$$q = D_f \gamma$$

where,

$$q = effective\ stress\ at\ the\ bottom\ of\ the\ foundation\ (psi)$$
$$D_f = depth\ of\ foundation\ (ft)$$
$$\gamma = unit\ weight\ of\ soil\ (lb/ft^3)$$

Terzaghi's bearing capacity equation has been modified while recognizing the three components from cohesion, surcharge and the soil weight that contribute to the ultimate bearing capacity. The soil was classified as SP-SM poorly graded sand with silt which has a cohesion value of zero. The bearing capacity factors were found using Table 6.1 - "Terzaghi's Bearing Capacity Factors," and can be seen in Appendix F. These factors were dependent on the angle of friction of the soil. The angle of friction for SP-SM is estimated to be 35 degrees, which is consistent with what is found in the Speedie Geotechnical Report.

Once the ultimate bearing capacity (q_u) was found it is necessary to determine the allowable bearing capacity (q_{all}) which is the load per unit area the foundation applies to the underlying soil when the structure is constructed. This was done by using equation 5.17 from the *"Principles of Foundation Engineering"* textbook.

Equation 5. 17: Allowable bearing pressure

$$q_{all} = \frac{q_u}{FS}$$

where,

$$q_{all} = allowable\ bearing\ pressure\ (psf)$$
$$q_u = ultimate\ bearing\ capacity\ (psf)$$
$$FS = factor\ of\ safety$$

A factor of safety of 3 was used to make up for any uncertainties associated with the shear strength parameters and the simplifications used in the bearing capacity theory.

After the allowable bearing capacity was calculated the maximum allowable load (Q) could then be calculated. The maximum allowable load is the downloading from the columns the foundation can handle. The column download was calculated to be 7418 lbs. The maximum allowable load was calculated using equation 5.18 below.

Equation 5. 18: Maximum allowable soil load

$$Q = q_{all}B$$

where,

$$Q = maximum\ allowable\ soil\ load\ (plf)$$
$$q_{all} = allowable\ bearing\ pressure\ (psf)$$
$$B = diameter\ of\ foundation\ (ft)$$

After the loading calculations were completed the reinforcement needed to be added. The minimum required area of steel was calculated using equation 5.19 below from the American Concrete Institution (ACI) 318-19. Once the minimum area of steel was calculated the minimum spacing and clear cover requirements needed to be checked. This was done using equation 5.19 and 5.20 below.

Equation 5. 19: Minimum required area of reinforcement

$$A_{s,min} = 0.0018A_g$$

where,

$$A_{s,min} = minimum\ required\ area\ of\ reinforcement\ (in^2)$$
$$A_g = gross\ cross\ sectional\ area\ (in^2)$$

The minimum required width of the foundation to compensate the minimum amount of reinforcement was checked using Equation 5.20.

Equation 5. 20: Minimum width of foundation

$$b_{w,min} = 2(clear\ cover) + 2(stirrup\ diameter) + ND + (N-1)(clear\ spacing) + 0.5$$

where,

$$b_{w,min} = minimum\ width\ of\ foundation\ (ft)$$
$$N = number\ of\ bars$$
$$D = bar\ diameter\ (in)$$
$$clear\ cover = 1.5"\ for\ normal\ exposure\ per\ ACI$$

$$stirrup\ diameter = 0.5\ inches\ for\ \#4\ bars$$
$$clear\ spacing = 1"\ minimum\ per\ ACI$$

5.2.6.2 Results

The ultimate bearing capacity of a 3 ft (L) x 3 ft (W) x 1 ft (D) square concrete foundation sitting on limestone bedrock was found to be 19366.27 pounds per square foot (psf).. The net allowable bearing capacity was determined to be 6353.34 psf. This value is close to the 6000 psf value given in the Speedie geotechnical report.

The maximum applied load was calculated to be 824.22 psf. All calculations can be seen in Appendix F.

The vertical reinforcement was calculated to be (3) #5 bars each way. The minimum area of steel was calculated to be 0.778 square inches. The spacing was set at 10.5" on center (O.C.) minimum. The shear reinforcement was calculated to be (3) #3 stirrups spaced at 10.5" O.C. reinforcement calculations can be seen below in Appendix H.
A summary of the foundation results can be seen in Table 5-7.

Table 5- 7: Foundation Results

Calculation	**Result**
Pier Depth Below Surface(ft)	2.5
Pier Height Above Surface (ft)	0.5
Pier Length and Width (ft)	3
Ultimate Bearing Capacity (psf)	19366.27
Net Allowable Bearing Capacity (psf)	6353.34
Maximum Applied Load (psf)	824.22
Minimum Required Area of Steel (in^2)	0.778
Vertical Reinforcement	(3) #5 bars at 10.5" O.C. each way
Shear Reinforcement	(3) #3 stirrups at 10.5" O.C.

5.2.7 Connection Design

5.2.7.1 Calculations

The Simpson Strong-Tie Wood Construction Connectors Catalog was used to determine the types of connections to use. This brand of connectors is very common among contractors and is widely available at local construction stores like Home Depot and Lowes. The down load, the uplift load, and the lateral load were determined for each of the connections as follows: 1) joist to beam, 2) beam to column, and 3) column to foundation. The loads at each connection can be seen in Appendix G. A summary of the end reactions can be seen in Table 5-8.

Table 5- 8: Connection Results

Calculation	Result
(1) Joist down load (lb)	576.234
(1) Joist uplift load (lb)	119.7
(3) Joist down load (lb)	1496.6
(3) Joist uplift load (lb)	329.175
End beam down load (lb)	3612.71
End beam uplift load (lb)	259.2
End beam lateral load (lb)	408
Middle beam down load (lb)	7225.42
Middle beam uplift load (lb)	518.4
Middle beam lateral load (lb)	816
Column down load (lb)	7418
Column uplift load (lb)	408
Column lateral load (lb)	518.4
Column moment (lb-ft)	4080

5.2.7.2 Results

For the joist to beam connection, the 2x12 joists will be connected to the beam using a modified HU212. The modification will be the seat of the HU212 being sloped at 3.5°. For the two (3) 2x12 joists nailed together, the joists will be connected to the beam using a modified HU212-3. This connection will have the same modification of a sloped seat of 3.5°. These can be seen below in Figure 5-5.

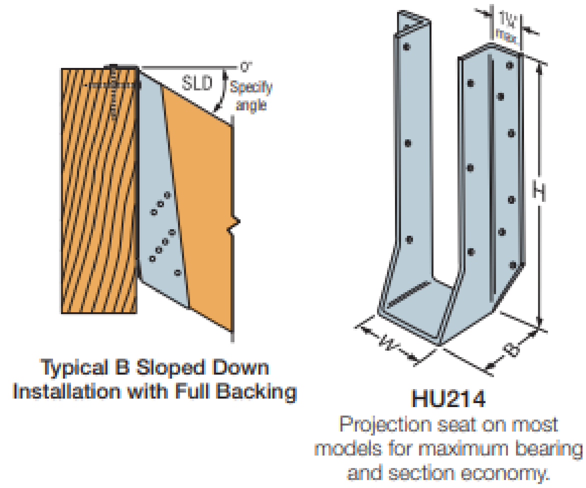

Figure 5- 5: Typical HU Joist Hanger [14]

For the beam to column connection, the middle columns and outer columns will have different connections. The beams bearing on the middle columns will be connected with a 1616HT each side. The beams bearing on the outer columns will be connected with an LCE4. These can be seen below in Figure 5-6.

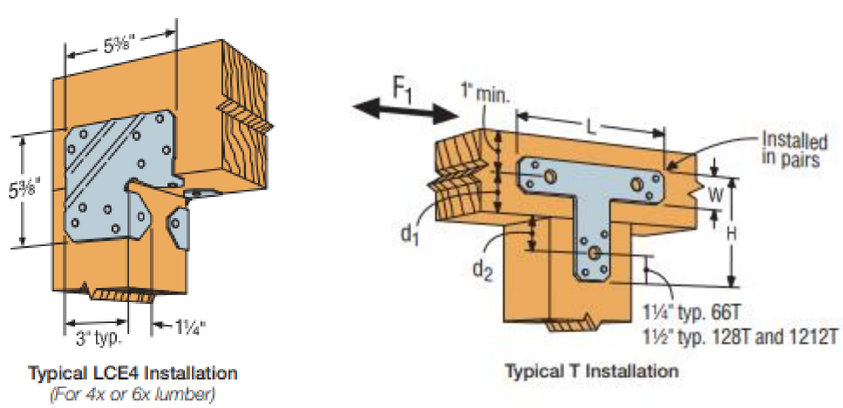

Figure 5- 6: Typical HT and LCE4 [14]

For the column to foundation connection, a MPB88Z with (2) 3/4" machine bolts will be used. This connection is placed in the wet concrete when the piers are poured. This can be seen below in Figure 5-7.

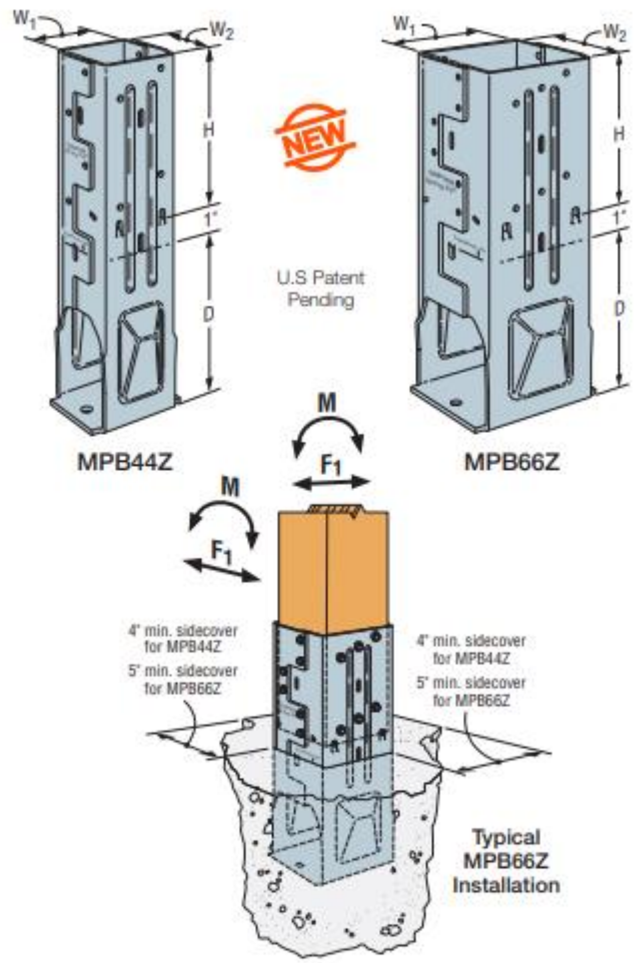

Figure 5- 7: MPB88Z [14]

5.2.8 Lateral Analysis

The lateral force resisting system for the ramada is cantilevered columns. The lateral capacity of the columns was already calculated and determined in the column section of the report. More specifically, the columns were 41.7% stressed in bending, and therefore pass the lateral check. The last piece of analysis for lateral calculations is determining the size and nailing of the plywood that will provide a small amount of lateral resistance. The Special Provisions for Wind and Seismic of the National Design Specifications for Wood was used to determine this. The thickness of the plywood depends on the total load being applied; a total max load of 51 psf. The design of plywood can be found in Appendix C. A summary of the plywood calculations can be seen in Table 5-9.

Table 5- 9: Lateral Plywood Results

Calculation	Results
Lateral load on structure (D+0.6Wx) (psf)	16
Lateral load on plywood (plf)	16
Allowable sheathing and Single-Floor grade, 6d nail size, 15/32 in panel thickness, 2 in face load (plf)	670
Ω	2
Plywood percent stress (%)	2.4

The plywood will be nailed directly to the top of the joists and will lie directly beneath the metal decking. The plywood will be 15/32" thick, spanning joist to joist in the north-south direction, and nailed down with 0.113" (6d) nails spaced 6" O.C.

6.0 Material Specifications

The metal roofing can be any type of metal that is 20 Gauge and corrugations spanning parallel to the roof slope. The plywood must be 15/32" thick Douglas Fir #2. The material for the joists will be Hem Fir #2. The material for the beams and columns will be Douglas Fir #2. All the connections will be Simpson Strong-Tie or equivalent. The concrete will have a minimum compressive strength of 4500 psi. The rebar will have a minimum yield strength of 60 ksi.

7.0 Costs of Design Implementation

A rough cost estimate was performed for the ramada materials. A table showing each element of the ramada and its price can be seen below in Table 7-1.

Table 7- 1: Approximate Material Estimate

Material	Quantity	Cost	Notes
2x12 Joists*	26	$489.00	joists 19' in length
8x12 Beams**	6	$4,684.00	beams 12' in length
8x8 Columns**	8	$3,803.00	(3) 10' in length, (3) 9' in length
Plywood*	456 square feet	$311.00	area of roof = 456 ft2
Metal Roofing***	456 square feet	$1,350.00	area of roof = 456 ft2
Philip Flat Head Sheet Metal Stainless Steel Screws	300 screws	$30.00	
Concrete****	14 cubic feet per pier	$520.00	1 pier=14 ft3 of concrete
Rebar*****	42 feet per pier	$336.00	each pier has 42 ft of rebar
HU212	52	$468.00	
HU212-3	4	$184.00	
1616HT	8	$298.50	
LCE4	8	$38.00	
MBP88Z	8	$1,592.00	
Clay	0.5 cubic yards	$40.00	
Class II Backfill	1.5 cubic yards	$90.00	
Total		$14,223.50	

*Home Depot website
**Boards and Beams website
***Roofing Calculator website
****HomeGuide website
*****Home Advisor website

The overall cost of the ramada materials is projected to be approximately $14,546. This is above the client original budget of $10,000. As this is a rough estimate retail stores were used to determine pricing which could lead to an increase in price. This estimate exceeds the client's budget by $4,546. This is due to the ramada being much larger than first expected. The client wished to cover the walkways along the sides of the kiln. This increased the length and width of the ramada by a significant amount. This caused the need for longer members and overall more members. The only way to reduce the cost of this project is to reduce the size of the ramada and therefore reduce the amount of materials needed. Originally the ramada was intended to have only 4 columns with larger members but it made it harder to find larger connections. The team then decided to add more columns to reduce the size of the members and have more options for connections.

8.0 Impacts Analysis

The design and construction of this design has many social, economic, and environmental impacts.

8.1 Social

There are numerous social impacts that are a result of this project. First, the ramada allows for students and faculty of the ceramics department to use the kiln during inclement weather conditions. This allows for a positive and communal experience of using the kiln. By providing cover from weather, students and faculty will be protected and therefore enjoy firing in the existing kiln.

Second, the aesthetics of the ramada will greatly impact the student's and faculty's view on ceramics at NAU. By expanding upon the aesthetics of the complex, the ceramics department increases their uniqueness and identity on campus. Ceramics students come to NAU to help facilitate their growth as an artist, and need an environment that strikes creativity and pride. This additional ramada will allow NAU Ceramics students to be proud of their college and aid in the motivation needed to complete a new piece of art. Lastly, the construction of the ramada by construction management students will continue to back the feeling of pride the students have in their college knowing their fellow peers used the knowledge being taught to them to bring happiness to others.

8.2 Environmental

There are numerous environmental impacts that are a result of this project. First, the area has no drainage design and therefore when it rains, the water runs off of the western parking lot and down the slope and into the existing ramada. The client has specifically noted that the area gets flooded during storm events. With the construction of this new ramada, the water is being rerouted to not interfere with the covered kiln. This provides a level of drainage control that the area lacked before. Although there is diversion of runoff from one structure, surrounding areas could potentially see an increase in runoff. This is due to the water landing on the metal roof and running off rather than landing on the kiln if the ramada were not there. Second, the height of the structure could cause disturbance to the wildlife living in the adjacent tree. Third, the construction of piers in the area could cause disturbance to the underlying soils and limestone. Depending on the type of tools used to dig the holes, the underlying soils will be disturbed.

Another environmental impact of this project is the increasing amounts of CO_2 that will be let out into the atmosphere due to increased usage of the Kiln. Before the ramda is placed the kiln is operating a minimum usage due to weather impacts, but once it is covered the kiln will be able to operate more days out of the year leading to more CO_2 being let out from the firing of the kiln. Due to the ramada being used more it will increase the usage of fuel. The increase of use of the kiln will cause more cutting down of more trees for fuel.

8.3 Economical

There are numerous economic impacts that are a result of this project. First, the existing kiln has the potential for damage and deterioration due to weathering. By protecting the kiln with a ramada, the kiln will have a longer lifespan and the university will save money rather than paying for repairs or replacements. Although the construction is an upfront cost with less maintenance repair over the years the profit of this project will increase.

Second, the ceramics department is paying to get this ramada built and is therefore assumed to be using some of the ceramic's student's tuition and fees to cover the cost. With the ability to use the kiln all year round the ceramic classes will increase in the availability. This will lead to more students having the opportunity to sign up for ceramic classes which would increase the money inflow in the ceramics complex. Another economic impact is the increase in the amount of students wanting to attend NAU's ceramics school due to the number of kilns that are available to use year round.

Lastly, the continuing commitment by NAU to keep the complex clean, safe, and updated may attract the attention of donors willing to donate their knowledge, money and time into the Ceramics Department to continue to foster the growth of the college.

9.0 Summary of Design Work

The original schedule was based on the estimated hours the team quantified for each task. This differs from the original estimated scheduling for the project. The proposed schedule can found in the Appendix Figure I-1. The actual schedule can be found in the appendix Figure I-2 which displays the team's GANTT chart that shows the task completion timeline for the project.
The major change in the schedule was due to COVID-19 the school year started and finished two weeks earlier. This caused the schedule to shift. Minor setbacks were encountered but did not deter the overall schedule of the project. A setback the team experienced was unable to perform surveying during the summer due to COVID-19. The surveying was pushed towards the beginning of the semester and was completed within a few weeks of starting. Another minor setback was the team did not have lab access to perform geotechnical analysis. The team originally planned to have geotechnical started by the end of August but didn't actually start until the 2nd week of September. This shift caused the geotechnical analysis to be completed by the 60% submittal instead of the 30% submittal. In order to keep the schedule on track, structural analysis started before the 30% submittal. The structural analysis was completed before the 90% submittal due to delays in calculations.

10.0 Summary of Staffing and Engineering Costs

In our original proposal the team estimated 786 hours to complete the ramada structure, seen in Table 10-1. This included the site visits, surveying, geotechnical and structural analysis, and design work for the project. The original cost of the engineering services was estimated to be $80,248 which also included supplies.

