2018 TIMBER ANALYSIS COMPETITION

CE404/504

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Introduction

The task for this project was to design the structure supporting the main roof loads of the Idaho Central Credit Union Arena to be built on the University of Idaho Campus. This arena is intended to showcase wood timber as the main structural material using a cantilever roof draped on large columns at the corners of the walkways. Wood has been used as a building material since the beginning of human time, however in the last hundred years humans have moved towards steel and concrete. This arena is intended to showcase this historic material as well as utilizing this environmentally friendly component. Not only is this arena 'green' but it is being placed in the center of Idaho's timber industry which will inevitably support the economy here in the Northwest.

For this project the University of Idaho students enrolled in CE 404/504 were broken into groups and were asked to take provided architectural drawings and start the structural design process. Every participant was challenged to interpret and arrange methods of several structural systems with a focus on innovation in wood design. In this report our team came up with three different designs for the main spans of the arena and have included our design and analysis for each.

Project Location

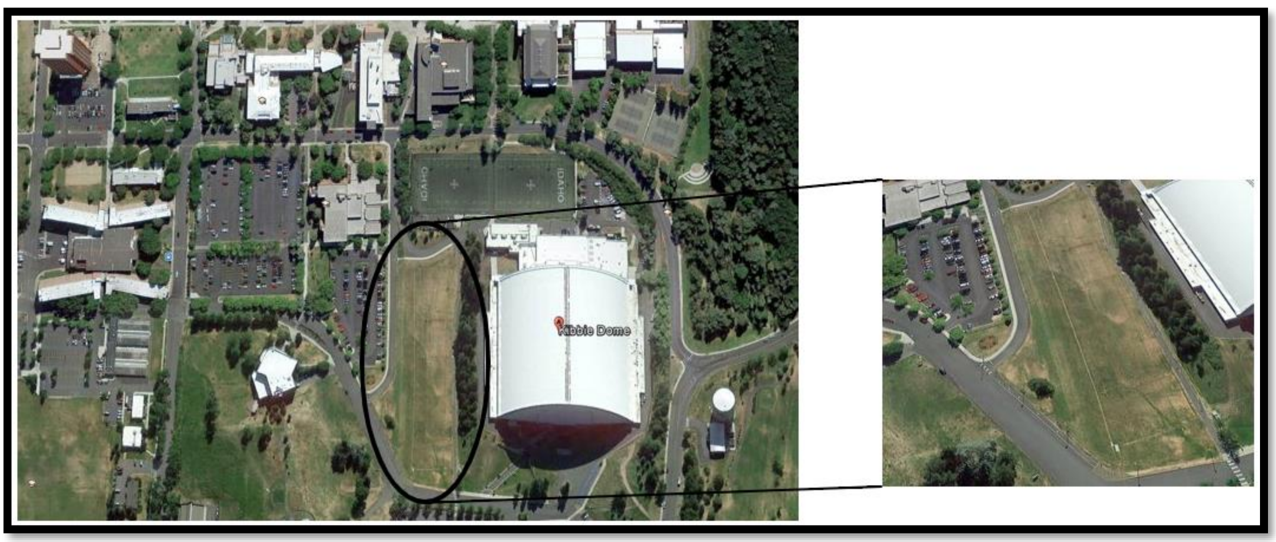


Figure 1: Aerial shot of proposed site

The basketball arena will be built in the parking lot north of the Kibbie Dome on the University of Idaho campus in Moscow. Figure 1 displays the project location above.

Load Calculations

The following dead, live, snow, and wind loads were calculated following the most up to date version of ASCE – 7. Detailed calculations and methods followed can be found in Appendix I of this report. A summary of all loadings are summarized in Table 1 below.

Table 1 - Calculated Loads Summary

Type of Loading	Load (psf)
Dead Load	22.00
Roof Live Load	12.00
Snow Load	27.00
Wind Load (Leeward)	9.25
Wind Load (Side Walls)	12.94
Wind Load (Windward Roof)	12.94
Wind Load (Leeward Roof)	5.54

The calculated loads were used to determine the applied moments and forces for all three designs in RISA. An assumption was made that the arena's roof angle would be a constant 12.24 degrees to calculate the wind and snow loads.

Primary Design

The most efficient and economical design had a North/South arc beam span of 138-feet and a vertical curve height of 12 feet. The East/West beams are 129-feet long with column supports 55-feet high as seen in Figure 2 below. The analysis for the structure and each individual component was completed in RISA. These components include the Lower (Tension) and Upper (Compression) cord, and vertical tensile steel. Also included in the analysis are graphs that display the moments and the axial forces at various points in the beam.

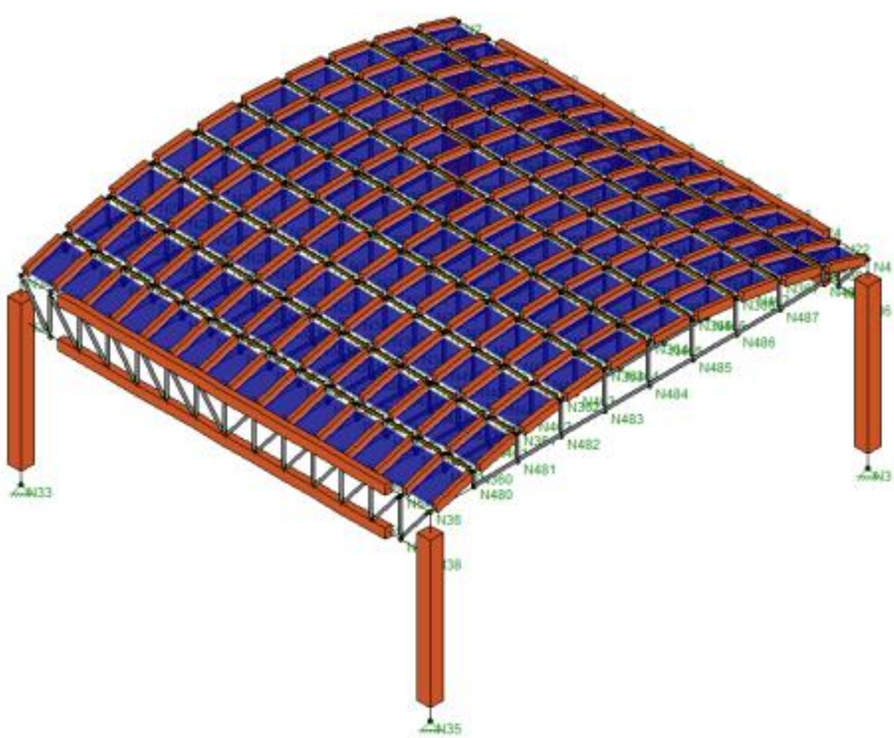


Figure 2: Primary RISA3D design

For the 129-ft span, the supporting main frame is a 16F-1.3E glulam truss which can be seen in Figure 3 below.

