Treasure Valley Scout Reservation

Structural Design and Design of East Lodge Hall

A Major Qualifying Project Report:

Submitted to Faculty of WORCESTER POLYTECHNIC INSTITUTE

in partial fulfillment of the requirements for the

Degree of Bachelor of Science By

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Sonia Banegas

Date: ______________________

Approved By:

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Professor Steven Van Dessel, Advisor

ABSTRACT

In 2015, The Treasure Valley Boy Scout Reservation reached to work with Worcester Polytechnic Institute to design a new addition to one of their facilities. This facility was the dining hall East Lodge. The facility contained some structural defects, such as lack of insulation and sufficient space to house the boy scouts. After an extensive structural evaluation, I was able to design a new addition to the building. The new addition addressed the lack of space and building layout problems of the current facility. The design won't be implemented, but it will serve as a reference for possible new changes.

AUTHORSHIP

Sonia Banegas:

My initial role of the project was to document the building. Various visits to the Treasure Valley Scout Reservation were required to document measurements and pictures of the building. Using the information from the site documentation, such as measurements and pictures, I conducted the structural analysis on the structural members of the East Lodge dining hall. The structural members included the truss structure, floor, walls, roof, and columns. After the structural analysis was completed, I produced the CAD drawings of the existing building. I also designed an alternative addition to the facility using AutoCad and Revit. I was also in charge of writing the report and making the final edits.

CAPSTONE DESIGN

Worcester Polytechnic Institute requires all students to fulfill a Capstone Design for their Major Qualifying Project (MQP). The MQP gives the students the opportunity to define a problem, develop a scope of work to address the problem, and determine design objectives. The Capstone Design provides students with a real-world experience of research and design within their major field. During this process of design completion, students get to apply the knowledge and skills they have gained in prior classes. This MQP has followed the requirements established by the Civil and Environmental Engineering Department.

The problem defined by this MQP was that the East Lodge facility contained some structural defects, such as lack of insulation and sufficient space to house the boy scouts. The scope of work involved conducting a structural evaluation on the facility in order to determine the structural stability of the current facility. Using the structural evaluation results, I was able to design a new addition to the building. The new addition addressed the lack of space and building layout problems of the current facility.

The results of the structural analysis determined the facility was structurally stable. This was a crucial part of the project, since determining the stability of the building structure is key to the safety of the people making use of the facility. Therefore, the new addition was designed based on the structural analysis results, to ensure that it would be structurally stable for the safety of the people using it.

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EXECUTIVE SUMMARY

The Treasure Valley Boy Scout Reservation is one of the scouting units supported by the Mohegan Council. The Reservation is located among four towns in Massachusetts; Spencer, Oakham, Rutland, and Paxton. It is composed of various acres of land that provides a diverse wildlife and ecosystem. The Reservation offers camps during the summer for the youth who want to enjoy valuable outdoor learning experience through their scouting programs.

The Treasure Valley Boy Scout Reservation is made up of many buildings that house the different activities of the scout programs. One of their main facilities is the East Lodge. This facility is located near Browning Pond. The East Lodge serves mainly as dining hall and gathering space during the summer camps.

The Treasure Valley Boy Scout Reservation requested assistance from Worcester Polytechnic Institute to help them with providing structural advice regarding their East Lodge building facility. The current state of the facility seemed structurally stable. One of the main problems is the lack of space to gather more than 200 people. The other problem regarding this facility was the poor condition of the kitchen space.

This MQP project focused on addressing the two main problems that the East Lodge scout facility faced. Before addressing these problems, an evaluation of the existing structure was conducted. Even though the current structure of the facility seemed structurally stable, a structural analysis was performed to ascertain load bearing capacity of the critical structural components of the building. The structural analysis focused on evaluating the main components of the building facility, which included the roof trusses, columns, roof boards, the floor, and the walls.

In order to conduct the structural analysis of East Lodge, a series of steps were followed. First, various visits were made to the camp site to document the building. The documentation included important information such as pictures and measurements. This detailed information was used to conduct the structural analysis of the main building components. The structural analysis followed specifications from the International Building Code.

Following the structural analysis performed, the two main problems of the East Lodge could be addressed; the lack of enough gathering space and the conditions of the current kitchen space. A new additional space to the existing East lodge was designed in order to solve these problems. The proposed design included the existing East Lodge dining hall, but removed the existing kitchen to make place for a new addition that included additional dining space, a new kitchen, new sanitary facilities, and a new entrance.

This newly proposed kitchen space would be bigger in order to provide service to the current East Lodge and to the new gathering space. It also included storage and cooling space for the food supplies. The new gathering area would be located next to the new kitchen space and have the same functions as the East Lodge; dining hall and gathering space. In addition, it would include a mechanical room and an office space. Also, as part of the new design, a deck at the back of the new East Lodge was included. This deck includes outside sitting space and could be used for other outdoor gathering activities.

The new addition to the East Lodge was designed using AutoCad and Revit. The design was proposed and presented to the members of the Mohegan Council and Treasure Valley Scout Reservation. The members’ feedback was taken into consideration for the final design. The members are evaluating other design and construction options as well. This new addition remains as one of their design alternatives for the future plans ahead regarding the East Lodge.

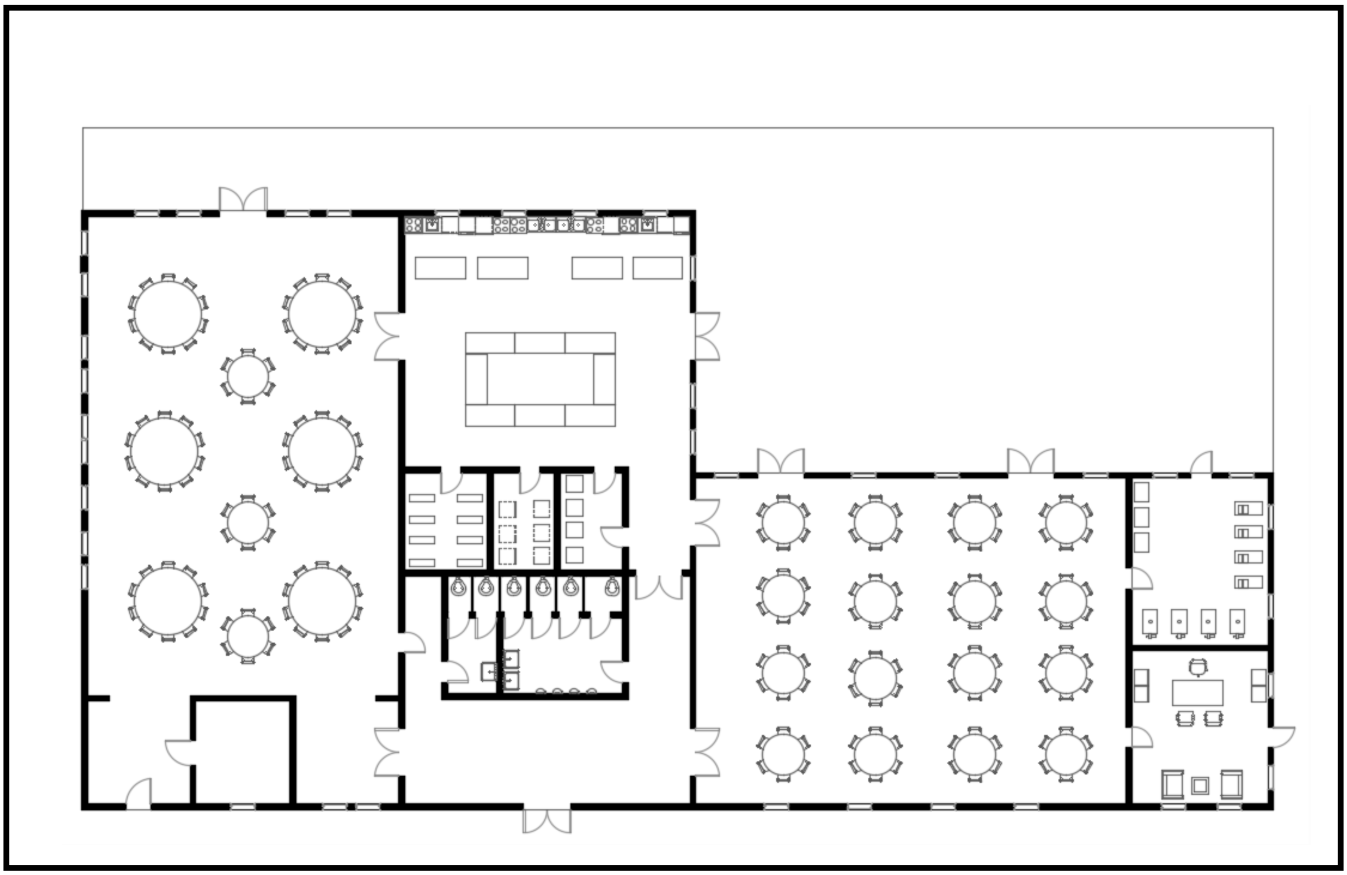


Figure 1 *New East Lodge facility floor plan*

1.0 INTRODUCTION

The Treasure Valley Boy Scout Reservation assistance from a Worcester Polytechnic Institute student to aid them with structural advice. The Reservation is made up of various facilities that house different activities. The facility to be structurally analyzed was chosen to be the East Lodge. The East Lodge houses dining services as well as gathering activities. In order to analyze the building facility, a series of steps were followed. First, various visits were made to the site to record important information such as pictures and measurements. This detailed information was used to make a structural analysis on the building which will be discussed later in this report. After reviewing the structural analysis, recommendations on how to improve the facility were made.

The scope of work involved conducting a structural evaluation on the facility in order to determine the structural stability of the current facility. Using the structural evaluation results, I was able to design a new addition to the building. The new addition addressed the lack of space and building layout problems of the current facility.

2.0 BACKGROUND

2.1 THE MOHEGAN COUNCIL

The Mohegan Council serves and supports scouting units in Worcester, Massachusetts. Their mission is to prepare the young community to make ethical decisions by teaching them the values of the Scout Oath and Law. They rely on more than a thousand volunteers in order to fulfill their mission. One of the Mohegan's Council facilities, is the Treasure Valley Boy Scout Reservation. They provide scout camps over the year to train young people who want to become a scout. Scouting represents an adventure, a challenge, and lessons for a person's lifetime. During the camps, young people enjoy themselves through different activities that reinforce their family values. (Mohegan, n.d.)

2.2 TREASURE VALLEY BOY SCOUT RESERVATION

The Treasure Valley Scout Reservation is located in Massachusetts among the towns of Paxton, Oakham, Rutland, and Spencer. It is made up of 1600 acres of woodland. It provides over 70 miles of hiking and biking trails, houses a variety of ecosystems and wildlife, and offers a nocturnal view of the solar system. The Treasure Valley Scout Reservation is divided into two main locations: the Treasure Valley East and the Treasure Valley West. (Mohegan, n.d.)

The Treasure Valley East is located on the east side of the Browning Pond. It is the home to the Boy Scout Resident Camp. It has several program areas such as the Carr Waterfront, shooting sports ranges, and has solar powered campsites provided by Worcester Polytechnic Institute. (Mohegan, n.d.)

The Treasure Valley West is located on the west side of the Browning Pond. Its program areas include shooting sports ranges, miles of trails, and it caters the Cub Scouts. (Mohegan, n.d.)

The Treasure Valley Scout Reservation offers camp programs during the summer. It is open during the rest of the year for several Scouting Events. Their buildings and summer campsites are available for rent for different events and non-Scouting groups. (Mohegan, n.d.)

2.3 FACILITIES

The Treasure Valley Boy Scout Reservation is made up of different facilities which house different activities. (Mohegan, n.d.)

Table 1 *Treasure Valley Scout Reservation facilities*

Venture Lodge	• Located on the West Side of the camp • Fully heated with running water • Contains two conference rooms and two fully equipped bathrooms • Contains rooms that hold up to 28 people for housing
Council Ring	• Located below the East Lodge, overlooking the Browning Pond • It is an outdoor amphitheater with a fire ring

East Lodge	• Holds 200 people • Serves as a dining hall and gathering space • Contains a kitchen, running water, and bathrooms
Probus	• Located near the Magee Visitor Center • Holds 10 people for housing • Contains electricity, an indoor wood burning stove, and an outdoor fire pit
West Lodge	• Located near the Venture Hall • Fully heated with running water and a kitchen • Upstairs, it contains a hall, two bathrooms, one office, and two bunks • Downstairs, it contains bunks that hold 22 people for housing
East Lodge (Trading Post)	• Located near the Pine Point and Browning Pond • Contains two rooms, electricity, and an indoor wood burning stove • Located near pit latrines and running water

Project Site

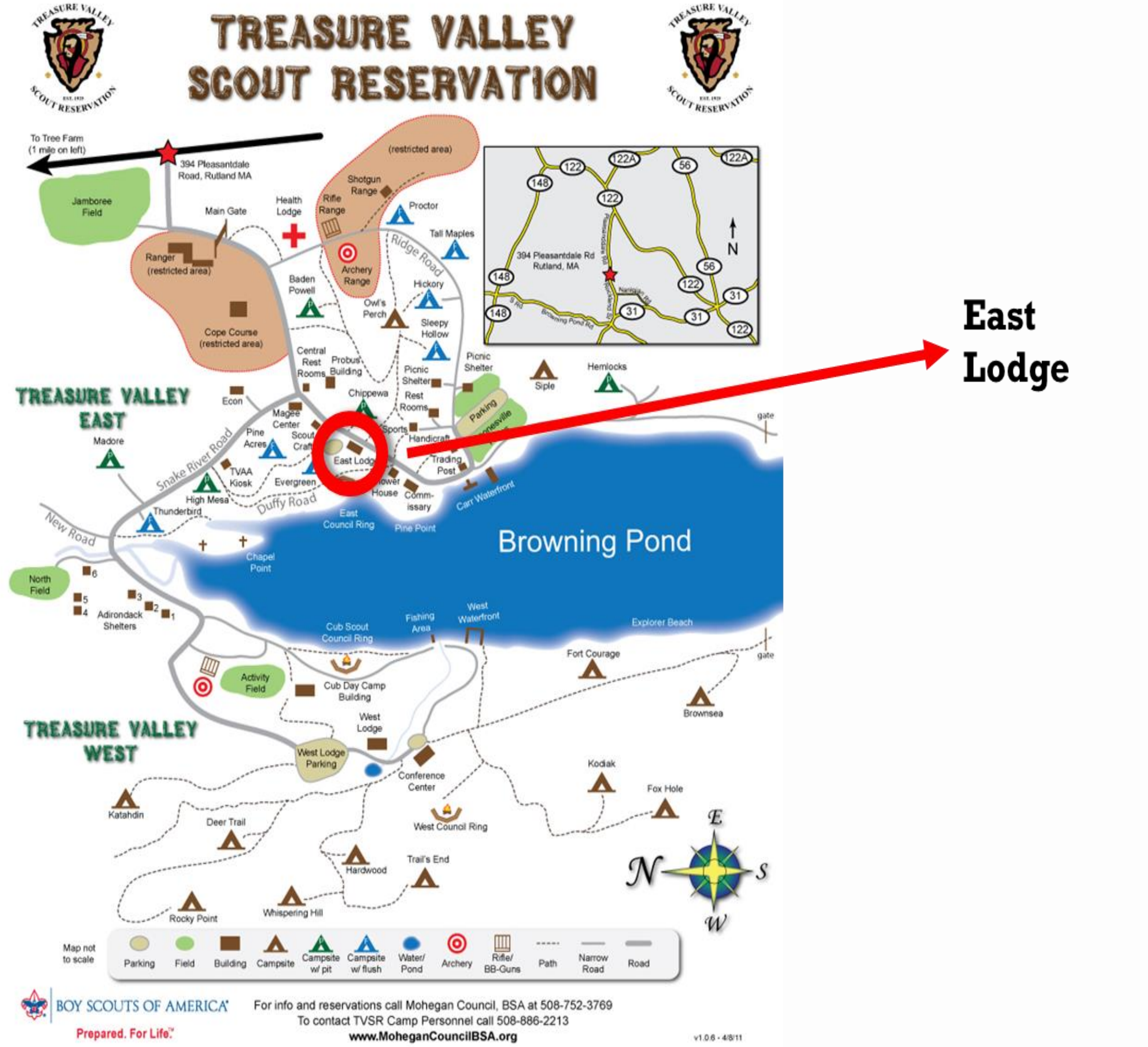


Figure 2 *Treasure Valley Scout Reservation Map*

Figure 3 *East Lodge*

Figure 4 *East Lodge*

Figure 5 *East Lodge*

Figure 6 *East Lodge*

Figure 7 *East Lodge*

Figure 8 *East Lodge*

3.0 STRUCTURAL ANALYSIS

A structural analysis was conducted for the East Lodge facility. The structural analysis evaluated the main components of the facility. The components were the truss, floor, walls, roof, and the columns. Building codes such as the International Building Code and the NDS for Wood Construction were used in order to perform calculations for the structural analysis. Calculations were performed to determine the following: loads acting on the structure, forces acting on the structure, stresses, deflections, and buckling.

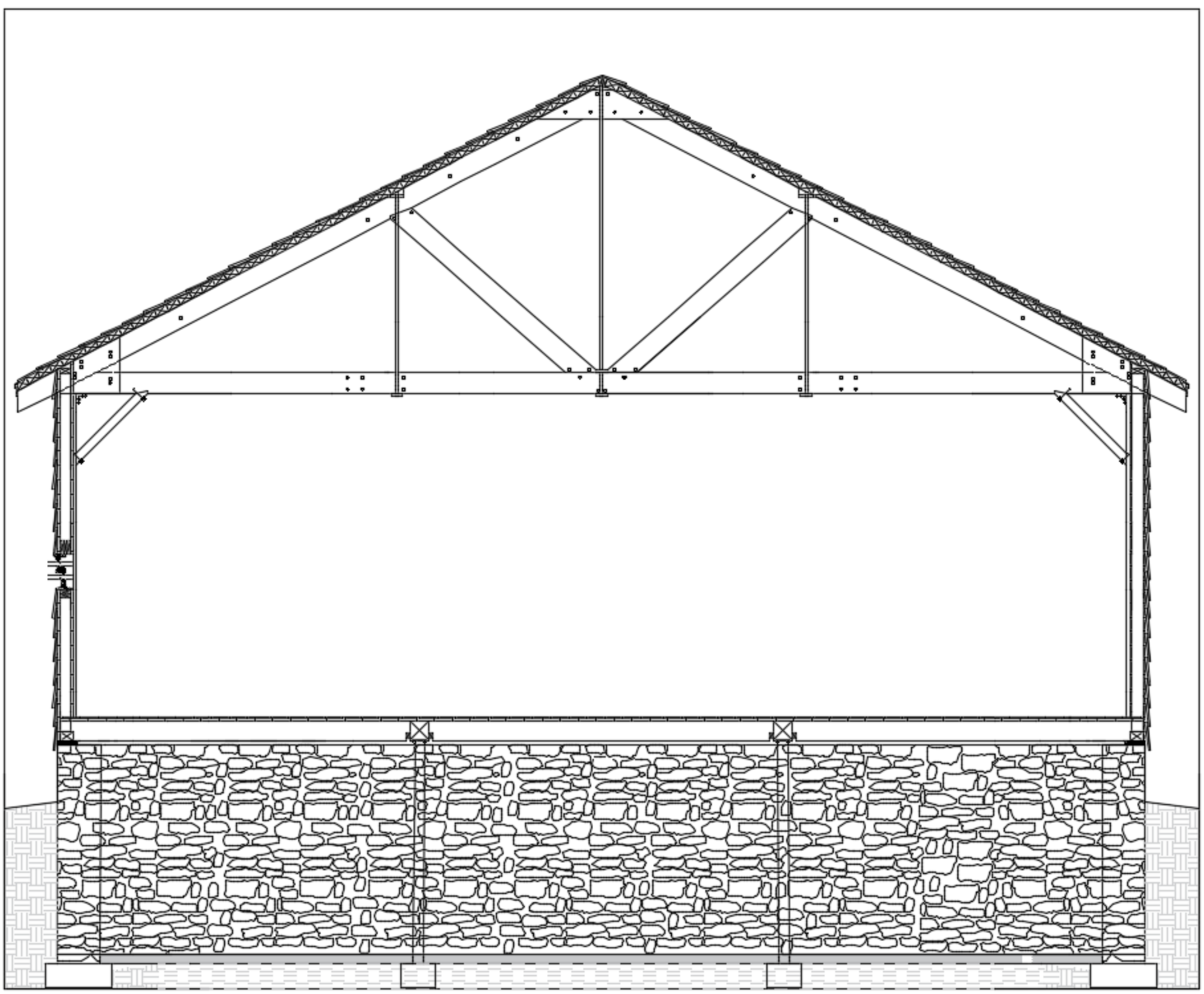


Figure 9 *East Lodge Section*

3.1 TRUSS

3.1.1 INTRODUCTION

Trusses are used in many civil and architectural engineering applications. These structures can be found in roofs, bridges, and steel buildings.

