

LATERAL DESIGN OF A 72'X120'X16' POST-FRAME BUILDING

Seismic-Governed Design Example

TABLE OF CONTENTS

<u>SECTION</u>	<u>PAGE</u>
▪ SECTION 1: BUILDING DESCRIPTION	3
▪ SECTION 2: ASCE 7-05 LOADING CALCULATIONS	6
▪ SECTION 3: DIAPHRAGM DESIGN	10
▪ SECTION 4: POST DESIGN	17
▪ SECTION 5: FOUNDATION DESIGN	18
▪ SECTION 6: CONNECTIONS	19
▪ SECTION 7: PURLIN AND GIRT DESIGN	25
▪ SECTION 8: OTHER DESIGN CONSIDERATIONS	26

<u>APPENDIX</u>	<u>APPENDIX PAGE</u>
▪ APPENDIX A	
A1: FRAME STIFFNESS	2
A2: POST DESIGN	3
▪ APPENDIX B: FOUNDATION DESIGN	1
▪ APPENDIX C: DESIGN OF CRITICAL CONNECTIONS	1
▪ APPENDIX D	
PURLIN DESIGN	2
GIRT DESIGN	9

Introduction

Design professionals can use this document to learn how to conduct the structural design of a post-frame building system. Architects or engineers can use this document to walk through the unique features of diaphragm design of post-frame building systems and the detailed engineering calculations for the structural design of a typical post-frame building.

This document—Lateral Design of a 72 ft. x 120 ft. x 16 ft Post-Frame Building: Seismic Governed Design Example—contains the engineering design procedures and detailed calculations to conduct the structural design of single story post-frame building located on a site where lateral design seismic loads exceed lateral design wind loads.

It begins with a general description of the post-frame building to be designed, followed by detailed descriptions and calculations of design loads, roof diaphragm panel in-plane shear strength and stiffness, shearwall panel in-plane shear strength and stiffness, the portion of the lateral wind load carried to ground by the post-frame, and the portion carried to ground by the roof diaphragm and shearwalls using an on-line computer program, the Diaphragm and Frame Interaction (DAFI) Calculator.

The structure has a 72 ft. clear span, is 120 ft. long, and has a 16 ft. eave height. The building has post-frames spaced 8 ft. on center along both sidewalls. Each post frame consists of wood sidewall columns attached directly to engineered, 2x metal-plate connected wood gable trusses with flat lower chords, and two equally sloped upper chords. The roof and walls are sheathed with 29 ga corrugated steel sheathing. Preservative treated laminated wood sidewall columns embedded directly into the ground provide the building foundation.

The design example continues with the structural design of the unique structural elements of the post-frame system, including the nail-laminated wood sidewall columns, the shallow embedded post foundation system, the wood sidewall girts, and the wood roof purlins. The document also includes the detailed procedures and calculations to determine the adequacy of the roof diaphragm panels and all the shearwall panels to carry the design in-plane shear loads.

The key structural connections required to ensure continuous load paths to ground are identified and detailed procedures and calculations for designing each of the key connections are provided. Finally, the document details the lateral and longitudinal bracing requirements for the building system.

The design example follows provisions of the 2009 International Building Code, the 2005 National Design Specification for Wood Construction, ASCE 7-05: Minimum Design Loads for Buildings and Other Structures, and the Post-Frame Building Design Manual. The appropriate sections of these design references are cited throughout the design example.

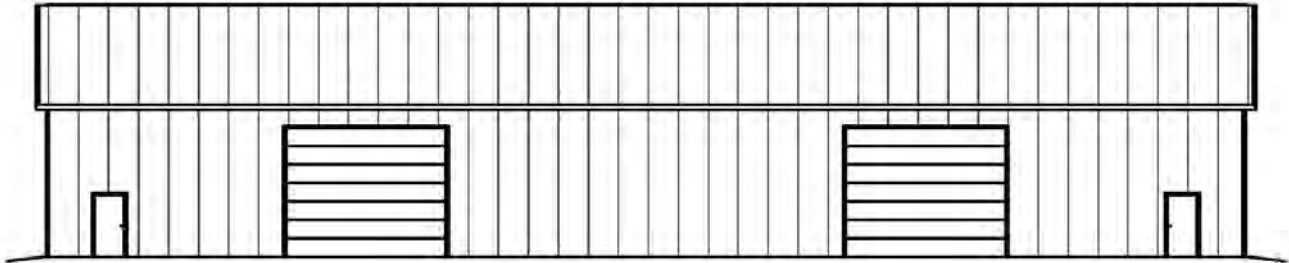
Section 1: Building Description

Seismic-Governed Design Example

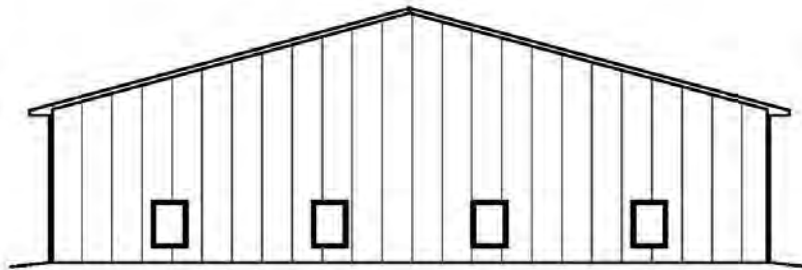
This is a design example in which a 72' wide x 120' long x 16' high Post-Frame building's lateral force resisting system is analyzed and designed for wind and seismic loading. The building is located in Western Kentucky, surrounded by closely spaced wind obstructions, it qualifies for surface roughness category B as defined in ASCE 7-05. This commercial building is comprised of the following structural members:

Structural Component	Description	Spacing
Sidewall Posts	3-ply 2x8 nail laminated column with structural finger joints; posts are embedded into ground; bottom treated for direct ground contact, top notched to accept truss	8 ft o/c
Endwall Posts	3-ply 2x8 nail laminated column with structural finger joints; posts are embedded into ground; bottom treated for direct ground contact	8 ft o/c
Foundation	24"Ø x 8" concrete footing with 24"Ø concrete collar poured around post, (1) #4x16" long rebar thru post at 8" above top of footing	8 ft o/c
Wall Girts	2x4 #2 SYP, each continuous over 2 spans, fastened to side of post with (2) 16d nails	24 in o/c
Roof Purlins	2x4 #2 SYP, on edge, fastened to truss with (1) 60d R.S. nail and (2) 16d toenails, each	24 in o/c
Roof Trusses	Metal plate connected wood trusses, 3.5/12 pitch top chord, 0/12 pitch bottom chord, attached to post directly	8 ft o/c
Roof/ Wall Sheathing	29 gage Grandrib 3 metal sheathing by Fabral, fasten to girts/purlins with #10x1" screws as per manufacturer's recommendations, no stitch screws	n/a

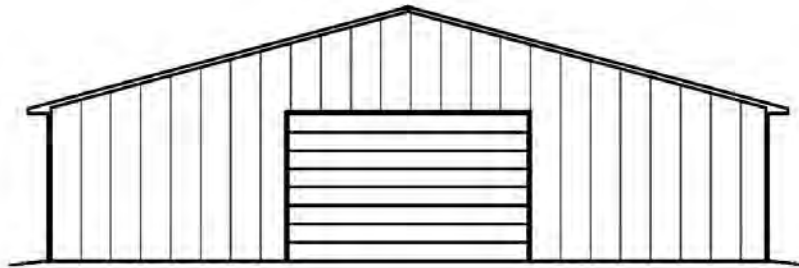
There are (4) 36"x60" windows equally spaced on one endwall and (1) 24'Wx14'H overhead door on opposite endwall; there are (2) 36"x60" windows and (2) 16'Wx14'H overhead doors and (1) 36"x80" man door in rear sidewall and (2) 36"x80" man doors and (2) 16'Wx14'H overhead doors in front sidewall. The exterior walls and roof are sheathed with Grandrib 3, 29 gage structural metal sheathing manufactured by Fabral. On the interior, the walls and ceiling are also sheathed with Grandrib 3, 29 gage structural metal sheathing by Fabral (see Figure 3A and 3B for panel profile and fastener pattern, test panel size and configuration, and in-plane shear strength and stiffness data). The ceiling is framed with 2x4 #2 SYP ceiling joists 24" o/c in between the trusses. The ceiling sheathing and roof sheathing have the same orientation. The building is heated and insulated with R-19 insulation in walls and R-30 insulation in the ceiling. There is a 12" roof overhang on the gable ends and 24" overhang on the sidewalls.