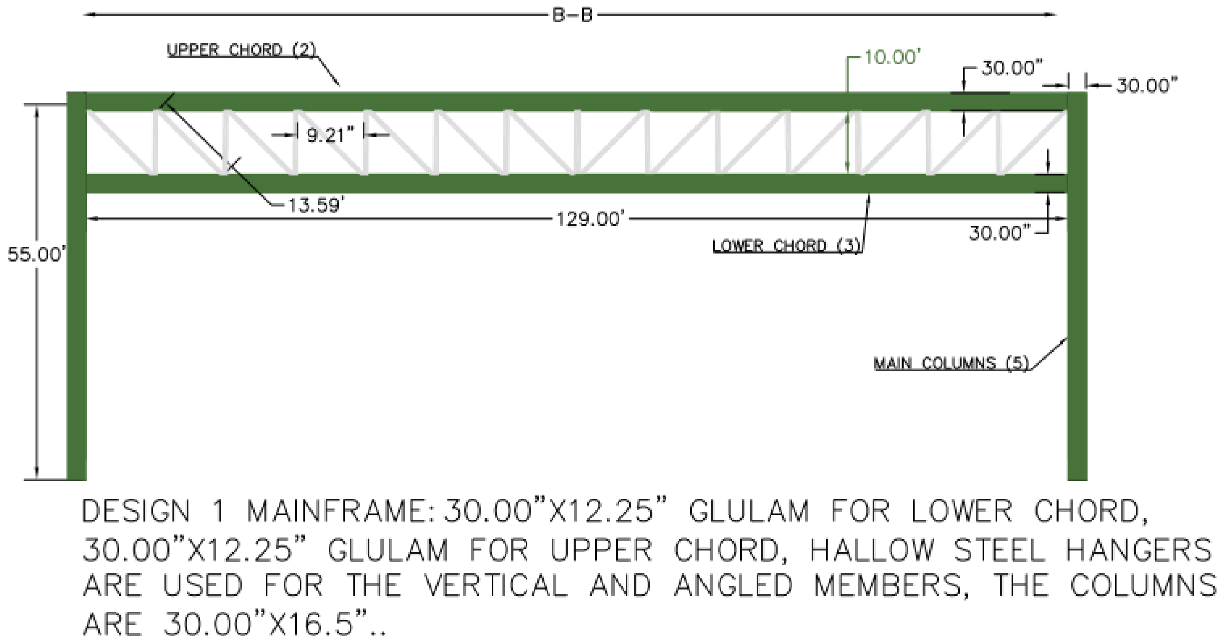


Figure 3: East/West Structural Framing

For the 129-ft span, RISA was utilized to calculate member load configurations. The maximum moment in the horizontal upper chord was calculated to be 208.8 kip-ft. In the lower chord, the maximum moment was found to be 258.4 kip-ft. The tensile steel members in the truss were found to carry a maximum load of 194.7 kips. The diagonal steel members were not sized due to a lack of education in compression steel design. The RISA calculations, along with corresponding shear and moment diagrams are shown in Figures 4, 5 and 6 below.

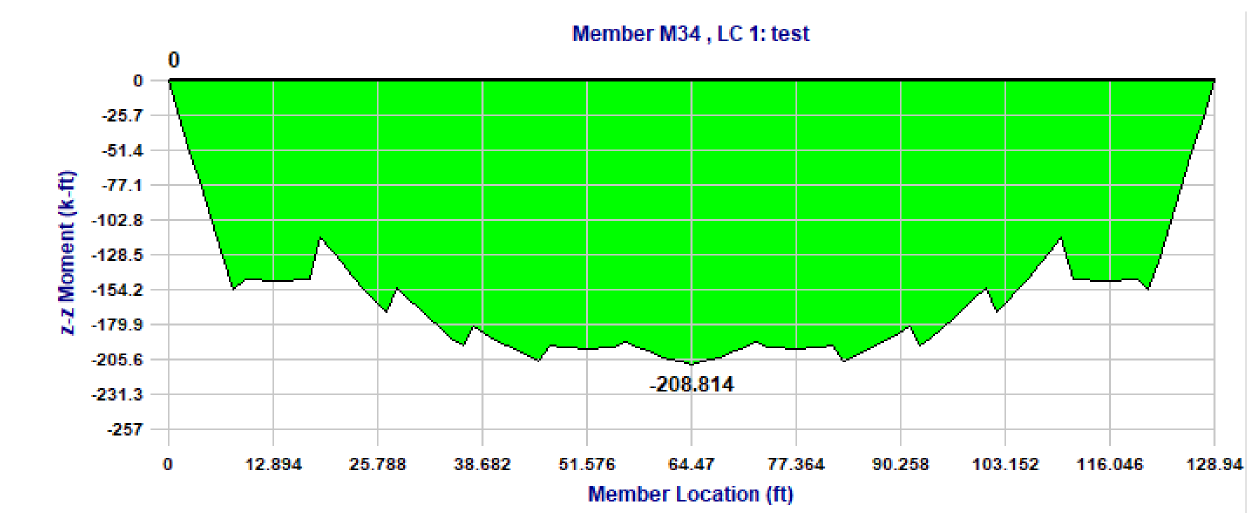


Figure 4: Horizontal Upper Chord Moment Diagram

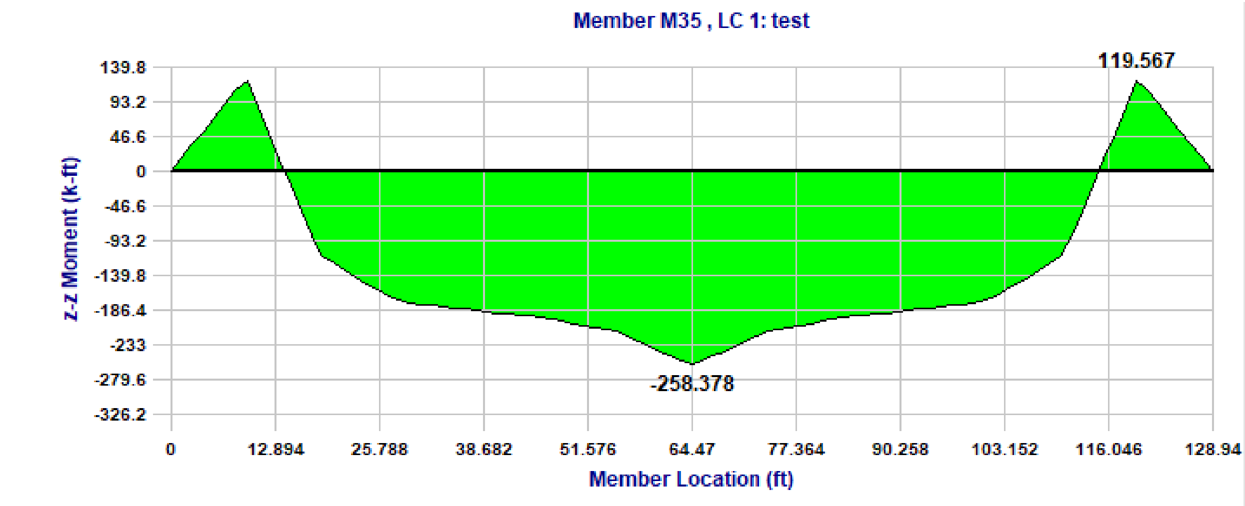


Figure 5: Horizontal Lower Cord Moment Diagram

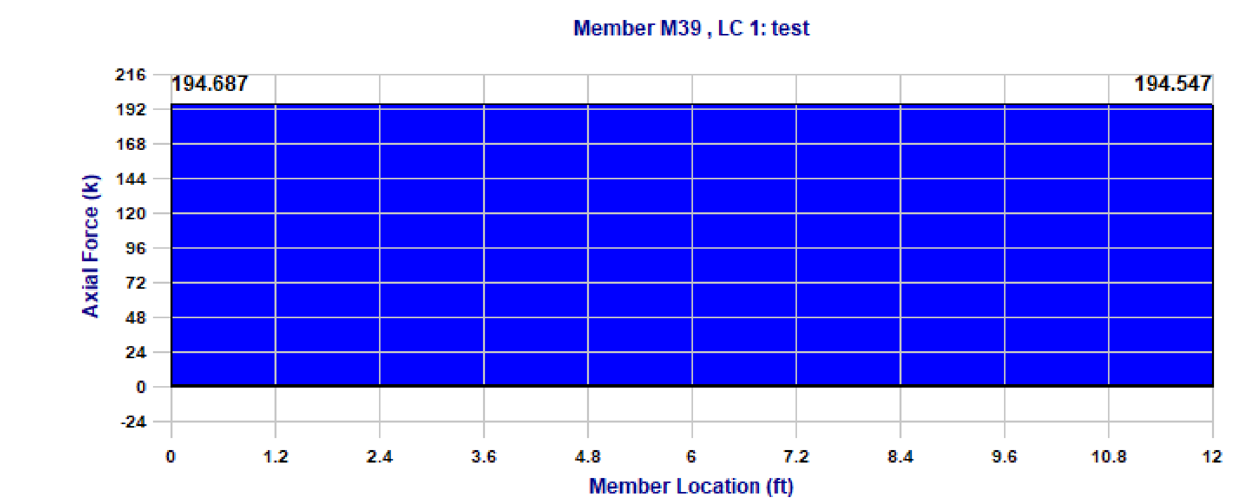


Figure 6: Steel Tensile Rods Axial Force Diagram

For the 138-ft span shown below the rafters will be a 16F-1.3E glulam arch beam with A-36 steel link members and a steel tie rod. A drawing of the design can be seen in Figure 7 below.