A truss is made up of various straight members that are linked to each other at the two ends of each of the members, and the members are linked together through the use of pin joints. For design purposes, it is assumed that the pin joints can’t carry or resist moments. The external loads that act on the truss are assumed to have an effect only at the joints. Therefore, for practical design purposes, all truss members are assumed to be subject to axial compression or tension forces only. (Noori, n.d.)

It is important to know how much load is being carried by each of the truss members. This is a crucial step when it comes to designing and building a new truss structure or when analyzing an existing truss. In both of these scenarios, the purpose is to know whether the members can support the forces or not. (Noori, n.d.)

There are different methods that can be used to analyze the forces acting on a truss structure, such as the Method of Joints or the Method of Sections. These methods assume that if the structure is in equilibrium, then the members of the structure will also be in equilibrium. (Noori, n.d.). There are also different graphical methods that can be used to analyze trusses, such as the Maxwell’s or Cremona Diagram methods. These graphical method are developed by the forces acting on each joint, which are drawn to scale for each joint. Then, the forces in each of the members are measured. (Daily, n.d.)

For this MQP, the Method of Sections was used. The Method of Sections consists of drawing an imaginary line through the truss, thus separating it into different sections. If the whole truss structure is in equilibrium, then each of the imaginary sections must also be in equilibrium. The following equations of equilibrium are used:

$$\sum F_x = 0 \qquad \sum F_y = 0 \qquad \sum M = 0$$

Using the method of sections, the following procedure of analysis has to be followed. First, the reaction forces at the supports of the truss have to be determined. Then, the imaginary cuts through the truss have to be made in order to draw the free-body diagrams for each cut section. After the free body diagrams are developed, the three equations of equilibrium are applied. (CE, n.d.)

3.1.2 TRUSS ANALYSIS

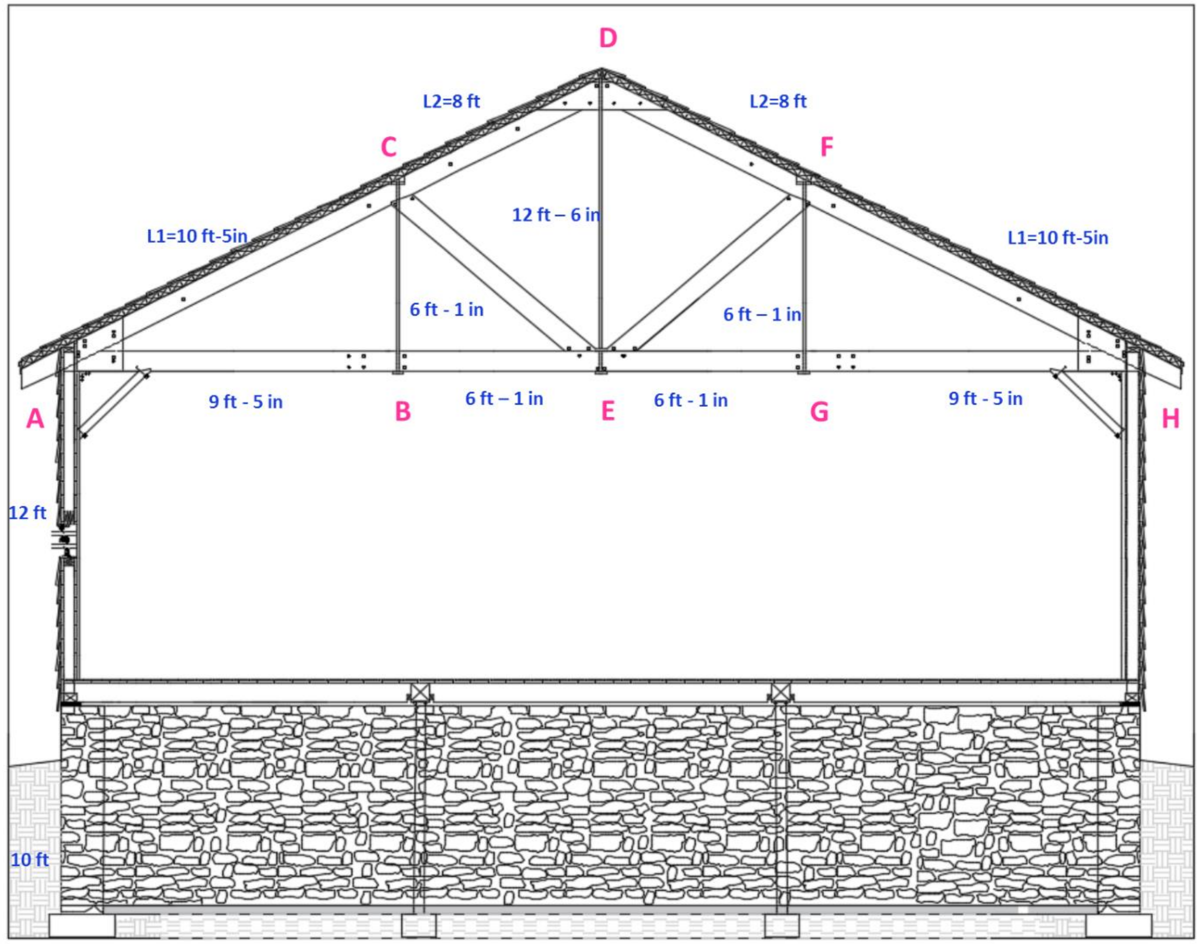


Figure 10 *Truss Section with measurements*

Figure 11 *Truss structure*

Figure 12 *Truss structure*

Figure 13 *Truss structure*

Figure 14 *Truss structure*

Figure: Truss structure

Figure 15 *Truss structure*

For the structural analysis of the truss, first I determined the loads acting on it. According to Chapter 16 Structural Design from the International Building Code, the following loads need to be taken into account based on the building's location: wind, snow, dead, and live loads; seismic loads were not taken into consideration. I first calculated each load individually and then I added them together. In order to calculate the loads acting on each of the truss nodes, first, I needed to determine the tributary areas for each node of the truss. I determined the tributary areas in both linear feet and squared feet.

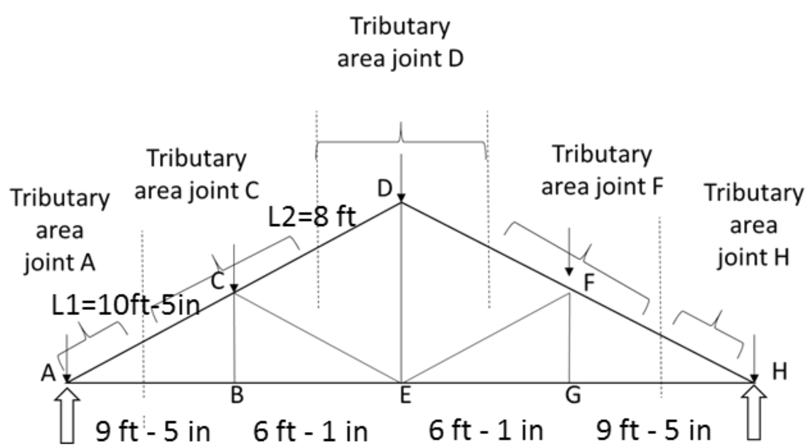


Figure 16 *Tributary areas for truss nodes*

<u>Node A (= node H)</u>

$(L_1 / 2) = (10.417/2) = 5.2085 \text{ ft} \times 12 \text{ ft} = 65.502 \text{ ft}^2$

<u>Node C (= node F)</u>

$[(L_1 / 2) + (L_2 / 2)] = (10.417/2) + (8/2) = 9.2085$ ft x 12 ft = 110.502 ft^2

Node D

$(L_2/2) + (L_2/2) = (8/2) + (8/2) = 8$ft x 12 ft = 96 ft^2

After calculating the tributary area of the truss, the values for dead, live, snow, and wind loads were determined.

3.1.3 DEAD LOAD

First, I calculated the dead load by using the information provided by the ASCE section 1603 on Dead Loads. The shingles of the roof were 2 pounds with a 5 psf planking. I added these two numbers to get a total of a 7 psf dead load on the roof.

3.1.4 LIVE LOAD

I determined the live load acting on the roof to be a 20 psf based on the Table 4-1 from the ASCE.

3.1.5 SNOW LOAD

I referred to the ASCE chapter 7 to define the following factors: the roof slope factor C_s, and the flat roof snow load P_f. These values where needed in order to calculate the snow load for a sloped roof P_s using the following equation:

$$P_s = C_S \times P_f$$

From ASCE Chapter 7.4.2: thermal factor $C_t = 1.1$ for cold roof, so $C_s = 1$

From ASCE Chapter 7.3.1: terrain C category so exposure factor C_e=1

From ASCE Chapter 7.3.3: importance factor of C_I=1

From ASCE Chapter 7.2: ground snow load p_g= 50 psf

From ASCE Chapter 7.3: equation 7.1 minimum value for low sloped roof $p_f = 0.7C_eC_tIp_g$

Substituting determined values for the above equation:

$$p_f = 0.7C_eC_tIp_g = 0.7 \times 1 \times 1.1 \times 1 \times 50 = 38.5 \text{ psf}$$

Then, I calculated the snow load for a sloped roof P_s using the following equation:

$$P_s = C_S \times P_f = 1 \times 38.5 \text{ psf} = 38.5 \text{ psf}$$

The total of 38.5 psf snow load was determined.

3.1.6 WIND LOAD

Mean roof height of 12 ft < 16 ft (least horizontal dimension)

From ASCE 6.5.10: equation 6-15 velocity pressure coefficient $q_z = 0.00256k_zk_{zt}k_dV^2I$

From ASCE 6.5.6: table 6-3 velocity pressure exposure coefficient $k_z = 0.85$

From ASCE 6.5.7.2: table 6-1 topographical factor $k_{zt} = 1$

From ASCE 6.5.4.4: wind directionality factor $k_d = 0.85$

From ASCE 6.5.11.1: figure 6-5 basic wind speed V = 110 mph

From ASCE 6.5.11.2.1: figure 6-6 category I = 1

Substituting the determined values in the following equation:

$q_z = 0.00256k_zk_{zt}k_dV^2I = 0.00256 \times 0.85 \times 1 \times 0.85 \times 110^2 \times 1 = 23$ psf

From ASCE 6.5.11.1: internal pressure coefficient G = 0.85 rigid foundation system

From ASCE 6.5.11.2: external pressure coefficient C_p = 0.2 for windward side and -0.6 for leeward side

From ASCE 7.6.17: design pressure $p = q_z \times G \times C_p$

Substituting the determined values in the following equation:

$p = q_z \times G \times C_p$ = 23 psf x 0.85 x 0.2 = 4 psf windward side

$p = q_z \times G \times C_p$ = 23 psf x 0.85 x -0.6 = -11.8 psf leeward side

Table 2 *Loads acting on the truss*

Type of Load	
Dead Load	7 psf
Live Load	20 psf
Snow Load	38.5 psf
Wind Load	4 psf from the west side & 11.8 psf from the east side (assumed a wind load of 12 psf for design calculations)

After the dead, live, snow, and wind load values were found, I calculated the total loads in pounds per feet. I did this by multiplying the length between two trusses by each load respectively. First, I multiplied the value of the dead load, 7 psf, by the length between two trusses, 12ft; to get a final result of 84 pounds per linear feet. Secondly, I multiplied the value of the snow load, 38.5 psf , times the length between two trusses, 12ft; to get a final result of 462 pounds per linear feet. Then, I multiplied the value of the live load, 20psf, by the length between two trusses, 12ft; to get a final result of 240 pounds per linear feet. Finally, I multiplied the value of the wind loads. 4 psf and 11.8 psf, by the length between two trusses, 12ft; to get a final results of 48 pounds per feet and 142 pounds per linear feet, respectively. For calculation purposes, I assumed a wind uniform load of 12 psf. This assumption was taken into consideration for the structural analysis of a symmetrical truss. The value of 12 psf would take into account the largest wind load that acted on the truss. So 12 psf multiplied by the length between two trusses, 12ft; resulted in 144 lb/ft.

After determining each of the loads acting on the truss, I added their values together to get a total load of **930 lb/ft = R_F (Resultant Load Force).**

Dead load 84 lb/ft + Live load 240 lb/ft + Snow load 462 lb/ft + Wind Load 144 lb/ft = **930 lb/ft**

Below is the truss with the labeled points and measurements that I used to determine the loads acting on each point of the truss.

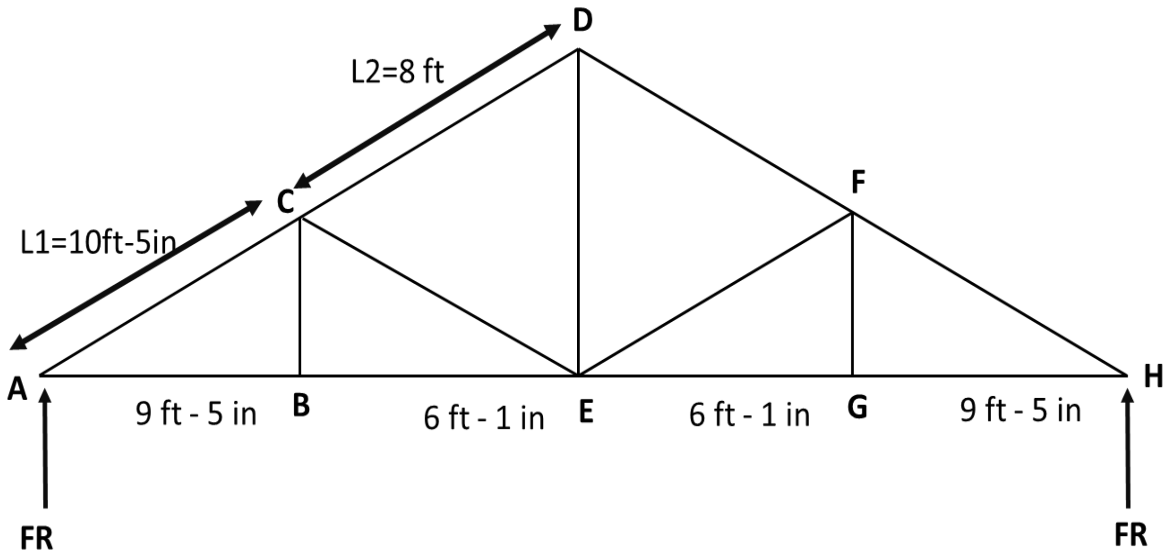


Figure 17 *Points on truss*

Then, I calculated the force acting on each point of the truss. Since the truss is symmetrical, I deducted that the force acting on points F and C to be equal; also the force acting on points A and H to be equal. I used the following equations to determine these forces:

$\mathbf{F_F = F_C}$ = R_F x [(L_1 / 2) + (L_2 / 2)] = 930 lb/ft x [(10.417 ft / 2) + (8 ft / 2)] = **8563.905 lb**

$\mathbf{F_D}$ = R_F x [(L_2 / 2) + (L_2 / 2)] = 930 lb/ft x 8 ft = **7440 lb**

$\mathbf{F_A = F_H}$ = R_F x (L_1 /2) = 930 lb/ft x (10.417 ft /2) = **4843.905 lb**

$\mathbf{F_R}$ = [F_A + F_D + F_F + F_C + F_H] / 2 =

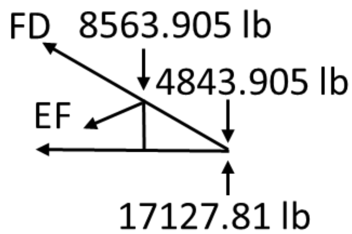
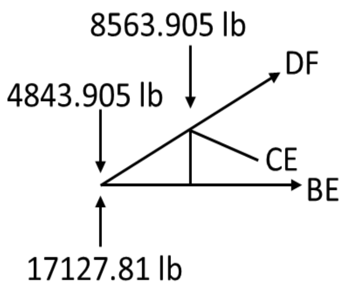
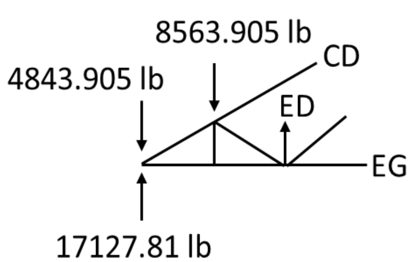
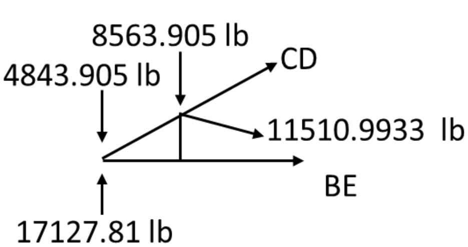
[4843.905 lb + 7440 lb + 8563.905 lb + 8563.905 lb + 4843.905 lb] / 2 = **17127.81 lb**

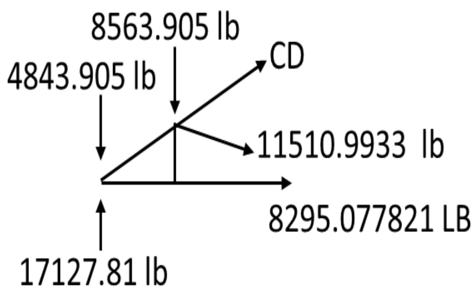
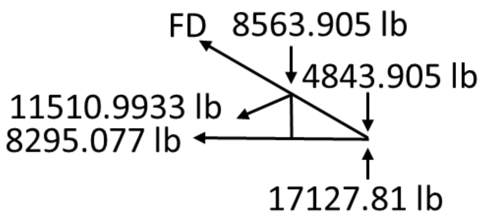
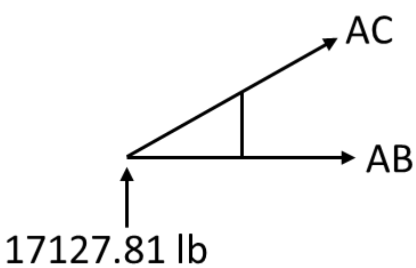
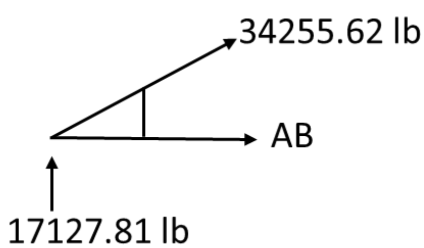
After determining the loads acting on each point on the truss, I used the Method of Sections to determine the forces of each truss member. The following equations of equilibrium were used:

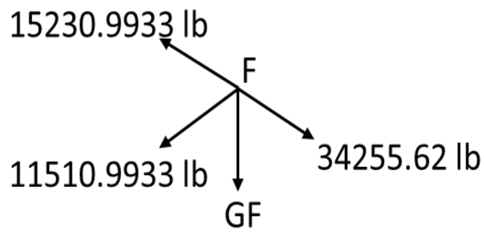
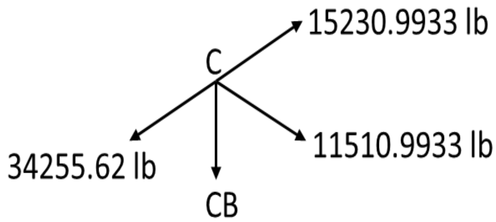
$\sum F_X = 0 \qquad \sum F_Y = 0 \qquad \sum M = 0$

Below is the table showing my results for each force acting on the sections of the truss.

Table 3 *Forces acting on the truss members calculated using Method of Sections*

Section	**Member**	**Force** (T) tension (C) compression
	EF	-11510.9933 lb (C)
	CE	-11510.9933 lb (C)
	ED	11510.99334 lb (T)
	BE	8295.077821 lb (T)
	GE	8295.077821 lb (T)

	CD	-15230.9933 lb (C)
	DF	-15230.9933 lb (C)
	AC	-34255.62 lb (C)
	HF	-34255.62 lb (C)
	AB	29666.23714 lb (T)
	HG	29666.23714 lb (T)

	GF	36073.73383 lb (T)
	CB	36073.73383 lb (T)

3.1.7 MEMBER STRESS

For the stress calculations, I used the stress formula σ = F/A.

F = member force

A = area of the member

For the area of the member, a reduction of 2/3 was applied to all members because only 2/3 of each member is continuous. This is shown in the section below.