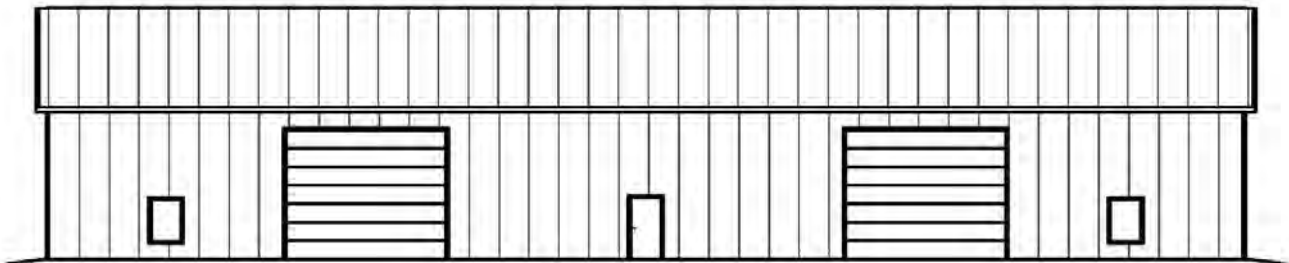
Front Elevation _____



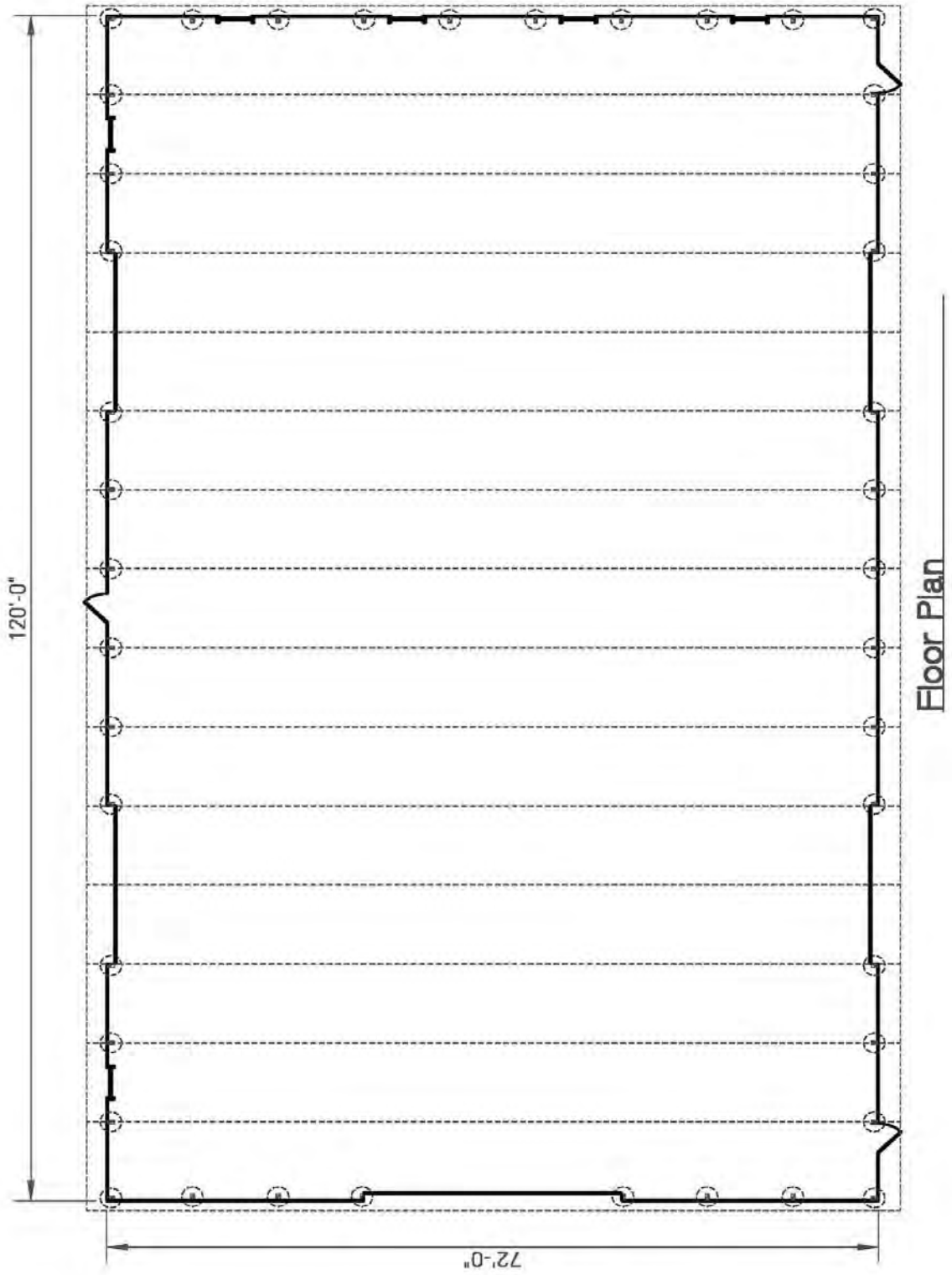
Right Elevation _____



Left Elevation _____



Rear Elevation _____



Section 2: ASCE 7-05 Load Calculations

The wind load calculations, including main wind force resisting system (MWFRS) and components and cladding (C&C) loads, are presented first; followed by dead, live and snow load calculations. Seismic loading is then calculated and compared to wind to see which controls the lateral design for strength and deflection. This section concludes with allowable lateral deflection criterion and controlling load combinations for various components.

2.1 Wind Design Method 2 - Analytical Procedure - Low-Rise Building (ASCE 7-05, 6.5, 6.5.12.2.2)

Building Inputs:

Length Parallel to Ridge, B	120	ft
Length Normal to Ridge, L	72	ft
Wall Height, z	16	ft
Post Sidewall Spacing, s	8	ft
Post Endwall Spacing, s	8	ft
Building Midheight, h	21.25	ft
Roof Pitch (rise per 12 units of run)	3.5	$\frac{12}{}$
Eave Overhang	2	ft

Calculation Inputs:

Basic Wind Speed, V	90	mph
Topographic Factor, K_{zt}	1.00	
Envelope:	<u>Enclosed Building</u>	
Wind Directionality Factor	0.85	
Building Category	II	
Exposure Category	B	

Definitions:

- Case A - Wind Direction Normal to Roof Ridge, Pressure Coefficients Vary With Roof Angle.
- Case B - Wind Direction Parallel to Ridge, Pressure Coefficients are Constant for all Roof Angles.
- Interior Zones - Zones 1 - 6 Below
- Edge Zones - Zones 1E - 6E Below

Intermediate Calculations:

Importance Factor, I	1.00	Table 6-1	Calculated Roof Angle	16.26	deg
Nom. Height of Atmospheric Boundary (z_g)	1200		Internal Press. Coefficient, G_{cpi}	0.18	-0.18
Vel. Press. Exp. Coefficient, K_z	0.635				
3-s Gust Speed Power Law Exponent (α)	7		Velocity Pressure, q_h	11.2	psf

2.1.1 Main Wind Force Resisting System: ASCE 7-05, Figure 6-18

Equation: $p = q_h[(G_{cpr})(G_{cpi})]$

	Case A				Case B			
	P (psf)				P (psf)			
	Gcpf	I*	II**	III~	Gcpf	I*	II**	III~
Zone 1: Windward Side Wall	0.50	3.6	7.6	5.6	0.40	2.5	6.5	4.5
Zone 2: Windward Roof	-0.69	-9.7	-5.7	-7.7	-0.69	-9.7	-5.7	-7.7
Zone 3: Leeward Roof	-0.45	-7.1	-3.1	-5.1	-0.37	-6.2	-2.1	-4.1
Zone 4: Leeward Side Wall	-0.40	-6.4	-2.4	-4.4	-0.29	-5.3	-1.2	-3.2
Zone 5: Gable Wall					-0.45	-7.0	-3.0	-5.0
Zone 6: Gable Wall					-0.45	-7.0	-3.0	-5.0
Zone 1E: Windward Side Wall Edge	0.75	6.4	10.4	8.4	0.61	4.8	8.8	6.8
Zone 2E: Windward Roof Edge	-1.07	-14.0	-10.0	-12.0	-1.07	-14.0	-10.0	-12.0
Zone 3E: Leeward Roof Edge	-0.65	-9.3	-5.3	-7.3	-0.53	-7.9	-3.9	-5.9
Zone 4E: Leeward Side Wall Edge	-0.59	-8.6	-4.6	-6.6	-0.43	-6.8	-2.8	-4.8

Wind load should not be less than 10 psf on vertical projection. (ASCE 7-05, 6.1.4.1)

Because the structure is less than 30ft high, the torsional cases 1T, 2T, 3T and 4T do not apply. (ASCE 7-05, Fig. 6-10, note 5)

2.1.2 Components and Cladding: ASCE 7-05, Figure 6-11

Equation: $p = qh[(G_{cp})-(G_{cpi})]$

Note: Only negative loads are shown because they are larger than positive and so control the design.