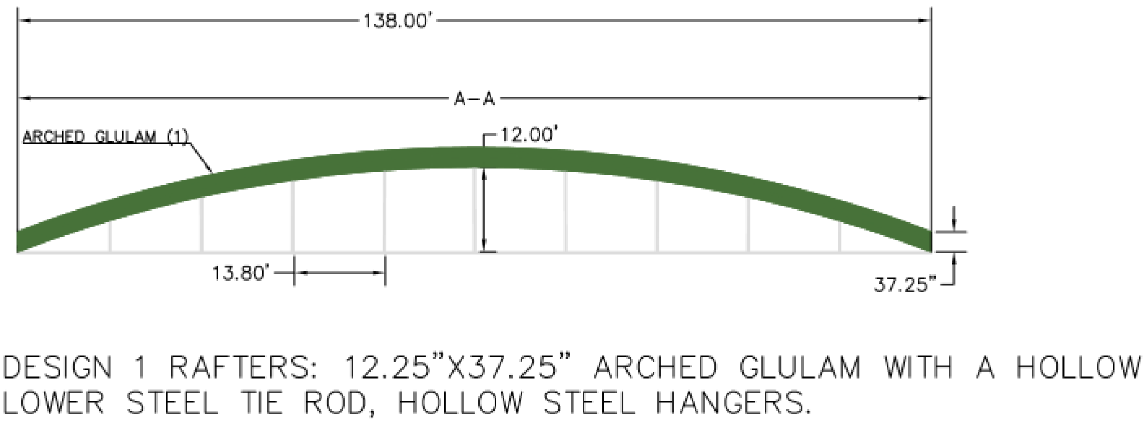


Figure 7: North/South Arch Glulam beam with Tie Rod

The maximum moment for the glulam arched beam was 402.1 kip-ft. The steel tie rod must carry a tensile load of 57.1 kips. The vertical steel compression members were not sized due to the scope of the project.

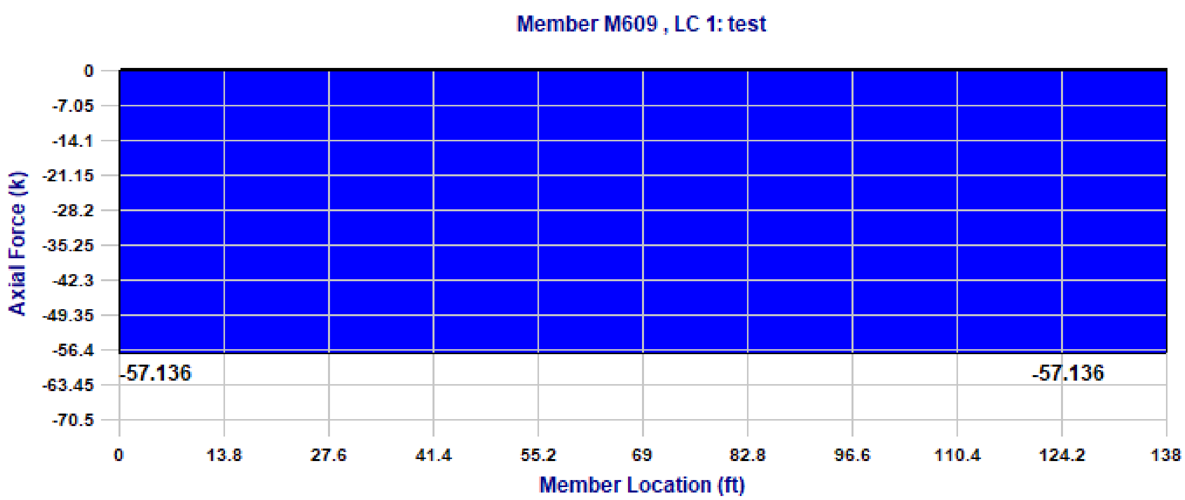


Figure 8 – Axial Force of Steel Compression Members

The purlins which can be seen in the plan view of the roof shown below in Figure 9 were also ran through RISA.

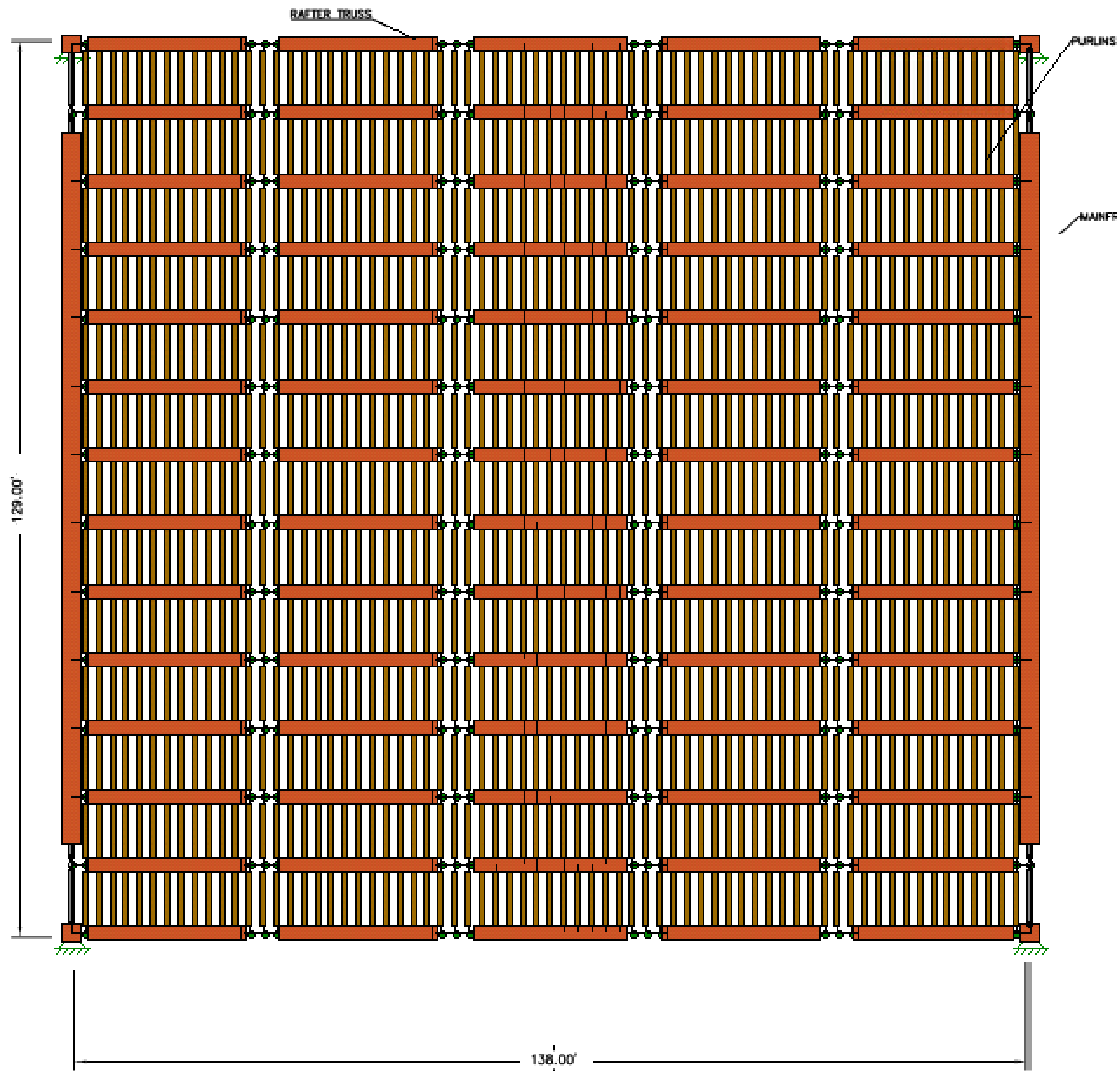


Figure 9 – Roof Plan View

It was found that the purlins would need to withstand a 72 lb-ft moment which can easily be done by an small glulam cross section (6”x2.5”).