Table 10- 1: Proposed Staffing Breakdown

Proposed Hours/Budget				
Personnel	Classification/Role	Hours	Hourly Rate	Cost
	Senior Engineer	72	$201	$14,504
	Engineer	183.5	$132	$24,310
	Engineer in Training	321.5	$99	$31,944
	Lab Technician	10	$47	$476
	Engineering Intern	129	$21	$2,805
	Administrative Assistant	70	$62	$4,359
	Total Personnel			**$78,398**
Supplies	Classification	Days	Daily Rate ($)	Cost
	Survey	1	$250	$250
	Geotechnical Equipment	1	$200	$200
	Geotechnical Lab	4	$350	$1400
	Total Supplies			$1850
Total	**Total Cost**			**$80,248**

The team accumulated a total of 621 hours to complete the project. The Table 10-2 below shows the distribution of hours and cost of each personnel. The total cost for engineering services and lab supplies totaled at $65,283. The change in hours came from various aspects in the project. Originally the team was looking to complete the project in 20 weeks but decreased to 16 weeks due to the pandemic. It was originally intended to complete surveying during the summer but there was no access to survey equipment, which shortened the project length. A lot more time was allocated to the plan set when in reality the team did not spend as much time on it. The scope of the hydraulic analysis also changed which caused a decrease in ours; Originally it was thought the hydraulic analysis would be more in depth. Another change came in the geotechnical analysis. The team was unable to perform the direct shear test due to the equipment being under maintenance. Another restriction that caused lab personnel hours to decrease is due to COVID-19, not all team members were able to be in the lab at once as the team took precautions.

Table 10- 2: Actual Staffing Breakdown

Actual Hours/Budget				
Personnel	Classification/Role	Hours	Hourly Rate	Cost
	Senior Engineer	65.75	$201	$13,245
	Engineer	200	$132	$26,496
	Engineer in Training	175.5	$99	$17,438
	Lab Technician	20	$47	$952
	Engineering Intern	115	$21	$2,500
	Administrative Assistant	45	$62	$2,802
	Total Personnel			**$63,433**
Supplies	Classification	Days	Daily Rate ($)	Cost
	Survey	1	$250	$250
	Geotechnical Equipment	1	$200	$200
	Geotechnical Lab	4	$350	$1400
	Total Supplies			$1850
Total	**Total Cost**			**$65,283**

The following tables display the hours broken down per role for each task. As expected, the senior engineer hours were kept as low as possible as they had the highest hourly rate. The team originally estimated that the engineer in training (EIT) would accumulate a majority of the hours, seen in Table 10-3. The bulk of the EIT's hours were to come from the project manager task as the project manager would mostly oversee the design of the project. The engineer and intern would come in a second due to assisting the EIT. Table 10-4 shows the team's accumulated hours broken down per role for each task. In the actual hours log, the team saw an increase of hours for the engineer while the EIT and intern decreased in hours. The engineer had a slight increase in most of the tasks than originally anticipated. The engineer saw more work than anticipated due to the lack of structural engineering knowledge the group had. The structural engineering courses at NAU are limited. Many of the calculations had to be led and taught by the engineer. Additionally, this project is heavily hand calculation based. The team did not have access to structural engineering programs and therefore, hand calculations were required.

Table 10- 3: Proposed Hours Broken Down per Role

Proposed Staff Breakdown per Task						
Task	Roles					
	SENG	ENG	EIT	LAB	INT	AA
Site Visit (hrs)	0	4	10.5	0	11.5	0
Geotechnical Analysis (hrs)	0	10	0	10	16	10
Structural Analysis (hrs)	10	33	55.5	0	25	0
Material Specification (hrs)	2	3	8	0	0	0
Plan Set/Cost Estimate (hrs)	12	12	22	0	7	0
Project Management (hrs)	48	121.5	225.5	0	69.5	60

Table 10- 4: Actual hours broken down per role

Actual Staff Breakdown per Task						
Task	Roles					
	SENG	ENG	EIT	LAB	INT	AA
Site Visit (hrs)	7.25	15	24	1	13.5	0
Geotechnical Analysis (hrs)	0	16.5	1	13	0	0
Structural Analysis (hrs)	21.5	39.5	46	0	17.5	0
Material Specification (hrs)	3	4	1	0	0	0
Planset/Cost Estimate (hrs)	4	8	3	0	12	0
Project Management (hrs)	30	117	100.5	6	72	45

11.0 Conclusion

The objective of this project was to provide shelter from weather for students and faculty who use a kiln at the Ceramics Complex at Northern Arizona University. A topographic survey was needed to create a topographic map and site map. Geotechnical analysis was needed to determine the soil classification and to determine the allowable soil bearing pressure of the soil around the kiln. Two alternatives were developed for the type of roof structure the ramada would have. The two alternatives were evaluated in a decision matrix to determine the best option. Wood structural analysis was performed to determine the sizes of wood members needed for the proposed ramada geometry. Foundation analysis was needed to determine the size of footings that the columns required to keep the structure stable. The project was completed on time and met the objectives of the project.

12.0 References

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Appendices

Appendix A- Geotech Testing

A-1 Atterberg Limits Data

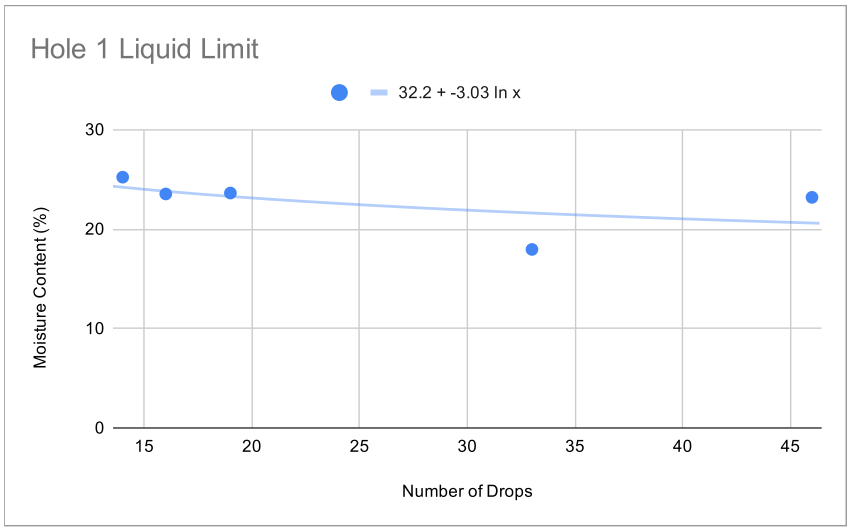
Hole 1:

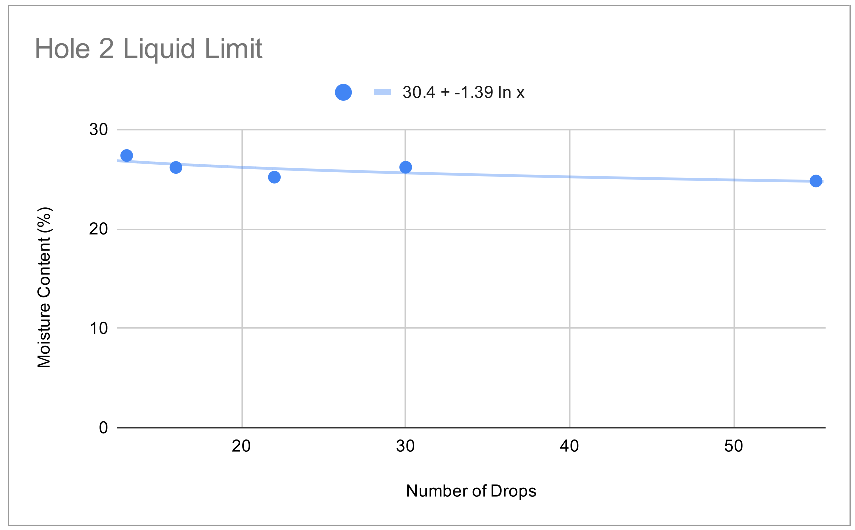
Sample	Mc (g)	Mm (g)	Md (g)	N	Moisture Content (%)
1	10.7	27.7	24.5	46	23.1884058
2	10.6	19.8	18.4	33	17.94871795
3	10.8	26.5	23.5	19	23.62204724
4	10.8	23.4	21	16	23.52941176
5	11	25.4	22.5	14	25.2173913

Atterberg Limit Results	
Plastic Limit	32.93
Liquid Limit	24.19
Pasticity Index	-8.75

Hole 2:

Sample	Mc (g)	Mm (g)	Md (g)	N	Moisture Content (%)
1	10.9	27	23.8	55	24.80620155
2	16.9	32.8	29.5	30	26.19047619
3	10.9	27.3	24	22	25.19083969
4	10.8	24.3	21.5	16	26.1682243
5	10.6	22.7	20.1	13	27.36842105





LL = 22.44680625 <-- N=25

LL = 25.9257626 <-- N=25

Average LL = 24.19

A-2 Sieve Analysis Data

Sieve #	Sieve Opening (mm)	Mass of Empty Sieve (g)	Mass of Sieve and retained sample (g)	Mass of Sample (g)	Rn	Sum Rn	Percent Finer
4	4.75	510.2	564.1	53.9	11.29268804	11.29268804	88.70731196
10	2	446.1	495.9	49.8	10.4336895	21.72637754	78.27362246
20	0.85	415.7	483	67.3	14.10014666	35.8265242	64.1734758
40	0.425	393.3	460.1	66.8	13.99539074	49.82191494	50.17808506
60	0.25	343.6	387.5	43.9	9.197569663	59.0194846	40.9805154
140	0.106	339.4	427.6	88.2	18.47894406	77.49842866	22.50157134
200	0.075	326.1	388.2	62.1	13.0106851	90.50911376	9.490886235
pan	0	369.9	415.2	45.3	9.490886235	100	0
				477.3			

40.5367771

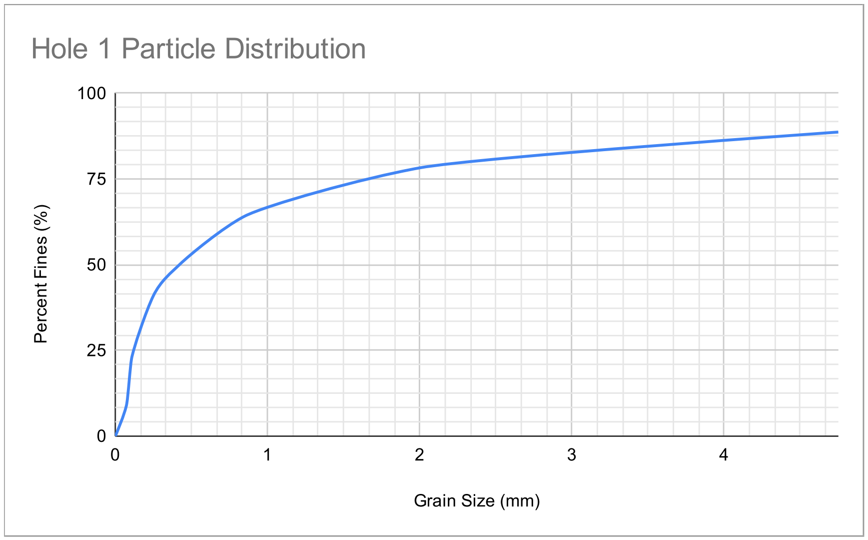
194.1	501.6	307.5
194.1	364.7	170.6
		478.1

6.680869446

evap

143.8	268.8	125	moist
143.8	200.6	56.8	dry

2% 0.1673290107



% sand	79.21642573
% gravel	11.29268804
% fines	9.490886235
D10	0.15
D30	0.25
D60	0.6
Cc	0.694
Cu	4

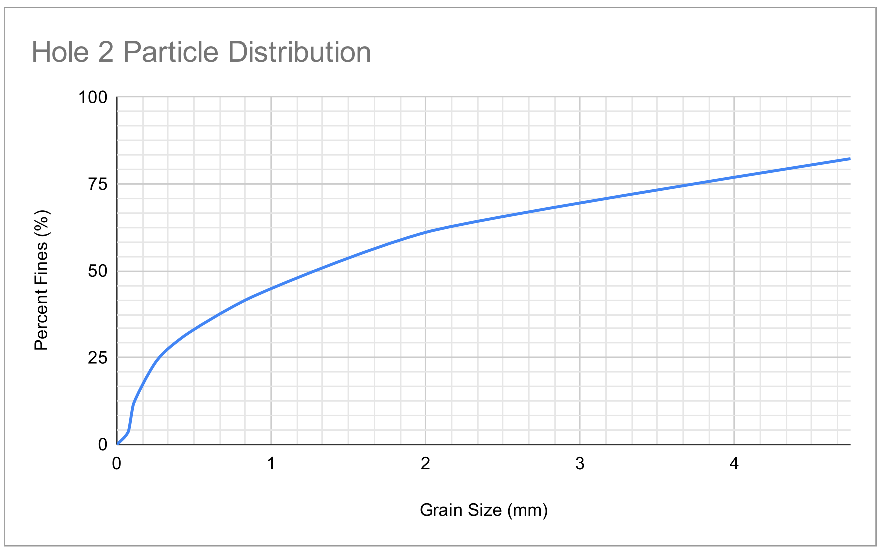
Hole 1 Particle Distribution	
% sand	79.21642573
% gravel	11.29268804
% fines	9.490886235
D10	0.15
D30	0.25
D60	0.6
Cc	0.6944444444
Cu	4

Sieve #	Sieve Opening (mm)	Mass of Empty Sieve (g)	Mass of Sieve and retained sample (g)	Mass of Sample (g)	Rn	Sum Rn	Percent Finer
4	4.75	520.7	619.9	99.2	17.69532644	17.69532644	82.30467356
10	2	447.3	566.2	118.9	21.20941848	38.90474492	61.09525508
20	0.85	395.4	502.6	107.2	19.12236889	58.02711381	41.97288619
40	0.425	390.6	452.7	62.1	11.07741705	69.10453086	30.89546914
60	0.25	345.7	386.3	40.6	7.242240457	76.34677132	23.65322868
140	0.106	338.5	407	68.5	12.21905102	88.56582233	11.43417767
200	0.075	318.6	361	42.4	7.563325009	96.12914734	3.870852658
pan	0	362.4	384.1	21.7	3.870852658	100	0
				560.6			

220.3	577.8	357.5
220.3	426.1	205.8
		563.3

evap	129.2	227.1	97.9	moist
	129.2	167.9	38.7	dry

2% 0.4793183029



% sand	78.43382091
% gravel	17.69532644
% fines	3.870852658
D10	0.15
D30	0.45
D60	1.55
Cc	0.871
Cu	10.33

Hole 2 Particle Distribution	
% sand	78.43382091
% gravel	17.69532644
% fines	3.870852658
D10	0.15
D30	0.45
D60	1.55
Cc	0.8709677419
Cu	10.33333333

Appendix B - Design Load Calculations and Results

B-1 Dead and Live Load Calculations

Per ASCE 7-16

Dead Load

Metal decking	2	psf	
5/8" Plywood Sheathing	2	psf	
Slope adjust	=SQRT(3.3^2+12^2)/12		<-- max slope of 3.3:12
	=ROUND(((B3+B4)*B6), 1)	psf	
Misc. +	1.9	psf	
	=B7+B9	psf	

="DL = "& B10 & " psf"

Live Load

Roof live load	20	psf	<-- Table 4.3-1

="Lr = "& B15 & " psf"

B-2 Dead and Live Load Results

Per ASCE 7-16

Dead Load

Metal decking	2	psf	
5/8" Plywood Sheathing	2	psf	
Slope adjust	1.037		<-- max slope of 3.3:12
	4.1	psf	
Misc. +	1.9	psf	
	6	psf	

DL = 6 psf

Live Load

Roof live load	20	psf	<-- Table 4.3-1

Lr = 20 psf

B-3 Snow Load Calculations

Per ASCE 7-16

Snow Load

Exposure Category	B		<-- Section 26.7.3
Roof Exposure	Partially exposed		<-- either fully exposed, partially exposed, or sheltered
Risk Category	II		<-- Table 1.5-1
Surface Roughness	B		<-- Section 26.7.2
pg	60	psf	<-- per City of Flagstaff
Exposure factor (Ce)	1		<-- Table 7.3-1 (exposure category B and partially enclosed)
Thermal factor (Ct)	1.2		<-- Table 7.3-2 (unheated and open air structures)
Importance factor (Is)	1		<-- Table 1.5-2 (risk category II)
pf	=0.7*B8*B9*B10*B11	psf	<-- flat roof snow load <-- Eqn
roof slope (3.3:12)	=ATAN(3.3/12)*180/PI()	degrees	
roof type	slippery		<-- slippery or non-slippery
Slope factor (Cs)	1		<-- Figure 7.4-1
ps	=CEILING(B13*B17,1)		<-- sloped roof snow load
	="SL = "& B19 & " psf"		

B-4 Snow Load Results

<u>Per ASCE 7-16</u>

Snow Load

Exposure Category	B		<-- Section 26.7.3
Roof Exposure	Partially exposed		<-- either fully exposed, partially exposed, or sheltered
Risk Category	II		<-- Table 1.5-1
Surface Roughness	B		<-- Section 26.7.2
pg	60	psf	<-- per City of Flagstaff
Exposure factor (Ce)	1		<-- Table 7.3-1 (exposure category B and partially enclosed)
Thermal factor (Ct)	1.2		<-- Table 7.3-2 (unheated and open air structures)
Importance factor (Is)	1		<-- Table 1.5-2 (risk category II)
pf	50.4	psf	<-- flat roof snow load <-- Eqn
roof slope (3.3:12)	15.376	degrees	
roof type	slippery		<-- slippery or non-slippery
Slope factor (Cs)	1		<-- Figure 7.4-1
ps	51		<-- sloped roof snow load