Effective Wind

Area: span length multiplied by an effective width that need not be less than one-third the span length.

Component: Wall Girts

Effective Area:	21.33	ft ²		
Zone	G_{cp}	P (psf)		
		I*	II**	III~
4: Wall Interior	-1.0	-13.7	-9.7	-11.7
5: Wall Edge	-1.3	-16.4	-12.4	-14.4

Component: Roof Purlins

Effective Area:	21.33	ft ²		
Zone	G_{cp}	P (psf)		
		I*	II**	III~
1: Roof Interior	-0.9	-11.7	-7.6	-9.6
2: Roof Edge	-1.5	-19.2	-15.2	-17.2
3: Roof Corners	-2.4	-28.9	-24.8	-26.8

* Internal Pressure Positive

** Internal Pressure Negative

~ Internal Pressure Zero

2.2 Dead Load Calculations

2.2.1 Wall Dead Load

3 psf

2.2.2 Roof Dead Load

TOP CHORD

BOTTOM CHORD

Steel Roofing 1 psf_{surface area}

Bottom Chord Load 5 psf

Purlins 1.25 psf_{surface area}

Bracing/Hardware 1.5 psf_{surface area}

Total Top Chord 3.75 psf_{surface area}

Top Chord Load On Horizontal Projection 3.75 psf_{horizontal projection}

TOTAL ROOF LOAD 9 psf

2.3 Live Load Calculations

2.3.1 Floor Live Load

Minimum Floor Live Load = n/a psf (ASCE 7-05, Table 4-1)

2.3.2 Minimum Roof Live Load

Top Chord 20 psf

Bottom Chord 0 psf

Total (on horizontal projection) 20 psf (ASCE 7-05, Table 4-1)

2.4 Snow Load Calculations

2.4.1 Flat-Roof Snow Load, pf

$$\text{Equation: } p_f = 0.7(C_e)(C_t)(I)(p_g)$$

Notes

Figure 7-1

Table 7-2 Partially exposed roof

Table 7-3 Heated building

Table 7-4

Ground Snow Load, p_g : 15 psf

Exposure Factor, C_e : 1.0

Thermal Factor, C_t : 1.1

Importance Factor, I : 1.0

p_f **11.6** psf

2.4.2 Sloped-Roof Snow Load, ps

$$\text{Equation: } p_s = (C_s)(p_f)$$

Notes

Figure 7-2

Roof Slope Factor, C_s : 1.00

p_s **11.6** psf

2.4.3 Unbalanced Roof Snow Load Hip and Gable Roofs

- 1) Required for Hip and Gable Roofs with a slope less than $70/W + 0.5$ and exceeding 70 degrees
- 2) The unbalanced snow load is applied to the leeward roof and windward roof as indicated

Inputs:

Horizontal Distance Eave to Ridge, W 38 ft

Equations: $W \leq 20 \text{ ft}$ (ASCE 7-05, Figure 7-5)

▪ 0 Windward Unbalanced

▪ $(p_g)(I)$ Leeward Unbalanced

$W > 20 \text{ ft}$

▪ $0.3 * p_s$ Windward Unbalanced

▪ $p_s + h_d(\gamma)/(\sqrt{S})$ Leeward Unbalanced, the latter extended from the ridge a distance of $[8(\sqrt{S})(hd)]/3$

Intermediate Calculations:

Roof Angle = 16.26 degrees $h_d = 1.73$ ft

$g = 16.0$ pcf $S = \text{Roof slope run for a rise of } 1 = 3.4286$

$P_{\text{unbal, leeward}} = 26.4 \text{ psf}$ for a distance of **8.5** ft from the ridge, then **11.6** psf to eaves

$P_{\text{unbal, windward}} = 3.5$ psf

2.5 Seismic Load Calculations

2.5.1 Building and Site Inputs:

Site Class **D**
 Seismic Force-Resisting System:
 Building Frame Systems (ASCE 7-05, Tbl 12.2-1)
Light-framed walls sheathed w/ steel sheets
 Seismic Design Category: **D** (ASCE 7-05, 11.6)

2.5.2 Calculation Inputs:

Spectral Response Acceleration, S_1 **0.57**
 Spectral Response Acceleration, S_s **1.74**
 Response Modification Factor, **R** **7**
 Height to Highest Level (ft), h_n **16**
 Weight of Structure (lbs), **W** **6240**
(9 psf)(72+2 +2 ft)(8 ft)+2(3 psf)(16 ft)(8 ft)
 Effective Weight of Structure (lbs), W_e **5760**
(9 psf)(72+2 +2 ft)(8 ft)+2(3 psf)(16 ft)(8 ft)(3/8)

2.5.3 Intermediate Seismic Calculations

Seismic Use Group **II**
 Occupancy Importance Factor, I_E **1**
 S_{ms} **1.740**
 S_{Ds} **1.160**
 S_{m1} **0.855**
 S_{D1} **0.570**
 C_T **0.02**
 C_u **1.4**
 Acceleration Site Coefficient, F_a **1**
 Velocity Site Coefficient, F_v **1.5**
 App. Fund. Period, T_a **0.16**
 Fundamental Period, **T** **0.22**
 Seismic Coefficient, C_s **0.166**
 $C_{s\ min}$ **0.051**
 $C_{s\ max}$ **0.364**
 Seismic Base Shear, $V = 1034$ lbs
 Lateral Seismic Force at Roof, $F_R = 955$ lbs
[$F_R = 1034 \times 5760 / 6240$]

2.5.4 Wind Resolved to Horizontal Point Load at Eave, F_w

Building Inputs: Load from Roof
 Fixity Factor **0.375**
 Wind Force with 10 psf min, $F_w = 1320$ lbs **840**
 Wind Force with Roof and Walls, $F_w = 256$ lbs **-223**
 Wind Force with Walls only, $F_w = 479$ lbs **0**
NOTE: The 10 psf minimum requirement of ASCE 7-05, 6.1.4.1 controls the design. The purpose of this design example is to show a complete seismic controlled design where the seismic lateral force at roof level is such that it controls both the strength and serviceability requirements (story drift criterion). For this reason, the 10 psf minimum requirement of ASCE 7-05 will be ignored in this example.
 Controlling Wind Load:
 Wind Force per Frame, $F_w = 479$ lbs

2.5.5 Seismic vs. Wind Comparison

Controlling Load Combination

$D+W$ **479 lbs**

$D+0.7E$ **668 lbs**

Strength Comparison

Seismic Controls Strength Design

Serviceability Comparison

$(D+0.7E)C_d > D+W$: **Seismic Controls Serviceability Requirements**

2.6 Story Drift and Wind Deflections

Allowable Story Drift = **0.02** h_{s1} (ASCE 7-05, Table 12.12-1)
 Story Height, $h_{s1} = 192$ in
Allowable Story Drift = 3.84 in
 Other Deflection Criterion = **1/120** (ASCE 7-05, Section 12.12.2 and IBC 2009, Table 1604.3)
Allowable Deflection at Eave = 3.2 in (cantilevered column deflection, IBC2009, Table 1604.3, Footnote i)
 Story Drift from Elastic Analysis, $\delta_{1e} = 0.71$ in (see DAFI outputs)
 Deflection Amplification Factor, $C_d = 4.5$ (ASCE 7-05, Tbl 12.2-1)
Calculated Story Drift at Eave, $\delta_1 = 3.17$ in
Story drift and deflection requirements are satisfied - calculated deflection is less than controlling allowable deflection

2.7 Controlling Load Combinations

Wall Posts	$D+L_r$	Post Foundation (Lateral Loading)	$D+0.7E$
Roof Diaphragm	$D+W, 0.6D+W, D+0.7E$	Post Foundation (Uplift)	$0.6D+W$
Endwall Shearwalls	$D+W, 0.6D+W, D+0.7E$	Wall Girts	$D+W$ and $0.6D+W$
Roof Truss	$D+L_r$ and $D+S$	Purlins	$0.6D+W, D+S, D+L_r$

2.8 Load Calculations Summary

Design MWFRS and C&C wind pressures and suctions were calculated for positive, negative and zero internal pressure conditions. The controlling lateral wind force to be resisted by the MWFRS was found by considering wind applied to roof and walls, wind applied to walls only, and 10 psf wind pressure applied to the vertical projection. The lateral force from seismic load was also calculated and compared to the wind force. The response modification factor, R , and deflection amplification factor, C_d , were chosen from ASCE 7-05 Table 12.2-1. The comparison between wind and seismic was performed for both strength and serviceability criterion using forces generated from the critical load combinations. The allowable lateral deflection at the eave was found by taking the eave height divided by 120 ($L/120$ limit). The actual deflection comes from the DAFI analysis in the Diaphragm Design section.