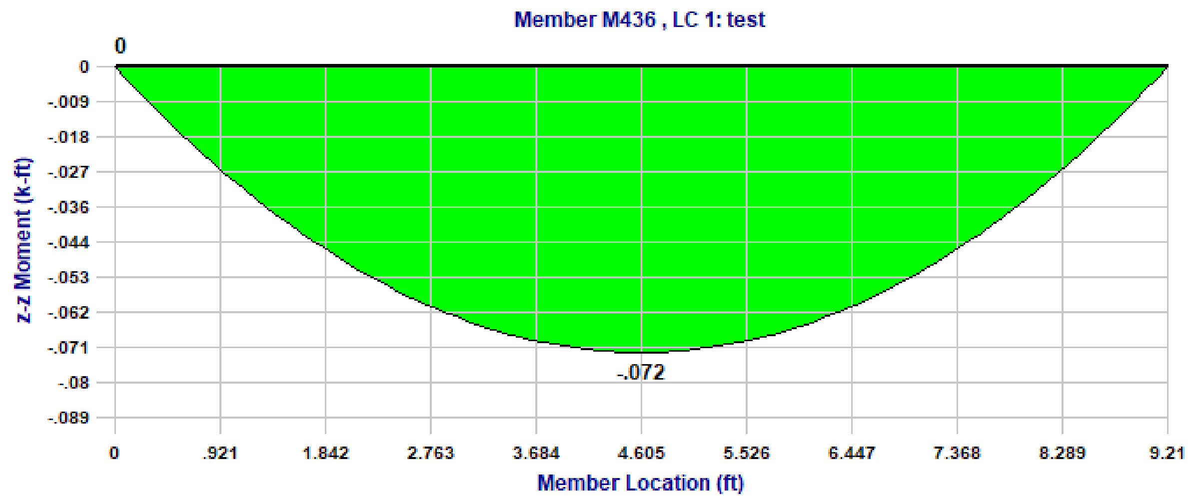


Figure 10 – Max Moment Diagram for Purlins

The major columns will be required to hold a compressive force of 293.1 kips, as shown in Figure 11 below.

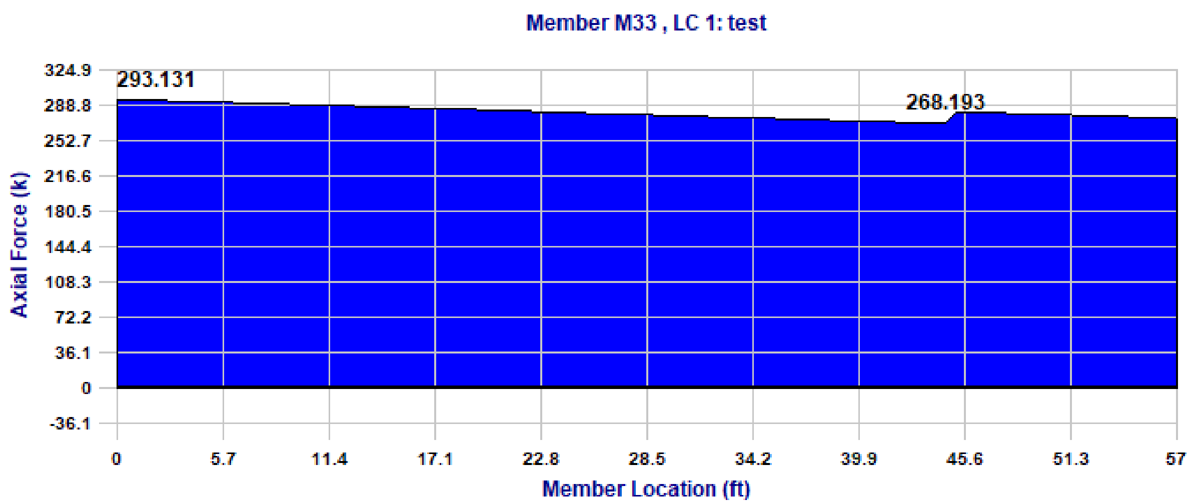


Figure 11 - Max Axial Force in Purlins

A MathCAD template was then created to calculate the required design dimensions for each of the different members in the design. The template was designed to follow NDS and ASD specifications and allowed for the most efficient design dimensions to be used in the design. The MathCAD template and calculations for each of the members are shown in Appendix I.

Utilization of the template allowed for the cross sections of all members to be determined. These dimensions are displayed in the drawings below.

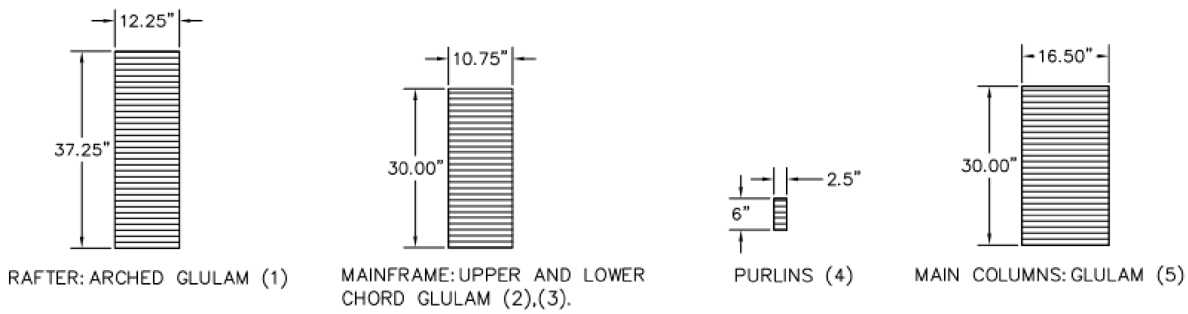


Figure 12 – All Glulam Beam Cross-Sections with 1.5" Thick Laminations

Associated Costs

Information on specific costs for a timber structure as large as the proposed basketball stadium is difficult to acquire. A glulam beam that is 130 feet long with dimensions 12.25"x37.25" is not a typical size that can be found on a timber supplier's catalogue. Due to this, this costs would have to be requested directly from the manufacture which we attempted to do with no success.

The member dimensions and their respective lengths were then used to determine the total cost of materials for the design. Industry standard labor and material estimates were sought out from several companies, however no timely response was given.

There are also a multitude of other expenses that should be accounted for outside of the raw materials. For example, transportation costs of such large beams depending on the location of the manufacturer will be a huge factor. To transport a 130 foot beam on a highway it would take an extended bed and also safety cars to follow in the rear. This would take special road permits and added safety procedures. Another consideration is the special treatment need for wood products. The manufacturer and/or supplier would have to apply special coatings and stains to insure that the end grains of each board will not absorb water or warp over time as this could greatly affect the structural qualities of the timber.

Industry standard labor and material estimates were sought out from several companies, however no timely response was given.

Deflection

Total maximum deflection was found in RISA to be approximately 9.1 inches which occurs in the middle of the structural rafting system. This is excessive for this structure but due to time constraints and the scope of the project the members were not further designed to negate deflection.

Alternative Designs

Two other alternative designs were created to determine the most cost effective and constructible design while also meeting the project's requirements. Both alternative designs were not selected for consideration due to difficulty in manufacturing and shipping, preliminary cost estimations were over budget, and aesthetic creativeness was limited in comparison to the chosen design. A similar process was followed from the above MathCAD and RISA procedures above for both alternatives. For both alternative designs the four supporting columns are typical to the primary design previously discussed in this report.

Alternative Design #1

Alternative Design #1 utilizes a 138-ft, 20”x40” 16F-1.3E glulam dual-arch beam that spans the East-West length of the arena. In the North-South direction, the proposed design calls for fourteen 129-ft, 20”x40” 16F-1.3E glulam arch beams spaced 9’11” O.C. Both arches extend to a maximum height of 12-ft above the four supporting columns. Purlins spanning the East-West direction were determined to be 6.75”x33” spaced 2’9” apart O.C. The rafting system is supported by four identical 30”x30” v-columns with supporting steel members (not sized). Figure 13 shows the isometric RISA drawing of the first alternative design.