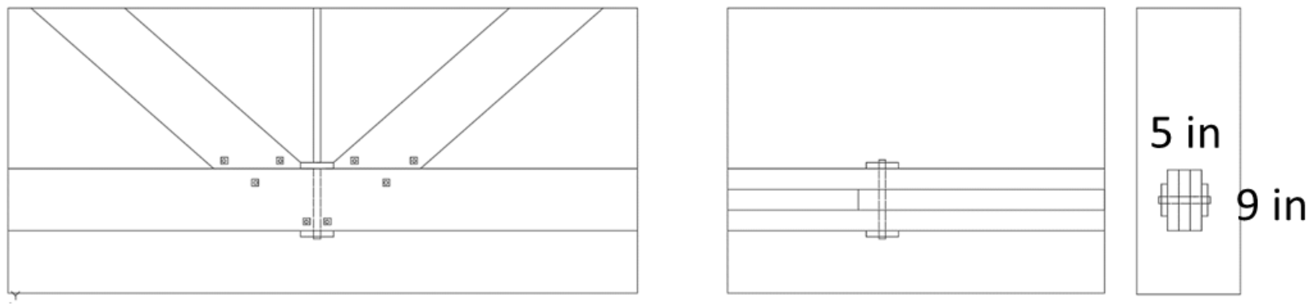


Figure 18 *Section of truss members*

Figure 19 *Truss structure*

Table 4 *Stress calculations*

Member	Force (lb)		Base (in)	Height(in)	Area (in^2)	2/3 Area reduction (in^2)	Stress (psi)	Stress [cross section area reduction] (psi)	Material
EF	-11511	compresion	5	9	45	30	-255.799851	-383.6997767	wood
CE	-11511	compresion	5	9	45	30	-255.799851	-383.6997767	wood
BE	8295.08	tension	5	9	45	30	184.3350627	276.502594	wood
GE	8295.08	tension	5	9	45	30	184.3350627	276.502594	wood
CD	-15231	compresion	5	9	45	30	-338.466518	-507.6997767	wood
DF	-15231	compresion	5	9	45	30	-338.466518	-507.6997767	wood
AC	-34256	compresion	5	9	45	30	-761.236	-1141.854	wood
HF	-34256	compresion	5	9	45	30	-761.236	-1141.854	wood
AB	29666.2	tension	5	9	45	30	659.2497142	988.8745713	wood
HG	29666.2	tension	5	9	45	30	659.2497142	988.8745713	wood
ED	11511	tension			0.7853982		14656.25187		steel
GF	36073.7	tension			0.7853982		45930.50444		steel
CB	36073.7	tension			0.7853982		45930.50444		steel

3.1.8 TRUSS ALLOWABLE STRESS CALCULATIONS

3.1.8.1 TRUSS WOOD MEMBERS

The National Design Specification (NDS) establishes the structural design standards for wood. Among the established specifications, we find the specification for Allowable Stress design. Values for wood strength are used to calculate the allowable strength and are modified with adjustment factors.

For the allowable stress calculation, I made use of the NDS for Wood Design Table 13.5.4A to determine the allowable stress for:

- tension parallel to the grain F_t
- compression parallel to the grain F_c

Table 5 *Representative Tabulated Design Values for Laminated Veneer Lumber*

TABLE 13.5.4A Representative Tabulated Design Values for Laminated Veneer Lumber[1]

		Design Values in Pounds Per Square Inch (psi)[3]							
					Compression Perpendicular to Grain $F_{c\perp}$		Horizontal Shear F_v		
					Load Direction		Load Direction		
Species	Grade	Extreme Fiber in Bending F_b	Tension Parallel to Grain[2] F_t	Compression Parallel to Grain[2] F_c	Parallel to glueline	Perpendicular to glueline	Parallel to glueline	Perpendicular to glueline	Modulus of Elasticity E
Douglas-Fir	2.0E	2800	1750	2725	750	480	285	175	2,000,000
Southern Pine	2.0E	2925	1805	3035	880	525	285	150	2,000,000

From the table above, I assumed the species of Douglas-Fir, grade 2.0. Then I extracted the values of:

- tension parallel to the grain F_t = 1750 psi
- compression parallel to the grain F_c = 2725 psi

3.1.8.2 TRUSS STEEL MEMBERS

The ultimate tensile strength is the maximum stress level reached in a tension test. The strength of a material is its ability to withstand external forces without breaking. (NDT, n.d.)

The truss members ED, GF, and CB are steel members under tension. In order to calculate the allowable stress for these members, I used the table below to determine the ultimate tensile strength. For calculation purposes, I chose the structural ASTM A36 steel, since it is the most common structural steel in the US. For the ultimate tensile strength, I chose the lowest value of 400 MPa.

400 MPa = 58.015 ksi = 58015 psi

Table 6 *Properties of steel*

Typical tensile strengths of some materials

Material	Yield strength (MPa)	Ultimate tensile strength (MPa)	Density (g/cm³)
Steel, structural ASTM A36 steel	250	400-550	7.8
Steel, 1090 mild	247	841	7.58
Human skin	15	20	2
Steel, 2800 Maraging steel[7]	2617	2693	8.00
Steel, AerMet 340[8]	2160	2430	7.86
Steel, Sandvik Sanicro 36Mo logging cable precision wire[9]	1758	2070	8.00
Steel, AISI 4130, water quenched 855 °C (1570 °F), 480 °C (900 °F) temper[10]	951	1110	7.85
Steel, API 5L X65[11]	448	531	7.8
Steel, high strength alloy ASTM A514	690	760	7.8

3.1.9 BUCKLING

Buckling occurs when a structural member is subjected to compressive forces. The Euler Buckling Load, $F = n\pi^2 EI/L^2$, is used to calculate the load at which a compression member buckles.

Euler buckling load $F = n\pi^2 EI/L^2$

n = fixed on both points

E = modulus of elasticity (lb/in^2)

L = length of member (inches)

I = moment of inertia (in^4)

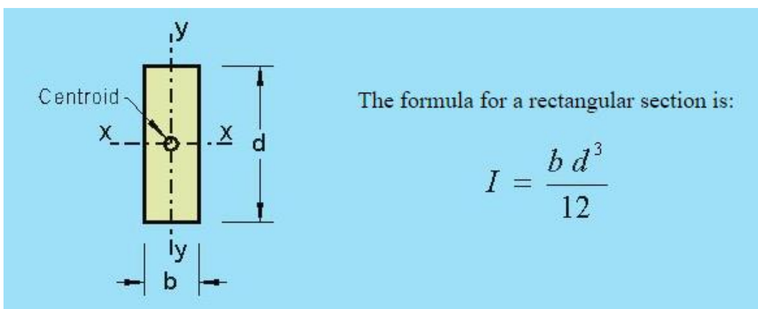


Figure 20 *Moment of Inertia*

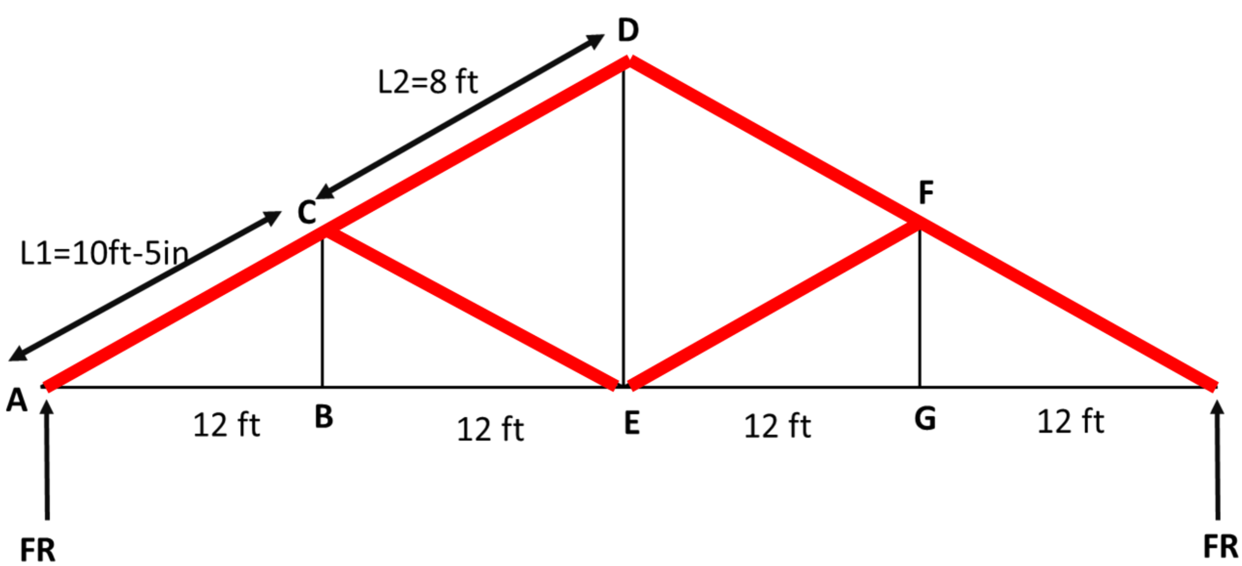


Figure 21 *Members under compression*

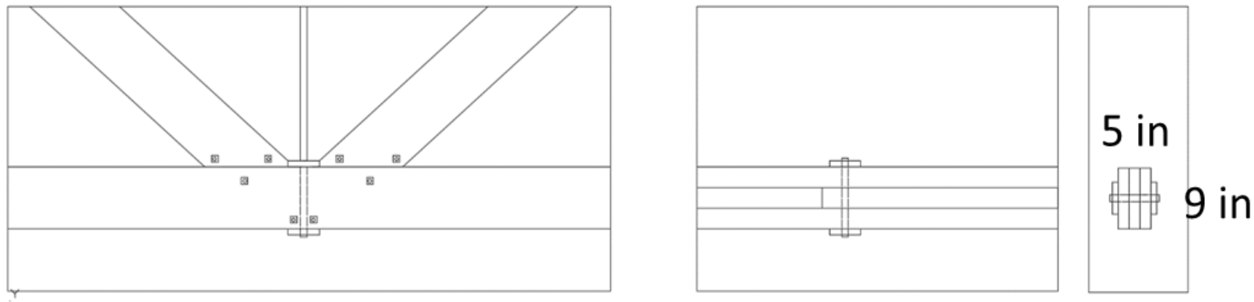


Figure 22 Truss section where b = 5 in and d = 9 in for I = $\frac{bd^3}{12}$

Table 7 *Euler Buckling Load calculations*

Member	n	E lb/in^2	Length in	Moment of Inertia in^4	F lb
EF	1	1900000	104.04	303.75	526221.9
CE	1	1900000	104.04	303.75	526221.9
CD	1	1900000	96	303.75	618055.1
DF	1	1900000	96	303.75	618055.1
AC	1	1900000	125.004	303.75	364520.4
HF	1	1900000	125.004	303.75	364520.4

Table 8 *Moment of inertia calculations*

Member	Base in	Height in	Length ft	Length in	Moment of Inertia in^4
EF	5	9	8.67	104.04	303.75
CE	5	9	8.67	104.04	303.75
ED	5	9	12.5	150	303.75
BE	5	9	6.083	72.996	303.75
GE	5	9	6.083	72.996	303.75
CD	5	9	8	96	303.75
DF	5	9	8	96	303.75
AC	5	9	10.417	125.004	303.75
HF	5	9	10.417	125.004	303.75
AB	5	9	9.417	113.004	303.75
HG	5	9	9.417	113.004	303.75
GF	5	9	6.083	72.996	303.75
CB	5	9	6.083	72.996	303.75

3.2 FLOOR

3.2.1 INTRODUCTION

The structural analysis of floors is less complicated than the analysis for trusses. The structural analysis for floors shows if the floor components are able to support the loading acting over them daily. (US, 2005)

After determining the span length for the floor beam, the load distribution factors have to be determined. The AASHTO provides with certain specifications and guidelines for determining this load distribution factor; which depend on the type of deck system that spans between the floor beams. (US, 2005)

For timber structures, the NDS (National Design Specifications) allows to neglect any concentrated loads within the beam depth of the beam supports. This loads are neglected because they are assumed to go through the beam supports without causing shear stresses. The NDS also specifies the handling of the undersides of floor beams; since this can reduce the capacity of the structural members. The use of bolts, side plates, and the use of a larger floor beam can be used to reduce the high capacity of shear force stresses. Failures due to shear in floor beams is not common, since the shear capacities is lower than the bending capacities in floor beams. (US, 2005)

3.2.2 FLOOR ANALYSIS

Figure 23 *East Lodge basement*

Figure 24 *Floor column structure*

Figure 25 *Floor beam and floor joists structure*

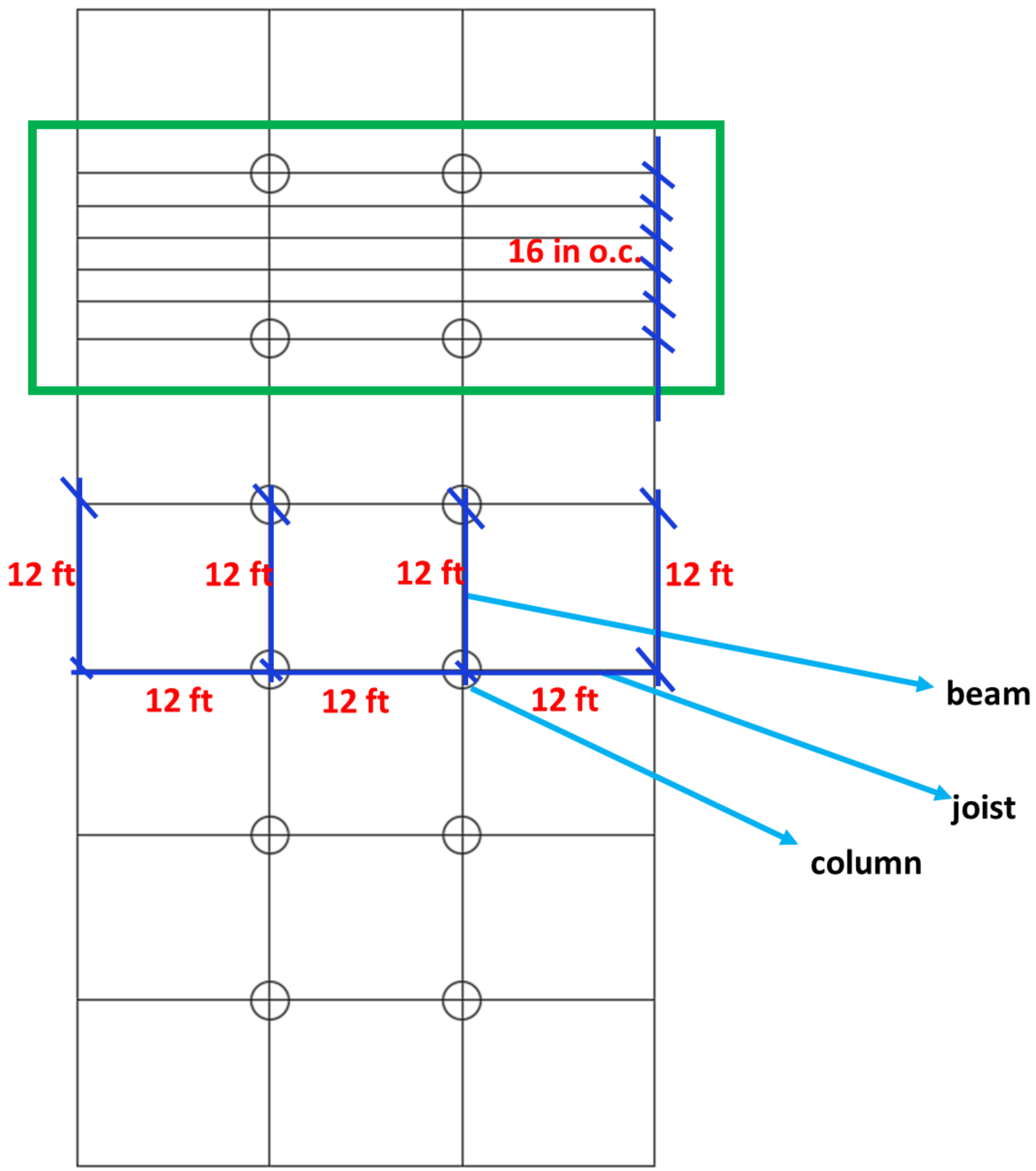


Figure 26 *Joists, Beams, and Columns layout*

3.2.3 FLOOR JOIST

From the Table Minimum Uniformly Distributed Live Loads adapted from SEI/ASCE 7-10: Minimum Design Loads for Buildings and Other Structures:

East Lodge facility building is an Assembly area with movable seats with a uniformly distributed load of 100 psf.

For the analysis of the floor, I started by calculating the tributary area of one floor joist.

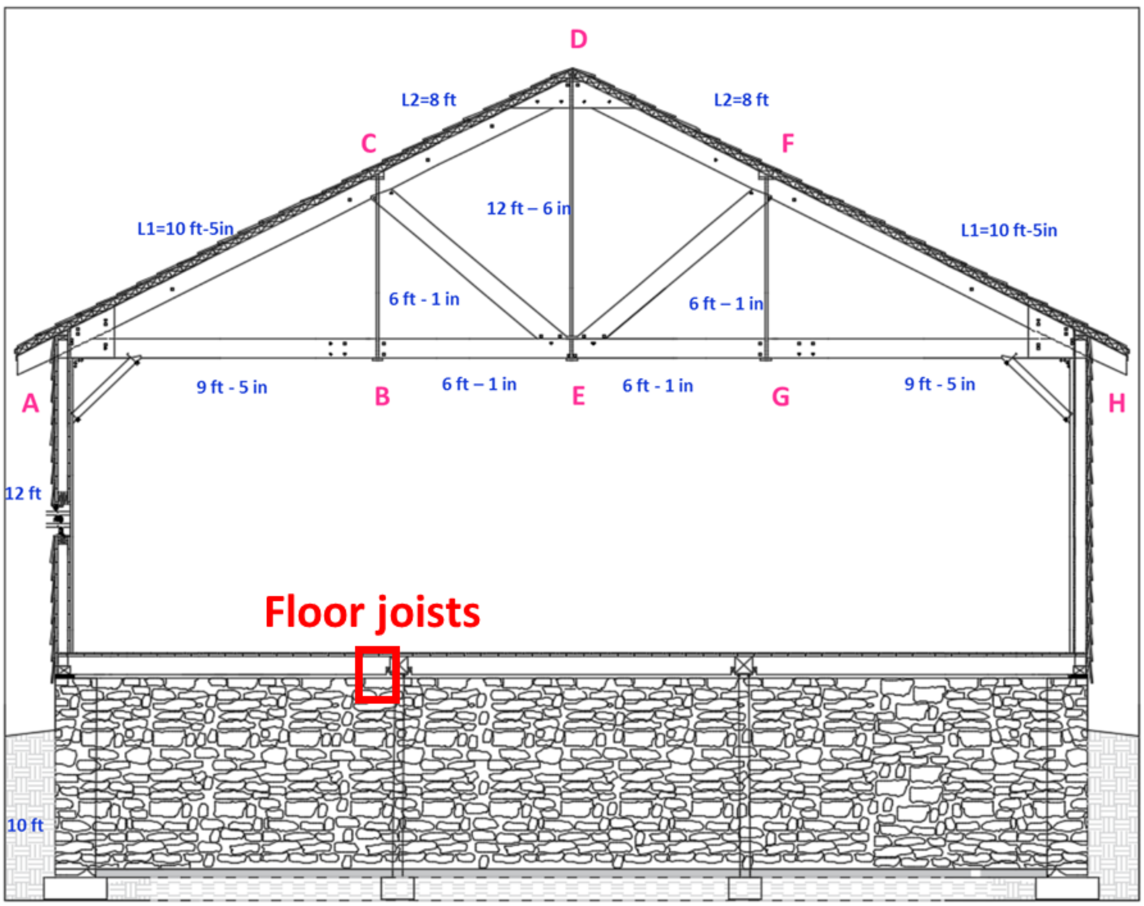


Figure 27 *Floor joists on section*

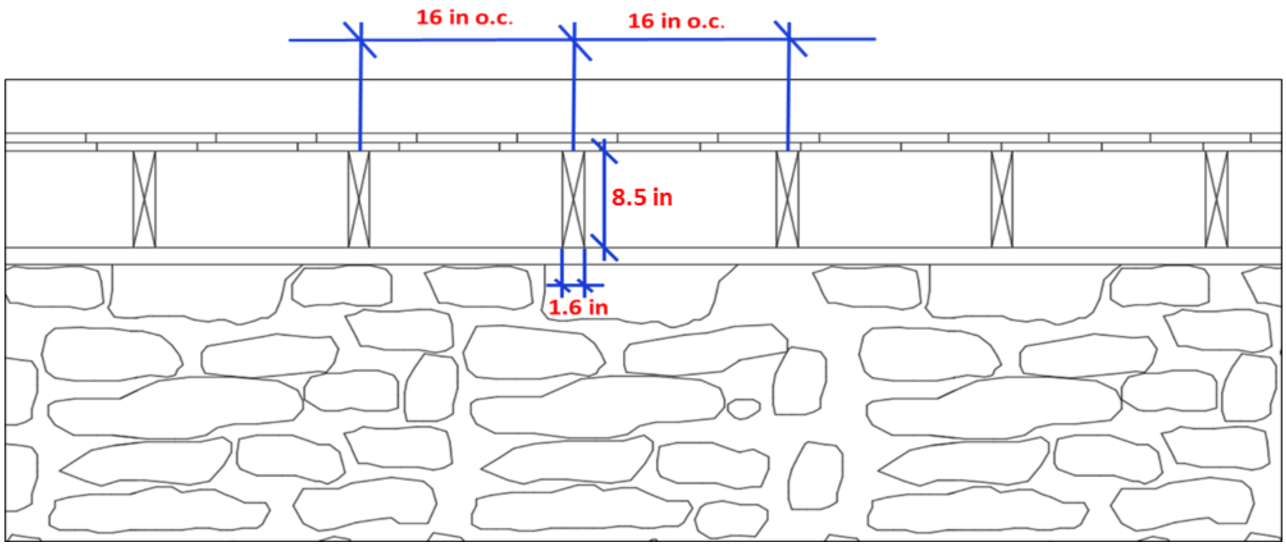


Figure 28 *Spacing between joists*

The floor joist are located 16 inches on center. Therefore each floor joist has a tributary area of:

8 in + 8 in = 16 inches

16 in / 12 ft = 1.33333 ft x 12 ft = 16 ft^2 tributary area per joist

Then, I calculated the total load acting on the joist. I did this by multiplying the tributary area of one floor joist, 16 ft^2, by the load supported by the floor joist, 100 lb/ft^2:

16 ft^2 x 100 lb/ft^2 = 1600 pounds

3.2.4 FLOOR BEAM

From the Table Minimum Uniformly Distributed Live Loads adapted from SEI/ASCE 7-10: Minimum Design Loads for Buildings and Other Structures:

East Lodge facility building is an Assembly area with movable seats with a uniformly distributed load of 100 psf.