The dead load of the different materials for the roof plane are listed based on surface area and then converted to the horizontal projection to give the top chord dead load. Snow load and unbalanced snow load is calculated for a heated building with insulated ceiling and the roof is considered to be partially exposed.

Section 3: Diaphragm Design

Walls and roof are sheathed with *Grandrib 3*, 29 gage structural metal sheathing manufactured by Fabral. Below are the properties of Grandrib 3 panels as provided in Table 6.1 of the "Post-Frame Building Design Manual" (PFBDM) by the National Frame Building Association (NFBA). The panel dimensions and fastening pattern are shown in Figure 3A. The panel properties were obtained using a cantilever test procedure as shown in Figure 3B.

- Ultimate Strength, P_u , lbf 3300
 - Allowable Shear Strength, v_a , lbf/ft 110
 - Effective In-Plane Stiffness, c , lbf/in 2920
 - Effective Shear Modulus, G , lbf/in 2190
- Reference: Lukens & Bundy, 1987*

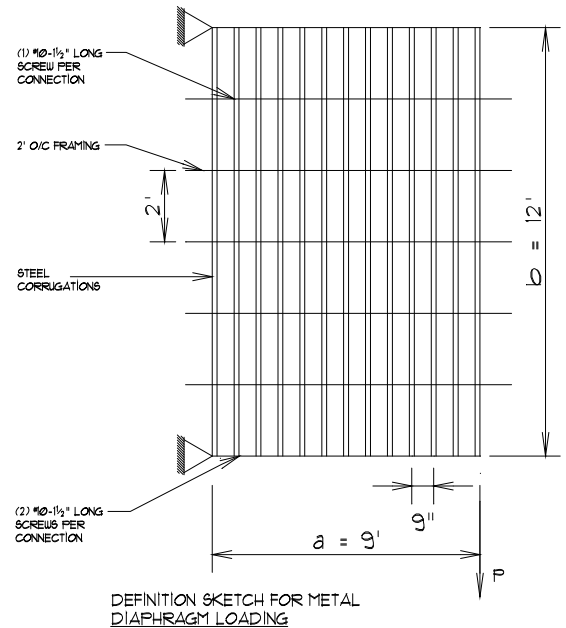
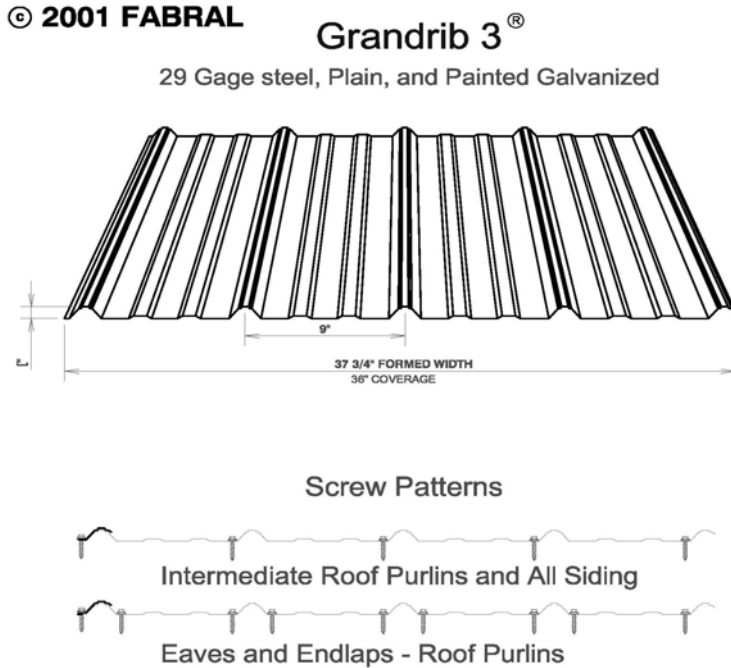


Figure 3A. Sheathing profiles and fastener patterns for the roof and wall panels.

Figure 3B. Test panel arrangement for determining in-plane shear strength and stiffness of diaphragm test panels.

3.1 Diaphragm Horizontal Roof Stiffness, C_h

The total horizontal shear stiffness, C_h , of the roof assembly is calculated by summing the horizontal shear stiffness values, $c_{h,1}$ and $c_{h,2}$, of the individual roof diaphragms including both roof slopes. The horizontal shear stiffness of an individual diaphragm, c_h , is obtained by adjusting the model diaphragm in-plane shear stiffness, c , for actual building size and roof slope.

$$C_h = c_{h,1 \text{ roof}} + c_{h,2 \text{ roof}} + c_{h,3 \text{ ceiling}}$$

$$c_{h,1 \text{ roof}} = c_{h,2 \text{ roof}} = G(\cos\theta_{\text{roof}})(b_{h,1 \text{ roof}}/s)$$

$$c_{h,3 \text{ ceiling}} = G(\cos\theta_{\text{ceiling}})(b_{h,1 \text{ ceiling}}/s)$$

$$\begin{aligned} b_{h,1 \text{ roof}} &= 38 \text{ ft} && (\text{half of the building's width} + \text{overhang}) \\ b_{h,1 \text{ ceiling}} &= 72 \text{ ft} && (\text{building's width}) \\ s &= 8 \text{ ft} && (\text{column spacing}) \end{aligned}$$

$$\begin{aligned} \theta_{\text{roof}} &= 16.26 \text{ degrees} \\ \theta_{\text{ceiling}} &= 0 \\ c_{h,1 \text{ roof}} &= 9986 \text{ lbf/in} \\ c_{h,2 \text{ roof}} &= 9986 \text{ lbf/in} \\ c_{h,3 \text{ ceiling}} &= 19710 \text{ lbf/in} \end{aligned}$$

$$C_h = 39683 \text{ lbf/in} \quad (\text{The total horizontal shear stiffness of roof diaphragm})$$

3.2 Frame Stiffness, k (See Visual Analysis Results in Appendix A)

The stiffness of the bare frame, k, is the ratio of the applied horizontal eave load divided by the resulting horizontal eave deflection. A computer analog of the frame consisting of two posts and the truss has been used to calculate this term. The post to soil interface

$$k = p/\Delta$$

p = horizontal load at eave
 Δ = frame displacement at eave

$$\begin{aligned} p &= 100 \text{ lbf} \quad (\text{applied to eave in Visual Analysis Model}) \\ \Delta &= 0.75 \text{ in} \quad (\text{resulting truss displacement in Visual Analysis Model}) \end{aligned}$$

$$k = 133.3 \text{ lbf/in} \quad (\text{bare frame stiffness})$$

3.3 Endwall Stiffness, k_e

The endwall stiffness, k_e , is calculated by summing the horizontal shear stiffness of the wall diaphragm and the bending stiffness of the endwall posts. The horizontal shear stiffness of the wall diaphragm is found by adjusting the model diaphragm shear stiffness for the actual size of endwall. The interior liner is constructed the same as the exterior siding and is included in the endwall calculations.

$$k_e = G(\cos\theta_{\text{exterior}})(b_{h,\text{exterior}}/s) + G(\cos\theta_{\text{interior}})(b_{h,\text{interior}}/s) + n(6EI/L^3)$$

	G=	2190 lbf/in	
	θ_{exterior} =	0.0 degrees	
	θ_{interior} =	0.0 degrees	
$b_{h1,\text{exterior}} = b_{h1,\text{interior}}$ =	60 ft		(length of endwall 1 minus door/window openings)
$b_{h2,\text{exterior}} = b_{h2,\text{interior}}$ =	48 ft		(length of endwall 2 minus door/window openings)
s =	16 ft		(s=length perpendicular to loading = wall height)
n_1 =	10 columns		(n_1 = number of columns in endwall 1)
n_2 =	8 columns		(n_2 = number of columns in endwall 2)
E=	1700000 psi		
I=	48 in ⁴		(moment of inertia about weak axis of each individual endwall column)
L=	16 ft		(column height)
k_{e1} =	17117 lbf/in		(stiffness of endwall with four windows)
k_{e2} =	13693 lbf/in		(stiffness of endwall with 24ft door)

3.4 Eave Load

The eave load, P_i , used in this analysis is the resultant lateral load from the controlling combination of design loads acting over the tributary area of the eave, and is applied as a concentrated load at the eave of each frame.