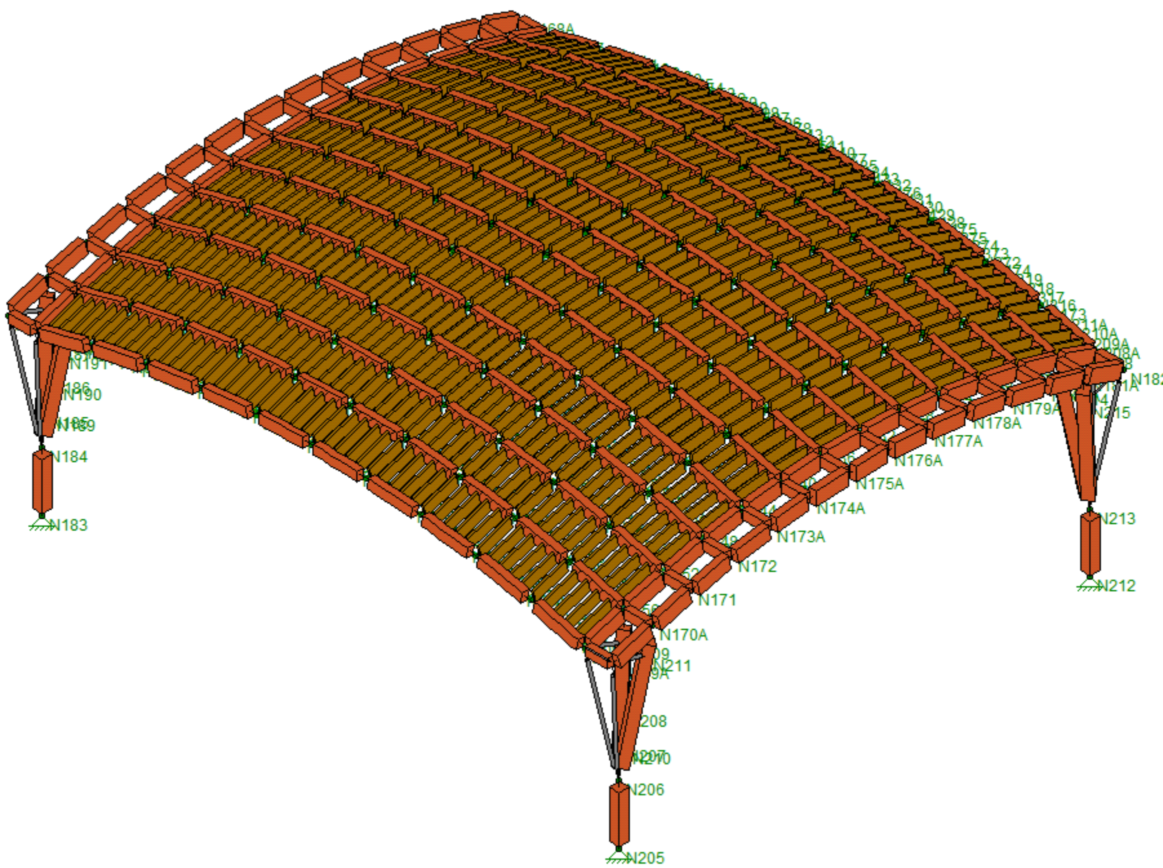


Figure 13 – Isometric Drawing of Alternative Design #1

Alternative Design #2

Alternative Design #2 utilizes a 138-ft, 20”x40” 16F-1.3E glulam dual-arch beam that spans the East-West length of the arena and includes a glulam cross-member sized 20”x30”. In the North-South direction, the design calls for fourteen 129-ft, 20”x30” 16F-1.3E glulam arch beams spaced 13’$\frac{2}{3}$” O.C. Both arches extend to a maximum height of 12-ft above the four supporting columns. Purlins spanning the East-West direction were determined to be 20”x30” spaced 10” apart O.C. The rafting system is supported by four identical 30”x30” v-columns. Figure 14 shows the isometric RISA drawing of the second alternative design.

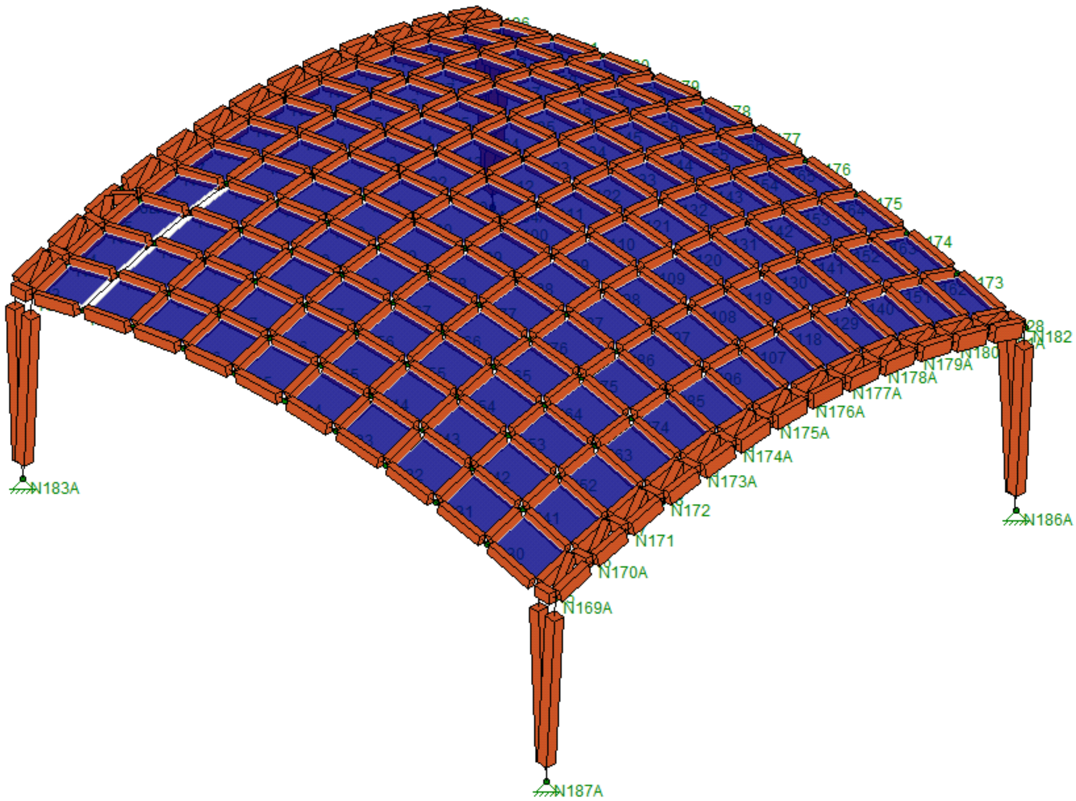


Figure 14 – Isometric Drawing of Alternative Design #2

The two alternative designs were not selected due to excessive sizing of the major glulam beams and the cost that would be involved with manufacturing and shipping. The primary design utilizes steel tie rods which help decrease the cross sections of the structural members in the 138-ft span. The 129-ft span in the primary design is made up of two glulam sections with steel truss members connecting the two. This allows for much smaller glulam arches which we believe are more in line with the goals set for the stadium from an architectural standpoint. Deflection in the alternative designs was also much greater, which is part of the reason that the thickness of the members increased so much from the primary design. The main structural component of the primary design is still glulam wood, but the combination of steel allows for smaller members that will significantly decrease costs without compromising aesthetics.

Conclusion

This project's primary objective was to design the structure supporting the main roof loads for the Idaho Central Credit Union Arena on the University of Idaho campus. Our two goals were to design a system that not only is cost effective and safe, but also showcases wood timber and its capability of being the main structural material in large structures. To accomplish this task, we designed three statical systems and determined which one would best accomplish these two objectives. By using multiple approaches we were able to procedurally learn what designs seemed to minimize member sizes and optimize the efficiency of individual members. Our final design included the most steel, however we estimated larger savings in structural materials, therefore helping shave costs. We determined that saving cost was more important than maximizing the percent of structural material being wood. We believe that the primary design is more aesthetically pleasing. It utilizes a composite approach with mixing of glulam and steel while being able to have smaller member sizes, which removes the 'bulkiness' of the two alternative designs. It also showcases different types of truss systems instead of utilizing as many large glulam trusses with no other supports.

Our group found this project very challenging. It forced us to use not only the skills we learned in class, but introduced us to structural designs programs such as RISA3D and SAP, where the group lacked any experience. This project was a great introduction to designing entire statical systems as opposed to individual member design.