Then I calculated the tributary area of one floor beam.

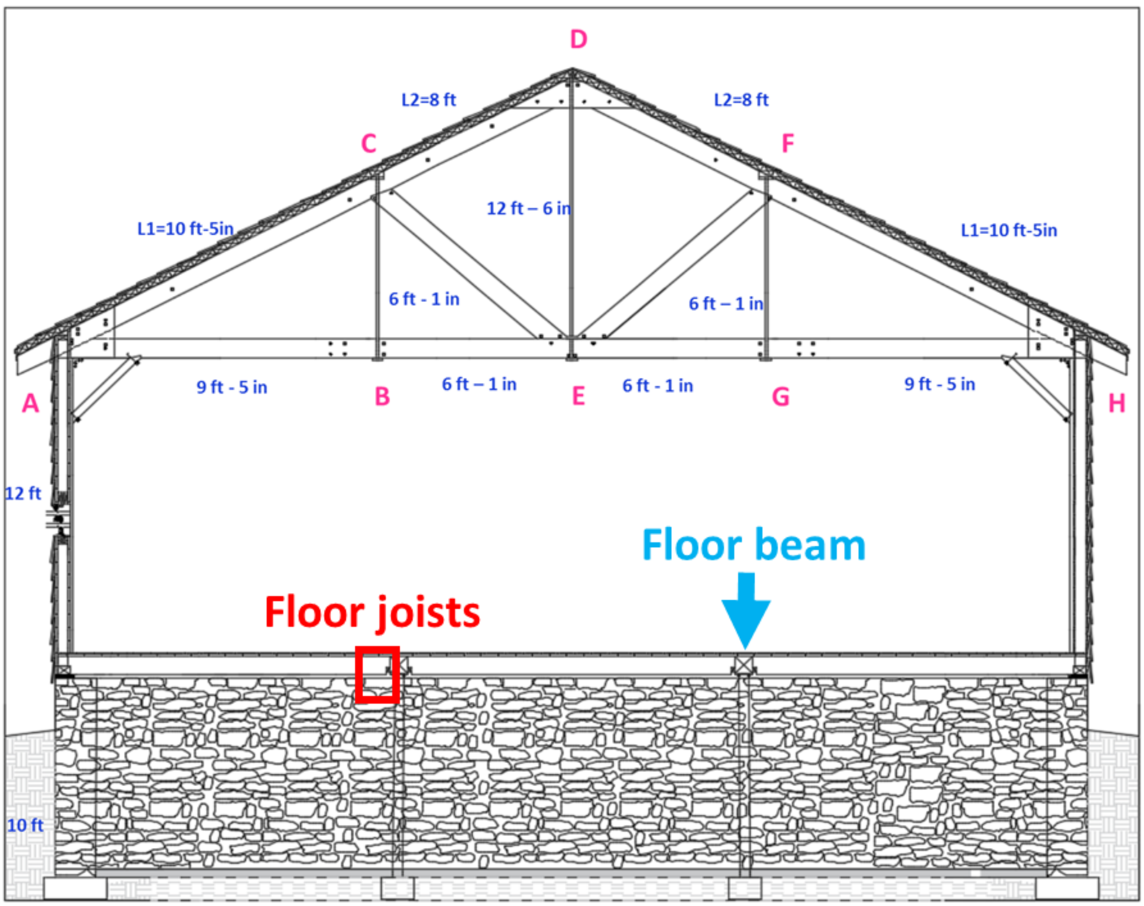


Figure 29 *Floor beam on section*

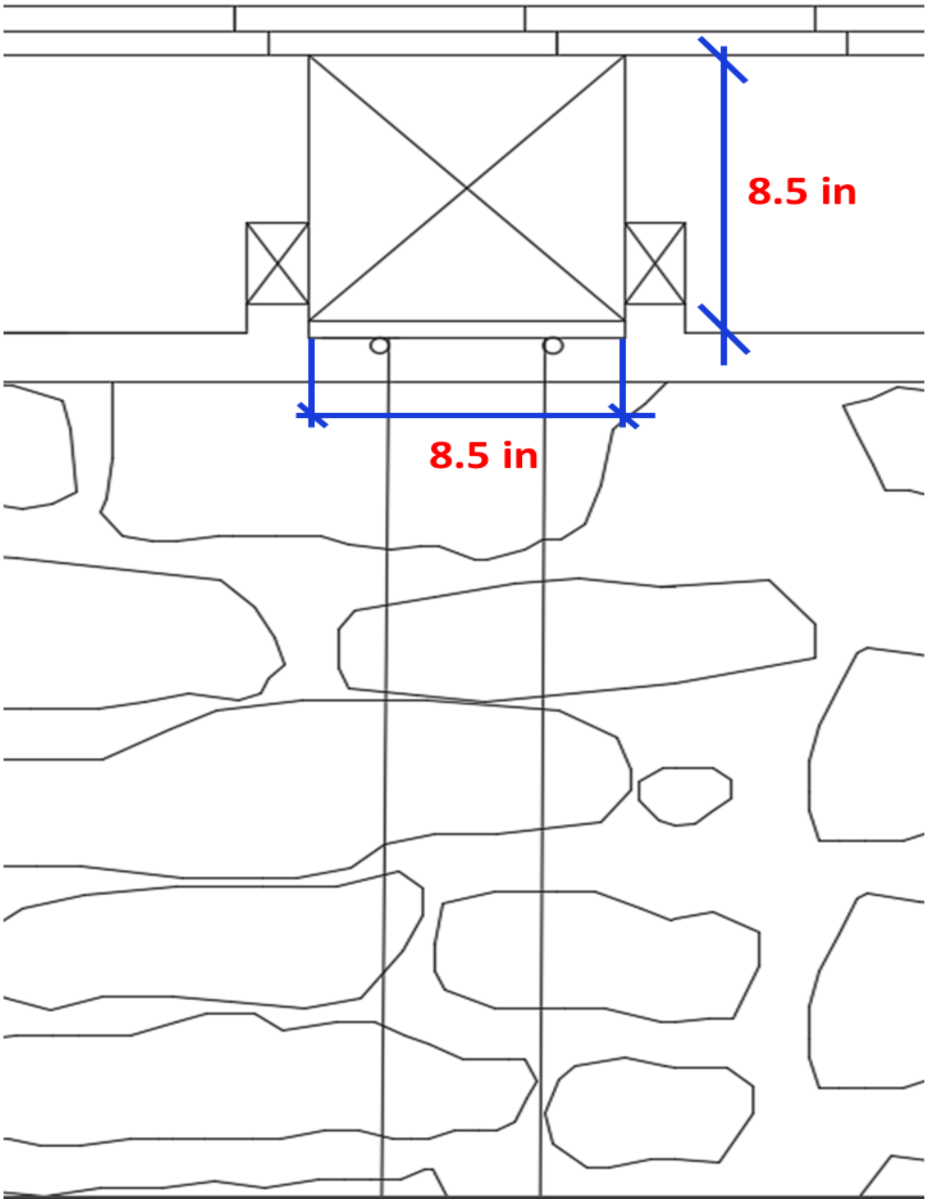


Figure 30 *Beam section*

The floor beams are located 12 feet on center. Therefore each floor beam has a tributary area of:

6 ft + 6 ft = 12 ft

12 ft x 12 ft = 144 ft^2 tributary area

Then, I calculated the total load acting on the beam. I did this by multiplying the tributary area of one floor beam, 144 ft^2, by the load supported by the floor beam, 100 lb/ft^2:

144 ft^2 x 100 lb/ft^2 = 14400 pounds

3.3 ROOF BOARDS

3.3.1 INTRODUCTION

A roof structure' structural stability depends on: appropriate lateral bracing to support the roof framing; appropriate roof framing members' design; and appropriate connections between the top of the wall and the roof structure. (Building, n.d.)

The structure of the roof is made up of internal bracing, roof framing, and roof decking/sheathing. These roof structure transmits the horizontal loads that act on the roof to the walls below. (Building, n.d.)

The roof sheathing supports loads such as snow, live, gravity, and vertical-uplift loads that are created by the wind pressures. Roof sheathing is located in the load path that is between the roof system and the foundation. Along with the roof framing, the roof sheathing transmits the lateral loads to the structure's shear walls. (Building, n.d.)

The roof framing resists the loads that are applied to the sheathing; support roof sheathing and decking; and transmit loads in the vertical direction to support walls. The roof framing is made up rafters made up of lumber or trusses. (Building, n.d.)

3.3.2 ROOF BOARDS ANALYSIS

From section 3.1.2 Truss Analysis in this report, Table 2, the loads acting on the roof are the following:

Dead load 7 psf + live load 20 psf + snow load 38.5 psf + wind load 12 psf = 77.5 psf total load acting on the roof

Table 9 *Loads acting on the truss*

Type of Load	
Dead Load	7 psf
Live Load	20 psf
Snow Load	38.5 psf
Wind Load	4 psf from the west side & 11.8 psf from the east side (assumed a wind load of 12 psf for design calculations)

For the analysis of the roof boards, I started by calculating the tributary area of one roof board.

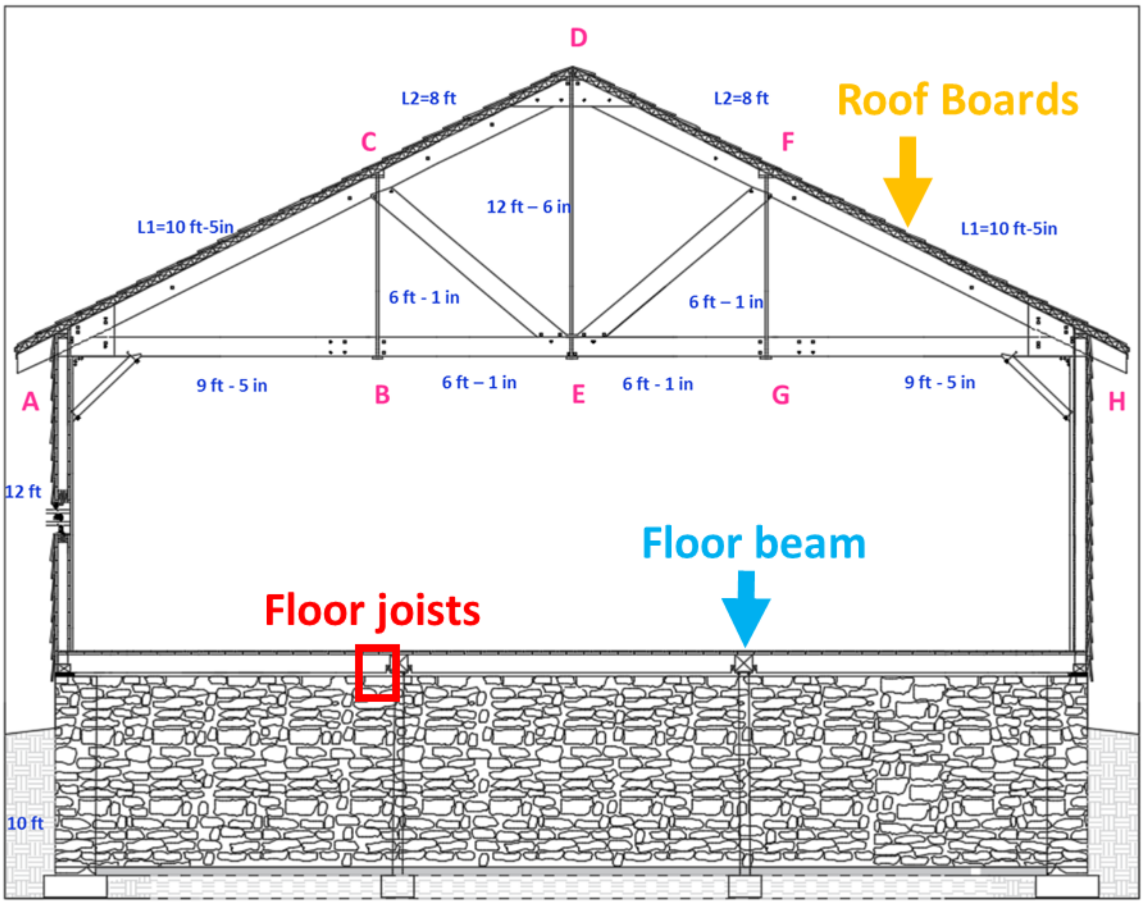


Figure 31 *Roof boards on section*

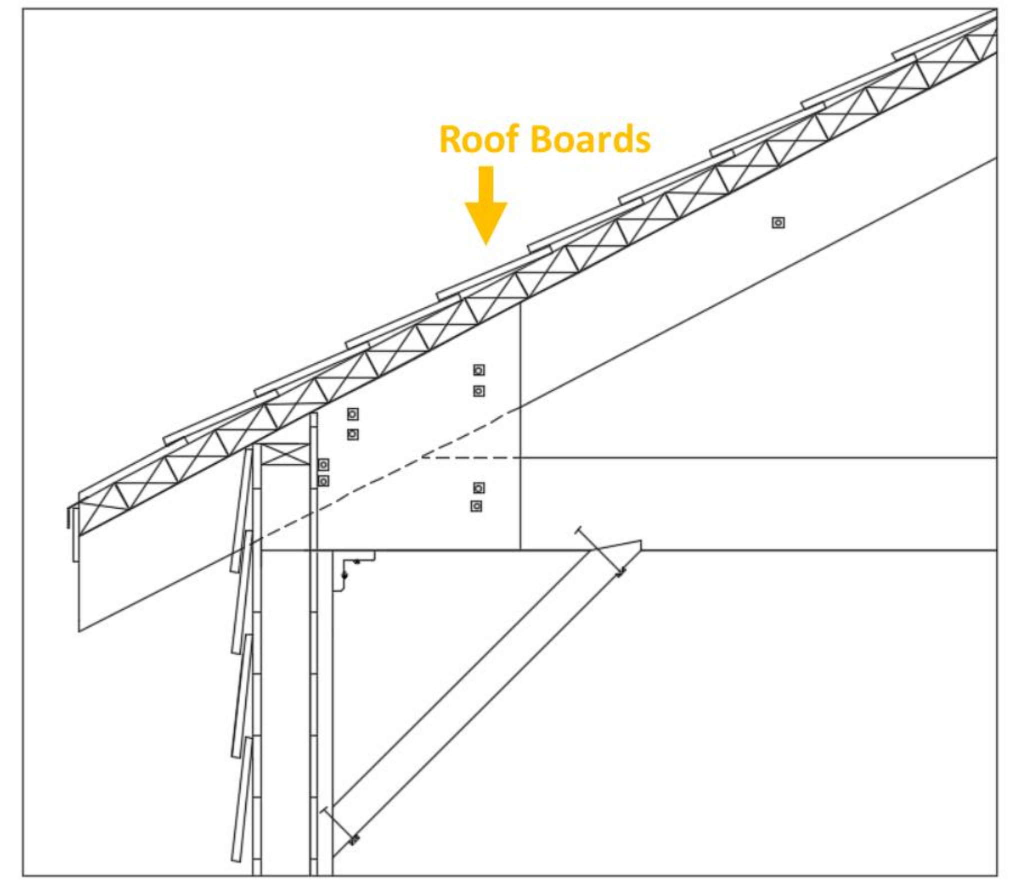


Figure 32 *Roof Boards section*

The roof boards are placed right next to each other. Each roof boards is 3 inches x 5.5 inches.

Therefore each roof board has a tributary area of:

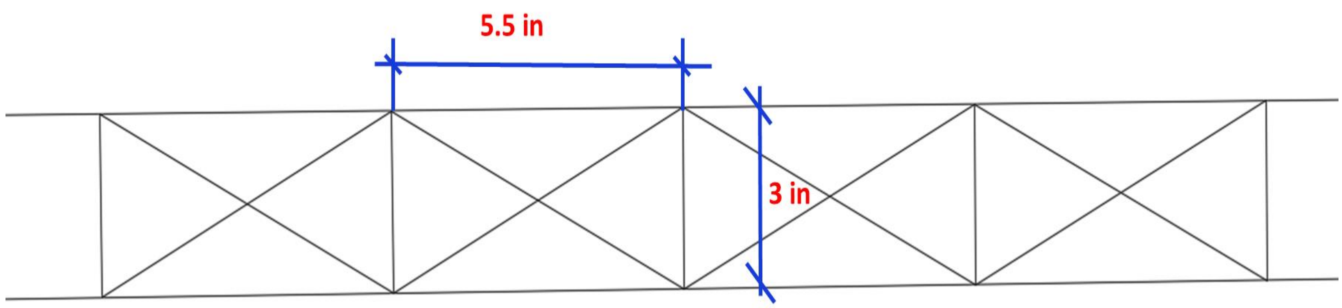


Figure 33 *Tributary area roof boards*

(2.75 in) + (2.75 in) = 5.5 inches

5.5 in / 12 in = 0.46 ft x 12 ft = 5.52 ft^2 tributary area

Then, I calculated the total load acting on a roof board. I did this by multiplying the tributary area of one roof board, 5.52 ft^2, by the load supported by the roof board, 77.5 lb/ft^2:

5.52 ft^2 x 77.5 lb/ft^2 = 427.8 pounds

3.4 BENDING STRESS

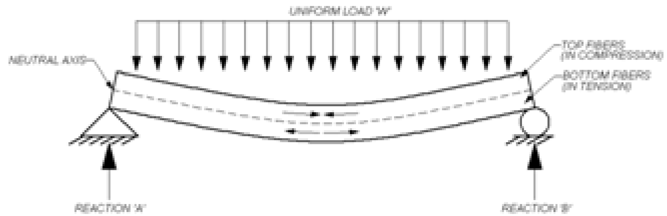


Figure 1-Beam diagram

Figure 34 *Beam undergoing uniform distributed load*

When a member is being loaded similar to that in the figure above, bending stress will occur. When a beam experiences a uniformly distributed load, the top fibers of the beam undergo a compressive stress. The stress at the horizontal plane of the neutral is zero. The bottom fibers of the beam undergo tensile stress. Therefore, the value of the bending stress will change linearly with distance from the neutral axis. (Engineering resources, 2007)

$$\sigma_b = \frac{My}{I}$$

σ_b – Bending stress

M – Calculated bending moment

y – Vertical distance away from the neutral axis

I – Moment of inertia around the neutral axis

Figure 35 *Bending stress equation*

3.4.1 FLOOR JOIST STRESS

In order to calculate the bending stress of the floor joist, I needed to determine the bending moment, moment of inertia around the neutral axis, and the vertical distance away from the neutral axis.