SL = 51 psf

B-5 Wind Load (C&C) Calculations

Per ASCE 7-16 | **MONOSLOPE**

Open Wind Load - Components and Cladding (Chpt 30)

h	=(9.5+10.5)/2	ft		
L	=SQRT(18^2+4.5^2	ft		
h/L	=H1/H2			

V (mph)	101		<-- ATC hazards	
Exposure Category	B		<-- Section 26.7.3	
Roof Exposure	Partially exposed		<-- either fully exposed, partially exposed, or sheltered	
Risk Category	II		<-- Table 1.5-1	
Surface Roughness	B		<-- Section 26.7.2	wooded area/numerous closely spaced obstructions
Wind directionality factor (Kd)	0.85		<-- Section 26.6	<-- Buildings-components and cladding
Topographic factor (Kzt)	1		<-- Section 26.8	given
Ground elevation factor (Ke)	=exp(-0.0000362*6900)		<-- Section 26.9	ground elevation above sea level zg = 6900 ft
Velocity pressure exposure coefficient (Kz or Kh)	=0.57		<-- Section 26.10.1	height above ground level is 0 ft
Velocity pressure (qh)	=0.00256*B11*B9*B8*B10*(B3^2)	psf	<-- Section 26.10.2	<-- Eqn 26.10-1 $q_z = 0.00256 K_z K_{zt} K_d K_e V^2$ (lb/ft^2); V in mi/h (26.10-1)
Gust effect factor (G)	0.85		<-- Section 26.11	given
Enclosure classification	open		<-- Section 26.12	no walls
max roof area	=18*24	ft2	(18' wide x 24' long)	
				roof angle (3.3:12 slope) =ATAN(3.3/12)*18(degrees
Net pressure coefficient (CN)	='C&C Interpolation'!C7		<-- Figure 30.7-1, interpolation between 7 deg & 15 deg, zone 3, =<a^2	
Net pressure coefficient (CN)	='C&C Interpolation'!C8		<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 3, >a^2,=<4.0a^2	
Net pressure coefficient (CN)	='C&C Interpolation'!C9		<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 3, >4.0a^2	
Net pressure coefficient (CN)	='C&C Interpolation'!D21		<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 3, =<a^2	
Net pressure coefficient (CN)	='C&C Interpolation'!D22		<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 3, >a^2,=<4.0a^2	
Net pressure coefficient (CN)	='C&C Interpolation'!D23		<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 3, >4.0a^2	

$$p = q_h G C_N \qquad (30.7\text{-}1)$$

WL	=ROUNDUP($B$12*$B$13*B18,0)	psf	Down Load for area =<a^2
WL	=ROUNDUP($B$12*$B$13*B19,0)	psf	Down Load for area >a^2,=<4.0a^2
WL	=ROUNDUP($B$12*$B$13*B20,0)	psf	Down Load for area >4.0a^2
WL	=ROUNDUP($B$12*$B$13*B21,0)	psf	Uplift for area =<a^2
WL	=ROUNDUP($B$12*$B$13*B22,0)	psf	Uplift for area >a^2,=<4.0a^2
WL	=ROUNDUP($B$12*$B$13*B23,0)	psf	Uplift for area >4.0a^2

Net pressure coefficient (CN)	='C&C Interpolation'!E7	<-- Figure 30.7-1, interpolation between 7 deg & 15 deg, zone 2, =<a^2
Net pressure coefficient (CN)	='C&C Interpolation'!E8	<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 2, >a^2,=<4.0a^2
Net pressure coefficient (CN)	='C&C Interpolation'!E9	<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 2, >4.0a^2
Net pressure coefficient (CN)	='C&C Interpolation'!F21	<-- Figure 30.7-1, interpolation between 7 deg & 15 deg, zone 2, =<a^2
Net pressure coefficient (CN)	='C&C Interpolation'!F22	<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 2, >a^2,=<4.0a^2
Net pressure coefficient (CN)	='C&C Interpolation'!F23	<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 2, >4.0a^2

$$p = q_h G C_N \qquad (30.7\text{-}1)$$

WL	=ROUNDUP($B$12*$B$13*B32,0)	psf	Down Load for area =<a^2
WL	=ROUNDUP($B$12*$B$13*B33,0)	psf	Down Load for area >a^2,=<4.0a^2
WL	=ROUNDUP($B$12*$B$13*B34,0)	psf	Down Load for area >4.0a^2
WL	=ROUNDUP($B$12*$B$13*B35,0)	psf	Uplift for area =<a^2
WL	=ROUNDUP($B$12*$B$13*B36,0)	psf	Uplift for area >a^2,=<4.0a^2

	WL	=ROUNDUP($B$12*$B$13*B37,0)	psf	Uplift for area >4.0a^2

Net pressure coefficient (CN)	='C&C Interpolation'!G7		<-- Figure 30.7-1, interpolation between 7 deg & 15 deg, zone 1, =<a^2
Net pressure coefficient (CN)	='C&C Interpolation'!G8		<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 1, >a^2,=<4.0a^2
Net pressure coefficient (CN)	='C&C Interpolation'!G9		<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 1, >4.0a^2
Net pressure coefficient (CN)	='C&C Interpolation'!H21		<-- Figure 30.7-1, interpolation between 7 deg & 15 deg, zone 1, =<a^2
Net pressure coefficient (CN)	='C&C Interpolation'!H22		<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 1, >a^2,=<4.0a^2
Net pressure coefficient (CN)	='C&C Interpolation'!H23		<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 1, >4.0a^2

	WL	=ROUNDUP($B$12*$B$13*B46,0)	psf	Down Load for area =<a^2
	WL	=ROUNDUP($B$12*$B$13*B47,0)	psf	Down Load for area >a^2,=<4.0a^2
$p = q_h G C_N$ (30.7-1)	WL	=ROUNDUP($B$12*$B$13*B48,0)	psf	Down Load for area >4.0a^2
	WL	=ROUNDUP($B$12*$B$13*B49,0)	psf	Uplift for area =<a^2
	WL	=ROUNDUP($B$12*$B$13*B50,0)	psf	Uplift for area >a^2,=<4.0a^2
	WL	=ROUNDUP($B$12*$B$13*B51,0)	psf	Uplift for area >4.0a^2

				Meet minimum requirement?
Zone 3	Max downward	=max(B25	psf	=if(C61>$
	Max uplift	=min(B28	psf	=if(-C62>$
Zone 2	Max downward	=max(B39	psf	=if(C63>$
	Max uplift	=min(B42	psf	=if(-C64>$
Zone 1	Max downward	=max(B53	psf	=if(C65>$ so use 16 psf
	Max uplift	=min(B56	psf	YES

Minimum wind load requirement	16	psf	<-- Section 27.1.5

B-6 Wind Load (C&C) Results

Per ASCE 7-16 | **MONOSLOPE**

Open Wind Load - Components and Cladding (Chpt 30)

h	10	ft
L	18.55397532	ft
h/L	0.5389680556	

V (mph)	101		<-- ATC hazards	
Exposure Category	B		<-- Section 26.7.3	
Roof Exposure	Partially exposed		<-- either fully exposed, partially exposed, or sheltered	
Risk Category	II		<-- Table 1.5-1	
Surface Roughness	B		<-- Section 26.7.2	wooded area/numerous closely spaced obstructions
Wind directionality factor (Kd)	0.85		<-- Section 26.6	<-- Buildings-components and cladding
Topographic factor (Kzt)	1		<-- Section 26.8	given
Ground elevation factor (Ke)	0.78		<-- Section 26.9	ground elevation above sea level zg = 6900 ft
Velocity pressure exposure coefficient (Kz or Kh)	0.57		<-- Section 26.10.1	height above ground level is 0 ft
Velocity pressure (qh)	9.86	psf	<-- Section 26.10.2	<-- Eqn 26.10-1 $q_z = 0.00256 K_z K_{zt} K_d K_e V^2$ (lb/ft^2); V in mi/h (26.10-1)
Gust effect factor (G)	0.85		<-- Section 26.11	given
Enclosure classification	open		<-- Section 26.12	no walls
max roof area	432	ft2	(18' wide x 24' long)	
			roof angle (3.3:12 slope)	15.37625125 degrees

Net pressure coefficient (CN)	3.549	<-- Figure 30.7-1, interpolation between 7 deg & 15 deg, zone 3, =<a^2
Net pressure coefficient (CN)	2.661	<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 3, >a^2,=<4.0a^2
Net pressure coefficient (CN)	1.774	<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 3, >4.0a^2
Net pressure coefficient (CN)	-4.316	<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 3, =<a^2
Net pressure coefficient (CN)	-3.123	<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 3, >a^2,=<4.0a^2
Net pressure coefficient (CN)	-2.049	<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 3, >4.0a^2

$$p = q_h G C_N \qquad (30.7\text{-}1)$$

WL	30	psf	Down Load for area =<a^2
WL	23	psf	Down Load for area >a^2,=<4.0a^2
WL	15	psf	Down Load for area >4.0a^2
WL	-37	psf	Uplift for area =<a^2
WL	-27	psf	Uplift for area >a^2,=<4.0a^2
WL	-18	psf	Uplift for area >4.0a^2

Net pressure coefficient (CN)	2.661	<-- Figure 30.7-1, interpolation between 7 deg & 15 deg, zone 2, =<a^2
Net pressure coefficient (CN)	2.661	<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 2, >a^2,=<4.0a^2
Net pressure coefficient (CN)	1.774	<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 2, >4.0a^2
Net pressure coefficient (CN)	-3.123	<-- Figure 30.7-1, interpolation between 7 deg & 15 deg, zone 2, =<a^2
Net pressure coefficient (CN)	-3.123	<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 2, >a^2,=<4.0a^2
Net pressure coefficient (CN)	-2.049	<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 2, >4.0a^2

WL	23	psf	Down Load for area =<a^2
WL	23	psf	Down Load for area >a^2,=<4.0a^2

$p = q_h G C_N$	(30.7-1)	WL	15	psf	Down Load for area >4.0a^2
		WL	-27	psf	Uplift for area =<a^2
		WL	-27	psf	Uplift for area >a^2,=<4.0a^2
		WL	-18	psf	Uplift for area >4.0a^2

Net pressure coefficient (CN)	1.774	<-- Figure 30.7-1, interpolation between 7 deg & 15 deg, zone 1, =<a^2
Net pressure coefficient (CN)	1.774	<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 1, >a^2,=<4.0a^2
Net pressure coefficient (CN)	1.774	<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 1, >4.0a^2
Net pressure coefficient (CN)	-2.049	<-- Figure 30.7-1, interpolation between 7 deg & 15 deg, zone 1, =<a^2
Net pressure coefficient (CN)	-2.049	<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 1, >a^2,=<4.0a^2
Net pressure coefficient (CN)	-2.049	<-- Figure 30.7-1,interpolation between 7 deg & 15 deg, zone 1, >4.0a^2

		WL	15	psf	Down Load for area =<a^2
		WL	15	psf	Down Load for area >a^2,=<4.0a^2
$p = q_h G C_N$	(30.7-1)	WL	15	psf	Down Load for area >4.0a^2
		WL	-18	psf	Uplift for area =<a^2
		WL	-18	psf	Uplift for area >a^2,=<4.0a^2
		WL	-18	psf	Uplift for area >4.0a^2

				Meet minimum requirement?	
Zone 3	Max downward	30	psf	YES	
	Max uplift	-37	psf	YES	
Zone 2	Max downward	23	psf	YES	
	Max uplift	-27	psf	YES	
Zone 1	Max downward	15	psf	NO	so use 16 psf
	Max uplift	-18	psf	YES	

Minimum wind load requirement	16	psf	<-- Section 27.1.5

B-7 MWFRS Wind Load Calculations

<u>Per ASCE 7-16</u>

MONOSLOPE

h	=(10.5+9.5)/2 ft
L	=SQRT(18^2+4.5^2 ft
h/L	=F1/F2

Open Wind Load - MWFRS (Chpt 27)

V (mph)	101		<-- ATC hazards	
Exposure Category	B		<-- Section 26.7.3	
Roof Exposure	Partially exposed		<-- either fully exposed, partially exposed, or sheltered	
Risk Category	II		<-- Table 1.5-1	
Surface Roughness	B		<-- Section 26.7.2	
Wind directionality factor (Kd)	0.85		<-- 26.6 and Table 26.6-1	<-- MWFRS
Topographic factor (Kzt)	1		<-- Section 26.8 and table in Fig. 26.8-1	
Ground elevation factor (Ke)	=exp(-0.0000362*6900)		<-- Section 26.9	
Velocity pressure exposure coefficient (Kz or Kh)	0.57		<-- Section 26.10.1	
Velocity pressure (qh)	=0.00256*B11*B9*B8*B10*(B3^2)	psf	<-- Section 26.10.2	<-- Eqn 26.10-1
Gust effect factor (G)	0.85		<-- Section 26.11	
Enclosure classification	open		<-- Section 26.12	
max roof area	=18*24	ft2	(18' wide x 24' long)	

Net pressure coefficient (CNW) - 0	='MWFRS Interpolation'!C3		<-- Fig. 27.3-4
Net pressure coefficient (CNL) - 0	='MWFRS Interpolation'!C4		<-- Fig. 27.3-5
Net pressure coefficient (CNW) - 180	='MWFRS Interpolation'!D3		<-- Fig. 27.3-6
Net pressure coefficient (CNL) - 180	='MWFRS Interpolation'!D4		<-- Fig. 27.3-7
WL	=$B$12*$B$13*B18	psf	
WL	=$B$12*$B$13*B19	psf	
WL	=$B$12*$B$13*B20	psf	
WL	=$B$12*$B$13*B21	psf	

Minimum wind load requirement	16	psf	<-- Section 27.1.5

B-8 MWFRS Wind Load Results

<u>Per ASCE 7-16</u>

	MONOSLOPE			h 10 ft
Open Wind Load - MWFRS (Chpt 27)				L 18.55397532 ft
V (mph)	101		<-- ATC hazards	h/L 0.5389680556
Exposure Category	B		<-- Section 26.7.3	
Roof Exposure	Partially exposed		<-- either fully exposed, partially exposed, or sheltered	
Risk Category	II		<-- Table 1.5-1	
Surface Roughness	B		<-- Section 26.7.2	
Wind directionality factor (Kd)	0.85		<-- 26.6 and Table 26.6-1	<-- MWFRS
Topographic factor (Kzt)	1		<-- Section 26.8 and table in Fig. 26.8-1	
Ground elevation factor (Ke)	0.78		<-- Section 26.9	
Velocity pressure exposure coefficient (Kz or Kh)	0.57		<-- Section 26.10.1	
Velocity pressure (qh)	9.86	psf	<-- Section 26.10.2	<-- Eqn 26.10-1
Gust effect factor (G)	0.85		<-- Section 26.11	
Enclosure classification	open		<-- Section 26.12	
max roof area	432	ft2	(18' wide x 24' long)	
Net pressure coefficient (CNW) - 0	-0.6		<-- Fig. 27.3-4	
Net pressure coefficient (CNL) - 0	-1.4		<-- Fig. 27.3-5	
Net pressure coefficient (CNW) - 180	-1		<-- Fig. 27.3-6	
Net pressure coefficient (CNL) - 180	0		<-- Fig. 27.3-7	
WL	-5.027	psf		
WL	-11.729	psf		
WL	-8.378	psf		
WL	0	psf		
Minimum wind load requirement	16	psf	<-- Section 27.1.5	

Appendix C- Plywood Design

C-1 Plywood Calculations

NDS SDPWS

Table 4.2C Nominal Unit Shear Capacities for Wood Frame-Diaphragms

Lateral load	51	psf	<-- D+0.45Wx+0.75S
Lateral Load	=B3*1	plf	=if(B5<B8, "OK"
Sheathing and Single-Floor grade, 6d nail size, 5/16 in panel thickness, 2 in face	=420/2	plf	<-- omega=2

Use: 5/16" plywood with 6" nail spacing

max spacing	=2.75*12	in	
nominal uniform load for 32/16	=90/1.6	psf	<-- omega=1.6

Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms

Unblocked Wood Structural Panel Diaphragms[1,2,3,4]

Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Supported Edges and Boundaries (in.)	A SEISMIC: 6 in. Nail Spacing at diaphragm boundaries and supporting members, Case 1, v_s (plf)			Cases 2,3,4,5,6, v_s (plf)			B WIND: 6 in. Nail Spacing at diaphragm boundaries and supporting members, Case 1, v_w (plf)	Cases 2,3,4,5,6, v_w (plf)
					v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_w (plf)	v_w (plf)
Structural I	6d	1-1/4	5/16	2	330	9.0	7.0	250	6.0	4.5	460	350
				3	370	7.0	6.0	280	4.5	4.0	520	390
	8d	1-3/8	3/8	2	480	8.5	7.0	360	6.0	4.5	670	505
				3	530	7.5	6.0	400	5.0	4.0	740	560
	10d	1-1/2	15/32	2	570	14	10	430	9.5	7.0	800	600
				3	640	12	9.0	480	8.0	6.0	895	670
Sheathing and Single-Floor	6d	1-1/4	5/16	2	300	9.0	6.5	220	6.0	4.0	420	310
				3	340	7.0	5.5	250	5.0	3.5	475	350
			3/8	2	330	7.5	5.5	250	5.0	4.0	460	350
				3	370	6.0	4.5	280	4.0	3.0	520	390
	8d	1-3/8	3/8	2	430	9.0	6.5	320	6.0	4.5	600	450
				3	480	7.5	5.5	360	5.0	3.5	670	505
			7/16	2	460	8.5	6.0	340	5.5	4.0	645	475
				3	510	7.0	5.5	380	4.5	3.5	715	530
			15/32	2	480	7.5	5.5	360	5.0	4.0	670	505
				3	530	6.5	5.0	400	4.0	3.5	740	560
	10d	1-1/2	15/32	2	510	15	9.0	380	10	6.0	715	530
				3	580	12	8.0	430	8.0	5.5	810	600
			19/32	2	570	13	8.5	430	8.5	5.5	800	600
				3	640	10	7.5	480	7.0	5.0	895	670

1. Nominal unit shear capacities shall be adjusted in accordance with 4.2.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.2.6. For specific requirements, see 4.2.7.1 for wood structural panel diaphragms. See Appendix A for common nail dimensions.
2. For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the

Table 3.2.2 Nominal Uniform Load Capacities (psf) for Roof Sheathing Resisting Out-of-Plane Wind Loads[1,3]

Sheathing Type[2]	Span Rating or Grade	Minimum Thickness (in.)	Strength Axis[4] Applied Perpendicular to Supports — Rafter/Truss Spacing (in.): 12	16	19.2	24	32	48
			Nominal Uniform Loads (psf)					
Wood Structural Panels (Sheathing Grades, C-C, C-D, C-C Plugged, OSB)	24/0	3/8	425	240	165	105	-	-
	24/16	7/16	540	305	210	135	-	-
	32/16	15/32	625	355	245	155	90	-
	40/20	19/32	955	595	415	265	150	-
	48/24	23/32	1160	805	560	360	200	90
Wood Structural Panels (Single Floor Grades, Underlayment, C-C Plugged)	16 o.c.	19/32	705	395	275	175	100	-
	20 o.c.	19/32	815	455	320	205	115	-
	24 o.c.	23/32	1085	610	425	270	150	-
	32 o.c.	7/8	1395	830	575	370	205	90
	48 o.c.	1-1/8	1790	1295	1060	680	380	170

C-1 Plywood Results

NDS SDPWS

Table 4.2C Nominal Unit Shear Capacities for Wood Frame-Diaphragms

Lateral load	51	psf	<-- D+0.45Wx+0.75S
Lateral Load	51	plf	OK
Sheathing and Single-Floor grade, 6d nail size, 5/16 in panel thickness, 2 in face	210	plf	<-- omega=2

Use: 5/16" plywood with 6" nail spacing

max spacing	33	in	
nominal uniform load for 32/16	56.25	psf	<-- omega=1.6

Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms

Unblocked Wood Structural Panel Diaphragms[1,2,3,4]

Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Supported Edges and Boundaries (in.)	A SEISMIC 6 in. Nail Spacing at diaphragm boundaries and supporting members						B WIND 6 in. Nail Spacing at diaphragm boundaries and supporting members	
					Case 1			Cases 2,3,4,5,6			Case 1	Cases 2,3,4,5,6
					v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_w (plf)	v_w (plf)
Structural I	6d	1-1/4	5/16	2	330	9.0	7.0	250	6.0	4.5	460	350
				3	370	7.0	6.0	280	4.5	4.0	520	390
	8d	1-3/8	3/8	2	480	8.5	7.0	360	6.0	4.5	670	505
				3	530	7.5	6.0	400	5.0	4.0	740	560
	10d	1-1/2	15/32	2	570	14	10	430	9.5	7.0	800	600
				3	640	12	9.0	480	8.0	6.0	895	670
Sheathing and Single-Floor	6d	1-1/4	5/16	2	300	9.0	6.5	220	6.0	4.0	420	310
				3	340	7.0	5.5	250	5.0	3.5	475	350
			3/8	2	330	7.5	5.5	250	5.0	4.0	460	350
				3	370	6.0	4.5	280	4.0	3.0	520	390
	8d	1-3/8	3/8	2	430	9.0	6.5	320	6.0	4.5	600	450
				3	480	7.5	5.5	360	5.0	3.5	670	505
			7/16	2	460	8.5	6.0	340	5.5	4.0	645	475
				3	510	7.0	5.5	380	4.5	3.5	715	530
			15/32	2	480	7.5	5.5	360	5.0	4.0	670	505
				3	530	6.5	5.0	400	4.0	3.5	740	560
	10d	1-1/2	15/32	2	510	15	9.0	380	10	6.0	715	530
				3	580	12	8.0	430	8.0	5.5	810	600
			19/32	2	570	13	8.5	430	8.5	5.5	800	600
				3	640	10	7.5	480	7.0	5.0	895	670

1. Nominal unit shear capacities shall be adjusted in accordance with 4.2.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.2.6. For specific requirements, see 4.2.7.1 for wood structural panel diaphragms. See Appendix A for common nail dimensions.
2. For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the

Table 3.2.2 Nominal Uniform Load Capacities (psf) for Roof Sheathing Resisting Out-of-Plane Wind Loads[1,3]

Sheathing Type[2]	Span Rating or Grade	Minimum Thickness (in.)	Strength Axis[4] Applied Perpendicular to Supports					
			Rafter/Truss Spacing (in.)					
			12	16	19.2	24	32	48
			Nominal Uniform Loads (psf)					
Wood Structural Panels (Sheathing Grades, C-C, C-D, C-C Plugged, OSB)	24/0	3/8	425	240	165	105	-	-
	24/16	7/16	540	305	210	135	-	-
	32/16	15/32	625	355	245	155	90	-
	40/20	19/32	955	595	415	265	150	-
	48/24	23/32	1160	805	560	360	200	90
Wood Structural Panels (Single Floor Grades, Underlayment, C-C Plugged)	16 o.c.	19/32	705	395	275	175	100	-
	20 o.c.	19/32	815	455	320	205	115	-
	24 o.c.	23/32	1085	610	425	270	150	-
	32 o.c.	7/8	1395	830	575	370	205	90
	48 o.c.	1-1/8	1790	1295	1060	680	380	170

Appendix D- Joists

D-1 Joists Bending Check Calculations

Bending Stress Fb:

Step 1	tributary width		12	in

Steps 2 & 3	57 lb	12 in	1 ft	=(57*(12/12))	lb	<-- distributed load along joist
	ft^2		12 in		ft	

Step 4	wL^2/8			
	w=	=F4		plf
	L=		19	ft
	Mmax=	=(D10*(D11^2))/8		lb-ft

Step 5	joist size		2x12		
	d		11.25	in	
	b		1.5	in	
	S	=(D16*(D15^2))/6		in3	
	S	=D17/1728		ft3	
	fb	=D12/D18		psf	<-- applied bending stress
	fb	**=D20/144**		psi	<-- applied bending stress

Step 6	Cd		1.15		
	Cm		1		
	Ct		1		
	Cl		1		
	CF		1		
	Cfu		1		
	Cc		1		
	Cr		1.15		
	Fb		850	psi	<-- HF #2
	Fb	=D32*144		psf	
	F'b	=D23*D24*D25*D26*D27*D28*D29*D30*D33		psf	<-- allowable bending stress
	F'b	**=D34/144**		psi	<-- allowable bending stress

Step 7	**=(D20/D34)*100**	**%**	**=IF(C37<100, "OK",**	**Use: 2x12 HF #2**

D-2 Joists Bending Check Results

Bending Stress Fb:

Step 1	tributary width		12	in	

Steps 2 & 3	57 lb	12 in	1 ft	57	lb	<-- distributed load along joist
	ft^2		12 in		ft	

Step 4	wL^2/8		
	w=	57	plf
	L=	19	ft
	Mmax=	2572.125	lb-ft

Step 5	joist size	2x12		
	d	11.25	in	
	b	1.5	in	
	S	31.640625	in3	
	S	0.01831054688	ft3	
	fb	140472.32	psf	<-- applied bending stress
	fb	**975.5022**	psi	<-- applied bending stress

Step 6	Cd	1.15		
	Cm	1		
	Ct	1		
	Cl	1		
	CF	1		
	Cfu	1		
	Cc	1		
	Cr	1.15		
	Fb	850	psi	<-- HF #2
	Fb	122400	psf	
	F'b	161874	psf	<-- allowable bending stress
	F'b	**1124.125**	psi	<-- allowable bending stress

Step 7	**86.77880327 %**	**OK**		**Use: 2x12 HF #2**

D-3 Joists Shear Check Calculations

Shear Stress Fv: **KNOWN :**

2x12		
b (in)		1.5
d (in)		11.25
b (ft)	**=C3/12**	
d(ft)	**=C4/12**	
w (lb/ft)	='Bending Check'!F4	
L (ft)		19

$$R = V \quad \ldots\ldots\ldots\ldots \quad = \frac{w\ell}{2}$$

$$V_x \quad \ldots\ldots\ldots\ldots \quad = w\left(\frac{\ell}{2} - x\right)$$

$$M_{max} \text{ (at center)} \quad \ldots\ldots\ldots \quad = \frac{w\ell^2}{8}$$

$$M_x \quad \ldots\ldots\ldots\ldots \quad = \frac{wx}{2}(\ell - x)$$

$$\Delta_{max} \text{ (at center)} \quad \ldots\ldots\ldots \quad = \frac{5w\ell^4}{384\,EI}$$

$$\Delta_x \quad \ldots\ldots\ldots\ldots \quad = \frac{wx}{24\,EI}(\ell^3 - 2\ell x^2 + x^3)$$

V	**wL/2**
V (lbs)	=(C7*C8)/2

Fv	3V/(2bd)
Fv (lb/ft^2)	=(3*C13)/(2*C5*C6)

Allowable Shear F'v= Fv(a)*Cd*Cm*Ct*Ci

Cd	1.15	snow load (most conservitive)
Cm	1	moisture < 19% for extended periods
Ct	1	t<100 degrees
Ci	1	is it incised?

Fv allowable (psi)		150	
Fv allowable (psf)	=C25*144		Hem-Fer

F'v (psf)	=C26*C20*C21*C22*C23

Stressed (%)	=(C16/C28)*100	OK

D-4 Joists Shear Check Results

Shear Stress Fv: **KNOWN :**

2x12	
b (in)	1.5
d (in)	11.25
b (ft)	**0.125**
d(ft)	**0.9375**
w (lb/ft)	57
L (ft)	19

$$R = V \;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\; = \frac{w\ell}{2}$$

$$V_x \;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\; = w\left(\frac{\ell}{2} - x\right)$$

$$M_{max} \text{ (at center) } .\;.\;.\;.\;.\;.\;.\;.\; = \frac{w\ell^2}{8}$$

$$M_x \;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\; = \frac{wx}{2}(\ell - x)$$

$$\Delta_{max} \text{ (at center) } .\;.\;.\;.\;.\;.\;.\;.\; = \frac{5w\ell^4}{384\,EI}$$

$$\Delta_x \;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\;.\; = \frac{wx}{24\,EI}(\ell^3 - 2\ell x^2 + x^3)$$

V **wL/2**

V (lbs)	541.5

Fv 3V/(2bd)

Fv (lb/ft^2)	6931.2

Allowable Shear F'v= Fv(a)*Cd*Cm*Ct*Ci

Cd	1.15	snow load (most conservitive)
Cm	1	moisture < 19% for extended periods
Ct	1	t<100 degrees
Ci	1	is it incised?

Fv allowable (psi)	150	
Fv allowable (psf)	21600	Hem-Fer

F'v (psf)	24840

Stressed (%)	27.90338164	OK

D-5 Joists Deflection Check Calculations

Delfection in inches:

W (lb/in)	='Bending Check'!F4/1	
L (in)	='Bending Check'!D11'	
E (psi)	1300000	<-- NDS supplement
I (in3)	=('Bending Check'!D16	
deflection (in)	=(5*C2*(C3^4))/(384*C	
allowable defl	L/240	
L/240	**=C3/240**	**=if(C7<C10, "OK", "NO GOOD")**

Now we need to chech the Deflection:

Using the Fornula for Deflection:

I = 177.907 in ^ 4 (See Section on Moment of Inertia)

E = 1190000 in ^ 4 (Modulus of Elasticity of Wood Selected)

Actual Deflect = 5 WL^4 / 384 EI = 0.22038 inches of Deflection Computed

The Deflections that are Permitted by the Code are:

Floor and Ceilings - L/360 = 0.4 inches

Roof < 3:12 - L/240 = 0.6 inches

Roof > 3:12 - L/180 = 0.8 inches

D-6 Joists Deflection Check Results

Delfection in inches:

W (lb/in)	4.75	
L (in)	228	
E (psi)	1300000	<-- NDS supplement
I (in3)	177.9785156	
deflection (in)	0.722371902	
allowable defl	L/240	
L/240	**0.95**	**OK**

Now we need to chech the Deflection:

Using the Fornula for Deflection:

I = 177.907 in ^ 4 (See Section on Moment of Inertia)

E = 1190000 in ^ 4 (Modulus of Elasticity of Wood Selected)

Actual Deflect = 5 WL^4 / 384 EI = 0.22038 inches of Deflection Computed

The Deflections that are Permitted by the Code are:

Floor and Ceilings - L/360 =	**0.4 inches**
Roof < 3:12 - L/240 =	**0.6 inches**
Roof > 3:12 - L/180 =	**0.8 inches**

D-7 2x12 Joist Calculations

Bending Stress Fb:	Mmax=	2731.5	lb-ft	
	typical joist sizes	2x12		
	d	11.25	in	
	b	1.5	in	
	S	=(D7*(D6^2))/6	in3	
	S	=D8/1728	ft3	
	Fb	=D3/D9	psf	<-- applied bending stress
	Cd	1.15		
	Cm	1		
	Ct	1		
	Cl	1		
	CF	1.3		
	Cfu	1		
	Cc	1		
	Cr	1.15		
	Fb	850	psi	
	Fb	=D23*144	psf	
	F'b	=D14*D15*D16*D17*D18*D19*D20*D21	psf	<-- allowable bending stress
	=(D11/D25)*100	**%**	**=IF(C27<100, "OK", "NO GOOD")**	**Use: (3) 2x12 HF #2**

Delfection in inches:	W (lb/in)	=80.08/12		
	L (in)	=19*12		
	E (psi)	1300000	<-- NDS supplement	
	I (in3)	=((D7*3)*(D6^3))/12		
	deflection (in)	=(5*C30*(C31^4))/(384*C32*C33)		
	allowable defl	L/240		
	L/240	**=C31/240**	**=if(C35<C38, "OK", "NO GOOD")**	

Actual Dimensions of Lumber for 2x12

b	1.5	in	=B42/12	ft
d	11.25	in	=B43/12	ft

V	wL/2		760.8	lb		
Fv	3V/2bd	=(3*C45)/(2*D42*D43)		lb/ft^2		
Cd	1.15	snow load				
Cm	1	moisture content less than 19%				
Ct	1	Flagstaff temperatures don't exceed 100 degrees				
Ci	1	not 100% sure but when in doubt choose 1				
allowable Fv	150	lb/in^2		=B53*144	lb/ft^2	Used Hem Fir - No. 2 Structural from Table 4A
F'v	=D53*B48*B49*E	lb/ft^2				
	=(C46/B55)*100	%		less than 100% = good		Use (3) 2x12 Hem Fir No. 2

D-8 2x12 Joist Results

Bending Stress Fb:

Mmax=	2731.5	lb-ft		

typical joist sizes	2x12		
d	11.25	in	
b	1.5	in	
S	31.640625	in3	
S	0.01831054688	ft3	
Fb	149176.32	psf	<-- applied bending stress

Cd	1.15		
Cm	1		
Ct	1		
Cl	1		
CF	1.3		
Cfu	1		
Cc	1		
Cr	1.15		
Fb	850	psi	
Fb	122400	psf	
F'b	210436.2	psf	<-- allowable bending stress

70.88909608 % **OK** **Use: (3) 2x12 HF #2**

Delfection in inches:

W (lb/in)	6.673333333	
L (in)	228	
E (psi)	1300000	<-- NDS supplement
I (in3)	533.9355469	
deflection (in)	0.3382897188	
allowable defl	L/240	
L/240	**0.95**	**OK**

Actual Dimensions of Lumber for 2x12

b	1.5	in	0.125	ft
d	11.25	in	0.9375	ft

V	wL/2		760.8	lb		
Fv	3V/2bd		9738.24	lb/ft^2		
Cd	1.15	snow load				
Cm	1	moisture content less than 19%				
Ct	1	Flagstaff temperatures don't exceed 100 degrees				
Ci	1	not 100% sure but when in doubt choose 1				
allowable Fv	150	lb/in^2	21600	lb/ft^2	Used Hem Fir - No. 2 Structural from Table 4A	
F'v	24840	lb/ft^2				
	39.20	%	less than 100% = good			Use (3) 2x12 Hem Fir No. 2

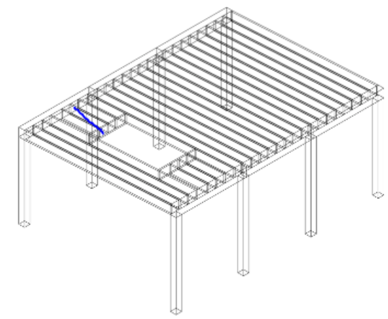
D-9 Short Joist Check Calculations

Bending Stress Fb:

Step 1	tributary width		1	ft		

Steps 2 & 3	57 lb	=D2	ft	=57*D5	lb	<-- distributed load along joist
	ft^2				ft	

Step 4	wL^2/8		
	w=	=F5	plf
	L=	=4+(8/12)	ft
	Mmax=	=(D11*(D12^2))/8	lb-ft



(4)

Step 5	typical joist sizes		2x12		
	d		11.25	in	
	b		1.5	in	
	S	=(D17*(D16^2))/6		in3	
	S	=D18/1728		ft3	
	Fb	=D13/D19		psf	<-- applied bending stress

Step 6	Cd		1.15		
	Cm		1		
	Ct		1		
	Cl		1		
	CF		1		
	Cfu		1		
	Cc		1		
	Cr		1.15		
	Fb		850	psi	
	Fb	=D33*144		psf	
	F'b	=D24*D25*D26*D27*D28*D29*D30*D3		psf	<-- allowable bending stress

Step 7	**=(D21/D35)*100**	**%**	**=IF(C37<100, "OK", "NO GOOD")**	**Use: 2x12 HF #2**

Delfection in inches:	W (lb/in)	=F5/12	
	L (in)	=D12*12	
	E (psi)	1300000	<-- NDS supplement
	I (in3)	=(D17*(D16^3))/12	
	deflection (in)	=(5*C40*(C41^4))/(384*C42*C4	
	allowable defl	L/240	
	L/240	**=C41/240**	**=if(C45<C48, "OK", "NO GOOD")**

Actual Dimensions of Lumber for 2x12

b	1.5	in	=B52/12	ft
d	11.25	in	=B53/12	ft
V	wL/2	=(F5*D12)/2	lb	
Fv	3V/2bd	=(3*C55)/(2*D52*D53)	lb/ft^2	
Cd	1.15	snow load		

Cm	1	moisture content less than 19%			
Ct	1	Flagstaff temperatures don't exceed 100 degrees			
Ci	1	not 100% sure but when in doubt choose 1			
allowable Fv	150	lb/in^2	=B63*144	lb/ft^2	Used Hem Fir - No. 2 Structural from Table 4A
F'v	=D63*B58*B59*B6	lb/ft^2			
	=(C56/B65)*100	%	less than 100% = good		Use: 2x12 Hem Fir No. 2
	end reactions	=(D11*D12)/2	lb		
	SW end reactions	7.3125	lb		
		=sum(C69:C70)	lb		

D-10 Short Joist Check Results

Bending Stress Fb:

Step 1	tributary width	1	ft			
Steps 2 & 3	57 lb	1	ft	57.0	lb	<-- distributed load along joist
	ft^2				ft	

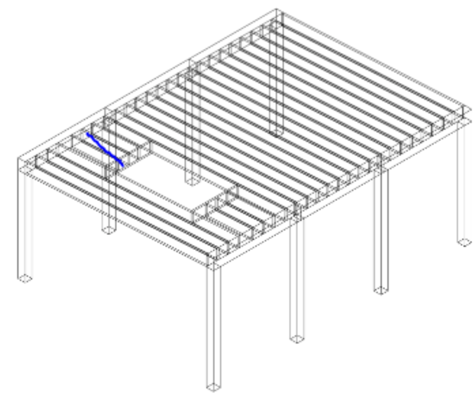
Step 4	wL^2/8		
	w=	57.0	plf
	L=	4.666666667	ft
	Mmax=	155.1666667	lb-ft

Step 5	typical joist sizes	2x12		
	d	11.25	in	
	b	1.5	in	
	S	31.640625	in3	
	S	0.01831054688	ft3	
	Fb	8474.168889	psf	<-- applied bending stress

Step 6	Cd	1.15		
	Cm	1		
	Ct	1		
	Cl	1		
	CF	1		
	Cfu	1		
	Cc	1		
	Cr	1.15		
	Fb	850	psi	
	Fb	122400	psf	
	F'b	161874	psf	<-- allowable bending stress

Step 7	**5.235040148 %**	**OK**	**Use: 2x12 HF #2**

(4)



Delfection in inches:

W (lb/in)	4.75	
L (in)	56	
E (psi)	1300000	<-- NDS supplement
I (in3)	177.9785156	
deflection (in)	0.002628896965	
allowable defl	L/240	