Appendices

Appendix I – Load Calculations

The roof dead load

Total roof load = 22 psf

Roof live loads

a) $L_r = 20\ R_1R_2$

R_1= 0.6 TA > 600 ft^2

R_2

F= 12 tanθ =12*14/64.5= 2.61

R_2= 1 from ASCE-7

L_r = 20*0.6*1= 12 psf

Snow load:

C_e = 0.9

P_g = 30

C_t=C_s=1

I=1.1

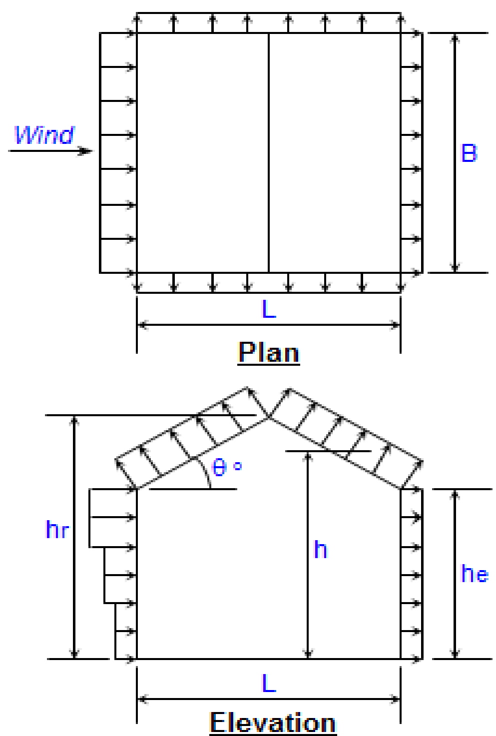
P_f=0.7 C_e C_t I P_g =27 psf

$P_{f\ min}$= 20 I= 22

S= 27 psf

Wind load:

Wind Direction =	Normal	(Normal to building)
Wind Speed, V =	90	mph (Wind Map, Figure 6-1)
Bldg. Classification =	II	(Table 1-1 Occupancy Cat.)
Exposure Category =	B	(Sect. 6.5.6)
Ridge Height, hr =	69.00	ft. (hr >= he)
Eave Height, he =	62.00	ft. (he <= hr)
Building Width =	129.00	ft.
Building Length =	138.00	ft.
Topo. Factor, Kzt =	1.00	
Direct. Factor, Kd =	0.85	(Tables)
Damping Ratio, β =	0.030	
Period Coef., Ct =	0.0200	



Resulting Parameters and Coefficients:

Roof Angle, θ =	6.19	deg.		L = 129 ft.
Mean Roof Ht., h =	62.00	ft. (h = he, for roof angle <=10 deg.)		B = 138 ft.
Windward Wall Cp =	0.80			
Leeward Wall Cp =	-0.50			
Side Walls Cp =	-0.70			
Roof Cp (zone #1) =	-0.90	-0.18	(zone #1 for 0 to h/2)	
Roof Cp (zone #2) =	-0.90	-0.18	(zone #2 for h/2 to h)	
Roof Cp (zone #3) =	-0.50	-0.18	(zone #3 for h to 2*h)	
Roof Cp (zone #4) =	-0.30	-0.18	(zone #4 for > 2*h)	
+GCpi Coef. =	0.18		(positive internal pressure)	
-GCpi Coef. =	-0.18		(negative internal pressure)	

If z <= 15 then: Kz = 2.01*(15/zg)^(2/α) , If z > 15 then: Kz = 2.01*(z/zg)^(2/α)

α =	7.00	zg =	1200	
Kh =	0.86	(Kh = Kz evaluated at z = h)		
I =	1.00	(Table 6-1)	(Importance factor)	

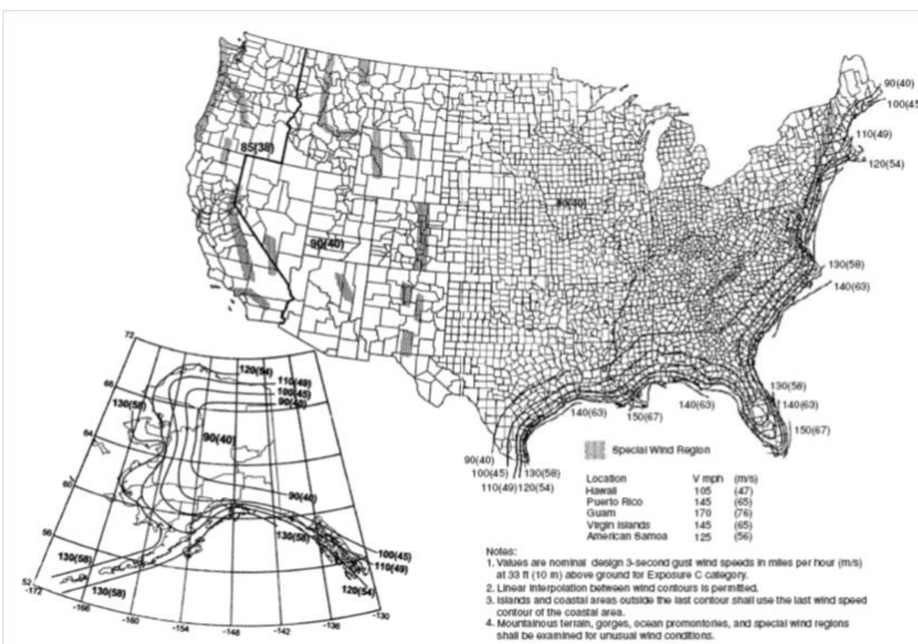
Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2*I

qh =	15.19	psf	qh = 0.00256*Kh*Kzt*Kd*V^2*I (qz evaluated at z = h)	
Ratio h/L =	0.481	freq., f =	2.263	hz.
Gust Factor, G =	0.825			

p = qz*G*Cp - qi*(+/-GCpi) for windward wall (psf), where: qi =qh

p = qh*G*Cp - qi*(+/-GCpi) for leeward wall, sidewalls, and roof (psf), where: qi = qh

Normal to Ridge Wind Load Tabulation for Buildings						
Surface	z	Kz	qz	Cp	p = Net Design Press. (psf)	
	(ft.)		(psf)		(w/ +GCpi)	(w/ -GCpi)
Windward Wall	0	0.57	10.13	0.80	3.95	9.42
	15.00	0.57	10.13	0.80	3.95	9.42
	20.00	0.62	11.00	0.80	4.52	9.99
	25.00	0.67	11.72	0.80	5.00	10.47
	30.00	0.70	12.35	0.80	5.42	10.89
	35.00	0.73	12.90	0.80	5.78	11.25
	40.00	0.76	13.41	0.80	6.11	11.58
	45.00	0.79	13.87	0.80	6.42	11.89
	50.00	0.81	14.29	0.80	6.70	12.17
	55.00	0.83	14.68	0.80	6.96	12.43
	60.00	0.85	15.05	0.80	7.20	12.67
For = hr:	69.00	0.89	15.67	0.80	7.61	13.08
For = he:	62.00	0.86	15.19	0.80	7.29	12.76
For = h:	62.00	0.86	15.19	0.80	7.29	12.76
Leeward Wall	All	-	-	-0.50	-9.00	-3.53
Side Walls	All	-	-	-0.70	-11.51	-6.04
Roof (zone #1) cond. 1	-	-	-	-0.90	-14.02	-8.55
Roof (zone #1) cond. 2	-	-	-	-0.18	-4.99	0.48
Roof (zone #2) cond. 1	-	-	-	-0.90	-14.02	-8.55
Roof (zone #2) cond. 2	-	-	-	-0.18	-4.99	0.48
Roof (zone #3) cond. 1	-	-	-	-0.50	-9.00	-3.53
Roof (zone #3) cond. 2	-	-	-	-0.18	-4.99	0.48
Roof (zone #4) cond. 1	-	-	-	-0.30	-6.50	-1.03
Roof (zone #4) cond. 2	-	-	-	-0.18	-4.99	0.48



Appendix II – Primary Design Calculations

Glulam Section Calculations - Primary Design

Note: All calculations for all three designs followed the latest version of the NDS. More detailed explanations on correction factors, methods, and constants can be found later in the appendix of this report. This section includes only the calculations for the primary design.