To calculate the bending moment, I used the following equation:

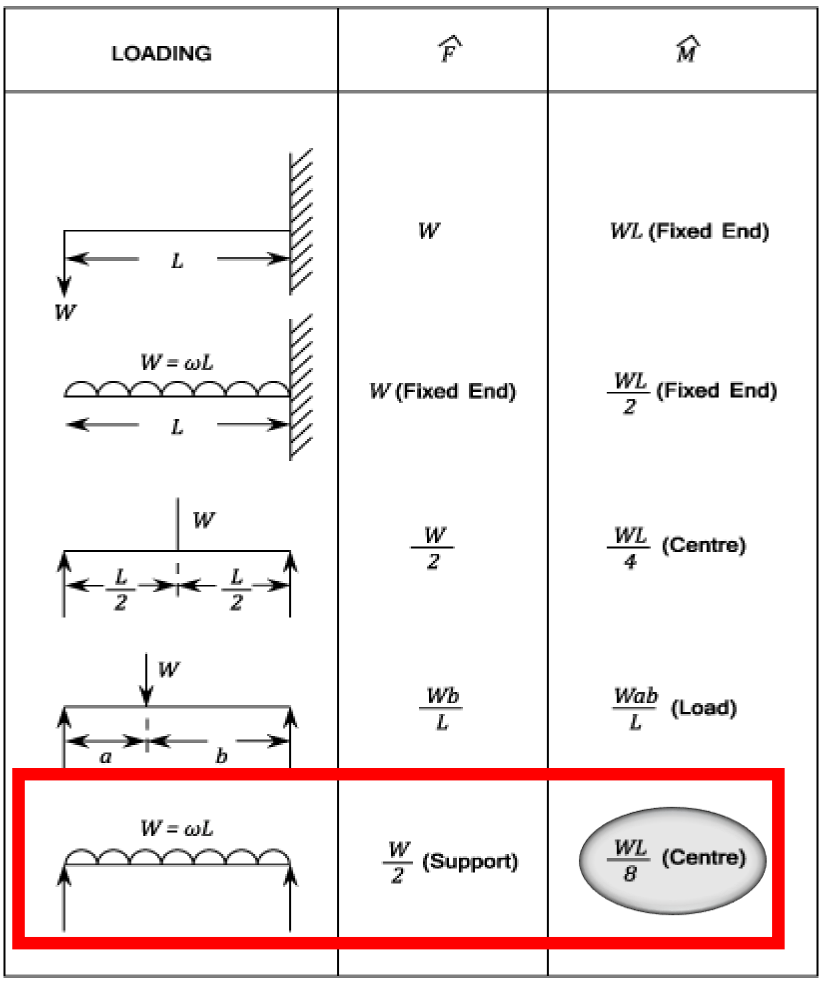


Figure 36 *Bending moment equations*

M = $\frac{WL}{8}$ = [(1600 lb) x (12 ft)] / 8

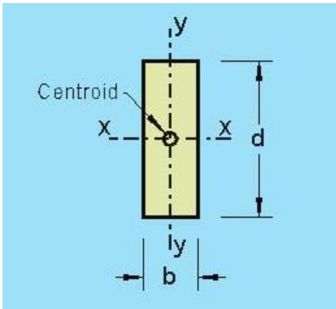
M = 2400 lb ft

Where:

W = load acting on the floor joist = 1600 lb

L = length of the floor joist = 12 ft

To calculate the moment of inertia, I used the following equation:



$$I = \frac{b\,d^3}{12}$$

Figure 37 *Moment of Inertia*

I calculated the moment of inertia around the Y axis of the floor joist.

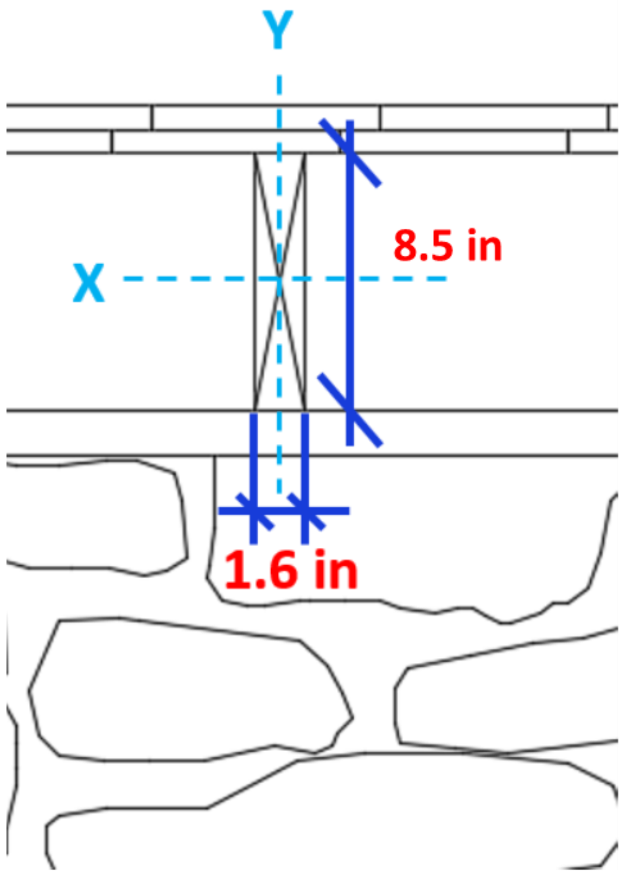


Figure 38 *Moment of inertia for floor joist*

I = $\frac{bd^3}{12}$ = [(1.6 in) $(8.5)^3$)] / 12 = 81.9 in^4

The vertical distance away from the Y axis is 4.25 inches.

Having determined the bending moment, moment of inertia around the Y axis, and the vertical distance away from the Y axis, I applied the bending stress formula:

$\sigma = \frac{My}{I}$ = [(2400 lb ft x 12 in) (4.25 in)] / 81.9 in^4 = 1495 psi

3.4.2 FLOOR BEAM STRESS

For the stress calculation of the floor beam, I followed the same process for calculating the stress in the floor joists.

In order to calculate the bending stress of the floor beams, I needed to determine the bending moment, moment of inertia around the neutral axis, and the vertical distance away from the neutral axis.

To calculate the bending moment, I used the following equation:

M = $\frac{WL}{8}$ = [(14400 lb) x (12 ft)] / 8

M = 21600 lb ft

Where:

W = load acting on the floor beam = 14400 lb

L = length of the floor beam = 12 ft

To calculate the moment of inertia, I used the following equation:

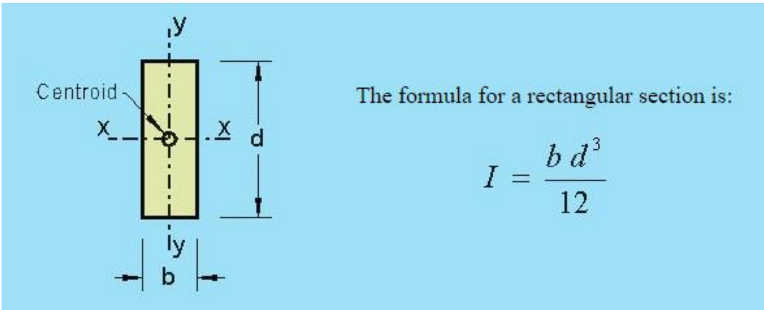


Figure 39 *Moment of Inertia*I = $\frac{bd^3}{12}$

I calculated the moment of inertia around the Y axis of the floor beam.

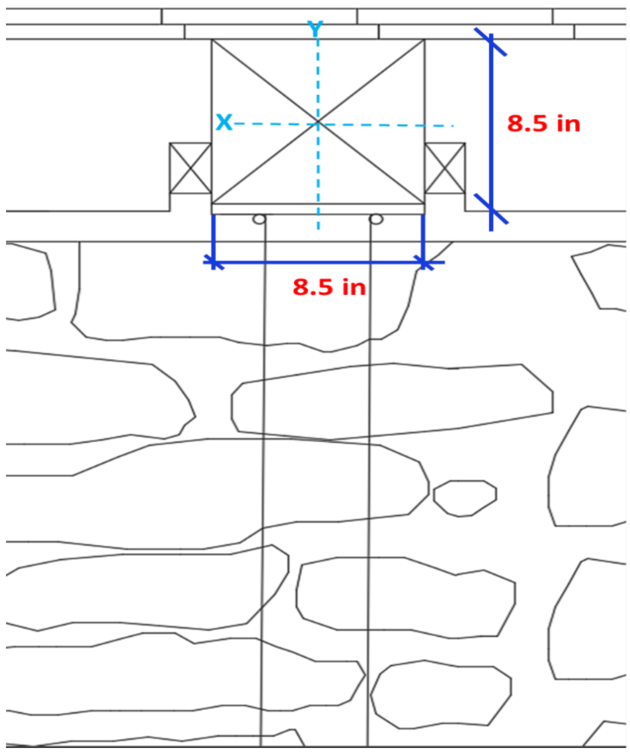


Figure 40 *Moment of inertia for floor beam*

I = $\frac{bd^3}{12}$ = [(8.5 in) $(8.5)^3$)] / 12 = 435 in^4

The vertical distance away from the Y axis is 4.25 inches.

Having determined the bending moment, moment of inertia around the Y axis, and the vertical distance away from the Y axis, I applied the bending stress formula:

$\sigma = \frac{My}{I}$ = [(21600 lb ft x 12 in) (4.25 in)] / 435 in^4 = 2532 psi

3.4.3 ROOF BOARDS STRESS

For the stress calculation of the roof boards, I followed the same process for calculating the stress in the floor joists and floor beams.

In order to calculate the bending stress of the roof boards, I needed to determine the bending moment, moment of inertia around the neutral axis, and the vertical distance away from the neutral axis.

To calculate the bending moment, I used the following equation:

$M = \frac{WL}{8}$ = [(427.8 lb) x 12 ft)] / 8

M = 641.7 lb ft

Where:

W = load acting on the roof board = 427.8 lb

L = length of the roof board = 12 ft

To calculate the moment of inertia, I used the following equation:

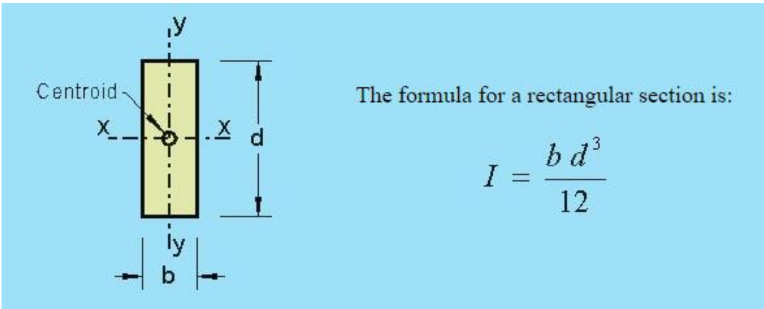


Figure 41 *Moment of Inertia* $I = \frac{bd\hat{}3}{12}$

I calculated the moment of inertia around the Y axis of the roof board.

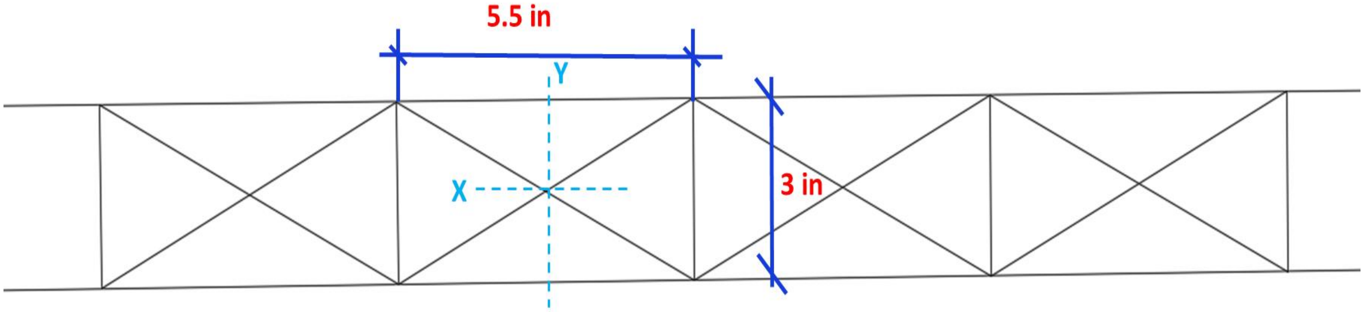


Figure 42 *Moment of inertia for roof board*

I = $\frac{bd\text{^}3}{12}$ = [(5.5 in) (3 in)^3)] / 12 = 12.4 in^4

The vertical distance away from the Y axis is 1.5 inches.

Having determined the bending moment, moment of inertia around the Y axis, and the vertical distance away from the Y axis, I applied the bending stress formula:

σ = $\frac{My}{I}$ = [(641.7 lb ft x 12 in) (1.5 in)] / 12.4 in^4 = 931.5 psi

Table 10 *stress results*

MEMBER	**STRESS**
Floor joist	1495 psi
Floor beam	2532 psi
Roof boards	931.5 psi

3.5 DEFLECTION

3.5.1 INTRODUCTION

The deformation of a beam can be defined in terms of its deflection from its original unloaded positon. (Civil, n.d.)

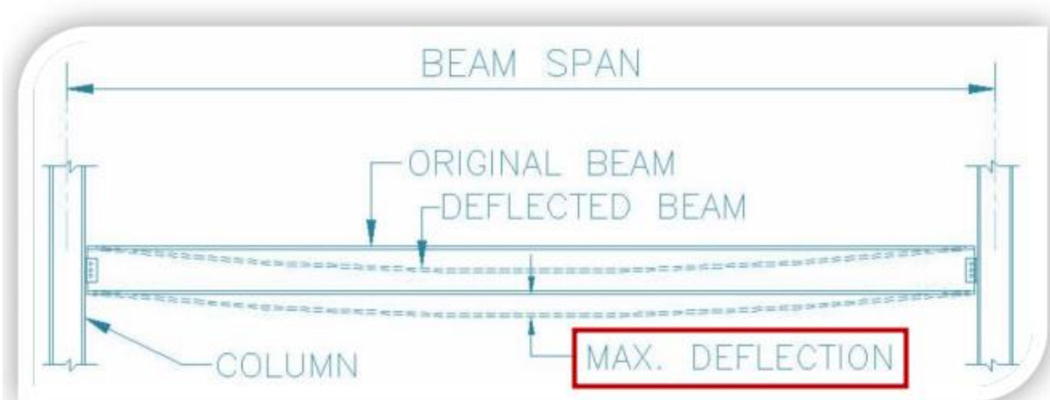


Figure 43 *Maximum deflection*

There are certain factors that are considered when calculating deflections. These factors are: span(L), load(w), beam shape, material properties(E and I) and end fixity(roller, fixed or hinge supports). (Civil, n.d.)

3.5.2 FLOOR AND ROOF BOARDS DEFLECTION

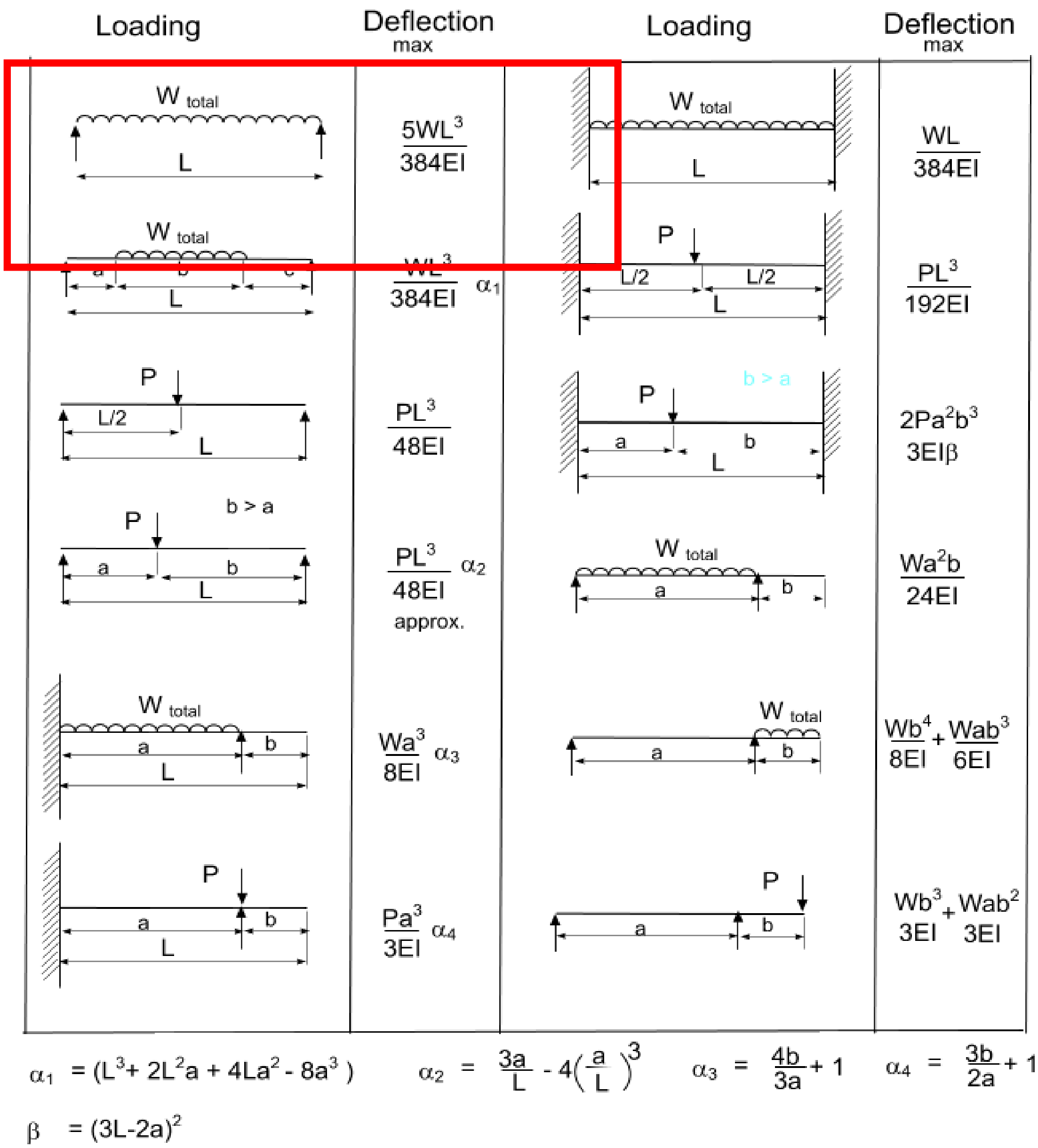


Figure 44 *Maximum deflection equations*

For calculating the deflection of the floor beams, floor joists, and roof boards, I used the highlighted equation in the figure above.

Table 11 *Deflection Calculations*

E	1900000	psi			
Member	W lb	L in	E psi	I in^4	Deflection in
Floor joist	1600	144	1900000	81.9	0.399768652
floor beam	14400	144	1900000	435	0.677401089
roof boards	427.8	144	1900000	12.4	0.705978947

3.6 COLUMNS

3.6.1 INTRODUCTION

Columns are vertical structural members that are located where an axial force acts parallel to the longitudinal axis. These vertical structures can have failures caused by buckling or crushing. Euler buckling load; the load at which buckling occurs in a column; is relevant to the ratio of the unsupported length of the column to its depth. NDS 3.7.1 provides information and specifications regarding the stability and compression of an axial compression member. (Design, n.d.)

There are two types of columns: simple columns and built-up columns. Simple columns are made up of one piece of lumber. Built-up columns are made up of many wood members put together with bolts or nails. (Design, n.d.)

Columns are important elements in structural frames and trusses. They support loads that are compressive from floors, roofs, or decks. Columns transfer the forces acting in the vertical direction to the foundations and ground below. The only loads that are applied on a column are axial loads. The loads on columns produce axial compressive stresses, since these loads are applied at the ends of the structural member. Other loads that act on a column are bending moments and transverse forces. (Mechanics, n.d.)

3.6.2 COLUMNS ANALYSIS

For the structural analysis of the columns, I first calculated the tributary area of the columns. I did this by multiplying half of the width of the roof, 36.25 ft/2, by the 12 ft, which is the total space the column covers (6 ft on each side of the column). The result of the tributary area of the column was 217.5 ft^2.

Then I calculated the load on the column by multiplying the tributary area by the addition of the live and dead loads. So I calculated the load by: 217.5 ft^2 x (20 psf + 7 psf) = 5873 pounds.

For design purposes, from table 5, Axial Compression Load Capacity for 8x8 and 8x10 Columns, from the Western Wood Products Association, I took the species to be Douglas fir larch for a column length of 12 ft. I used this information to determine the column capacity of an 8x8 column, which was 43513 pounds.

3.6.3 FLOOR COLUMNS

The columns located in the floor were not analyzed. I would recommend to conduct a structural analysis on these columns. The analysis would provide more information regarding the stability of the floor structure.