L/240	**0.2333333333**	**OK**

Actual Dimensions of Lumber for 2x12

b	1.5	in	0.125	ft
d	11.25	in	0.9375	ft

V	wL/2	133	lb
Fv	3V/2bd	1702.40	lb/ft^2

Cd	1.15	snow load
Cm	1	moisture content less than 19%
Ct	1	Flagstaff temperatures don't exceed 100 degrees
Ci	1	not 100% sure but when in doubt choose 1

allowable Fv	150	lb/in^2	21600	lb/ft^2	Used Hem Fir - No. 2 Structural from Table 4A

F'v	24840	lb/ft^2

6.85 % less than 100% = good

Use: 2x12 Hem Fir No. 2

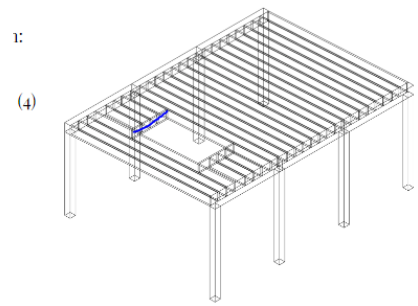
end reactions	133	lb
SW end reactions	7.3125	lb
	140.3125	lb

D-11 Short Joist #2 Check Calculations

Bending Stress Fb:

Steps 2 & 3	='Short Joist Check'!C71		=C5/D6	lb	<-- distributed load along joist
		1 ft		ft	

Step 4	wL^2/8		
	w=	=F5	plf
	L=	=3+(8/12)	ft
	Mmax=	=(D11*(D12^2))/8	lb-ft



Step 5	typical joist sizes		2x12		
	d		11.25	in	
	b		1.5	in	
	S	=(D17*(D16^2))/6		in3	
	S	=D18/1728		ft3	
	Fb	=D13/D19		psf	<-- applied bending stress

Step 6	Cd		1.15		
	Cm		1		
	Ct		1		
	Cl		1		
	CF		1		
	Cfu		1		
	Cc		1		
	Cr		1.15		
	Fb		850	psi	
	Fb	=D33*144		psf	
	F'b	=D24*D25*D26*D27*D28*D29*D30*I		psf	<-- allowable bending stress

Step 7	**=(D21/D35)*100**	**%**	**=IF(C37<100, "OK", "NO GOOD")**	**Use: 2x12 HF #2**

Delfection in inches:

W (lb/in)	=F5/12	
L (in)	=D12*12	
E (psi)	1300000	<-- NDS supplement
I (in3)	=(D17*(D16^3))/12	
deflection (in)	=(5*C40*(C41^4))/(384*C4:	
allowable defl	L/240	
L/240	**=C41/240**	**=if(C45<C48, "OK", "NO GOOD")**

Actual Dimensions of Lumber for 2x12

b	1.5	in	=B52/12	ft
d	11.25	in	=B53/12	ft
V	wL/2	=(F5*D12)/2	lb	
Fv	3V/2bd	=(3*C55)/(2*D52*D53)	lb/ft^2	
Cd	1.15	snow load		
Cm	1	moisture content less than 19%		
Ct	1	Flagstaff temperatures don't exceed 100 degrees		
Ci	1	not 100% sure but when in doubt choose 1		

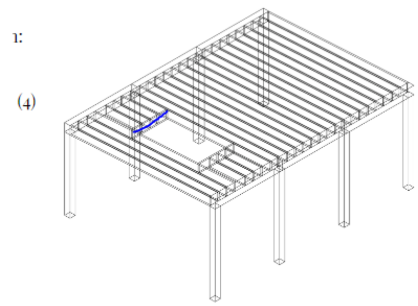
allowable Fv	150	lb/in^2	=B63*144	lb/ft^2	Used Hem Fir - No. 2 Structural from Table 4A
F'v	=D63*B58*B59*B60*	lb/ft^2			
	=(C56/B65)*100	%	less than 100% = good		Use: 2x12 Hem Fir No. 2
	end reactions	=(F5*D12)/2	lb		
	SW end reactions	6.71	lb		
		=sum(C69:C70)	lb	at 4'-8" and 12'-5" on (3) 2x12 joists	

D-12 Short Joist #2 Check Results

Bending Stress Fb:

Steps 2 & 3	140.3125		140.3 lb	<-- distributed load along joist
		1 ft	ft	

Step 4	wL^2/8	
	w=	140.3 plf
	L=	3.666666667 ft
	Mmax=	235.8029514 lb-ft



Step 5	typical joist sizes	2x12	
	d	11.25 in	
	b	1.5 in	
	S	31.640625 in3	
	S	0.01831054688 ft3	
	Fb	12877.98519 psf	<-- applied bending stress

Step 6	Cd	1.15	
	Cm	1	
	Ct	1	
	Cl	1	
	CF	1	
	Cfu	1	
	Cc	1	
	Cr	1.15	
	Fb	850 psi	
	Fb	122400 psf	
	F'b	161874 psf	<-- allowable bending stress

Step 7	**7.95556123 %**	**OK**	**Use: 2x12 HF #2**

Delfection in inches:

W (lb/in)	11.69270833	
L (in)	44	
E (psi)	1300000	<-- NDS supplement
I (in3)	177.9785156	
deflection (in)	0.002466344108	
allowable defl	L/240	
L/240	**0.1833333333**	**OK**

Actual Dimensions of Lumber for 2x12

b	1.5	in	0.125	ft
d	11.25	in	0.9375	ft

V	wL/2	257.2395833	lb
Fv	3V/2bd	3292.67	lb/ft^2

Cd	1.15	snow load
Cm	1	moisture content less than 19%
Ct	1	Flagstaff temperatures don't exceed 100 degrees
Ci	1	not 100% sure but when in doubt choose 1

allowable Fv	150	lb/in^2		21600	lb/ft^2	Used Hem Fir - No. 2 Structural from Table 4A
F'v	24840	lb/ft^2				
	13.26	%	less than 100% = good			Use: 2x12 Hem Fir No. 2
end reactions		257.2395833	lb			
SW end reactions		6.71	lb			
		263.9495833	lb		at 4'-8" and 12'-5" on (3) 2x12 joists	

Appendix E- Beams

E-1 Beam Design Calculations

Hem Fir Density (pcf)	31.2	Joist Volume (ft3)	=(1.5/12)*(11.25/12)*B2	weight of joist (lb)	=B1*D1	rxn at joist support from weight (lb)	=F1/2
joist length (ft)	19	joist spacing (ft)	1				
max roof load on joist (psf)	=57*D2	rxn at joist support from roof load (lb)	=(B3*D2*B2)/2	rxn at joist support (lb)	**=D3+H1**		

Bending Stress Fb:

Step 1	tributary width	=18/2	ft			
Steps 2 & 3	=F3	lb		=C9/D2	lb	<-- distributed load along beam
			1 ft		ft	
Step 4	wL^2/8					
	w=	=C9	plf			
	L=	8	ft			
	Mmax=	=(D15*(D16^2))/8	lb-ft			
Step 5	size	8x12				
	b	7.5	in			
	d	11.5	in			
	S	=(D20*(D21^2))/6	in3			
	S	=D22/1728	ft3			
	Fb	=D17/D23	psf	<-- applied bending stress		
		=D25/144	psi			
Step 6	Cd	1.15				
	Cm	1				
	Ct	1				
	CL	1	<-- lesser of CL and CV used (do not use simultaneously)			
	CV	=((21/12)^(1/20))*((12/D21)^(1/20))*((5.125/D20)^(1/20))	<-- lesser of CL and CV used (do not use simultaneously)			
	Cfu	1				
	Cc	1				
	Ci	1				
	Fb	875	psi	<-- DF #2 (beams and stringers)		
	Fb	=D37*144	psf			
	F'b	=D28*D29*D30*D31*D33*D34*D35*D38	psf	<-- allowable bending stress		
		=D39/144	psi			
Step 7	**=(D25/D39)*100**	**%**	**=IF(C41<100, "OK", "NO GOOD")**	**Use: 8x12 DF #2**		

Shear Stress Fv:

	Shear	**3.4.2**		
b	=D20	in	=B46/12	ft
d	=D21	in	=B47/12	ft
V	wL/2	=(D15*D16)/2	lb	
Fv	3V/2bd	=(3*C49)/(2*D46*D47)	lb/ft^2	
Cd	1.15			
Cm	1			
Ct	1			
Ci	1			
allowable Fv	265	lb/in^2	=B57*144	lb/ft^2
F'v	=D57*B52*B53*B54*B55	lb/ft^2		
	=(C50/B59)*100	**%**		**Use 8x12 DF #2**

Deflection

	W (lb/in)	=D15/12	
	L (in)	=D16*12	
	E (psi)	=1.8*(10^6)	<-- NDS supplement
	I (in3)	=(B46*(B47^3))/12	
	deflection (in)	=(5*C65*(C66^4))/(384*C67*C68)	
	allowable defl	L/240	
	L/240	**=C66/240**	**=if(C70<C73, "OK", "NO GOOD")**

E-2 Beam Design Results

Hem Fir Density (pcf)	31.2	Joist Volume (ft3)	2.2265625	weight of joist (lb)	69.46875	rxn at joist support from weight (lb)	34.734375
joist length (ft)	19	joist spacing (ft)	1				
max roof load on joist (psf)	57	rxn at joist support from roof load (lb)	541.5	rxn at joist support (lb)	**576.234375**		

Bending Stress Fb:

Step 1	tributary width	9	ft

Steps 2 & 3	576.2	lb		576.234375	lb	<-- distributed load along beam
			1 ft		ft	

Step 4	wL^2/8		
	w=	576.2	plf
	L=	8	ft
	Mmax=	4609.875	lb-ft

Step 5	size	8x12		
	b	7.5	in	
	d	11.5	in	
	S	165.3125	in3	
	S	0.09566695602	ft3	
	Fb	48186.70095	psf	<-- applied bending stress
		334.6298677	psi	

Step 6	Cd	1.15		
	Cm	1		
	Ct	1		
	CL	1	<-- lesser of CL and CV used (do not use simultaneously)	
	CV	1.011	<-- lesser of CL and CV used (do not use simultaneously)	
	Cfu	1		
	Cc	1		
	Ci	1		
	Fb	875	psi	<-- DF #2 (beams and stringers)
	Fb	126000	psf	
	F'b	144900	psf	<-- allowable bending stress
		1006.25	psi	
Step 7	**33.25514213 %**		**OK**	**Use: 8x12 DF #2**

Shear Stress Fv: **Shear** **3.4.2**

b	7.5	in	0.625	ft
d	11.5	in	0.9583	ft
V	wL/2	2304.9375 lb		
Fv	3V/2bd	5772.37 lb/ft^2		
Cd	1.15			
Cm	1			
Ct	1			
Ci	1			
allowable Fv	265	lb/in^2	38160	lb/ft^2
F'v	43884	lb/ft^2		

13.15 % **Use 8x12 DF #2**

Deflection

W (lb/in)	48.01953125	
L (in)	96	
E (psi)	1800000	<-- NDS supplement
I (in3)	950.546875	
deflection (in)	0.03103813265	
allowable defl	L/240	
L/240	**0.4**	**OK**

Appendix F- Columns

F-1 End Column Design Calculations

Reaction at beam support from loading	=((577.00625*12)/2)	lb	<-- 2 times the loading of the beams
Axial load acting down onto column	=B1+(H2*2)	lb	--> add beam self weight
Lateral wind load	16	psf	
Tributary width	12	ft	
Lateral wind load acting on column	=B3*B4	plf	

DF #2 density (pcf)	33.1	rxn from SW (lb)	=(F2*F7)/2
Beam size:		Column size:	
width (in)	7.5	width (in)	7.5
length (in)	11.5	length (in)	7.5
beam length (ft)	8	height (ft)	10
beam volume (ft^3)	=(F4/12)*(F5/12)*F		

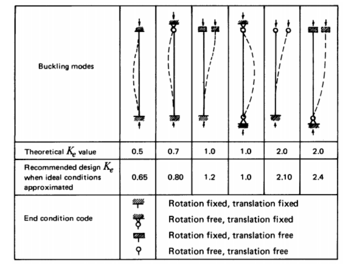
Bearing Check:

Check 8x8

fc=P/A			
P	=B2	lb	
A	=H4*H5	in2	
fc	=B9/B10	psi	
Fc (axially loaded)	1650	psi	<-- DF/DF 24F-V4
CD	1.15		
CM	1		
Ct	1		
CF	1		
Ci	1		
CP	=G20-((G21-G22)^(1/2		
F'c	=B13*B14*B15*B16*B	psi	
fc/F'c (%)	**=(B11/B21)*100**	**=IF(B23>100, "NO GOOD", "OK")**	

CP:		
FCE	=(0.822*G14)/((G15	
F*c	=B13*B14*B15*B16	psi
c	0.9	
E'min	='Middle ASCE 7-1	psi
le	=G17*G18	in
d	=H4	in
Ke	2.4	
l	=H6*12	in
1st term	=(1+(G11/G12))/(2*	
2nd term	=((1+(G11/G12))/(2	
3rd term	=(G11/G12)/G13	

Table G1 Buckling Length Coefficients, K_e



Shear Check:

KNOWN :

8x8	
b (in)	=H4
d (in)	=H5
b (ft)	**=C29/12**
d(ft)	**=C30/12**
P (lb)	=B69
L (ft)	=H6

V	**V=P**
V (lbs)	=C33

Fv	3V/(2bd)
Fv (lb/ft^2)	=(3*C37)/(2*C31*C32)

Allowable Shear	F'v= Fv(a)*Cd*Cm*Ct*Ci	
Cd	1.15	snow load (most conservitive)
Cm	1	moisture < 19% for extended periods
Ct	1	t<100 degrees
Ci	1	is it incised?
Fv allowable (psi)	230	
Fv allowable (psf)	=C48*144	<-- DF/DF 24F-V4

F'v (psf)	=C49*C43*C44*C45*C46	
Stressed (%)	**=(C40/C51)*100**	**OK**

Deflection Check:

P (lb)	=C37
L (in)	=H6*12
I (in3)	=(C29*(C30^3))/12
E (psi)	=1.6*(10^6)
deflection (in)	=(B57*(B58^3))/(3*B60
L/240	**=B58/240**

=if(B62<B64, "OK", "NO GOOD")

Bending Check #1:			
Lateral load at top of column	=51*4*1	lb	<-- D+0.45Wx+0.75S
Max moment (P*L)	=B69*H6	lb-ft	
Column size	8x8		
b	=H4	in	
d	=H5	in	
b	=B73/12	ft	
d	=B74/12	ft	
S	=(B73*(B74^2))/6	in3	
S	=(B75*(B76^2))/6	ft3	
fb	=B70/B78	psf	
fb	=B80/144	psi	
Cd	1.15		
Cm	1		
Ct	1		
Cl	1		
CF	1		
Cfu	1		
Cc	1		
Cr	1		
Fb	1450	psi	<-- DF/DF 24F-V4
F'b	=B91*B90*B89*B88*B	psi	
fb/F'b (%)	**=(B81/B92)*100**	**=if(B94>100, "NO GOOD", "OK")**	

Bearing Check: 3.10 of NDS		
Axial load on column	=B9	lb
Bearing area	=H4*H5	in2
Applied load	=B97/B98	psi
F*c (psi)	**=G12**	**=if(B98<B101, "OK", "NO GOOD")**

F-2 End Column Design Results

Reaction at beam support from loading	3462.0375	lb	<-- 2 times the loading of the beams
Axial load acting down onto column	3620.641667	lb	--> add beam self weight
Lateral wind load	16	psf	
Tributary width	12	ft	
Lateral wind load acting on column	192	plf	

DF #2 density (pcf)	33.1	rxn from SW (lb)	79.30208333
Beam size:		Column size:	
width (in)	7.5	width (in)	7.5
length (in)	11.5	length (in)	7.5
beam length (ft)	8	height (ft)	10
beam volume (ft^3)	4.8		

Bearing Check:

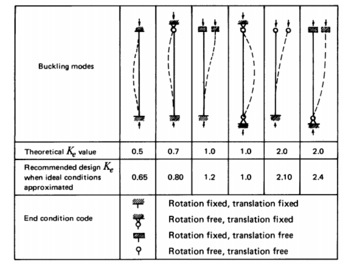
Check 8x8

fc=P/A		
P	3620.641667	lb
A	56.25	in2
fc	64.36696296	psi
Fc (axially loaded)	700	psi
CD	1.15	
CM	1	
Ct	1	
CF	1	
Ci	1	
CP	0.5291	
F'c	425.9655195	psi
fc/F'c (%)	**15.11083879**	**OK**

CP:

FCE	473.836263	
F*c	805	psi
c	0.9	
E'min	850000	psi
le	288	in
d	7.5	in
Ke	2.4	
l	120	in
1st term	0.8825647088	
2nd term	0.7789204652	
3rd term	0.6540183064	

Table G1 Buckling Length Coefficients, K_e



Shear Check:

KNOWN :

8x8

b (in)	7.5
d (in)	7.5
b (ft)	**0.625**
d(ft)	**0.625**
P (lb)	204
L (ft)	10

V	**V=P**
V (lbs)	204

Fv	3V/(2bd)
Fv (lb/ft^2)	783.36

Allowable Shear	F'v= Fv(a)*Cd*Cm*Ct*Ci	
Cd	1.15	snow load (most conservitive)
Cm	1	moisture < 19% for extended periods
Ct	1	t<100 degrees
Ci	1	is it incised?
Fv allowable (psi)	230	
Fv allowable (psf)	33120	<-- DF/DF 24F-V4
F'v (psf)	38088	
Stressed (%)	**2.056710775**	**OK**

Deflection Check:

P (lb)	204
L (in)	120
I (in3)	263.671875
E (psi)	1600000
deflection (in)	0.278528
L/240	**0.5** **OK**

Bending Check #1:			
Lateral load at top of column	204	lb	<-- D+0.45Wx+0.75S
Max moment (P*L)	2040	lb-ft	
Column size	8x8		
b	7.5	in	
d	7.5	in	
b	0.625	ft	
d	0.625	ft	
S	70.3125	in3	
S	0.04069010417	ft3	
fb	50135.04	psf	
fb	348.16	psi	
Cd	1.15		
Cm	1		
Ct	1		
Cl	1		
CF	1		
Cfu	1		
Cc	1		
Cr	1		
Fb	1450	psi	<-- DF/DF 24F-V4
F'b	1667.5	psi	
fb/F'b (%)	**20.87916042**	**OK**	

Bearing Check: 3.10 of NDS		
Axial load on column	3620.642	lb
Bearing area	56.25	in2
Applied load	64.367	psi
F*c (psi)	**805**	**OK**

F-3 Middle Column Design Calculations

Reaction at beam support from loading	=((577.00625*12)/2)*2	lb	<-- 2 times the loading of the beams
Axial load acting down onto column	=B1+(H2*2)	lb	--> add beam self weight