Seismic Eave Load, P_i = 668 lbs (see Section 2.5.5 Seismic vs. Wind Comparison)

3.5 Summary of DAFI Inputs

DAFI (Diaphragm and Frame Interaction) is a computer program for calculating the distribution of horizontal loads among the individual post-frames and roof diaphragm sections of a building. It can be used to analyze diaphragm action in buildings in which bay spacings vary, the stiffness of individual post-frames differ, endwalls are not assumed infinitely rigid, and/or the stiffness of individual diaphragms are not the same.

A windows version of this program is available as a free download from the National Frame Building Association website (nfba.org). It allows data to be entered using a special screen editor. The data can be saved to and later recalled from an input data file.

Roof Diaphragm Shear Stiffness:	39683 lbf/in	$(C_1 = C_2 = C_3 = \dots = C_{15})$
Endwall 1 Shear Stiffness:	17117 lbf/in	
Endwall 2 Shear Stiffness:	13693 lbf/in	
Interior Frame Stiffness:	133.3 lbf/in	$(k_2 = k_3 = k_4 = \dots = k_{14})$
Eave Load on Interior Frame:	668 lbf	

3.6 Summary of DAFI Outputs

DAFI FRAME ANALYSIS OUTPUTS

Frame Number	Frame Stiffness	Applied Load	Horizontal Displacement	Load Resisted Frame	by	Fraction of Applied Load
1	17117	334	0.27	4594.9		13.757
2	133.3	668	0.38	50.1		0.075
3	133.3	668	0.47	62.3		0.093
4	133.3	668	0.54	72.5		0.109
5	133.3	668	0.61	80.7		0.121
6	133.3	668	0.65	87.0		0.130
7	133.3	668	0.68	91.3		0.137
8	133.3	668	0.70	93.6		0.140
9	133.3	668	0.71	94.0		0.141
10	133.3	668	0.69	92.5		0.138
11	133.3	668	0.67	89.0		0.133
12	133.3	668	0.63	83.6		0.125
13	133.3	668	0.57	76.3		0.114
14	133.3	668	0.50	66.9		0.100
15	133.3	668	0.42	55.6		0.083
16	13693	334	0.32	4329.6		12.963

DAFI DIAPHRAGM ANALYSIS OUTPUTS

Diaphragm Number	Diaphragm Stiffness	Shear	Displacement	Shear	Load
1	39683		0.1073743		4260.94
2	39683		0.0918034		3643.03
3	39683		0.0765407		3037.37
4	39683		0.0615352		2441.9
5	39683		0.0467364		1854.64
6	39683		0.0320946		1273.61
7	39683		0.0175606		696.86
8	39683		0.0030856		122.45
9	39683		0.011379		451.55
10	39683		0.0258819		1027.07
11	39683		0.0404717		1606.04
12	39683		0.0551974		2190.4
13	39683		0.0701086		2782.12
14	39683		0.0852553		3383.18
15	39683		0.1006883		3995.61

3.7 Interpretation of DAFI Outputs

Controlling Frame Number = 9	(Resists the most load compared to other frames)
Deflection of Frame = 0.71 in	
Load Resistance by Frame = 94.0 lbf	
Resisting Force by Diaphragm, Q = 574.0 lbf	(Eave Load Minus Load Resistance by Frame)
Shear Load in Endwall 1 = 4594.9 lbf	DAFI Output: Load Resisted by Frame 1
Shear Load in Endwall 2 = 4329.6 lbf	DAFI Output: Load Resisted by Frame 16; Wall with 24ft Door
Horizontal Diaphragm Shear = 4260.9 lbf	DAFI Output: Largest Diaphragm Shear Load

3.8 Endwall Shear Strength Check

The endwalls are sheathed inside and out with *Grandrib 3*, 29 gage structural metal sheathing manufactured by Fabral. The sheathing is fastened to 2x4 girts with #10x1" screws 6" o/c at edges and 12" o/c at all intermediate framing. Though the testing was done with purlins placed on edge, it is a reasonable assumption that the purlins with flat orientation will yield equal or better results. The testing was done with 2x4 No.2 DFL *purlins*, fastened to *rafters* with (1) 60d spike and (2) 10d toenails. Table 6.1 of the PFBDM is silent on the controlling failure mode, whether it was in the wood portion of the test assembly or the steel panels. To be conservative, it is assumed that the failure is in the steel panels and the load duration factor, $C_D = 1.0$ is applied in the calculations. Ref: *Lukens & Bundy, 1987, as presented in Table 6.1 (Test Assembly #6) of the PFBDM by NFBA.*

$$k_c = G(\cos\theta_{\text{exterior}})(b_{h,\text{exterior}}/s) + G(\cos\theta_{\text{interior}})(b_{h,\text{interior}}/s) + n(6EI/L^3) \quad (\text{endwall stiffness})$$

$$G(\cos\theta_{\text{exterior}})(b_{h1,\text{exterior}}/s) = 8213 \text{ lbf/in} \quad (\text{stiffness provided by exterior sheathing of endwall 1})$$

$$G(\cos\theta_{\text{interior}})(b_{h1,\text{interior}}/s) = 8213 \text{ lbf/in} \quad (\text{stiffness provided by interior sheathing of endwall 1})$$

$$n(6EI/L^3) = 691.7 \text{ lbf/in} \quad (\text{stiffness provided by columns of endwall 1})$$

$$G(\cos\theta_{\text{exterior}})(b_{h2,\text{exterior}}/s) = 6570 \text{ lbf/in} \quad (\text{stiffness provided by exterior sheathing of endwall 2})$$

$$G(\cos\theta_{\text{interior}})(b_{h2,\text{interior}}/s) = 6570 \text{ lbf/in} \quad (\text{stiffness provided by interior sheathing of endwall 2})$$

$$n(6EI/L^3) = 553.4 \text{ lbf/in} \quad (\text{stiffness provided by columns of endwall 2})$$

$$\text{Shear Load in Endwall 1, } V_{\text{max},1} = 4595 \text{ lbf} \quad \text{DAFI Output: Load Resisted by Frame 1}$$

$$\text{Shear Load in Endwall 2, } V_{\text{max},2} = 4330 \text{ lbf} \quad \text{DAFI Output: Load Resisted by Frame 16; Wall with 24' Door}$$

$$\text{Allowable Shear Strength, } v_a = 110 \text{ lbf/ft} \quad (\text{from Table 6.1 test assembly #6 of PFBDM})$$

Endwall	Wall Component	Stiffness of Wall Component	Total Stiffness of Endwall	Load Ratio	Total Load on Component	Shear Load on Component, v_{max}	Allowable Load on Component, v_a
		(lbf/in)	(lbf/in)		(lbs)	(lb/ft)	(lb/ft)
1	Exterior Sheathing	8213	17117	0.480	2205	37	110
	Interior Sheathing	8213	17117	0.480	2205	37	110
	Bare Frame	692	17117	0.040	186	n/a	n/a
2	Exterior Sheathing	6570	13693	0.480	2077	43	110
	Interior Sheathing	6570	13693	0.480	2077	43	110
	Bare Frame	553	13693	0.040	175	n/a	n/a

$$v_{\text{max}} \leq v_a \quad \text{<ok>} \quad (\text{actual shear in endwalls is less than allowable shear})$$

3.9 Sidewall Shear Strength Check

The lateral seismic load parallel to the ridge line can be calculated by multiplying seismic lateral load on one bay by the number of bays in the building. Because seismic calculations are based on the effective weight of the structure, the direction of the load used in original seismic calculations is irrelevant. It may be noted that the weight of the gable end walls is not accounted for, however, their contribution is extremely insignificant and so is ignored in this design example.