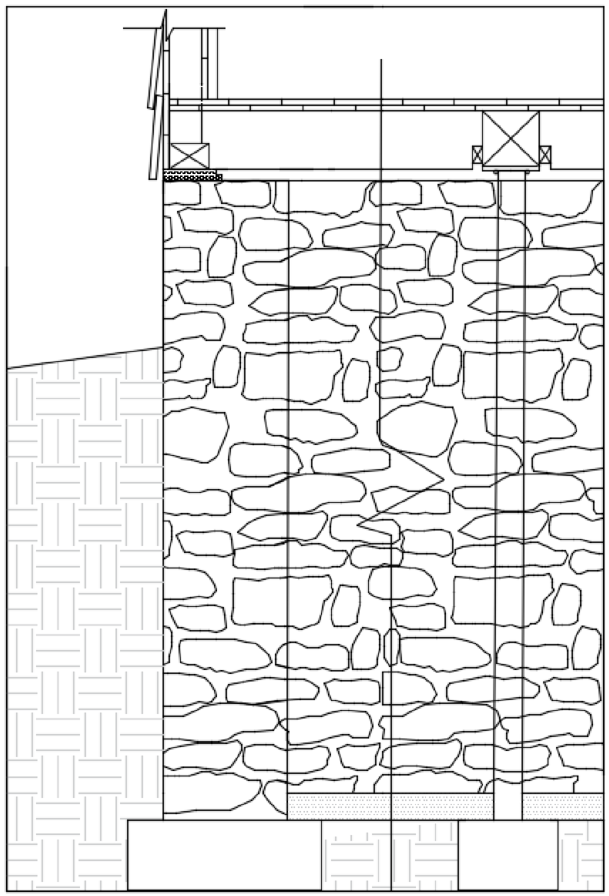


Figure 45 *Floor columns on section*

4.0 STRUCTURAL ANALYSIS CONCLUSION

4.1 TRUSS

For a further analysis of the truss, I would recommend to conduct a structural analysis on the wood joints and steel connections shown below. This would provide more information regarding the stability of the truss.

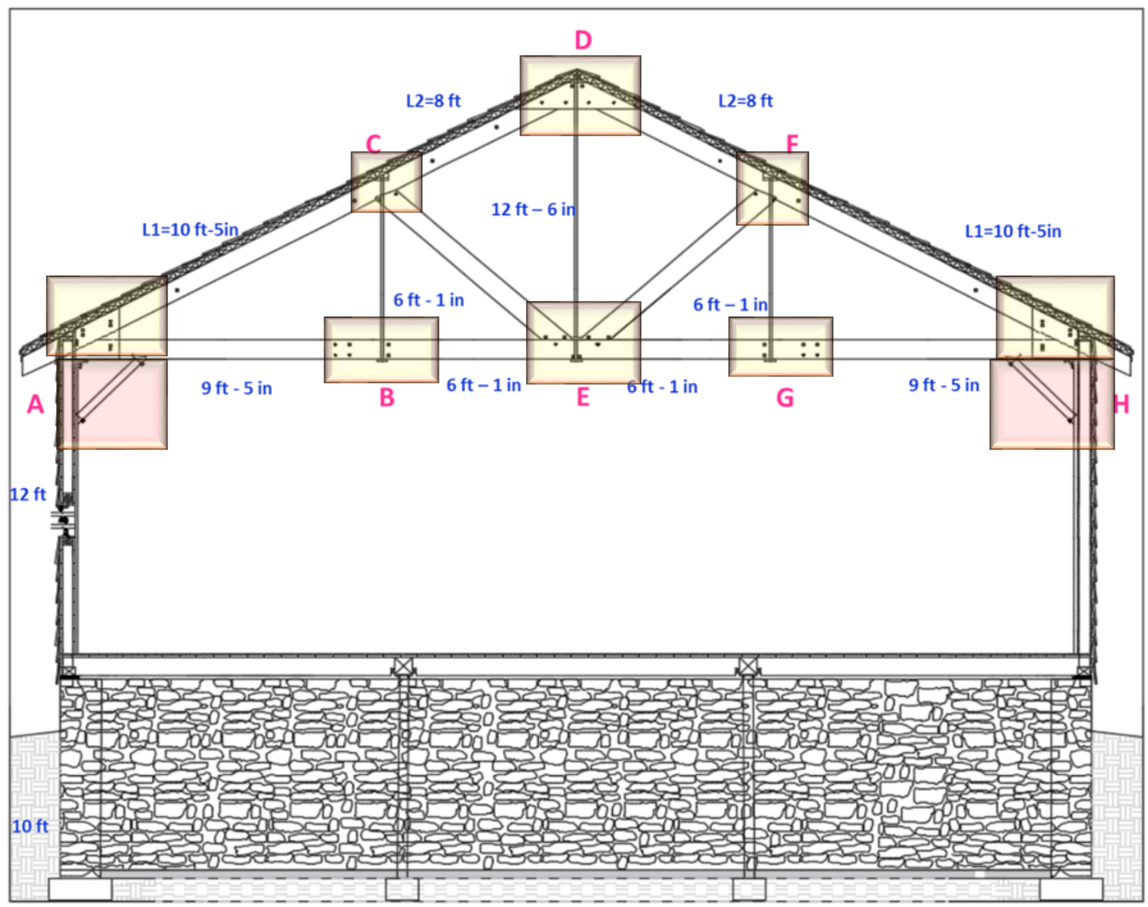


Figure 46 *Truss section with highlighted steel and wood connections*

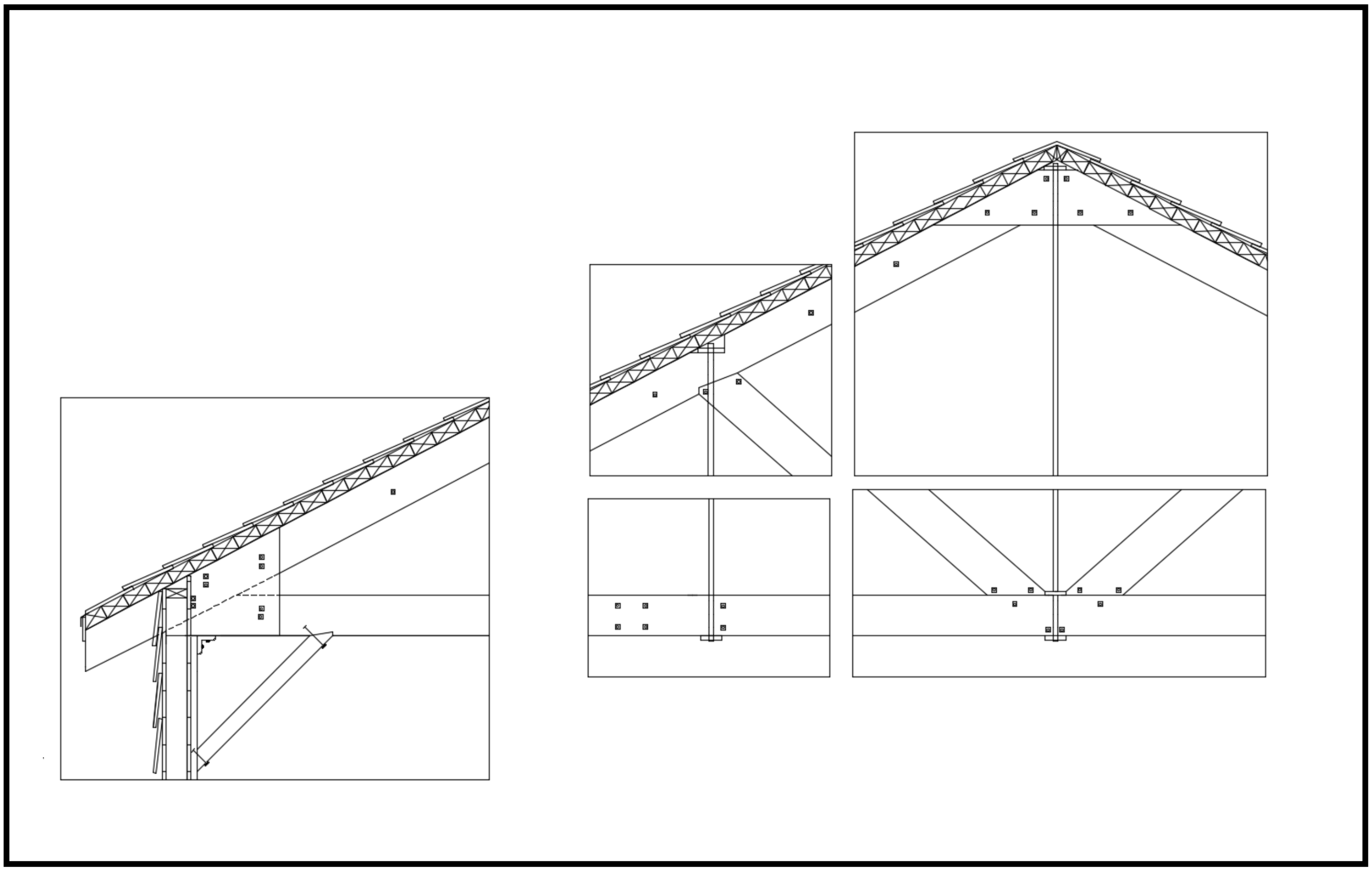


Figure 47 *Steel and Wood connections on section*

4.2 DEFLECTION CONCLUSION

After determining the deflections values for the floor joist, floor beam, and roof boards, I proceeded to determine the deflection limit for each structural member.

From the IBC Chapter 16 Section 1604.3, Table 1604.3 Deflection Limits, I used the following formula.

Table 12 *Deflection Limits*

CONSTRUCTION	*L*	*S* or *W* [f]	*D* + *L* [d, g]
Roof members: [e]			
Supporting plaster or stucco ceiling	*l*/360	*l*/360	*l*/240
Supporting nonplaster ceiling	*l*/240	*l*/240	*l*/180
Not supporting ceiling	*l*/180	*l*/180	*l*/120
Floor members	*l*/360	—	*l*/240
Exterior walls and interior partitions:			
With plaster or stucco finishes	—	*l*/360	—
With other brittle finishes	—	*l*/240	—
With flexible finishes	—	*l*/120	—
Farm buildings	—	—	*l*/180
Greenhouses	—	—	*l*/120

For the roof boards, I used the formula L/180, where L is the length of the roof board:

144 inches / 180 = 0.8 inches

For the floor members, beams and joist, where L is the same length for both members:

144 inches / 360 = 0.4 inches

Table 13 *Deflection calculations*

E	1900000	psi			
Member	W lb	L in	E psi	I in^4	Deflection in
Floor joist	1600	144	1900000	81.9	0.399768652
floor beam	14400	144	1900000	435	0.677401089
roof boards	427.8	144	1900000	12.4	0.705978947

Comparing the deflection limit results with the actual deflection, only the floor beam exceeds the deflection limit.

Deflection limit 0.4 inches < 0.677401089 inches actual deflection

I would recommend to decrease the design load of 100 psf to a load of 50 psf acting on the floor beams and floor joists. The occupancy of the building would remain the same, but the amount of people allowed in the building would need to be restricted. Currently, 200 people are allowed inside the facility. I would recommend to restrict the amount of people inside the facility to 100. By reducing the load, the beam will be below the deflection limit of 0.4 inches. This is shown in the table below:

Table 14 *Deflection calculations with reduced load acting on beams and joists*

E	1900000	psi			
Member	W lb	L in	E psi	I in^4	Deflection in
Floor joist	1600	144	1900000	81.9	0.399768652
floor beam	7200	144	1900000	435	0.338700544
roof boards	427.8	144	1900000	12.4	0.705978947

4.3 STRESS CONCLUSION

Table 15 *Stress results for Wood members*

Member	Force lb		Base in	Height in	Area in^2	2/3 Area reduction in^2	Stress psi	Stress [cross section area reduction] psi
EF	-11511	compresion	5	9	45	30	-255.799851	-383.6997767
CE	-11511	compresion	5	9	45	30	-255.799851	-383.6997767
BE	8295.08	tension	5	9	45	30	184.3350627	276.502594
GE	8295.08	tension	5	9	45	30	184.3350627	276.502594
CD	-15231	compresion	5	9	45	30	-338.466518	-507.6997767
DF	-15231	compresion	5	9	45	30	-338.466518	-507.6997767
AC	-34256	compresion	5	9	45	30	-761.236	-1141.854
HF	-34256	compresion	5	9	45	30	-761.236	-1141.854
AB	29666.2	tension	5	9	45	30	659.2497142	988.8745713
HG	29666.2	tension	5	9	45	30	659.2497142	988.8745713

The allowable stress calculated for wood members were:

- tension parallel to the grain $F_t = 1750$ psi
- compression parallel to the grain $F_c = 2725$ psi

Comparing the allowable stresses to the actual stress on the table below, none of the wood members exceed their respective allowable stress. This means that the wood members of the truss are stable.

Table 16 *Stress and Allowable Stress wood members*

Member	Force (lb)		Base (in)	Height(in	Area (in^2)	2/3 Area reduction (in^2	Stress (psi)	Stress [cross section area reduction] (psi)	Material	Allowable stress (psi)
EF	-11511	compresion	5	9	45	30	-255.799851	-383.6997767	wood	2725
CE	-11511	compresion	5	9	45	30	-255.799851	-383.6997767	wood	2725
BE	8295.08	tension	5	9	45	30	184.3350627	276.502594	wood	1750
GE	8295.08	tension	5	9	45	30	184.3350627	276.502594	wood	1750
CD	-15231	compresion	5	9	45	30	-338.466518	-507.6997767	wood	2725
DF	-15231	compresion	5	9	45	30	-338.466518	-507.6997767	wood	2725
AC	-34256	compresion	5	9	45	30	-761.236	-1141.854	wood	2725
HF	-34256	compresion	5	9	45	30	-761.236	-1141.854	wood	2725
AB	29666.2	tension	5	9	45	30	659.2497142	988.8745713	wood	1750
HG	29666.2	tension	5	9	45	30	659.2497142	988.8745713	wood	1750

The allowable stress calculated for steel members was 58015 psi. Comparing this value with the table below, none of the members exceed the allowable stress.

Table 17 *Stress and Allowable stress steel members*

Member	Force (lb)		Area (in^2)	Stress (psi)	Material	Allowable stress (psi)
ED	11511	tension	0.785398163	14656.25187	steel	58015
GF	36073.7	tension	0.785398163	45930.50444	steel	58015
CB	36073.7	tension	0.785398163	45930.50444	steel	58015

4.4 FLOOR BEAMS AND FLOOR JOISTS CONCLUSION

From the Table Minimum Uniformly Distributed Live Loads adapted from SEI/ASCE 7-10: Minimum Design Loads for Buildings and Other Structures; a load of 100 psf was determined to be acting on the facility.

The structural analysis conducted on the floor beams and floor joists, was solely based on the uniform load 100 psf acting on the facility. The analysis did not take into consideration the weight of the joist and floor deck.

5.0 DESIGN

The two main problems of the East Lodge were the lack of enough gathering space and the conditions of the current kitchen space. A new additional space to the existing East lodge was designed in order to solve these problems. The proposed design included the existing East Lodge dining hall, but removed the existing kitchen to make place for a new addition that included additional dining space, a new kitchen, new sanitary facilities, and a new entrance.

This newly proposed kitchen space would be bigger in order to provide service to the current East Lodge and to the new gathering space. It also included storage and cooling space for the food supplies. The new gathering area would be located next to the new kitchen space and have the same functions as the East Lodge; dining hall and gathering space. In addition, it would include a mechanical room and an office space. Also, as part of the new design, a deck at the back of the new East Lodge was included. This deck includes outside sitting space and could be used for other outdoor gathering activities.

The new addition to the East Lodge was designed using AutoCad and Revit. The design was proposed and presented to the members of the Mohegan Council and Treasure Valley Scout Reservation. The members' feedback was taken into consideration for the final design. The members are evaluating other design and construction options as well. This new addition remains as one of their design alternatives for the future plans ahead regarding the East Lodge.