DF #2 density (pcf)	33.1	rxn from SW (lb)	=(F2*F7)/2
Beam size:		Column size:	
width (in)	7.5	width (in)	7.5
length (in)	11.5	length (in)	7.5
beam length (ft)	6	height (ft)	10
beam volume (ft^3)	=(F4/12)*(F5/12)*F		

Bearing Check:

Check 8x8

fc=P/A			
P	=B2	lb	
A	=H4*H5	in2	
fc	=B9/B10	psi	
Fc (axially loaded)	1650	psi	<-- DF/DF 24F-V4
CD	1.15		
CM	1		
Ct	1		
CF	1		
Ci	1		
CP	=G20-((G21-G22)^(1/2))		
F'c	=B13*B14*B15*B16*B17*B18*B19	psi	
fc/F'c (%)	**=(B11/B21)*100**	**=IF(B23>100, "NO GOOD", "OK")**	

CP:

FCE	=(0.822*G14)/((G15	
F*c	=B13*B14*B15*B16	psi
c	0.9	
E'min	='Middle ASCE 7-1(	psi
le	=G17*G18	in
d	=H4	in
Ke	2.4	
l	=H6*12	in
1st term	=(1+(G11/G12))/(2*	
2nd term	=((1+(G11/G12))/(2	
3rd term	=(G11/G12)/G13	

Shear Check:

KNOWN :

8x8	
b (in)	=H4
d (in)	=H5
b (ft)	**=C29/12**
d(ft)	**=C30/12**
P (lb)	=B69
L (ft)	=H6

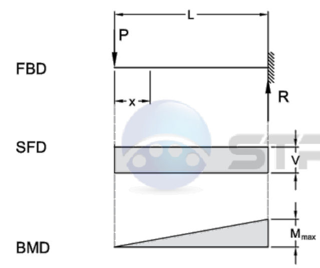
V	**V=P**
V (lbs)	=C33

Fv	3V/(2bd)
Fv (lb/ft^2)	=(3*C37)/(2*C31*C32)

Allowable Shear		F'v= Fv(a)*Cd*Cm*Ct*Ci	
	Cd	1.15	snow load (most conservitive)
	Cm	1	moisture < 19% for extended periods
	Ct	1	t<100 degrees
	Ci	1	is it incised?
Fv allowable (psi)		230	
Fv allowable (psf)		=C48*144	<-- DF/DF 24F-V4
F'v (psf)		=C49*C43*C44*C45*C46	
Stressed (%)		**=(C40/C51)*100**	**OK**

Deflection Check:

P (lb)	=C37	
L (in)	=H6*12	
I (in3)	=(C29*(C30^3))/12	
E (psi)	=1.6*(10^6)	
deflection (in)	=(B57*(B58^3))/(3*B60*B59)	
L/240	**=B58/240**	**=if(B62<B64, "OK", "NO GOOD")**



$R = V \dots\dots = P$

$M_{max}(\text{at fixed end}) \dots\dots = PL$

$M_x \dots\dots = Px$

$\Delta_{max}(\text{at free end}) \dots\dots = \frac{PL^3}{3EI}$

$\Delta_x \dots\dots = \frac{P}{6EI}(2L^3 - 3L^2x + x^3)$

Bending Check #1:

Lateral load at top of column	=51*8*1	lb	<-- D+0.45Wx+0.75S
Max moment (P*L)	=B69*H6	lb-ft	
Column size	8x8		
b	=H4	in	
d	=H5	in	
b	=B73/12	ft	
d	=B74/12	ft	
S	=(B73*(B74^2))/6	in3	
S	=(B75*(B76^2))/6	ft3	
fb	=B70/B78	psf	
fb	=B80/144	psi	
Cd	1.15		
Cm	1		
Ct	1		
Cl	1		
CF	1		
Cfu	1		
Cc	1		
Cr	1		
Fb	1450	psi	<-- DF/DF 24F-V4
F'b	=B91*B90*B89*B88*B87*B86*B85*B84*B83	psi	
fb/F'b (%)	**=(B81/B92)*100**	**=if(B94>100, "NO GOOD", "OK")**	

Bearing Check: 3.10 of NDS

Axial load on column	=B9	lb
Bearing area	=H4*H5	in2
Applied load	=B97/B98	psi
F*c (psi)	**=G12**	**=if(B98<B101, "OK", "NO GOOD")**

F-4 Middle Column Design Results

Reaction at beam support from loading	6924.075	lb	<-- 2 times the loading of the beams
Axial load acting down onto column	7043.028125	lb	--> add beam self weight

DF #2 density (pcf)	33.1	rxn from SW (lb)	59.4765625
Beam size:		Column size:	
width (in)	7.5	width (in)	7.5
length (in)	11.5	length (in)	7.5
beam length (ft)	6	height (ft)	10
beam volume (ft^3)	3.6		

Bearing Check:

Check 8x8

fc=P/A		
P	7043.028125	lb
A	56.25	in2
fc	125.2093889	psi
Fc (axially loaded)	700	psi
CD	1.15	
CM	1	
Ct	1	
CF	1	
Ci	1	
CP	0.5291	
F'c	425.9655195	psi
fc/F'c (%)	**29.3942545**	**OK**

CP:

FCE	473.836263	
F*c	805	psi
c	0.9	
E'min	850000	psi
le	288	in
d	7.5	in
Ke	2.4	
l	120	in
1st term	0.8825647088	
2nd term	0.7789204652	
3rd term	0.6540183064	

Shear Check:

KNOWN :

8x8	
b (in)	7.5
d (in)	7.5
b (ft)	**0.625**
d(ft)	**0.625**
P (lb)	128
L (ft)	10

V	**V=P**
V (lbs)	128

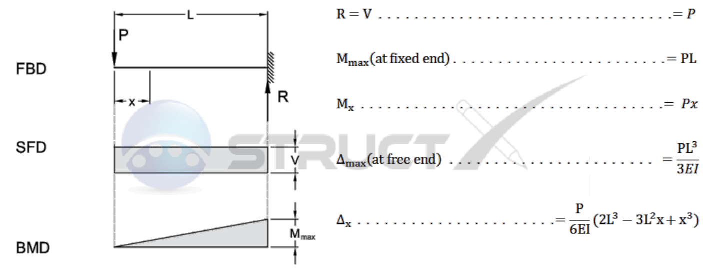
Fv	3V/(2bd)
Fv (lb/ft^2)	491.52

Allowable Shear		F'v= Fv(a)*Cd*Cm*Ct*Ci	
	Cd	1.15	snow load (most conservitive)
	Cm	1	moisture < 19% for extended periods
	Ct	1	t<100 degrees
	Ci	1	is it incised?

Fv allowable (psi)	170	
Fv allowable (psf)	24480	
F'v (psf)	28152	
Stressed (%)	**1.745950554**	**OK**

Deflection Check:

P (lb)	128	
L (in)	120	
I (in3)	263.671875	
E (psi)	1300000	
deflection (in)	0.2150925128	
L/240	**0.5**	**OK**



Bending Check #1:

Lateral load at top of column	128	lb	<-- D+0.6Wx
Max moment (P*L)	1280	lb-ft	
Column size	8x8		
b	7.5	in	
d	7.5	in	
b	0.625	ft	
d	0.625	ft	
S	70.3125	in3	
S	0.04069010417	ft3	
fb	31457.28	psf	
fb	218.4533333	psi	
Cd	1.15		
Cm	1		
Ct	1		
Cl	1		
CF	1		
Cfu	1		
Cc	1		
Cr	1		
Fb	750	psi	
F'b	862.5	psi	
fb/F'b (%)	**25.32792271**	**OK**	

Bearing Check: 3.10 of NDS

Axial load on column	7043.028	lb
Bearing area	56.25	in2
Applied load	125.209	psi
F*c (psi)	**805**	**OK**

F-5 End Column ASCE Check Calculations

In a cantilevered column system, stability of mass at the top is provided by one or more columns with base fixity acting as a single-degree-of-freedom system.

Cantilever column systems are essentially a special class of moment-resisting frame, except that they do not possess the redundancy and overstrength that most moment-resisting frames derive from sequential formation of yield or plastic hinges. Where a typical moment-resisting frame must form multiple plastic hinges in members to develop a yield mechanism, a cantilever column system develops hinges only at the base of the columns to form a mechanism. As a result, their overstrength is limited to that provided by material overstrength and by design conservatism.

It is permitted to construct cantilever column structures using any of the systems that can be used to develop moment frames, including ordinary and special steel; ordinary, intermediate, and special concrete; and timber frames. The system limitations for cantilever column systems reflect the type of moment frame detailing provided but with a limit on structural height, h_n, of 35 ft (10.7 m).

The value of R for cantilever column systems is derived from moment-resisting frame values where R is divided by Ω_0 but is not taken as less than 1 or greater than 2 1/2. This range accounts for the lack of sequential yielding in such systems. C_d is taken as equal to R, recognizing that damping is quite low in these systems and inelastic displacement of these systems is not less

12.2.5.2 Cantilever Column Systems. Cantilever column systems are permitted as indicated in Table 12.2-1 and as follows. The required axial strength of individual cantilever column elements, considering only the load combinations that include seismic load effects, shall not exceed 15% of the available axial strength, including slenderness effects.

Foundation and other elements used to provide overturning resistance at the base of cantilever column elements shall be designed to resist the seismic load effects, including overstrength of Section 12.4.3.

D+0.525Ev+0.525Eh+0.75S	44.08	psf					
Joists	=(G18*1*19)/2	lb	SW	34.73	lb	plf on beam	=(G19+J19)/1
Beams	=(((M19*9)+J20)/2)	lb	SW	='Middle Column	lb		
Total load	=G20	lb					

Bearing Check:

Check 8x8

fc=P/A		
P	=G21	lb
A	=G23*G24	in2
fc	=B26/B27	psi
Fc (axially loaded)	='Middle Column Calcs'!B13	psi
CD	1.15	
CM	1	
Ct	1	
CF	1	
Ci	1	
CP	=G37-((G38-G39)^(1/2))	
F'c	=B30*B31*B32*B33*B34*B35*B36	psi
fc/F'c (%)	**=(B28/B38)*100**	**=IF(B40>16.5, "NO GOOD", "OK")**

b	7.5	
d	7.5	
CP:		
FCE	=(0.822*G31)/((G32/G33)^2)	
F*c	=B30*B31*B32*B33*B34*B35	psi
c	0.9	
E'min	=0.85*(10^6)	psi
le	=G34*G35	in
d	=G24	in
Ke	2.4	
l	=10*12	in
1st term	=(1+(G28/G29))/(2*G30)	
2nd term	=((1+(G28/G29))/(2*G30))^2	
3rd term	=(G28/G29)/G30	

F-6 End Column ASCE Check Results

In a cantilevered column system, stability of mass at the top is provided by one or more columns with base fixity acting as a single-degree-of-freedom system.

Cantilever column systems are essentially a special class of moment-resisting frame, except that they do not possess the redundancy and overstrength that most moment-resisting frames derive from sequential formation of yield or plastic hinges. Where a typical moment-resisting frame must form multiple plastic hinges in members to develop a yield mechanism, a cantilever column system develops hinges only at the base of the columns to form a mechanism. As a result, their overstrength is limited to that provided by material overstrength and by design conservatism.

It is permitted to construct cantilever column structures using any of the systems that can be used to develop moment frames, including ordinary and special steel; ordinary, intermediate, and special concrete; and timber frames. The system limitations for cantilever column systems reflect the type of moment frame detailing provided but with a limit on structural height, h_n, of 35 ft (10.7 m).

The value of R for cantilever column systems is derived from moment-resisting frame values where R is divided by Ω_0 but is not taken as less than 1 or greater than 2 1/2. This range accounts for the lack of sequential yielding in such systems. C_d is taken as equal to R, recognizing that damping is quite low in these systems and inelastic displacement of these systems is not less

12.2.5.2 Cantilever Column Systems. Cantilever column systems are permitted as indicated in Table 12.2-1 and as follows. The required axial strength of individual cantilever column elements, considering only the load combinations that include seismic load effects, shall not exceed 15% of the available axial strength, including slenderness effects.

Foundation and other elements used to provide overturning resistance at the base of cantilever column elements shall be designed to resist the seismic load effects, including overstrength of Section 12.4.3.

D+0.525Ev+0.525Eh+0.75S	44.08	psf					
Joists	418.76	lb	SW	34.73	lb	plf on beam	453.49
Beams	2070.443281	lb	SW	59.4765625	lb		
Total load	2070.443281	lb					
b	7.5						
d	7.5						

Bearing Check:

Check 8x8

fc=P/A		
P	2070.443281	lb
A	56.25	in2
fc	36.80788056	psi
Fc (axially loaded)	700	psi
CD	1.15	
CM	1	
Ct	1	
CF	1	
Ci	1	
CP	0.5291	
F'c	425.9655195	psi
fc/F'c (%)	**8.641046955**	**OK**

CP:

FCE	473.836263	
F*c	805	psi
c	0.9	
E'min	850000	psi
le	288	in
d	7.5	in
Ke	2.4	
l	120	in
1st term	0.8825647088	
2nd term	0.7789204652	
3rd term	0.6540183064	

F-7 Middle Column ASCE Check Calculations

In a cantilevered column system, stability of mass at the top is provided by one or more columns with base fixity acting as a single-degree-of-freedom system.

Cantilever column systems are essentially a special class of moment-resisting frame, except that they do not possess the redundancy and overstrength that most moment-resisting frames derive from sequential formation of yield or plastic hinges. Where a typical moment-resisting frame must form multiple plastic hinges in members to develop a yield mechanism, a cantilever column system develops hinges only at the base of the columns to form a mechanism. As a result, their overstrength is limited to that provided by material overstrength and by design conservatism.

It is permitted to construct cantilever column structures using any of the systems that can be used to develop moment frames, including ordinary and special steel; ordinary, intermediate, and special concrete; and timber frames. The system limitations for cantilever column systems reflect the type of moment frame detailing provided but with a limit on structural height, h_n, of 35 ft (10.7 m).

The value of R for cantilever column systems is derived from moment-resisting frame values where R is divided by Ω_0 but is not taken as less than 1 or greater than 2 1/2. This range accounts for the lack of sequential yielding in such systems. C_d is taken as equal to R, recognizing that damping is quite low in these systems and inelastic displacement of these systems is not less

12.2.5.2 Cantilever Column Systems. Cantilever column systems are permitted as indicated in Table 12.2-1 and as follows. The required axial strength of individual cantilever column elements, considering only the load combinations that include seismic load effects, shall not exceed 15% of the available axial strength, including slenderness effects.

Foundation and other elements used to provide overturning resistance at the base of cantilever column elements shall be designed to resist the seismic load effects, including overstrength of Section 12.4.3.

D+0.525Ev+0.525Eh+0.75S		44.08	psf				
Joists	=(G18*1*19)/2		lb	SW	34.73 lb	plf on beam	=(G19+J19)/1
Beams	=(((M19*9)+J20)/2)		lb	SW	='Middle Column lb		
Total load	=G20*2		lb				
b		7.5					
d		7.5					

Bearing Check:		
Check 8x8		
fc=P/A		
P	=G21	lb
A	=G23*G24	in2
fc	=B26/B27	psi
Fc (axially loaded)	='Middle Column Calcs'!B13	psi
CD	1.15	
CM	1	
Ct	1	
CF	1	
Ci	1	
CP	=G37-((G38-G39)^(1/2))	
F'c	=B30*B31*B32*B33*B34*B35*B36	psi
fc/F'c (%)	**=(B28/B38)*100**	**=IF(B40>16.5, "NO GOOD", "OK")**

CP:		
FCE	=(0.822*G31)/((G32/G33)^2)	
F*c	=B30*B31*B32*B33*B34*B35	psi
c	0.9	
E'min	=0.85*(10^6)	psi
le	=G34*G35	in
d	=G24	in
Ke	2.4	
l	=10*12	in
1st term	=(1+(G28/G29))/(2*G30)	
2nd term	=((1+(G28/G29))/(2*G30))^2	
3rd term	=(G28/G29)/G30	

F-8 Middle Column ASCE Check Results

In a cantilevered column system, stability of mass at the top is provided by one or more columns with base fixity acting as a single-degree-of-freedom system.

Cantilever column systems are essentially a special class of moment-resisting frame, except that they do not possess the redundancy and overstrength that most moment-resisting frames derive from sequential formation of yield or plastic hinges. Where a typical moment-resisting frame must form multiple plastic hinges in members to develop a yield mechanism, a cantilever column system develops hinges only at the base of the columns to form a mechanism. As a result, their overstrength is limited to that provided by material overstrength and by design conservatism.

It is permitted to construct cantilever column structures using any of the systems that can be used to develop moment frames, including ordinary and special steel; ordinary, intermediate, and special concrete; and timber frames. The system limitations for cantilever column systems reflect the type of moment frame detailing provided but with a limit on structural height, h_n, of 35 ft (10.7 m).

The value of R for cantilever column systems is derived from moment-resisting frame values where R is divided by Ω_0 but is not taken as less than 1 or greater than 2 1/2. This range accounts for the lack of sequential yielding in such systems. C_d is taken as equal to R, recognizing that damping is quite low in these systems and inelastic displacement of these systems is not less

12.2.5.2 Cantilever Column Systems. Cantilever column systems are permitted as indicated in Table 12.2-1 and as follows. The required axial strength of individual cantilever column elements, considering only the load combinations that include seismic load effects, shall not exceed 15% of the available axial strength, including slenderness effects.

Foundation and other elements used to provide overturning resistance at the base of cantilever column elements shall be designed to resist the seismic load effects, including overstrength of Section 12.4.3.