Known Values of Glulam Arch Beam

$L = 139\ ft$ $\quad R_{curvature} := 177.04ft$ or $R_{curvature} = 2124.5 \cdot in$

$Thickness_{Lamination} := 1.5in$

$d := 37.5$ Depth of Bending Member, in

$b := 12.25$ Width of Bending Member, in

$x := 10$ For all other wood species

Dead, Live, Wind, & Snow Loads

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$C_M := 1$

$C_t := 1$

$C_i := 1$

$C_F := NA$

$C_L := 1$

$C_{fu} := 1$

$C_D := 1.6$

$C_{vr} := .72$

$$C_V := \left(\frac{21}{L}\right)^{\frac{1}{x}} \left(\frac{12}{d}\right)^{\frac{1}{x}} \left(\frac{5.125}{b}\right)^{\frac{1}{x}}$$

$\boxed{C_V = 0.682}$ must be less than or equal to 1

$$C_C := 1 - 2000 \cdot \left(\frac{Thickness_{Lamination}}{R_{curvature}}\right)^2$$

$\boxed{C_C = 0.999}$

Moment and Shear of Main Glulam Arch Beam

$M_{actual} := 402144 lb \cdot ft$ $V_{actual} := 34540 lb$ (Determined from RISA Analysis)

Design Values

$b := 12.25 in$ $f_b := 1600 psi$ $I := 53830 in^4$

$d := 37.5 in$ $y := 18.75 in$

Moment

$$f'_b := f_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_V \cdot C_{fu} \cdot C_C \cdot C_i$$

$$\boxed{f'_b = 1744.4 \cdot psi}$$

$$M_{allow} := f'_b \cdot \frac{I}{y} \qquad \boxed{M_{allow} = 417332.6 \cdot ft \cdot lbf}$$

Shear

$f_v := 300 psi$

$A := 36.75 in^2$

$$f'_v := f_v \cdot C_D \cdot C_M \cdot C_t \cdot C_{vr}$$

$$f'_v = 345.6 \cdot psi$$

$$V_{allow} := f'_v \cdot \frac{2A}{3} \qquad \boxed{V_{allow} = 105840 lb}$$

From the above calculations, 12.25"x37.5" 16F-1.3E Glued Laminated Softwood Timber arch beams will be sufficent in supporting the applied moment and shear loads for the main span of 138-ft.

Tensile Steel (Tie Rod)

The LRFD and ASD tensile yielding strengths ϕT_n and T_n/Ω, are given by:

$$T_n = F_y A_g$$

$$\phi = 0.9 \qquad \Omega = 1.67$$

Where:

T_n: nominal tensile yield strength, kip

F_y: material yield strength, ksi

A_g: gross cross-sectional area, in^2

$T_n := 57136\,lb$ (Determined from RISA analysis)

$F_y := 36300\,psi$ $\quad \Omega := 1.67$

$A_g := \dfrac{T_n}{(\Omega \cdot F_y)}$ $\quad \boxed{A_g := .94\,in^2}$

From the above calculations, an A36 steel tie rod with a cross-sectional area of .94 in^2 is sufficient in supporting the tensile load. Size up to nearest industry standard size.

Angled Truss Members (Steel)

Due to the scope of the project revolving around timber design, and lack of knowledge in steel design among the group, the angled truss member sizing step was omitted.

Lower Cord of Glulam Truss

General Known Values

$L := 129ft$ $Thickness_{Lamination} := 1.5in$

Dead, Live, Snow, & Wind Loads   Page 62 NDS Supplement

$C_M := 1$ $b := 12.25in$

$C_t := 1$ $d := 30in$

$C_i := 1$

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

$C_L := 1$ $\boxed{C_V = 0.7}$ must be less than or equal to 1

$C_{fu} := 1$

$C_D := 1.6$

$C_C := 1$

$N_{actual} := 547698\ lb$ $M_{actual} := 258378\ ft \cdot lbf$ (Determined from RISA analysis)

Design Values

$b := 12.25in$ $I := 27560in^4$

$d := 30in$ $y := 15in$

$A := 367.5in^2$

$f_t := 1100psi$

$f'_t := f_t \cdot C_D \cdot C_M \cdot C_t$ $f'_t := 1760psi$

Moment

$f_b := 1600psi$

$f'_b := f_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_V \cdot C_{fu} \cdot C_C \cdot C_i$

$f'_b = 1785.5 \cdot psi$

$N_{allow} := f'_t \cdot A$ $\boxed{N_{allow} := 646800psi}$ $M_{allow} := f'_b \cdot \frac{I}{y}$ $\boxed{M_{allow} = 273382.9 \cdot ft \cdot lbf}$

From the above calculations, a 12.25"x30" 16F-1.3E Glued Laminated Softwood Timber beam will be sufficent in supporting the applied load on the lower cord of the truss.

Upper Cord of Glulam Truss

General Known Values

$L := 129ft$ $L_e := 1656in$ $Thickness_{Lamination} := 1.5in$

Dead & Live & Snow Loads

$C_M := 1$ $C_F := NA$ $C_D := 1.6$ $E_{min} := 900000\ psi$

$C_t := 1$ $C_L := 1$ $C_C := 1$ $E'_{min} := 900000\ psi$

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

Calculated Values

$M_{actual} := 208800\ ft \cdot lbf$ (Determined from RISA analysis)

Design Values

$b := 12.25in$

$d := 30in$

Moment

$f_b := 1600psi$

$f'_b := f_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_V \cdot C_{fu} \cdot C_C \cdot C_i$

$f'_b = 1785.5 \cdot psi$

$M_{allow} := f'_b \cdot \frac{I}{y}$ $\boxed{M_{allow} = 273382.9 \cdot ft \cdot lbf}$

From the above calculations, a 12.5"x30" 16F-1.3E Glued Laminated Softwood Timber beam will be sufficent in supporting the applied load on the upper cord of the truss. The upper cord was sized the same as the lower cord for a more aesthetically pleasing design and for consistency in manufacturing.

Vertical Truss Member (Tension)

The LRFD and ASD tensile yielding strengths ϕT_n and T_n/Ω, are given by:

$$T_n = F_y A_g$$

$$\phi = 0.9 \qquad \Omega = 1.67$$

Where:

T_n: nominal tensile yield strength, kip

F_y: material yield strength, ksi

A_g: gross cross-sectional area, in^2

$T_n := 194690 lb$ (Determined from RISA analysis)

$F_y := 36300 psi$ $\quad \Omega := 1.67$

$A_g := \dfrac{T_n}{(\Omega \cdot F_y)}$ $\quad \boxed{A_g := 3.21 in^2}$

From the above calculations, an A36 steel angled member with a cross-sectional area of 3.21 in^2 is sufficient in supporting the applied load, T_n. Size up to nearest industry standard size.

Purlin Joists

$L := 10$

$Thickness_{Lamination} := 1.5in$ $\quad$ $R_{curvature} := 0in$

$d := 6$ Depth of Member, in $\quad$ $y := 3in$

$b := 2.5$ Width of Member, in $\quad$ $I := 45in^4$

$x := 10$ For all other species $\quad$ $A := 15in^2$

Dead & Live & Snow Loads $\quad$ Page 62 NDS Supplement

$C_M := 1$

$C_t := 1$

$C_i := 1$ $\quad$ $C_V := \left(\frac{21}{L}\right)^{\frac{1}{x}} \left(\frac{12}{d}\right)^{\frac{1}{x}} \left(\frac{5.125}{b}\right)^{\frac{1}{x}}$

$C_F := NA$

$C_L := 1$ $\quad$ $\boxed{C_V := 1}$ must be less than or equal to 1

$C_{fu} := 1$

$C_D := 1.6$

$C_{vr} := 1$

Moment

$f'_b := f_b \cdot C_D \cdot C_M \cdot C_t \cdot C_L \cdot C_V \cdot C_{fu} \cdot C_C \cdot C_i$

$f_b := 1600psi$ $\quad$ $M_{actual} := 72ft \cdot lbf$ $\quad$ (Determined from RISA Analysis)

$M_{allow} := f'_b \cdot \frac{I}{y}$ $\quad$ $\boxed{M_{allow} = 3200 \cdot ft \cdot lbf}$

Shear

$V_{actual} := 31lb$ $\quad$ (Determined from RISA Analysis)

$f_v := 300psi$ $\quad$ $f'_v := f_v \cdot C_D \cdot C_M \cdot C_t \cdot C_{vr}$

$\boxed{f'_v = 480 \cdot psi}$

$V_{allow} := f'_v \cdot \frac{2A}{3}$ $\quad$ $\boxed{V_{allow} = 4800lb}$

From the above calculations, 2.5"x6" 16F-1.3E Glued Laminated Softwood Timber purlin rafters will be sufficent in supporting the applied M_{actual} and V_{actual} loads. The size selected is more than capable of supporting the applied loads, however the NDS specifies a 2.5"x6" glulam beam as the smallest selectable size and was thus chosen for design.