5.1 FINAL DESIGN

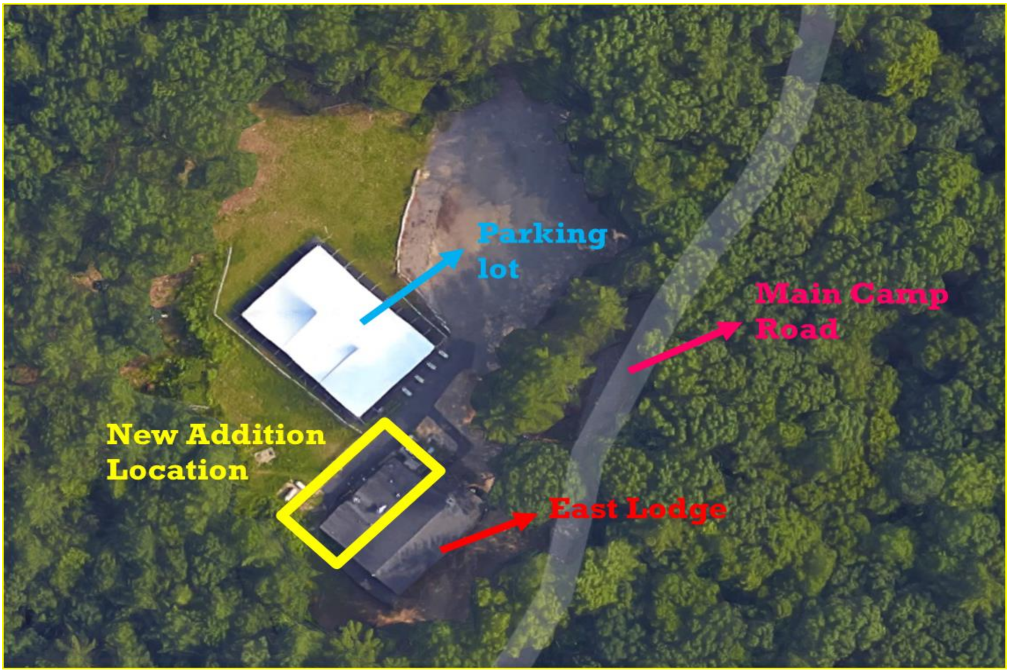


Figure 48 *Site Plan*

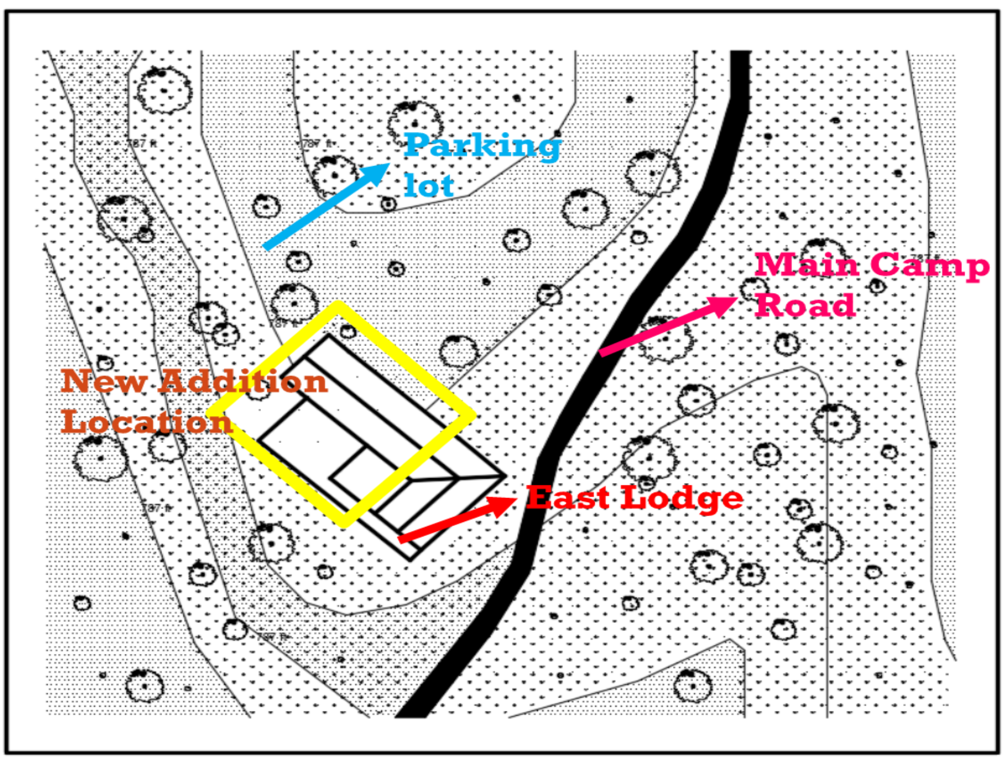


Figure 49 *Site Plan*

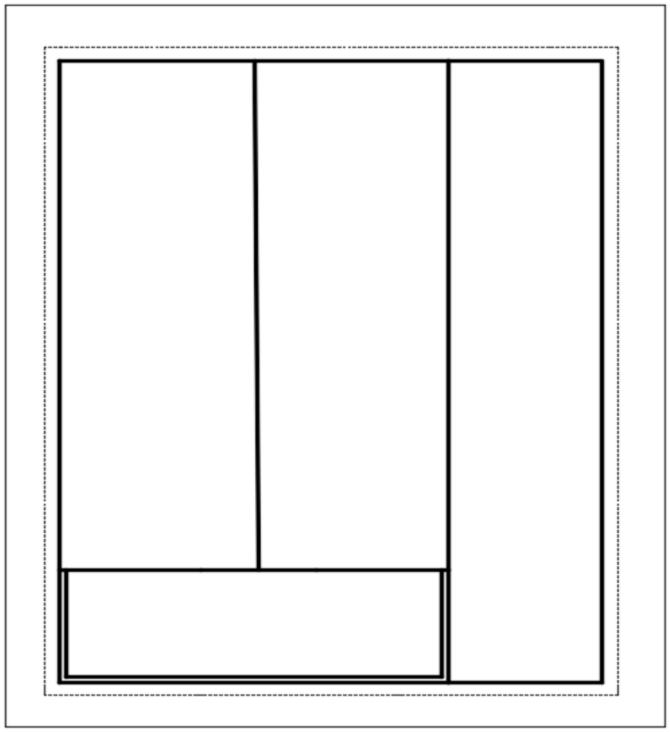


Figure 50 *Current East Lodge roof plan*

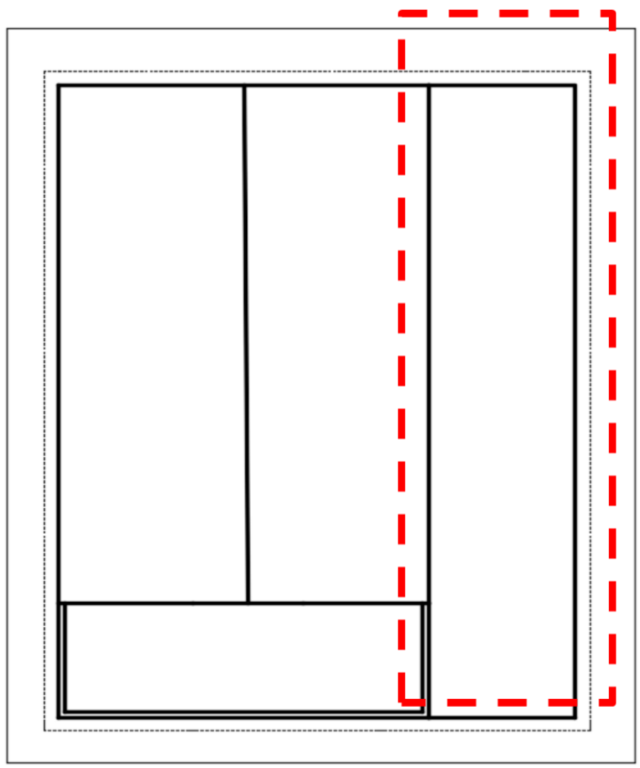


Figure 51 *Current East Lodge roof plan highlighting kitchen area*

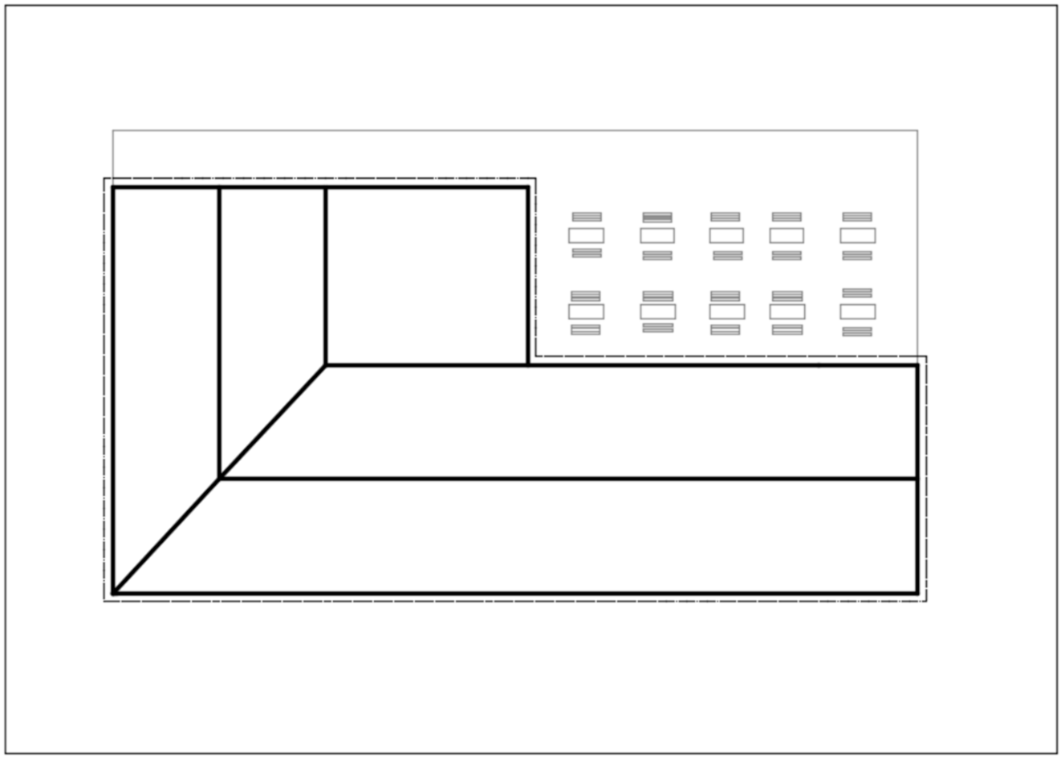


Figure 52 *East lodge with new addition roof plan*

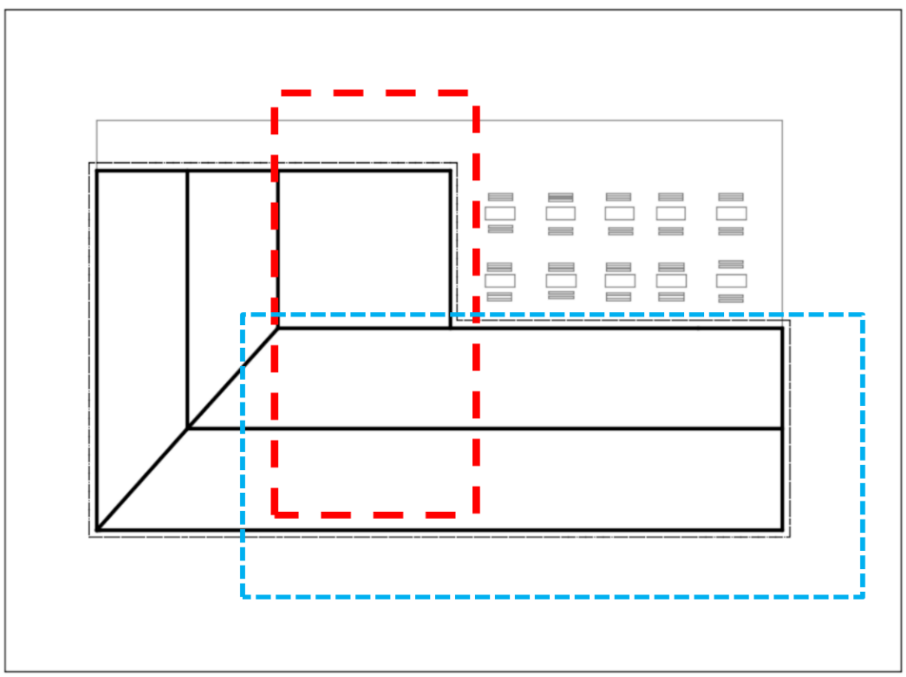


Figure 53 *East Lodge roof plan highlighting new addition location*

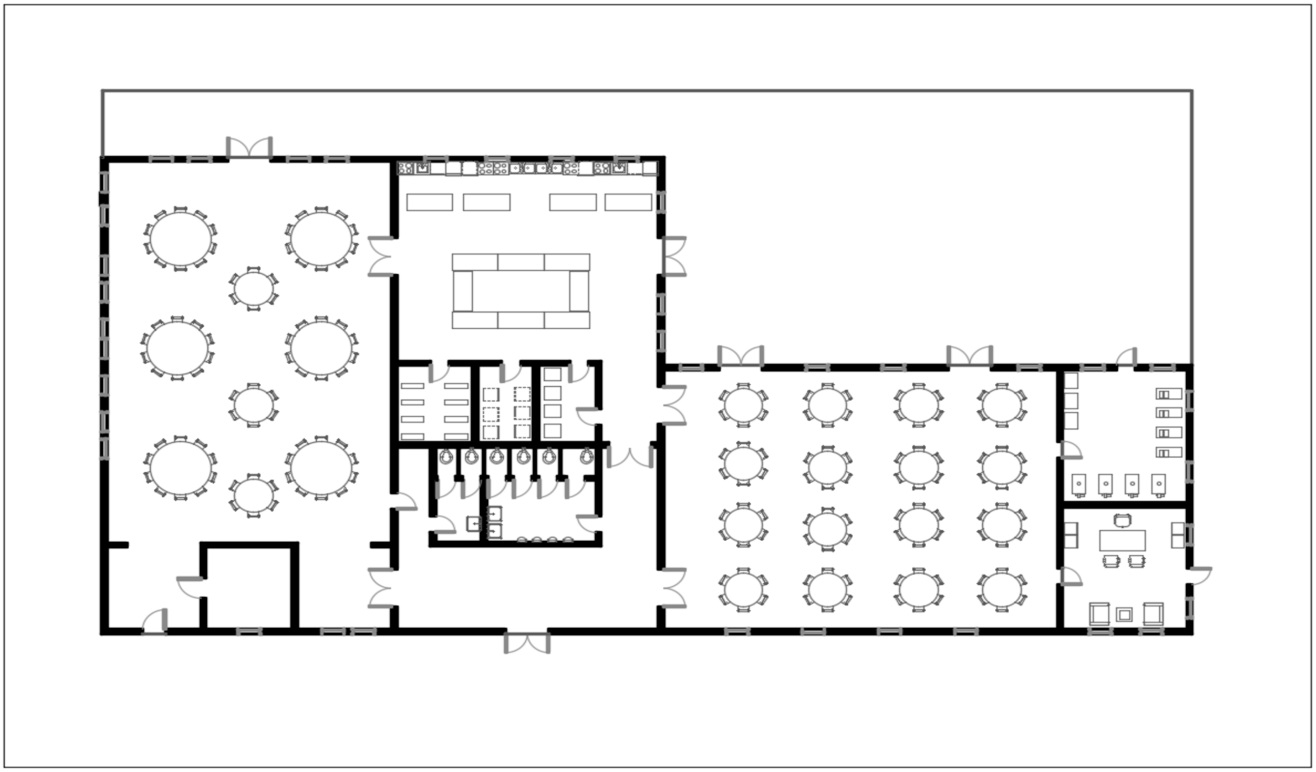


Figure 54 *East Lodge with new addition Floor plan*

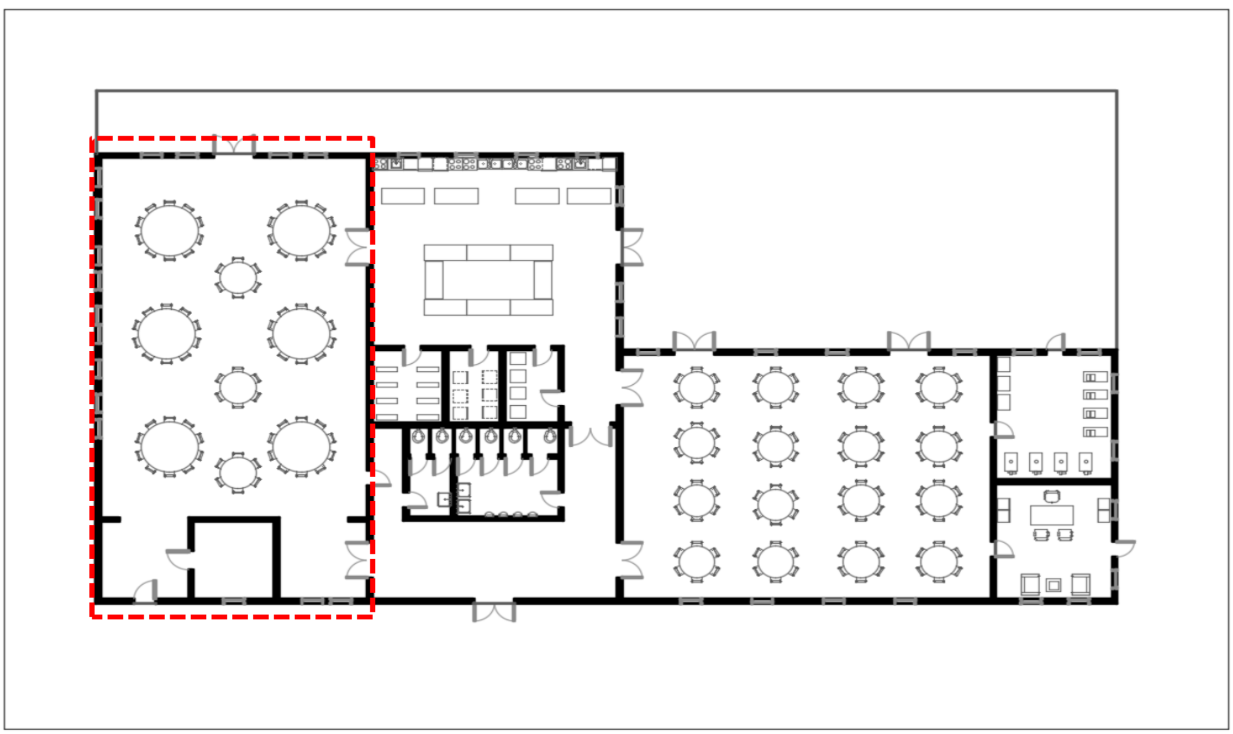
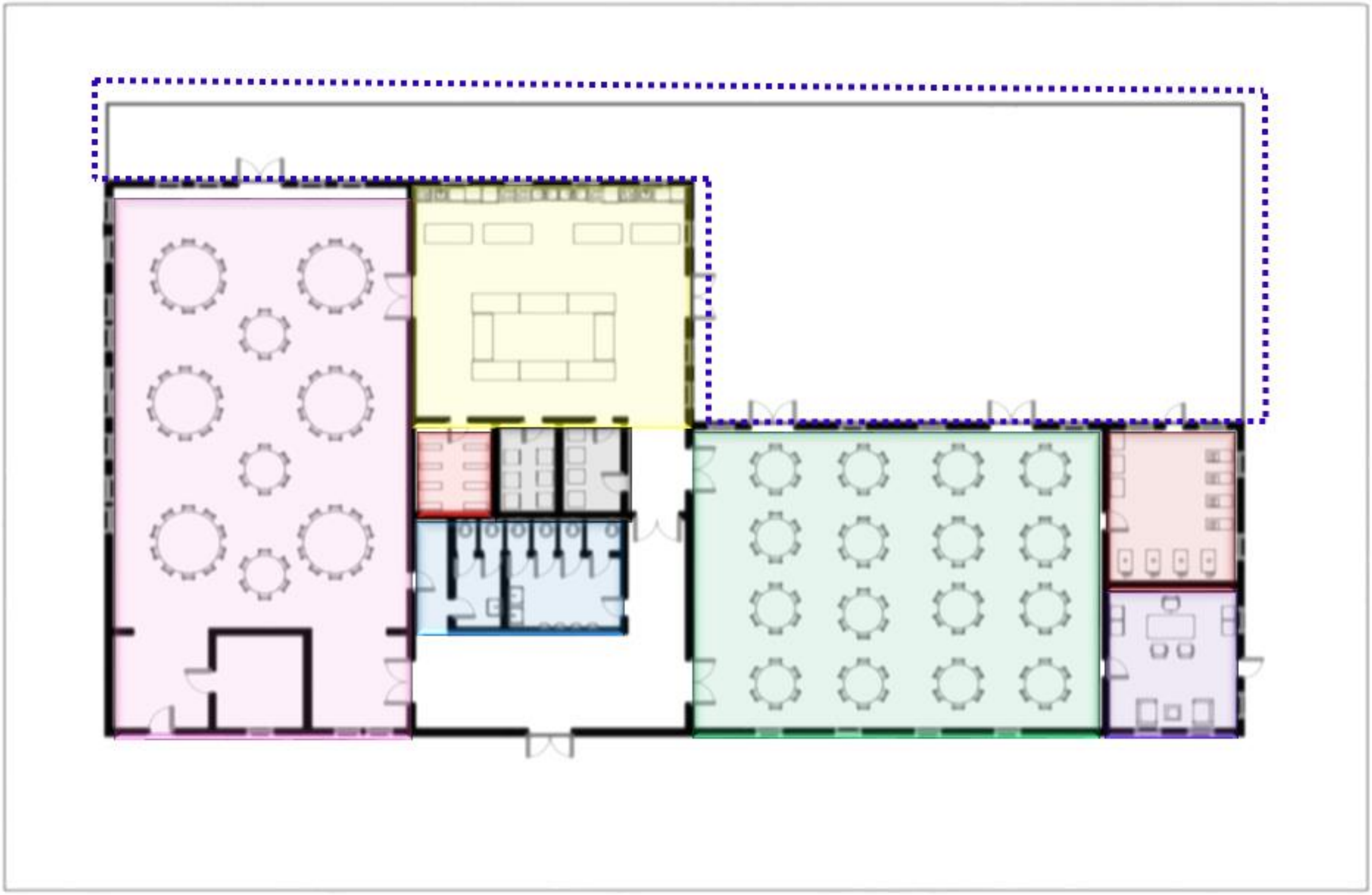


Figure 55 *East Lodge with new addition Floor plan highlighting current East Lodge floor plan*



Mechanical room
Office space
Dining hall
Kitchen
Restrooms
Storage room
East Lodge
Dishwashing/Cooling room
.... Outside Deck