Allowable Shear Strength, $v_{a, in}$ =	110 lbf/ft	(Exterior Sheathing, $C_D = 1.0$)
Allowable Shear Strength, $v_{a, out}$ =	110 lbf/ft	(Interior Liner, $C_D = 1.0$)
Allowable Shear Strength, v_a =	220 lbf/ft	(Interior Liner, $C_D = 1.0$)
Lateral Seismic Force at Roof, F_R =	668 lbs	(for one bay)
Number of Bays in the Building =	15	
Seismic Force, $F_{R, longitudinal}$ =	10023 lbs	(total seismic load on the building)
Effective Sidewall Length, $L_{sidewall}$ =	79 ft	(building length minus door and window openings)
Actual Shear Load, $v_{sidewall}$ =	63 lbf/ft	$F_{R, longitudinal} / L_{sidewall} / 2walls$

$v_{sidewall} \leq v_a$	<ok>	(actual shear in sidewall is less than allowable shear)
-------------------------	------	---------------------------------------------------------

In a typical post-frame building endwalls are the controlling shear walls. There are cases, however, when sidewalls are the critical shear walls, especially in wide buildings that are short in length. In those situations a more thorough analysis is required, in which a roof and sidewall stiffness and possibly torsional effects on the overall building envelope are considered.

3.10 Roof Diaphragm Shear Strength Check

The roof is also sheathed with *Grandrib 3*, 29 gage structural metal sheathing manufactured by Fabral. The sheathing is fastened to 2x4 purlins with #10x1" screws 6" o/c at edges and 12" o/c at all intermediate framing. The testing was done with 2x4 No.2 DFL purlins, fastened to rafters with (1) 60d spike and (2) 10d toenails. Ref: *Lukens & Bundy, 1987, as presented in Table 6.1 (Test Assembly #6) of the PFBDM by NFBA.*

Allowable Shear Strength, v_a =	110 lbf/ft	
Max Shear, $V_{max, horizontal}$ =	4261 lbf	<i>DAFI Diaphragm Analysis Output</i>
$C_{h, roof} = c_{h,1 roof} + c_{h,2, roof} =$	19973 lbf/in	(horizontal stiffness provided by roof sheathing)
$C_{h, ceiling} =$	19710 lbf/in	(horizontal stiffness provided by ceiling sheathing)
$L_{s, roof} =$	79.17 ft	(width of building plus overhangs divided by cosine of roof angle)
$L_{s, ceiling} =$	72 ft	(width of building)
$v_{max, inplane} = V_{max, in-plane} / L_s$		(shear load in plane of the roof or ceiling sheathing)

Roof Component	Horizontal Stiffness of Component (lbf/in)	Total Horizontal Stiffness of Diaphragm (lbf/in)	Load Ratio	Max Horizontal Load on Component, $V_{max, horizontal}$ (lbs)	θ (deg)	Max Load in Plane of Component, $V_{max, in-plane}$ (lbs)	Shear Load in Plane of Component, v_a (lb/ft)	Allowable Shear Load, v_a (lb/ft)
Roof Sheathing	19973	39683	0.503	2145	16.26	2234	28	110
Ceiling Sheathing	19710	39683	0.497	2116	0	2116	29	110

$v_{max} \leq v_a$	<ok>	(actual shear in diaphragm is less than allowable shear)
--------------------	------	----------------------------------------------------------

3.11 Roof Diaphragm Chord Forces

The roof diaphragm acts like a deep beam where the ends of the beam are assumed to be fixed. The diaphragm of this building consists of two individual roof diaphragms, one on each side of ridge, and of one ceiling diaphragm. The bending forces in each diaphragm are resisted by roof purlins and ceiling joists. Because only the edge purlins and ceiling joists are fastened together to provide a continuous tensile resistance to tension chord of diaphragms, it can be conservatively assumed that the intermediate purlins and ceiling joists provide zero contribution to the bending resistance of diaphragms. The load is applied to the diaphragm at eave, and redistributed to individual diaphragms according to their stiffness. For simplicity, the load resistance contribution of frames is ignored. The purlins are fastened together at splices with a single [HTP37Z Simpson](#) plate fastened to the side with 10dx1-1/2 nails; the ceiling joists are fastened together at splice with a single [MSTA21 Simpson](#) strap, fastened to the bottom edge with 10dx3" nails.

It should be noted that typical roof diaphragm deflection consists of bending deflection, shear deflection of sheathing panel, deflection due to nail slip, and deflection due to slip in chord connection splices. Because the diaphragm stiffness in this example is based on a sample test, it can be assumed that all of these deflection contributors, with exception of the deflection due to slip in chord connection splices, are accounted for. It is further assumed that the deflection due to slip in chord connection splices is minimal and is an insignificant contributor to the overall diaphragm deflection.

$$c_{h,1 \text{ roof}} = 9986 \text{ lbf/in} \quad (\text{horizontal stiffness of roof diaphragm 1})$$

$$c_{h,2 \text{ roof}} = 9986 \text{ lbf/in} \quad (\text{horizontal stiffness of roof diaphragm 2})$$

$$c_{h,3 \text{ ceiling}} = 19710 \text{ lbf/in} \quad (\text{horizontal stiffness of ceiling diaphragm})$$

$$C_h = 39683 \text{ lbf/in} \quad (\text{total combined horizontal stiffness diaphragm})$$

$$\text{Load Ratio to Each Diaphragm, } p = c_{h,x}/C_h \quad (\text{individual diaphragm stiffness divided by total diaphragm stiffness})$$

$$\text{Load on Roof Diaphragm, } w = 83.52 \text{ lbf/ft} \quad (\text{eave load, } P_i, \text{ divided by column spacing, } s)$$

$$\text{End Moment, } M_{ex} = w_x L_x^2 / 12 \quad (\text{controls the design})$$

$$\text{Midspan Moment, } M_{mx} = w_x L_x^2 / 24 \quad (\text{moment equation for a beam with fixed end supports})$$

$$\text{Tension/ Compression Force, } T_x = M_{ex} / b_x \quad (\text{controlling moment divided by depth of individual diaphragm})$$

$$\text{HTP37z Simpson Strap} = 1600 \text{ lbs} \quad (\text{allowable tension capacity as specified by the manufacturer})$$

$$\text{MSTA21 Simpson Strap} = 1505 \text{ lbs} \quad (\text{allowable tension capacity as specified by the manufacturer})$$

Diaphragm Component	Load Ratio, p	Diaphragm Length, L_x (ft)	Diaphragm Depth, b_x (ft)	Load on Diaphragm, w_x (lb/ft)	Moment in Diaphragm, M_x (lb-ft)	Tension/ Compression Force, T_x (lbs)	Allowable Tension Load T_a (lbs)
Roof Diaphragm 1	0.252	120	38	21.0	25223	664	1850
Roof Diaphragm 2	0.252	120	38	21.0	25223	664	1850
Ceiling Diaphragm	0.497	120	72	41.5	49783	691	1505

$$P_x \leq P_a \quad \text{<ok>} \quad (\text{actual tension load is less than allowable tension load})$$

Section 4: Post Design

4.1 Loads

Dead Load, $DL =$	9 psf	
Roof Live Load, $LL_r =$	20 psf	
Snow Load, $SL_{\text{balanced}} =$	11.6 psf	
Snow Load, $SL_{\text{unbalanced, windward}} =$	3.5 psf	
Snow Load, $SL_{\text{unbalanced, leeward}} =$	26.4 psf, for a distance of 8.5 ft from ridge, then 11.6 psf	
Seismic Eave Load, $P_i =$	955 lbs	(Lateral seismic force, F_R , applied at eave of frame)
Diaphragm Restraining Force, $Q_s =$	-820 lbs	(need to modify diaphragm resistance if it is to be presented as seismic load $Q_s = Q/0.7$)
Resulting Load $P_r =$	135 lbs	