Glulam Straight Collumns

General Known Values

$L := 45ft$ $\quad L_e := 540in$ $\quad (Thickness_{Lamination} := 1.5in)$

Dead & Live & Snow Loads

$(C_M := 1)$ $\quad (C_F := NA)$ $\quad (C_D := 1.6)$ $\quad (E_{min} := 900000)$

$(C_t := 1)$ $\quad (C_L := 1)$ $\quad (C_C := 1)$ $\quad (E'_{min} := 900000)$

$(C_i := 1)$ $\quad (C_{fu} := 1)$ $\quad c := .9$ $\quad f_c := 1100\ psi$

Calculated Values

$$\left(F_{cE} := \frac{(.822 \cdot E'_{min})}{\left(\frac{L_e}{b}\right)^2}\right) \qquad F_{cE} := 690.7\ psi$$

$$(F_{cstar} := f_c \cdot C_D \cdot C_M \cdot C_t) \qquad (F_{cstar} := 1760)\ psi$$

$$\left(C_p := \frac{1 + \left(\frac{F_{cE}}{F_{cstar}}\right)}{2c} - \sqrt{\left[\frac{1 + \left(\frac{F_{cE}}{F_{cstar}}\right)}{2.c}\right]^2 - \frac{\frac{F_{cE}}{F_{cstar}}}{c}}\right)$$

$C_p := .371$ Column Stability Factor

$C_{actual} := 293131lb$ (Calculated from RISA Analysis)

Design Values

$b := 16.5in$ $\quad d := 30in$ $\quad A := 495in^2$ $\quad (f_c := 1100psi)$

$(f'_c := f_c \cdot C_D \cdot C_M \cdot C_t \cdot C_p)$ $\quad f'_c := 652.3psi$

$C_{allow} := f'_c \cdot A$ $\quad \boxed{C_{allow} := 322886.7psi}$

From the above calculations,16.5"x30" 16F-1.3E Glued Laminated Softwood Timber columns (4 columns in total, all typical) will be sufficent in supporting the applied compression loads.

Appendix III - Adjustment Factors

Wet Service Factor, C_M – 1

Page 62 (NDS Supp.): Moisture Content does not exceed 16% for an extended time period.

Temperature Factor, C_t - 1

Page 37 (NDS): Structural Members will not experience sustained exposure to elevated temperature up to 150 degrees Fahrenheit.

Load Duration Factor, C_D – 1.6

Page 36 (NDS): All reference design values except modulus of elasticity, E, modulus of elasticity for beam and column stability, E_{min}, and compression perpendicular to grain, shall be multiplied by load duration factors. **A construction load of 1.6 is used.**

Beam Stability Factor, C_L – 1

Page 37 (NDS): Shall not apply simultaneously with the volume factor C_V

Flat Use Factor, C_{fu}

Page 62 (NDS Supp.): Beams are orientated in the strong axis so factor does not apply

Stress Interaction Factor, C_I – 1

Page 38 (NDS): No structural members in compression and tension are tapered.

Shear Reduction Factor, C_{vr} – 1.72

Page 38 (NDS): Structural members satisfy one of the following:

1. Non-Prismatic Members
2. Subject to impact or repetitive cyclic loading
3. Notched
4. **Members used in connection**

Volume Factor, C_V – Reference equation below

Page 62 (NDS Supp.)

When structural glued laminated timber members are loaded in bending about the x-x axis, the reference bending design values, $\mathbf{F_{bx}}^{+}$, and $\mathbf{F_{bx}}^{-}$, shall be multiplied by the following volume factor:

$$C_V = \left(\frac{21}{L}\right)^{1/x}\left(\frac{12}{d}\right)^{1/x}\left(\frac{5.125}{b}\right)^{1/x} \leq 1.0 \qquad (5.3\text{-}1)$$

where:

L = length of bending member between points of zero moment, ft

d = depth of bending member, in.

b = width (breadth) of bending member. For multiple piece width layups, b = width of widest piece used in the layup. Thus, $b \leq 10.75"$.

x = 20 for Southern Pine

x = 10 for all other species

Curvature Factor, C_C

For curved portions of bending members, the reference bending design value shall be multiplied by the following curvature factor:

$$C_c = 1 - (2000)(t / R)^2 \qquad (5.3\text{-}3)$$

where:

t = thickness of laminations, in.

R = radius of curvature of inside face of member, in.

$t/R \leq 1/100$ for hardwoods and Southern Pine

$t/R \leq 1/125$ for other softwoods

The curvature factor shall not apply to reference design values in the straight portion of a member, regardless of curvature elsewhere.

Column Stability Factor, C_P

3.7.1.1 When a compression member is supported throughout its length to prevent lateral displacement in all directions, $C_P = 1.0$.

3.7.1.2 The effective column length, ℓ_e, for a solid column shall be determined in accordance with principles of engineering mechanics. One method for determining effective column length, when end-fixity conditions are known, is to multiply actual column length by the appropriate effective length factor specified in Appendix G, $\ell_e = (K_e)(\ell)$.

3.7.1.3 For solid columns with rectangular cross section, the slenderness ratio, ℓ_e/d, shall be taken as the larger of the ratios ℓ_{e1}/d_1 or ℓ_{e2}/d_2 (see Figure 3F) where each ratio has been adjusted by the appropriate buckling length coefficient, K_e, from Appendix G.

3.7.1.4 The slenderness ratio for solid columns, ℓ_e/d, shall not exceed 50, except that during construction ℓ_e/d shall not exceed 75.

3.7.1.5 The column stability factor shall be calculated as follows:

$$C_P = \frac{1+(F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1+(F_{cE}/F_c^*)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}} \quad (3.7\text{-}1)$$

where:

F_c^* = reference compression design value parallel to grain multiplied by all applicable adjustment factors except C_P (see 2.3), psi

$$F_{cE} = \frac{0.822\ E_{min}'}{(\ell_e/d)^2}$$

c = 0.8 for sawn lumber

c = 0.85 for round timber poles and piles

c = 0.9 for structural glued laminated timber, structural composite lumber, and cross-laminated timber

F' Values

Calculated following ASD standards:

Table 5.3.1 Applicability of Adjustment Factors for Structural Glued Laminated Timber

	ASD only	ASD and LRFD										LRFD only		
	Load Duration Factor	Wet Service Factor	Temperature Factor	Beam Stability Factor [1]	Volume Factor [1]	Flat Use Factor	Curvature Factor	Stress Interaction Factor	Shear Reduction Factor	Column Stability Factor	Bearing Area Factor	Format Conversion Factor K_F	Resistance Factor ϕ	Time Effect Factor
$F_b' = F_b$ x	C_D	C_M	C_t	C_L	C_V	C_{fu}	C_c	C_I	-	-	-	2.54	0.85	λ
$F_t' = F_t$ x	C_D	C_M	C_t	-	-	-	-	-	-	-	-	2.70	0.80	λ
$F_v' = F_v$ x	C_D	C_M	C_t	-	-	-	-	-	C_{vr}	-	-	2.88	0.75	λ
$F_{rt}' = F_{rt}$ x	C_D	C_M[2]	C_t[2]	-	-	-	-	-	-	-	-	2.88	0.75	λ
$F_c' = F_c$ x	C_D	C_M	C_t	-	-	-	-	-	-	C_P	-	2.40	0.90	λ
$F_{c\perp}' = F_{c\perp}$ x	-	C_M	C_t	-	-	-	-	-	-	-	C_b	1.67	0.90	-
$E' = E$ x	-	C_M	C_t	-	-	-	-	-	-	-	-	-	-	-
$E_{min}' = E_{min}$ x	-	C_M	C_t	-	-	-	-	-	-	-	-	1.76	0.85	-

1. The beam stability factor, C_L, shall not apply simultaneously with the volume factor, C_V, for structural glued laminated timber bending members (see 5.3.6). Therefore, the lesser of these adjustment factors shall apply.
2. For radial tension, F_{rt}, the same adjustment factors (C_M and C_t) for shear parallel to grain, F_v, shall be used.