Figure 56 *New East Lodge Facility*

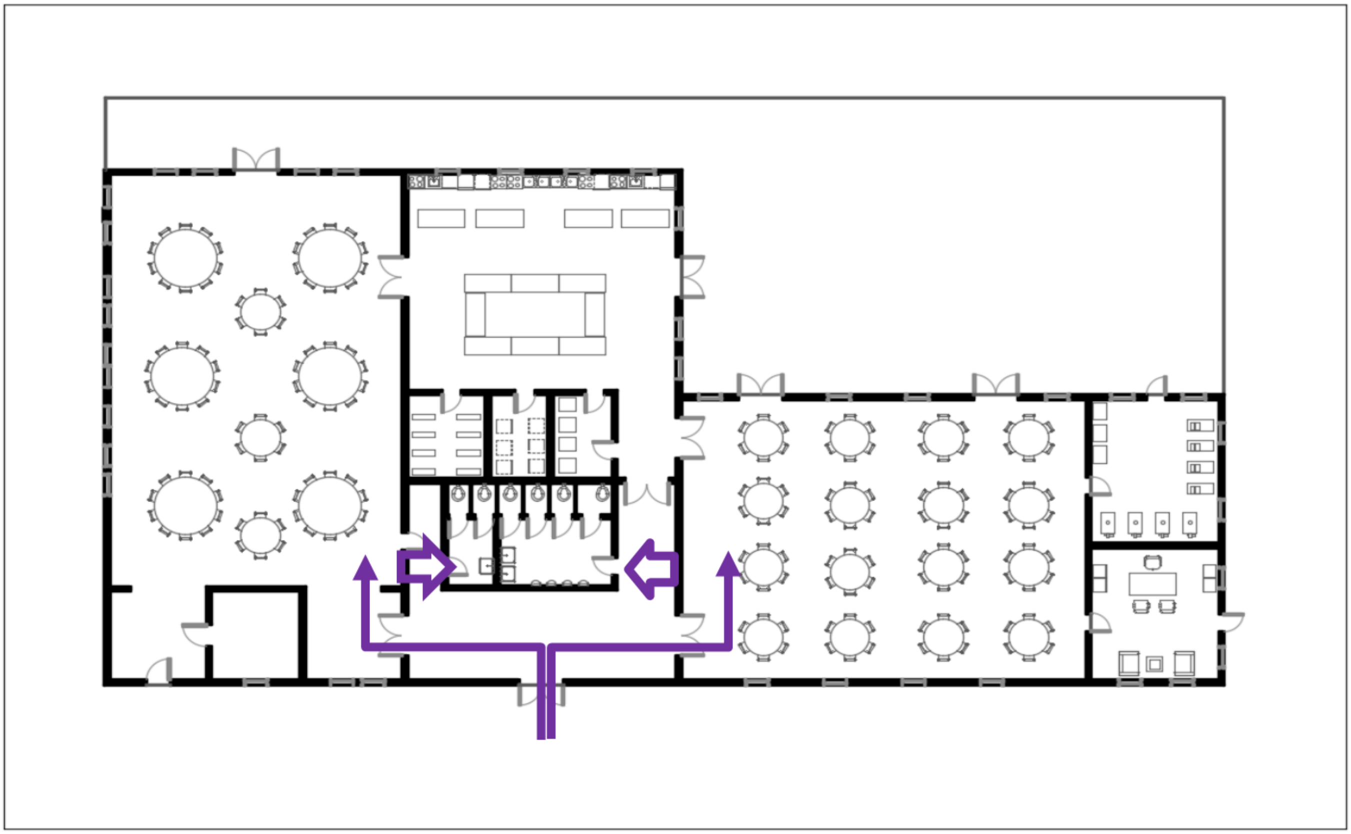


Figure 57 *New addition highlighting common entrance and restrooms*

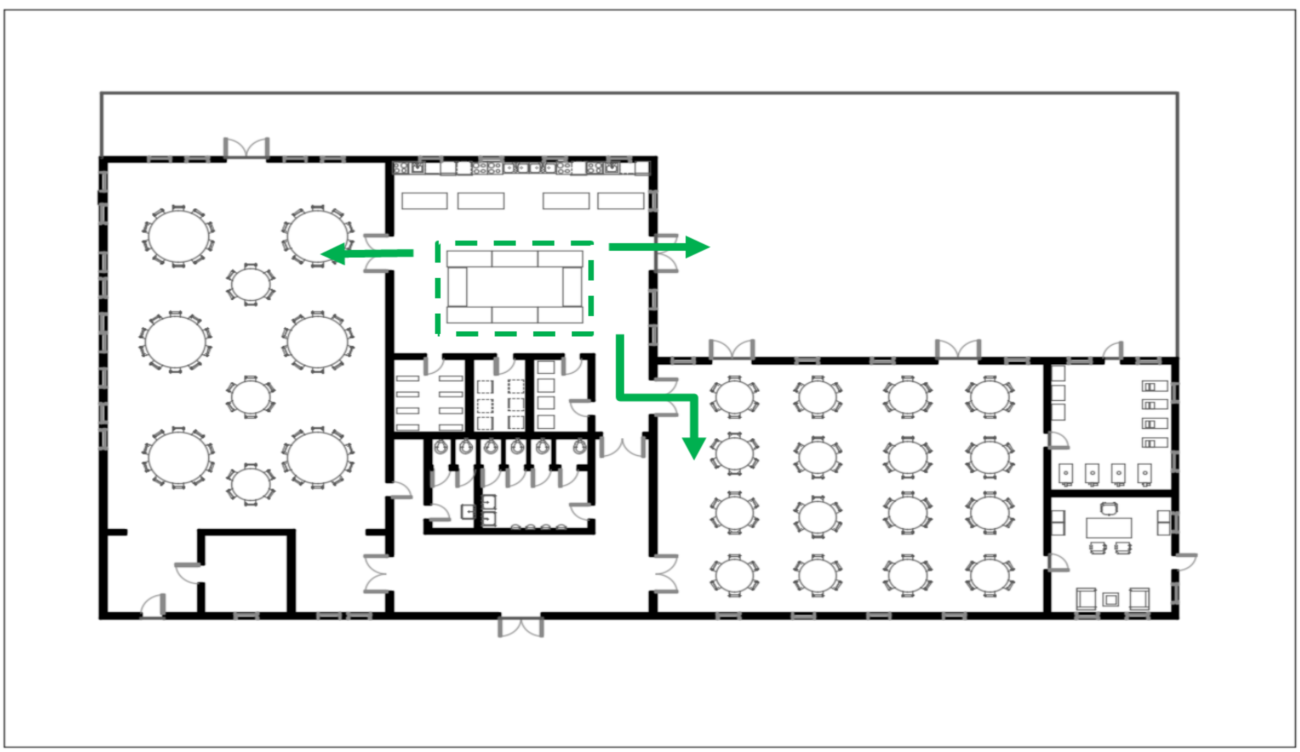


Figure 58 *New addition highlighting kitchen space*

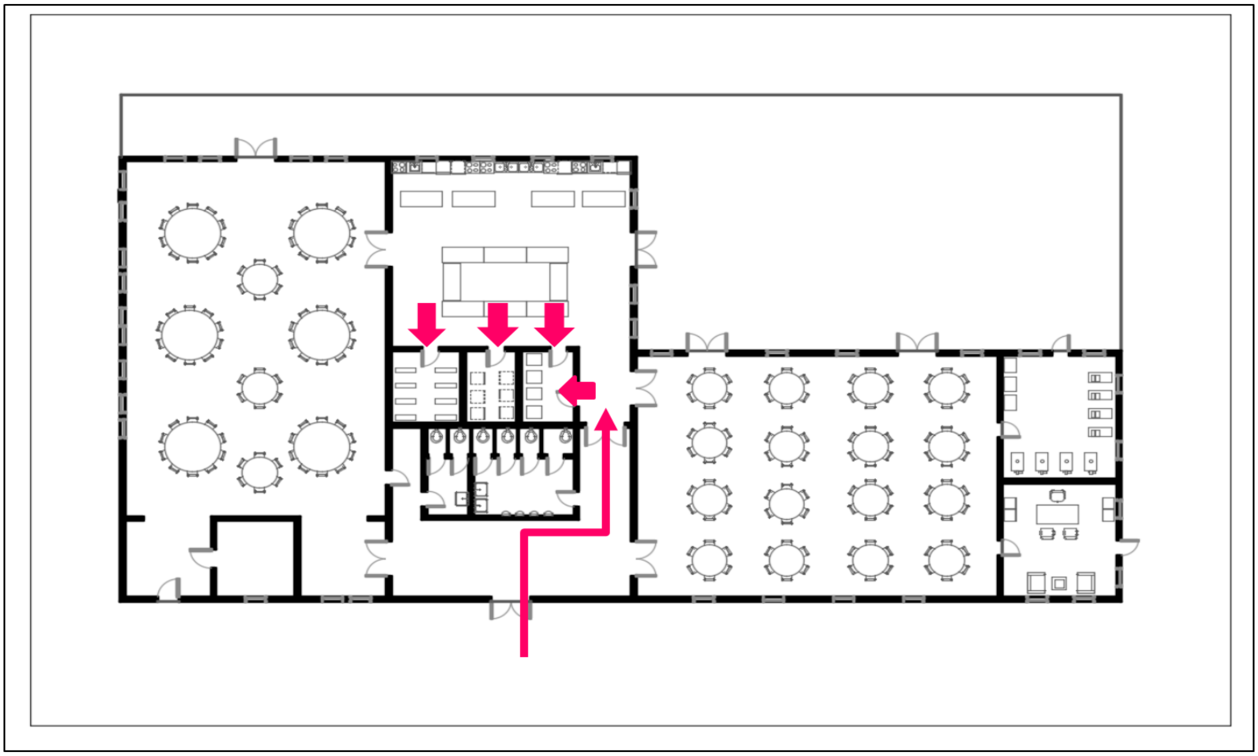


Figure 59 *New addition highlighting loading and cooling space*

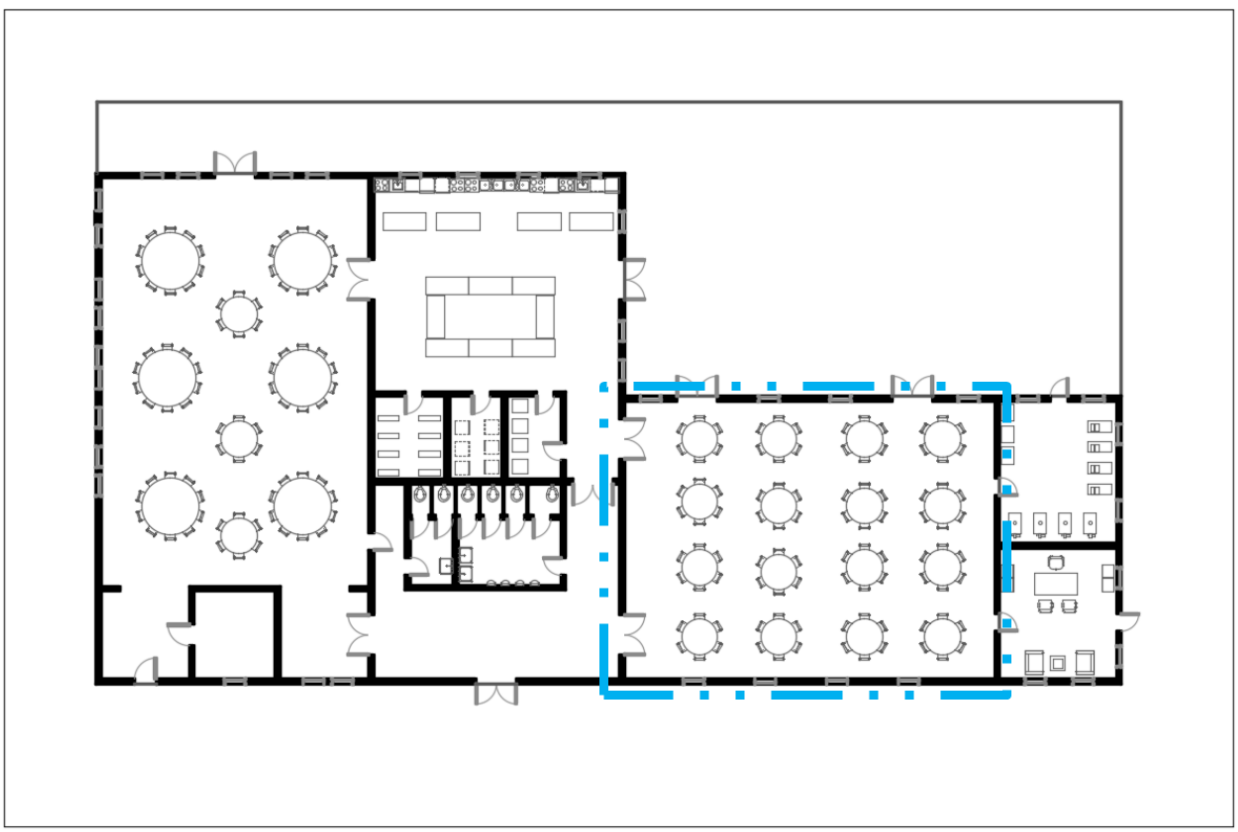


Figure 60 *New addition highlighting dining and gathering space*

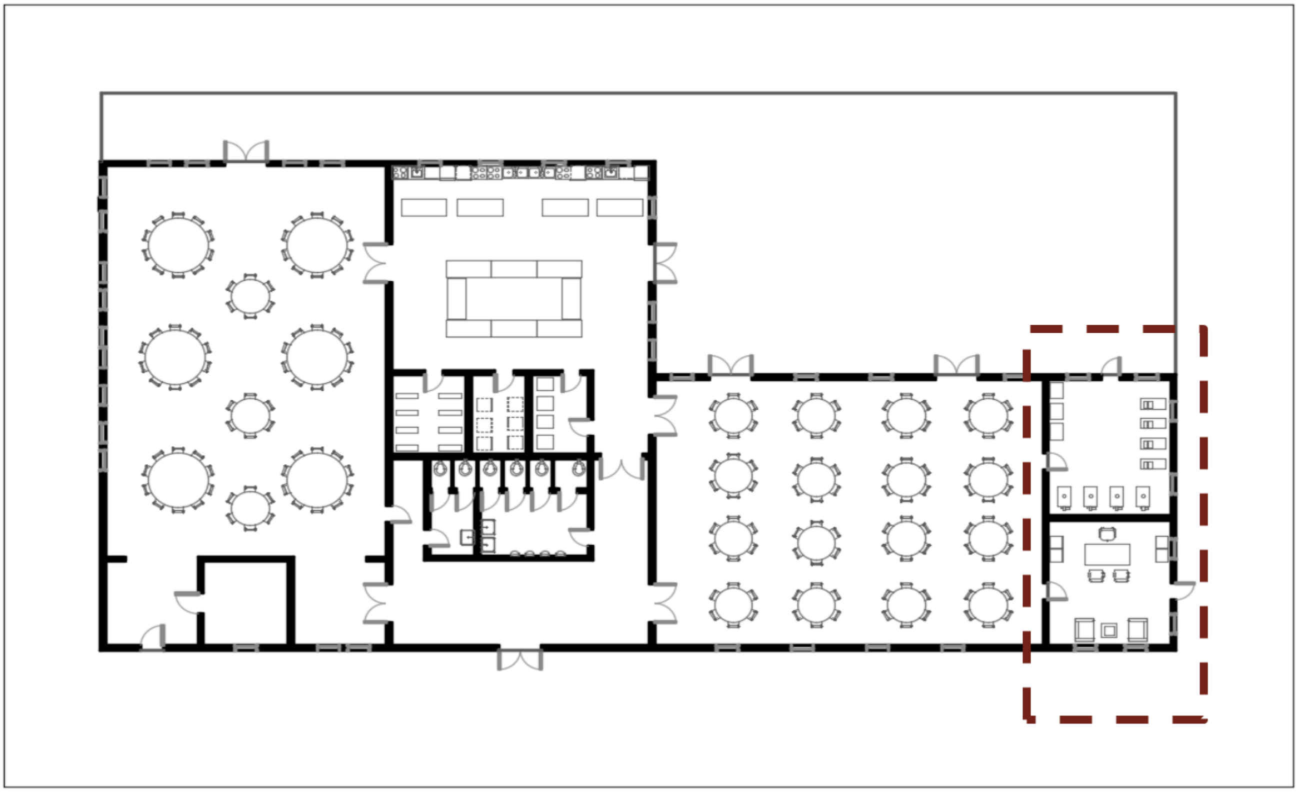


Figure 61 *New addition highlighting office and mechanical space*

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American Society of Civil Engineers ASCE Chapter 7.3.1

American Society of Civil Engineers ASCE Chapter 7.3.3

American Society of Civil Engineers ASCE Chapter 7.2

American Society of Civil Engineers ASCE Chapter 7.3 equation 7.1

American Society of Civil Engineers ASCE 6.5.10: equation 6-15

American Society of Civil Engineers ASCE 6.5.6: table 6-3

American Society of Civil Engineers ASCE 6.5.7.2: table 6-1

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American Society of Civil Engineers ASCE 6.5.11.1

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American Society of Civil Engineers ASCE 6.5.11.2

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US Department of Transportation, April 2005, Force and Stress Analysis Issues; Floor Beams. Extracted from https://www.fhwa.dot.gov/publications/research/infrastructure/structures/04098/12.cfm

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APPENDIX

APPENDIX A TRUSS METHOD OF SECTIONS EXCEL SPREADSHEET

84	lb/ft		L1 ft	L2 ft
462	lb/ft		10.417	8
240	lb/ft			
144	lb/ft			
930	lb/ft			
		Ff=Fc	8563.905	lb
		Fd	7440	lb
		Fa=Fh	4843.905	lb
		Fr	17127.81	lb
		METHOD OF SECTIONS		
	EF	∑MH=0	-11510.9933	lb
	CE	∑MA=0	-11510.9933	lb
	ED	∑MA=0	11510.99334	lb
	BE	∑MC=0	8295.077821	lb
	GE	∑MF=0	8295.077821	lb
	CD	∑Fy=0	-15230.9933	lb
	DF	∑Fy=0	-15230.9933	lb
	AC	∑Fy=0	-34255.62	lb
	HF	∑Fy=0	-34255.62	lb
	AB	∑Fx=0	29666.23714	lb
	HG	∑Fx=0	29666.23714	lb
	GF	∑Fy=0	36073.73383	lb
	CB	∑Fy=0	36073.73383	lb

APPENDIX B STRESS FOR TRUSS EXCEL SPREADSHEET

Member	Force (lb)		Base (in)	Height(in)	Area (in^2)	2/3 Area reduction (in^2)	Stress (psi)	Stress [cross section area reduction] (psi)	Material
EF	-11511	compresion	5	9	45	30	-255.799851	-383.6997767	wood
CE	-11511	compresion	5	9	45	30	-255.799851	-383.6997767	wood
BE	8295.08	tension	5	9	45	30	184.3350627	276.502594	wood
GE	8295.08	tension	5	9	45	30	184.3350627	276.502594	wood
CD	-15231	compresion	5	9	45	30	-338.466518	-507.6997767	wood
DF	-15231	compresion	5	9	45	30	-338.466518	-507.6997767	wood
AC	-34256	compresion	5	9	45	30	-761.236	-1141.854	wood
HF	-34256	compresion	5	9	45	30	-761.236	-1141.854	wood
AB	29666.2	tension	5	9	45	30	659.2497142	988.8745713	wood
HG	29666.2	tension	5	9	45	30	659.2497142	988.8745713	wood
ED	11511	tension			0.7853982		14656.25187		steel
GF	36073.7	tension			0.7853982		45930.50444		steel
CB	36073.7	tension			0.7853982		45930.50444		steel

APPENDIX C MOMENT OF INERTIA EXCEL SPREADSHEET

Member	Base in	Height in	Length ft	Length in	Moment of Inertia in^4
EF	5	9	8.67	104.04	303.75
CE	5	9	8.67	104.04	303.75
ED	5	9	12.5	150	303.75
BE	5	9	6.083	72.996	303.75
GE	5	9	6.083	72.996	303.75
CD	5	9	8	96	303.75
DF	5	9	8	96	303.75
AC	5	9	10.417	125.004	303.75
HF	5	9	10.417	125.004	303.75
AB	5	9	9.417	113.004	303.75
HG	5	9	9.417	113.004	303.75
GF	5	9	6.083	72.996	303.75
CB	5	9	6.083	72.996	303.75

APPENDIX D TRUSS BUCKLING EXCEL SPREADSHEET

Member	n	E lb/in^2	Length in	Moment of Inertia in^4	F lb
EF	1	1900000	104.04	303.75	526221.9
CE	1	1900000	104.04	303.75	526221.9
CD	1	1900000	96	303.75	618055.1
DF	1	1900000	96	303.75	618055.1
AC	1	1900000	125.004	303.75	364520.4
HF	1	1900000	125.004	303.75	364520.4

APPENDIX E DEFLECTION EXCEL SPREADSHEET

E	1900000	psi			
Member	W lb	L in	E psi	I in^4	Deflection in
Floor joist	1600	144	1900000	81.9	0.399768652
floor beam	14400	144	1900000	435	0.677401089
roof boards	427.8	144	1900000	12.4	0.705978947

APPENDIX F STRESS AND ALLOWABLE STRESS EXCEL SPREADSHEET

Member	Force (lb)		Base (in)	Height(in)	Area (in^2)	2/3 Area reduction (in^2	Stress (psi)	Stress [cross section area reduction] (psi)	Material	Allowable stress (psi)
EF	-11511	compresion	5	9	45	30	-255.799851	-383.6997767	wood	2725
CE	-11511	compresion	5	9	45	30	-255.799851	-383.6997767	wood	2725
BE	8295.08	tension	5	9	45	30	184.3350627	276.502594	wood	1750
GE	8295.08	tension	5	9	45	30	184.3350627	276.502594	wood	1750
CD	-15231	compresion	5	9	45	30	-338.466518	-507.6997767	wood	2725
DF	-15231	compresion	5	9	45	30	-338.466518	-507.6997767	wood	2725
AC	-34256	compresion	5	9	45	30	-761.236	-1141.854	wood	2725
HF	-34256	compresion	5	9	45	30	-761.236	-1141.854	wood	2725
AB	29666.2	tension	5	9	45	30	659.2497142	988.8745713	wood	1750
HG	29666.2	tension	5	9	45	30	659.2497142	988.8745713	wood	1750

APPENDIX G STRESS AND ALLOWABLE STRESS STEEL MEMBERS EXCEL SPREADSHEET

Member	Force (lb)		Area (in^2)	Stress (psi)	Material	Allowable stress (psi)
ED	11511	tension	0.785398163	14656.25187	steel	58015
GF	36073.7	tension	0.785398163	45930.50444	steel	58015
CB	36073.7	tension	0.785398163	45930.50444	steel	58015

APPENDIX H DEFLECTION WITH LOAD DECREASE EXCEL SPREADSHEET

E	1900000	psi			
Member	W lb	L in	E psi	I in^4	Deflection in
Floor joist	1600	144	1900000	81.9	0.399768652
floor beam	7200	144	1900000	435	0.338700544
roof boards	427.8	144	1900000	12.4	0.705978947

APPENDIX I LATERAL STABILITY WALL HAND CALCULATIONS

Lateral Stability walls

wood Shear wall capacity

Table 4.3A (SDPWS)

- 8d (fastener Type & Size)
- 7/16" minimum nominal panel thickness
- 1-3/18 min fastener penetration in framing member
- 980 at 4 in Spacing

} calculation purposes

wall capacity $= \frac{980 \text{ plf}}{2} = 490 \text{ plf}$

wall length is 58 ft so total capacity 58(490) = 28420 lbs

ASD capacity wall is 350plf

the overturning force required is $H = 350\text{plf} \times 58\text{ft}$

$= 20,300 \text{ lbf}$