D+0.525Ev+0.525Eh+0.75S	44.08	psf					
Joists	418.76	lb	SW	34.73	lb	plf on beam	453.49
Beams	2070.443281	lb	SW	59.4765625	lb		
Total load	4140.886563	lb					
b	7.5						
d	7.5						

Bearing Check:		
Check 8x8		
fc=P/A		
P	4140.886563	lb
A	56.25	in2
fc	73.61576111	psi
Fc (axially loaded)	700	psi
CD	1.15	
CM	1	
Ct	1	
CF	1	
Ci	1	
CP	0.5291	
F'c	425.9655195	psi
fc/F'c (%)	**17.28209391**	**NO GOOD**

CP:		
FCE	473.836263	
F*c	805	psi
c	0.9	
E'min	850000	psi
le	288	in
d	7.5	in
Ke	2.4	
l	120	in
1st term	0.8825647088	
2nd term	0.7789204652	
3rd term	0.6540183064	

Appendix G- Connections

G-1 Joist to Beam Connection Calculations

Total down load	=576.234	lb	<-- 2x12	HU212 allowable down snow load	=1680*F11	lb	=if(F1>B1, "OK", "NO GOOD")
Total uplift	=B11	lb	<-- 2x12	HU212 allowable uplift	=1135*F12	lb	=if(F2>B2, "OK", "NO GOOD")

Total down load	=((156.8*19)/2)+7	lb	<-- (3) 2x12	HU212-3 allowable down snow load	=2685*F11	lb	=if(F4>B4, "OK", "NO GOOD")
Total uplift	=B17	lb	<-- (3) 2x12	HU212-3 allowable uplift	=1135*F12	lb	=if(F5>B5, "OK", "NO GOOD")

2x12:

Uplift	12.6	psf
Joist trib	1	ft
Joist length	19	ft
Uplift	=(B8*B9*B10)/2	lb

(3) 2x12:

Uplift	12.6	psf
Joist trib	2.75	ft
Joist length	19	ft
Uplift	=(B14*B15*B16)/2	lb

Use: HU212 modified for slope down for single 2x12 joists
Use: HU212-3 modified for slope down for (3) 2x12 joists

reduction factor for down load	0.65
redcuction factor for uplift	0.65

U/HU/HUC Series Modifications and Associated Load Reduction Factors

Seat			Flange	Fastener Substitutions	
Seat Sloped Up or Down 45° Max.	Seat Skewed 67½° Max.[3] for W ≤ 6 45° Max. for W ≥ 6	Seat Sloped and Skewed	One or Both HU Flanges Concealed[2]	16d Stainless-Steel Nails	Other Fastener Substitutions
1.00	W ≤ 3 9/16 use 1.00 W > 3 9/16 use 0.80	0.80	1.00 (normal) 0.80 (when sloped and skewed)	Ring shank (all conditions) 1.00 Smooth shank (normal seat) 1.00 Smooth shank (modified seat[1]) 0.50	16d → 16d x 2½" 1.00 16d → 10d 0.84 16d → 10d x 1½" 0.64

1. Modified seat is sloped, skewed or both. If sloped only or skewed only, use a smooth shank stainless steel reduction of 0.65.
2. For both flanges concealed, W must be at least 2 5/16". To order ask for HUCXXX.
For skewed HUC, only flange on acute side is concealed.
3. Skews over 50° require a square-cut joist.

G-2 Joist to Beam Connection Results

Total down load	576.234	lb	<-- 2x12	HU212 allowable down snow load	1092	lb	OK
Total uplift	119.7	lb	<-- 2x12	HU212 allowable uplift	737.75	lb	OK

Total down load	1496.6	lb	<-- (3) 2x12	HU212-3 allowable down snow load	1745.25	lb	OK
Total uplift	329.175	lb	<-- (3) 2x12	HU212-3 allowable uplift	737.75	lb	OK

Use: HU212 modified for slope down for single 2x12 joists
Use: HU212-3 modified for slope down for (3) 2x12 joists

2x12:

Uplift	12.6	psf
Joist trib	1	ft
Joist length	19	ft
Uplift	119.7	lb

reduction factor for down load	0.65
redcuction factor for uplift	0.65

(3) 2x12:

Uplift	12.6	psf
Joist trib	2.75	ft
Joist length	19	ft
Uplift	329.175	lb

U/HU/HUC Series Modifications and Associated Load Reduction Factors

Seat			Flange	Fastener Substitutions	
Seat Sloped Up or Down 45° Max.	Seat Skewed 67½° Max.[3] for W ≤ 6 45° Max. for W ≥ 6	Seat Sloped and Skewed	One or Both HU Flanges Concealed[2]	16d Stainless-Steel Nails	Other Fastener Substitutions
1.00	W ≤ 3⁹⁄₁₆ use 1.00 W > 3⁹⁄₁₆ use 0.80	0.80	1.00 (normal) 0.80 (when sloped and skewed)	Ring shank (all conditions) 1.00 Smooth shank (normal seat) 1.00 Smooth shank (modified seat[1]) 0.50	16d → 16d x 2½" 1.00 16d → 10d 0.84 16d → 10d x 1½" 0.64

1. Modified seat is sloped, skewed or both. If sloped only or skewed only, use a smooth shank stainless steel reduction of 0.65.
2. For both flanges concealed, W must be at least 2⁵⁄₁₆". To order ask for HUCXXX.
For skewed HUC, only flange on acute side is concealed.
3. Skews over 50° require a square-cut joist.

G-3 Beam to Column Connection Calculations

Total down load	=7225.42/2	lb	end columns	LCE4 allowable uplift	1905	lb	=if(F1>B2, "OK",
Total uplift	=B16	lb	end columns	LCE4 allowable lateral	1425	lb	=if(F2>B3, "OK",
Lateral	=B11	lb	end columns				

Total down load	7225.42	lb	middle columns	1616HT allowable uplift	2585	lb	=if(F5>B2, "OK",
Total uplift	=B16*2	lb	middle columns	1616HT allowable F1	815	lb	=if(F6>B3, "OK",
Lateral	=B11*2	lb	middle columns				

Use: 1616HT for center columns each side
Use: LCE4 for end columns each side

Wind load	51	psf
Lateral	=51*8*1	lb
Uplift	7.2	psf
Trib	9	ft
Beam length	8	ft
Uplift for 1 beam end	=(B13*B14*B15)/	lb

G-4 Beam to Column Connection Results

Total down load	3612.71	lb	end columns	LCE4 allowable uplift	1905	lb	OK
Total uplift	259.2	lb	end columns	LCE4 allowable lateral	1425	lb	OK
Lateral	408	lb	end columns				

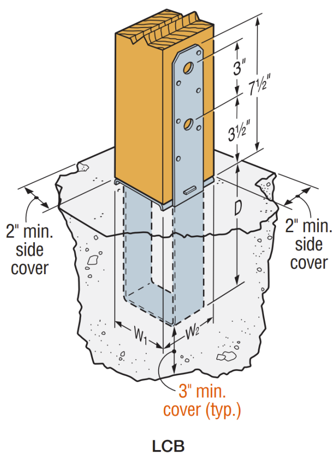
Total down load	7225.42	lb	middle columns	1616HT allowable uplift	2585	lb	OK
Total uplift	518.4	lb	middle columns	1616HT allowable F1	815	lb	OK
Lateral	816	lb	middle columns				

Use: 1616HT for center columns each side
Use: LCE4 for end columns each side

Wind load	51	psf
Lateral	408	lb
Uplift	7.2	psf
Trib	9	ft
Beam length	8	ft
Uplift for 1 beam end	259.2	lb

G-5 Column to Footing Connection Calculations

Total down load	7418	lb
Total shear load	408	lb
Total uplift	='Beam to Column'!B6	lb
Total moment	4080	lb-ft



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Concrete Allowables:

MPB88Z allowable uplift	6,100	lb	=if(F2>B3, "OK", "NO GOOD")
MPB88Z allowable shear	4875	lb	=if(F3>B2, "OK", "NO GOOD")
MPB88Z allowable moment	4525	lb-ft	=if(F4>B4, "OK", "NO GOOD")

Wood Assembly Allowables:

MPB88Z allowable down load	17585	lb	=if(F7>B1, "OK", "NO GOOD")
MPB88Z allowable moment	4525	lb-ft	=if(F8>B4, "OK", "NO GOOD")

Use: MPB88Z

G-6 Column to Footing Connection Results

Total down load	7418	lb
Total shear load	408	lb
Total uplift	518.4	lb
Total moment	4080	lb-ft

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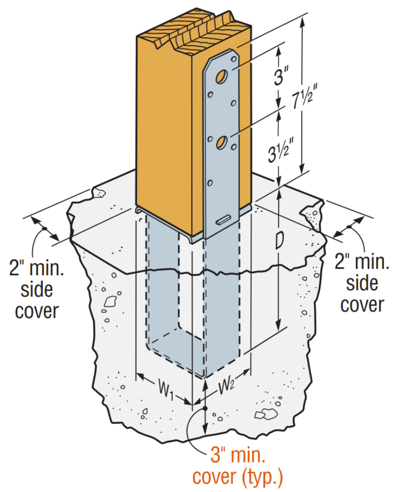


LCB

Concrete Allowables:

MPB88Z allowable uplift	6,100	lb	OK
MPB88Z allowable shear	4875	lb	OK
MPB88Z allowable moment	4525	lb-ft	OK

Wood Assembly Allowables:

MPB88Z allowable down load	17585	lb	OK
MPB88Z allowable moment	4525	lb-ft	OK

Use: MPB88Z

Appendix H: Foundation Design

H-1 Bearing Design for Foundations Calculations

***We will specify 4000 psi concrete in drawings (per ACI 19.2.1.1)**

qu:			
c'	0	<-- little to no fines or clay	
angle	35	<-- from geotech report	
Nc	57.75		
Nq	41.44		
Ngamma	45.41		
unit weight	122.5	<--- estimate found online	
Df	3	ft	
Diameter (B)	2	ft	<-- Can change
Factor of Safety	3		
CASE 3	**d>B**		
q	=B8*B9	lbs/ft^2	Df*gamma
qu	=(1.3*B3*B5+B17*B6)+(0.3*B8*B10*B7)	lbs/ft^2	
qallowable	=B19/B12	lbs/ft^2	
q net	=B19-B17	lbs/ft^2	
q allowable (net)	=(B19-B17)/B12	lbs/ft^2	
Q	=B21*B10	LBS/FT	<- MAX ALLOWABLE LOAD
Reinforcing:	0.01Ag<x<0.08Ag	<-- temperature and shrinkage	
	diameter (in)	=B10*12	
	Ag (in2)	=(pi()*(C31^2))/4	
	As,min (in2)	=C32*0.01	
	As,max (in2)	=0.08*C32	
	Bar Diameter (in)	0.75	<-- #6 bars
	area of bar (in2)	=0.25*3.14*(C36^2)	
	Required bars	=ROUNDUP(C33/C37, 0.1)	
	Area of rebar (in2)	=C37*C38	
	min spacing (in)	1	
	cc (in)	2	
	#4 stirrup diameter (in)	0.5	
	bw min (in)	=(2*C41)+(2*C42)+(C38*C36)+((C38-1)*C40)+0.5	=if(C43<C31, "OK", "NO GOOD")
Shear reinforcement:	0.75(f'c)^(1/2)(bw*s/fyt)	=0.75*sqrt(4000)*((24*C49)/60000)	
	greater of		
	50(bw*s/fyt)	=50*((24*C49)/60000)	
	Min. reinforcement required (in2)	=max(C47,C45)	
	s (in)	=36/4	(d/4)
	4(f'c)^(1/2)bwd	=4*SQRT(4000)*C31*(B9*12)	
Vs,req	Vu/Phi-Vc	=C53/0.75-C52	
Vc	2sqrt(4000)bwd	=2*4000^0.5*B9*B10	
Vu		=G3/2	
2 #3's stirups	0.22	in	>0.18in^2

total down load	7418	lb	
area of foundation	=((B10^2)*pi())/	ft2	
	=G3/G4	psf	=if(G5<B2
2nd Check:			
qu 1st term	=1.3*B3*B5		
qu 2nd term	=B8*B9*B6		=G14/4
qu 3rd term	=0.4*B8*B7	*B	=G15/4
qu=	2225.09B+15229.2		
qall=	556.2725B+3807.3		
7418/B^2=	556.2725B+3807.3		
B (ft)	1.28		

H-2 Bearing Design for Foundations Results

***We will specify 4500 psi concrete in drawings (per ACI 19.2.1.1)**

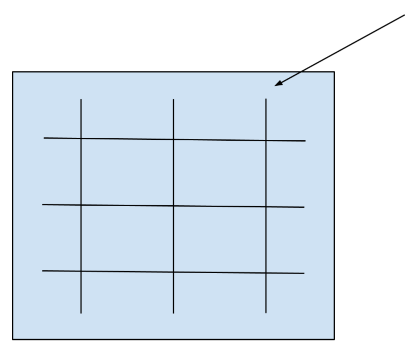
qu:			
c'	0	<-- little to no fines or clay	
angle	35	<-- from geotech report	
Nc	57.75		
Nq	41.44		
Ngamma	45.41		
unit weight	122.5	<--- estimate found online	
Df	2.5	ft	
Width (B)	3	ft	<-- Can change
Factor of Safety	3		

q	306.25	lbs/ft^2	Df*gamma
qu	19366.27	lbs/ft^2	
qallowable	6455.423333	lbs/ft^2	
q net	19060.02	lbs/ft^2	
q allowable (net)	6353.34	lbs/ft^2	
Q	19366.27	LBS/FT	<- MAX ALLOWABLE LOAD

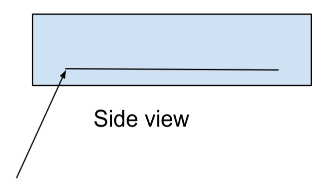
total down load	7418	lb	
area of foundation	9	ft2	
	824.2222222	psf	OK

2nd Check:			
qu 1st term	0		
qu 2nd term	12691		3172.75
qu 3rd term	2225.09	*B	556.2725
qu=	2225.09B+15229.2		
qall=	556.2725B+3807.3		
7418/B^2=	556.2725B+3807.3		
B (ft)	1.28		

Reinforcing:	0.0018Ag	<-- temperature and shrinkage	
	width (in)	36	
	Ag (in2)	432	
	As,min (in2)	0.778	
	Bar Diameter (in)	0.625	<-- #5 bars
	area of bar (in2)	0.307	
	Required bars	3	
	Area of rebar (in2)	0.920	
	min spacing (in)	1	
	cc (in)	2	
	#4 tie diameter (in)	0.5	
	bw min (in)	9.375	OK
Shear reinforcement:	0.75(f'c)^(1/2)(bw*s/fyt	0.199	
	greater of		
	50(bw*s/fyt)	0.21	
Min. reinforcement required (in2)		0.21	
s (in)		10.5	(d/4)
	4(f'c)^(1/2)bwd	273220.7898	
Vs,req	Vu/Phi-Vc	3996.650035	
Vc	2sqrt(4000)bwd	948.6832981	
Vu		3709	
2 #4's stirups	0.4	in	>0.21in^2



Bird’s eye



Side view

H-3 Sliding Check Calculations

Sliding Check:

Lateral load	1700	lb	
B	='Bearing Capacity'!B10	ft	
Df	='Bearing Capacity'!B9	ft	
Lateral load	=B2/(B3*B4)	psf	
Lateral resistance	350	pcf	
Lateral resistance	=B7*B3	psf	=IF(B5<B8, "OK", "NO GOOD")

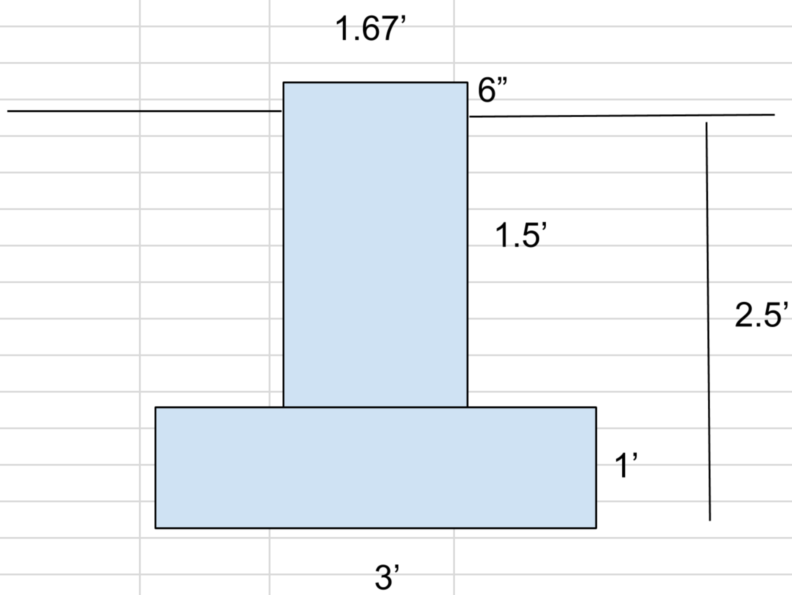
H-4 Sliding Check Results

Sliding Check:

Lateral load	1700	lb	
B	3	ft	
Df	2.5	ft	
Lateral load	226.67	psf	
Lateral resistance	350	pcf	
Lateral resistance	1050	psf	OK

H-5 Uplift Check Calculations

Uplift Check:			
Weight of concrete	=G17	lb	
Weight of soil	=G25	lb	
Weight resisting uplift	=B2+B3	lb	
Uplift from column	518.4	lb	=if(B4>B6, "OK", "NO GOOD")



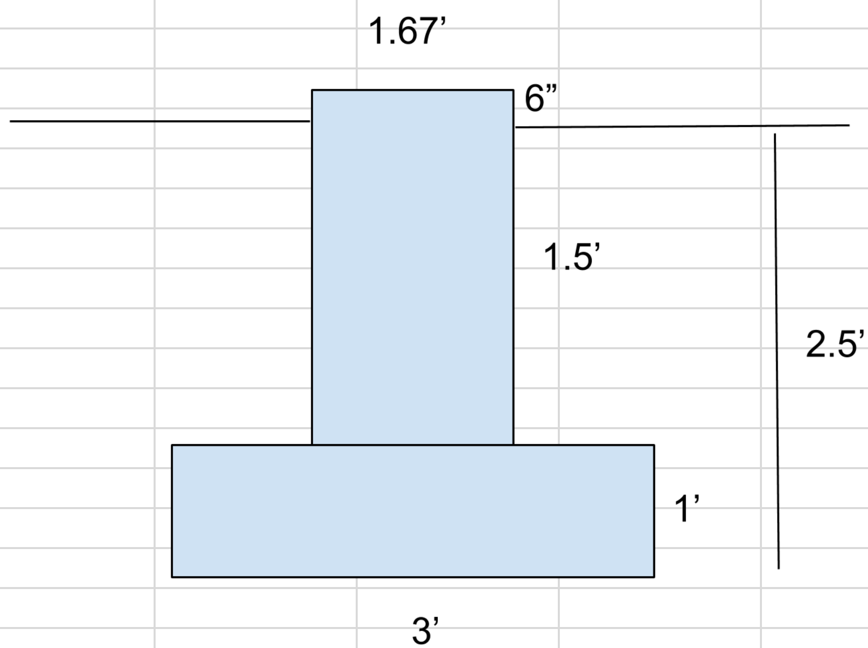
unit weight of concrete	150	lb/ft3
Simpson:		
L	1.67	ft
W	1.67	ft
D	2	ft
Volume of concrete	=G5*G6*G7	ft3
Ours:		
L	3	ft
W	3	ft
D	1	ft
Volume of concrete	=G11*G12*G13	ft3
Total volume of concrete	=G8+G14	ft3
Weight of concrete	=G3*G16	lb

unti weight of soil	='Bearing Capacity'!B8	lb/ft3
3x3 area	=G11*G12	ft2
1.67x1.67 area	=G5*G6	ft2
area of soil	=G20-G21	ft2
height of soil	1.5	ft2
volume of soil	=G22*G23	ft3
weight of soil	=G19*G24	lb