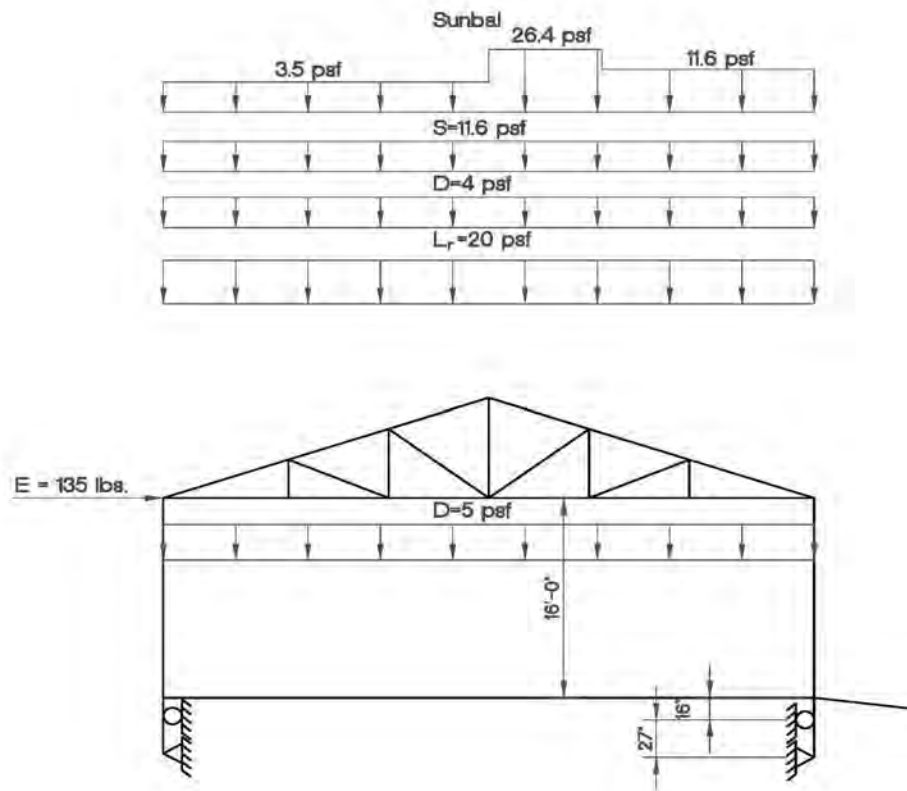


Figure 4A. Structural analog of post-frame with non-constrained foundation.

4.2 Results

Detailed post design calculations are provided in Appendix A

Post Size =	3-ply 2x8, nail laminated post with structural glued finger joints	
Grade =	#1 southern yellow pine, pressure preservative treated at the embedded end	
$V_{\text{max, dry}} =$	33.7 lb	(at grade)
$M_{\text{max, dry}} =$	539 lb-ft	(at grade)
$V_{\text{max, wet}} =$	261 lb	(below grade)
$M_{\text{max, wet}} =$	569 lb-ft	(below grade)
$P_{\text{max}} =$	9092 lb	($D+L_r$)
Actual/Allowable Unity =	0.45	(controlling axial load combination: $D+L_r$)

The post is sized adequately for the required loading

Section 5: Foundation Design

ANSI/ASAE EP486.1 Defines non-constrained foundation as a case in which "Post foundation rotation and horizontal The post embedment below grade is 4 ft; a 24" diameter x 24" high concrete collar is poured around the post, on top of

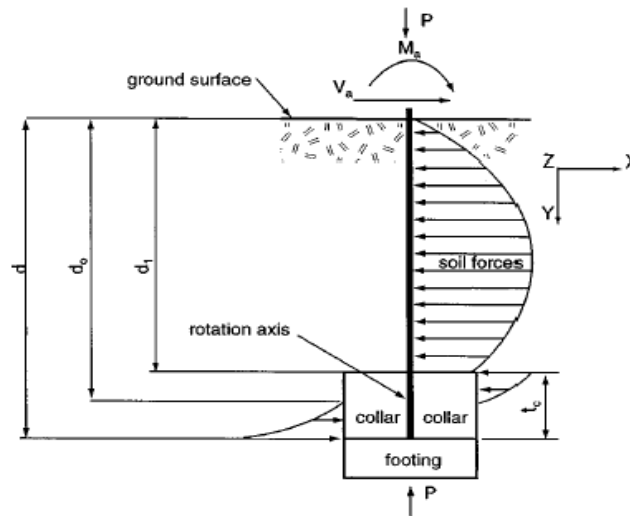


Figure 5A. FBD of non-constrained post foundation with concrete collar from ANSI/ASAE EP486.1.

5.1 Post Reactions at Grade Level

$P_D = 3012$ lbs	(Vertical dead load reaction from Visual Analysis model)
$P_{Lr} = 6080$ lbs	(Vertical roof live load reaction from Visual Analysis model)
$P_S = 5985$ lbs	(Vertical snow load reaction from Visual Analysis model)
$Q_{\text{uplift}} = 2959$ lbs	(Wind service load on footing calculated using tributary area)
$V_a = V_{\text{max, dry}} = 33.7$ lbs	(Shear force applied at grade level, from Visual Analysis model)
$M_a = M_{\text{max, dry}} = 539$ lbs	(Moment reaction applied at grade level, from Visual Analysis model)
$P = P_{\text{max}} = 9092$ lb	(Vertical foundation load, $D+L_r$)
$Q_{\text{net}} = 1152$ lb	(Net uplift load on foundation, $0.6D+W$)

5.2 Results

Detailed foundation calculations are provided in Appendix B

Minimum Post Embedment, $d = 2.3$ ft	(minimum calculated embedment depth)
Actual Post Embedment, $d_a = 4.0$ ft	(calculated)
Applied Vertical Soil Pressure, $S_a = 2894$ psf	(calculated)
Allowable Soil Pressure, $S_v = 3867$ psf, (93.33 % increases are applied per EP486.1, Table 1, Footnote 4)	
Calculated Uplift Resistance, $U = 2395$ lbs	(resistance consists of weight of concrete collar + weight of soil cone)
$d \leq d_a$ <ok>	(required post embedment is less than actual post embedment)
$S_a \leq S_v$ <ok>	(actual vertical soil bearing pressure is less than the allowable)
$Q_{\text{net}} \leq U$ <ok>	(net uplift load is less than the calculated footing uplift resistance)

Section 6: Connections

Detailed Connection Calculations Are Provided In Appendix C

6.1 List of Critical Connections

In this example only connections critical to the lateral force resistance system are analyzed:

- Truss to Post Connection - Vertical shear due to truss uplift
- Truss to Post Connection - Horizontal shear due to post top end reactions
- Truss to Header Connection - Truss uplift
- Endwall Ceiling Ledger to Posts Connection - Shear between ceiling diaphragm and end shear wall
- Skirt Board to Posts Connection - Shear between shear wall and posts at grade level
- Purlin to Purlin at Splice Connection - Tension load between diaphragm tension chords
- Ceiling Joist to Ceiling Joist at Splice Connection - Tension load between diaphragm tension chords
- Post to Concrete Collar Connection - Vertical shear due to post uplift

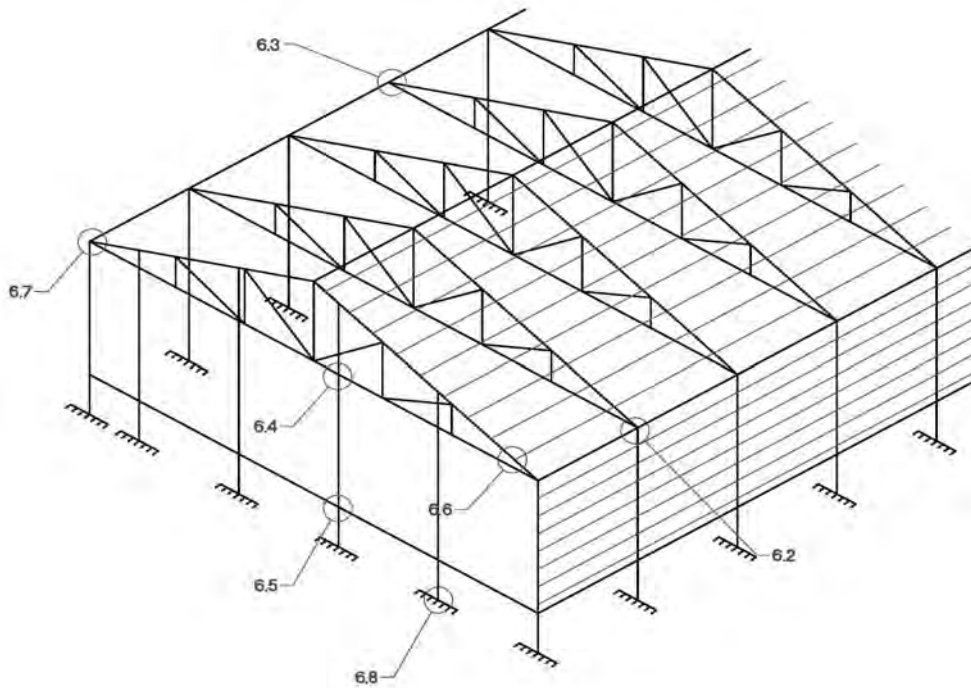


Figure 6A. Critical connection locations.