H-6 Uplift Check Results

Uplift Check:			
Weight of concrete	2186.67	lb	
Weight of soil	1141.289625	lb	
Weight resisting uplift	3327.959625	lb	
Uplift from column	518.4	lb	OK

unit weight of concrete	150	lb/ft3
Simpson:		
L	1.67	ft
W	1.67	ft
D	2	ft
Volume of concrete	5.5778	ft3
Ours:		
L	3	ft
W	3	ft
D	1	ft
Volume of concrete	9	ft3
Total volume of concrete	14.5778	ft3
Weight of concrete	2186.67	lb

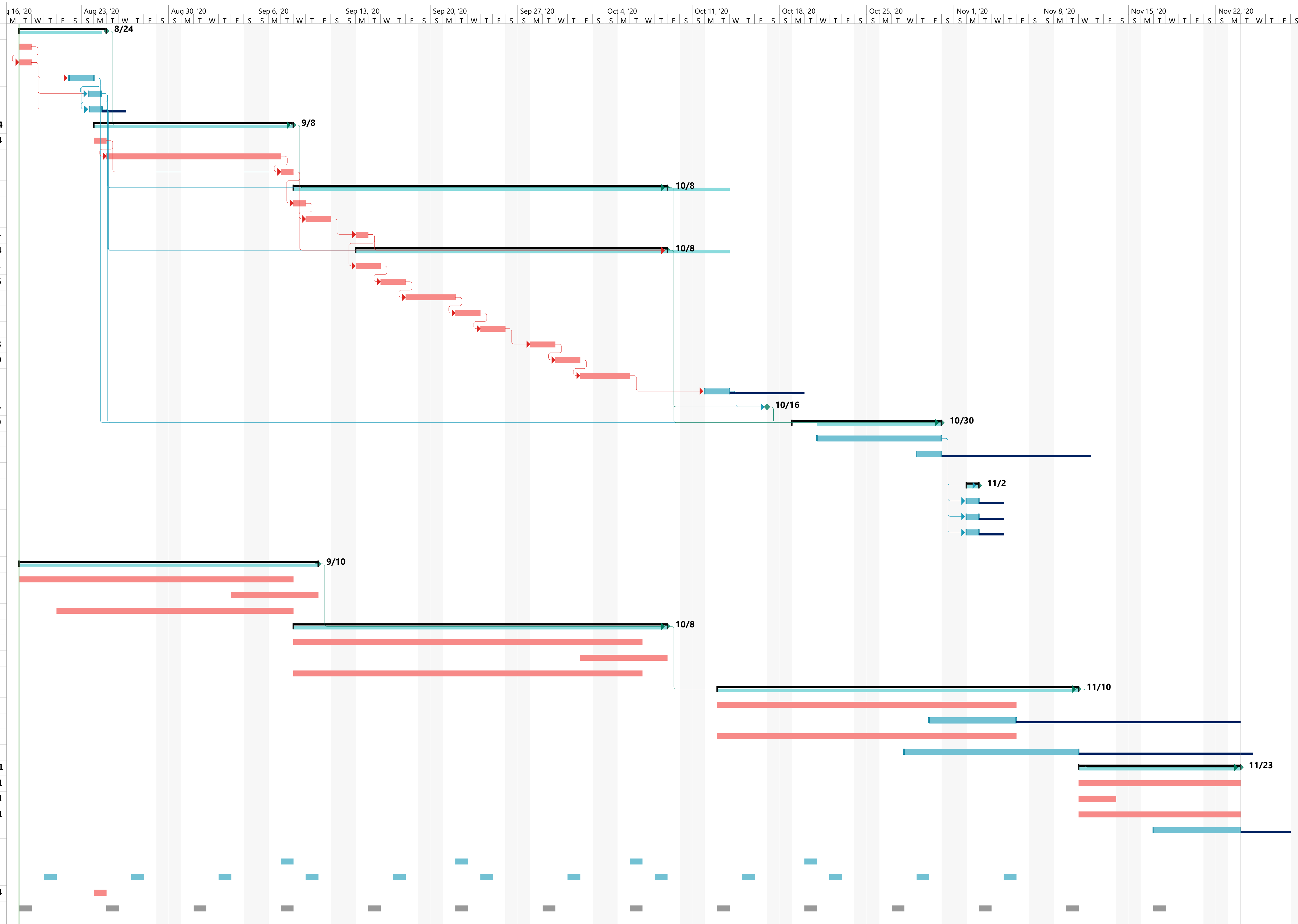


unti weight of soil	122.5	lb/ft3
3x3 area	9	ft2
1.67x1.67 area	2.7889	ft2
area of soil	6.2111	ft2
height of soil	1.5	ft2
volume of soil	9.31665	ft3
weight of soil	1141.289625	lb

Appendix I - Schedules

I-1 Original Schedule

ID	Task Name	Duration	Start
1	**Task 1: Analyze Existing Site**	**7 days**	**Tue Aug 18**
2	1.1: Pre-Site Visit Research	1 day	Tue Aug 18
3	1.2: Site Visit	1 day	Tue Aug 18
4	1.3: Survey	2 days	Sat Aug 22
5	1.4: Topographic Map	1 day	Sun Aug 23
6	1.5: Hydraulic Drainage Analysis	1 day	Sun Aug 23
7	**Task 2: Geotechnical Analysis**	**12 days**	**Mon Aug 24**
8	2.1: Soil Collection	1 day	Mon Aug 24
9	2.2: Lab Testing	10 days	Tue Aug 25
10	2.3: Soil Classification	1 day	Tue Sep 8
11	**Task 3: Structural Analysis**	**22 days**	**Wed Sep 9**
12	3.1: Examination of Existing Structures	1 day	Wed Sep 9
13	3.2: Develop Alternatives	2 days	Thu Sep 10
14	3.3: Ramada Geometry	1 day	Mon Sep 14
15	**3.4: Proposed Design Analysis**	**19 days**	**Mon Sep 14**
16	3.4.1: Design Loads	2 days	Mon Sep 14
17	3.4.2: Decking	2 days	Wed Sep 16
18	3.4.3: Trusses/Joists	2 days	Fri Sep 18
19	3.4.4: Beams	2 days	Tue Sep 22
20	3.4.5: Columns	2 days	Thu Sep 24
21	3.4.6: Foundations	2 days	Mon Sep 28
22	3.4.7: Connections	2 days	Wed Sep 30
23	3.4.8: Lateral Analysis	2 days	Fri Oct 2
24	3.4.9: Decision Matrix	2 days	Mon Oct 12
25	**Task 4: Material Specifications**	**3 days**	**Wed Oct 14**
26	**Task 5: Site Design**	**10 days**	**Mon Oct 19**
27	5.1: Plan Set	8 days	Wed Oct 21
28	5.2: Cost Estimate	2 days	Thu Oct 29
29	**Task 6: Project Management**	**72 days**	**Tue Aug 18**
30	**6.1: Project Impacts**	**1 day**	**Mon Nov 2**
31	6.1.1: Environmental Impacts	1 day	Mon Nov 2
32	6.1.2: Economic Impacts	1 day	Mon Nov 2
33	6.1.3: Social Impacts	1 day	Mon Nov 2
34	**6.2: Project Devilerables**	**55 days**	**Tue Sep 8**
35	**6.2.1: 30% Submittal**	**20 days**	**Tue Aug 18**
36	6.2.1.1: 30% Report	18 days	Tue Aug 18
37	6.2.1.2: 30% Presentation	5 days	Fri Sep 4
38	6.2.1.3: 30% Plan Set	15 days	Fri Aug 21
39	**6.2.2: 60% Submittal**	**22 days**	**Wed Sep 9**
40	6.2.2.1: 60% Report	20 days	Wed Sep 9
41	6.2.2.2: 60% Presentation	5 days	Fri Oct 2
42	6.2.2.3: 60% Plan Set	20 days	Wed Sep 9
43	**6.2.3: 90% Submittal**	**21 days**	**Tue Oct 13**
44	6.2.3.1: 90% Report	18 days	Tue Oct 13
45	6.2.3.2: Practice Presentation	5 days	Fri Oct 30
46	6.2.3.3: 90% Plan Set	18 days	Tue Oct 13
47	6.2.3.4: 90% Website	10 days	Wed Oct 28
48	**6.2.4: Final Submittal**	**9 days**	**Wed Nov 11**
49	6.2.4.1: Final Report	9 days	Wed Nov 11
50	6.2.4.2: Final Presentation	3 days	Wed Nov 11
51	6.2.4.3: Final Plan Set	9 days	Wed Nov 11
52	6.2.4.4: Final Website	5 days	Tue Nov 17
53	**6.3: Meetings**	**72 days**	**Tue Aug 18**
54	**6.3.1: Technical Advisor Meetings**	**31 days**	**Tue Sep 8**
59	**6.3.2: Grading Instructor Meetings**	**58 days**	**Thu Aug 20**
72	**6.3.3: Client Meetings**	1 day	Mon Aug 24
73	**6.3.4: Team Meetings**	**68 days**	**Tue Aug 18**
88	**6.4: Resource Management**	72 days	Tue Aug 18

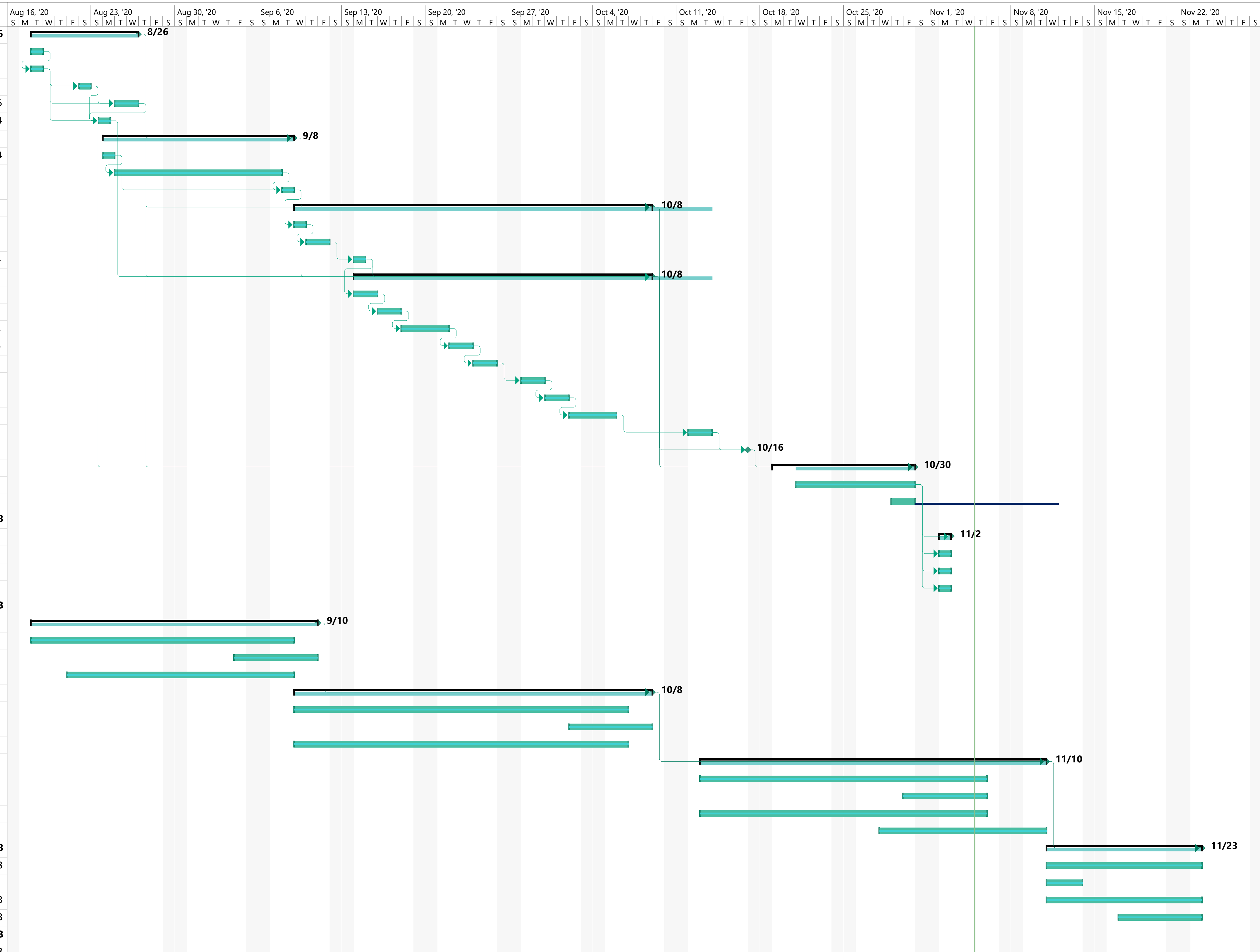


Project: UPDATED SCHEDULE
Date: Tue Aug 18

Task	Summary	Inactive Milestone	Duration-only	Start-only	External Milestone	Critical Split	Slack
Split	Project Summary	Inactive Summary	Manual Summary Rollup	Finish-only	Deadline	Progress	
Milestone	Inactive Task	Manual Task	Manual Summary	External Tasks	Critical	Manual Progress	

I-2 Final Schedule

ID	Task Name	Duration	Start	Finish
1	**Task 1: Analyze Existing Site**	**9 days**	**Tue Aug 18**	**Wed Aug 26**
2	1.1: Pre-Site Visit Research	1 day	Tue Aug 18	Tue Aug 18
3	1.2: Site Visit	1 day	Tue Aug 18	Tue Aug 18
4	1.3: Survey	1 day	Sat Aug 22	Sat Aug 22
5	1.4: Topographic Map	2 days	Tue Aug 25	Wed Aug 26
6	1.5: Hydraulic Drainage Analysis	1 day	Sun Aug 23	Mon Aug 24
7	**Task 2: Geotechnical Analysis**	**12 days**	**Mon Aug 24**	**Tue Sep 8**
8	2.1: Soil Collection	1 day	Mon Aug 24	Mon Aug 24
9	2.2: Lab Testing	10 days	Tue Aug 25	Mon Sep 7
10	2.3: Soil Classification	1 day	Tue Sep 8	Tue Sep 8
11	**Task 3: Structural Analysis**	**22 days**	**Wed Sep 9**	**Thu Oct 8**
12	3.1: Examination of Existing Structures	1 day	Wed Sep 9	Wed Sep 9
13	3.2: Develop Alternatives	2 days	Thu Sep 10	Fri Sep 11
14	3.3: Ramada Geometry	1 day	Mon Sep 14	Mon Sep 14
15	**3.4: Proposed Design Analysis**	**19 days**	**Mon Sep 14**	**Thu Oct 8**
16	3.4.1: Design Loads	2 days	Mon Sep 14	Tue Sep 15
17	3.4.2: Decking	2 days	Wed Sep 16	Thu Sep 17
18	3.4.3: Trusses/Joists	2 days	Fri Sep 18	Mon Sep 21
19	3.4.4: Beams	2 days	Tue Sep 22	Wed Sep 23
20	3.4.5: Columns	2 days	Thu Sep 24	Fri Sep 25
21	3.4.6: Foundations	2 days	Mon Sep 28	Tue Sep 29
22	3.4.7: Connections	2 days	Wed Sep 30	Thu Oct 1
23	3.4.8: Lateral Analysis	2 days	Fri Oct 2	Mon Oct 5
24	3.4.9: Decision Matrix	2 days	Mon Oct 12	Tue Oct 13
25	**Task 4: Material Specifications**	**3 days**	**Wed Oct 14**	**Fri Oct 16**
26	**Task 5: Site Design**	**10 days**	**Mon Oct 19**	**Fri Oct 30**
27	5.1: Plan Set	8 days	Wed Oct 21	Fri Oct 30
28	5.2: Cost Estimate	2 days	Thu Oct 29	Fri Oct 30
29	**Task 6: Project Management**	**72 days**	**Tue Aug 18**	**Mon Nov 23**
30	**6.1: Project Impacts**	**1 day**	**Mon Nov 2**	**Mon Nov 2**
31	6.1.1: Environmental Impacts	1 day	Mon Nov 2	Mon Nov 2
32	6.1.2: Economic Impacts	1 day	Mon Nov 2	Mon Nov 2
33	6.1.3: Social Impacts	1 day	Mon Nov 2	Mon Nov 2
34	**6.2: Project Devilerables**	**55 days**	**Tue Sep 8**	**Mon Nov 23**
35	**6.2.1: 30% Submittal**	**20 days**	**Tue Aug 18**	**Thu Sep 10**
36	6.2.1.1: 30% Report	18 days	Tue Aug 18	Tue Sep 8
37	6.2.1.2: 30% Presentation	5 days	Fri Sep 4	Thu Sep 10
38	6.2.1.3: 30% Plan Set	15 days	Fri Aug 21	Tue Sep 8
39	**6.2.2: 60% Submittal**	**22 days**	**Wed Sep 9**	**Thu Oct 8**
40	6.2.2.1: 60% Report	20 days	Wed Sep 9	Tue Oct 6
41	6.2.2.2: 60% Presentation	5 days	Fri Oct 2	Thu Oct 8
42	6.2.2.3: 60% Plan Set	20 days	Wed Sep 9	Tue Oct 6
43	**6.2.3: 90% Submittal**	**21 days**	**Tue Oct 13**	**Tue Nov 10**
44	6.2.3.1: 90% Report	18 days	Tue Oct 13	Thu Nov 5
45	6.2.3.2: Practice Presentation	5 days	Fri Oct 30	Thu Nov 5
46	6.2.3.3: 90% Plan Set	18 days	Tue Oct 13	Thu Nov 5
47	6.2.3.4: 90% Website	10 days	Wed Oct 28	Tue Nov 10
48	**6.2.4: Final Submittal**	**9 days**	**Wed Nov 11**	**Mon Nov 23**
49	6.2.4.1: Final Report	9 days	Wed Nov 11	Mon Nov 23
50	6.2.4.2: Final Presentation	3 days	Wed Nov 11	Fri Nov 13
51	6.2.4.3: Final Plan Set	9 days	Wed Nov 11	Mon Nov 23
52	6.2.4.4: Final Website	5 days	Tue Nov 17	Mon Nov 23
53	**6.3: Meetings**	**72 days**	**Tue Aug 18**	**Mon Nov 23**
86	**6.4: Resource Management**	72 days	Tue Aug 18	Mon Nov 23



Project: UPDATED SCHEDULE
Date: Thu Nov 5

Task	Summary	Inactive Milestone	Duration-only	Start-only	External Milestone	Critical Split
Split	Project Summary	Inactive Summary	Manual Summary Rollup	Finish-only	Deadline	Progress
Milestone	Inactive Task	Manual Task	Manual Summary	External Tasks	Critical	Manual Progress
						Slack