6.2 Truss to Post Connection

Trusses are placed in a pocket created by notching the center lamination at top of post. The exterior laminations are then extended to the top of top chord of truss, and fastened to truss with (5) 16d common wire nails on each side. This connection is designed to resist vertical and horizontal shear loads. The vertical shear load, or load from truss uplift, can be calculated using tributary areas and roof wind pressures, or can be provided by a truss designer. In this example the uplift loads are calculated using tributary areas and wind pressures.

Vertical Shear Load and Design:

Tributary Width =	304 ft ²	
Roof Wind Pressures, q_{wr} =	-9.7 psf	
Effective Dead Load =	4.5 psf	(50% of design dead load)
Net Uplift Force =	-2139 lbs	(0.6D+W)
Allowable Shear Capacity =	2275 lbs	(Detailed connection calculations are provided in Appendix C)

Horizontal Shear Load and Design:

The end shear at top of post equals the calculated horizontal reaction of the critical frame. This reaction is the sum of two (2)

$$\text{Seismic Eave Load, } P_i = 955 \text{ lbs}$$

$$\text{Diaphragm Restraining Force, } Q_s = -820 \text{ lbs}$$

$$\text{Shear at Top of Frame 9 Post, } V_9 = 67 \text{ lbs}$$

$$\text{Allowable Shear Capacity} = 2275 \text{ lbs}$$

$$V_9 = (P_i + Q_s) / 2 \text{ posts (this is service load)}$$

(Detailed connection calculations are provided in Appendix C)

The results show that seismic load does not control this connection design. Please refer to the Wind-Governed Design example for wind controlled design.

6.3 Truss to Glulam Header Connection

Truss is fastened to glulam header with (2) H10A Simpson hurricane ties, one tie on each side of beam. The specific gravities of

$$\text{Tributary Width} = 304 \text{ ft}^2$$

$$\text{Roof Wind Pressures, } q_{wr} = -9.7 \text{ psf}$$

$$\text{Effective Dead Load} = 4.5 \text{ psf}$$

(50% of design dead load)

$$\text{Net Uplift Force} = -2139 \text{ lbs}$$

(0.6D+W)

$$\text{Allowable Shear Capacity} = 2280 \text{ lbs}$$

(Detailed connection calculations are provided in Appendix C)

6.4 Endwall Ceiling Ledger to Posts Connection

The shear load from roof diaphragm is transferred to endwall truss and then to wall sheathing. This load path does not directly rely

$$\text{Number of Posts} = 10$$

$$\text{Number of 16d Nails per Post} = 4$$

$$\text{Maximum Shear, } V_{\text{max, horizontal}} = 2116 \text{ lbs}$$

($V_{\text{max, horiz.}}$ from Section 3.10 Roof Diaphragm Shear Strength Check)

$$\text{Allowable Shear Capacity} = 9830 \text{ lbs}$$

(Detailed connection calculations are provided in Appendix C)

6.5 Endwall Skirt Board to Post Connection

There is a #2 SYP skirt board on the exterior side of end wall, and a 2x4 #1 SYP bottom girt on the interior side of end wall. The

$$\text{Number of Posts} = 10$$

$$\text{Number of 16d Nails per Post} = 4$$

$$\text{Maximum Shear, } V_{\text{max, exterior}} = 2205 \text{ lbs}$$

(V_{max} from Section 3.8 Endwall Shear Strength Check)

$$\text{Maximum Shear, } V_{\text{max, interior}} = 2205 \text{ lbs}$$

(V_{max} from Section 3.8 Endwall Shear Strength Check)

$$\text{Allowable Shear Capacity, } V_a = 6881 \text{ lbs}$$

(Detailed connection calculations are provided in Appendix C)

6.6 Purlin to Purlin at Splice and to Endwall Truss Connection

The edge purlins serve as tension and compression chords of the roof diaphragm. In order to provide a continuity in tensile resistance in the tension chord of the diaphragm, the purlins must be fastened together at each splice. In this design a HTP37Z Simpson strap is used at each purlin splice. The edge purlin must also be fastened to endwall truss to transfer loads into sidewall sheathing. In addition to (1) 60d R.S. nail, a 2x4x10 inch wood block is attached to truss and purlin. To provide adequate withdrawal capacity, the block is attached to truss with (8) #8x3" wood screws, four (4) screws at top of top chord of truss and four (4) screws at bottom of top chord. The purlin is fastened to the block with (5) 16d nails. This connection must be at all edge purlins on each side of the ridge line; there are the total of four (4) purlins and eight (8) of such connections in the building.

Maximum Tension Force, T_{\max} = 664 lbs *(T_x from Section 3.11 Roof Diaphragm Chord Forces)*

Allowable Tension Capacity, T_a = 1850 lbs *(Detailed connection calculations are provided in Appendix C)*

Allowable Tension Capacity, T_a = 1513 lbs

6.7 Ceiling Joist to Ceiling Joist Splice and to Corner Post Connection

The edge ceiling joists serve as tension and compression chords of the ceiling diaphragm. In order to provide a continuity in tensile resistance in the tension chord of the diaphragm, the ceiling joists must be fastened together at each splice. In this design a MSTA21 Simpson strap is used at each purlin splice. The edge ceiling joists must also be fastened to corner posts to transfer loads into sidewall sheathing.

Maximum Tension Force, T_{\max} = 691 lbs *(T_x from Section 3.11 Roof Diaphragm Chord Forces)*

Allowable Tension Capacity, T_a = 1505 lbs *(Detailed connection calculations are provided in Appendix C)*

Allowable Tension Capacity, T_a = 1474 lbs

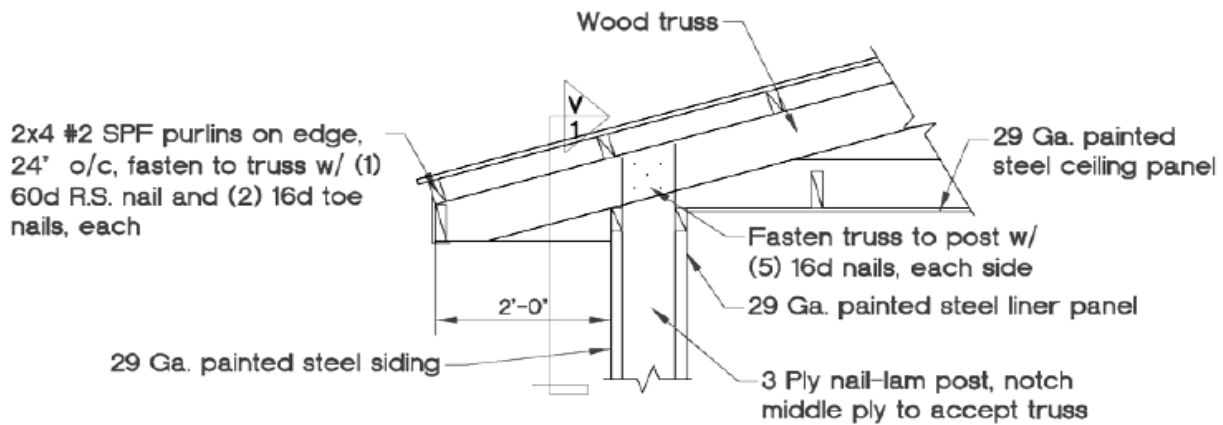
6.8 Post to Concrete Collar Connection

Each post is connected below grade to a concrete collar (backfill) with (1) #4 x 16 inch hot dipped galvanized rebar. This connection

Net Uplift Force, Q_{net} = 1152 lbs *(from Section 5.1 Post Reactions at Grade Level)*

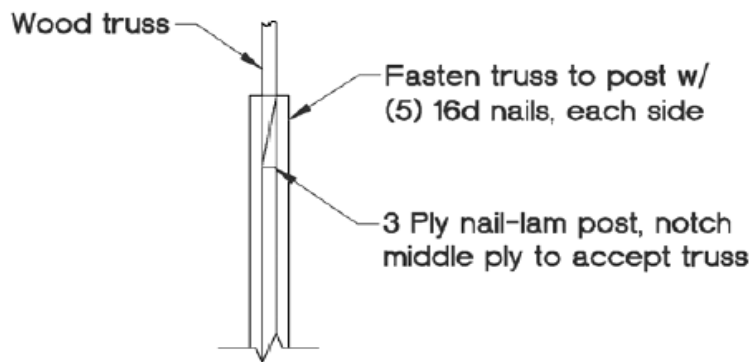
Number of Reinforcing Bars = 1

Allowable Shear Capacity, $Z' = Z(C_D)(C_M) = 1720(1.6)(0.7) = 1926$ lbs



Truss Connection

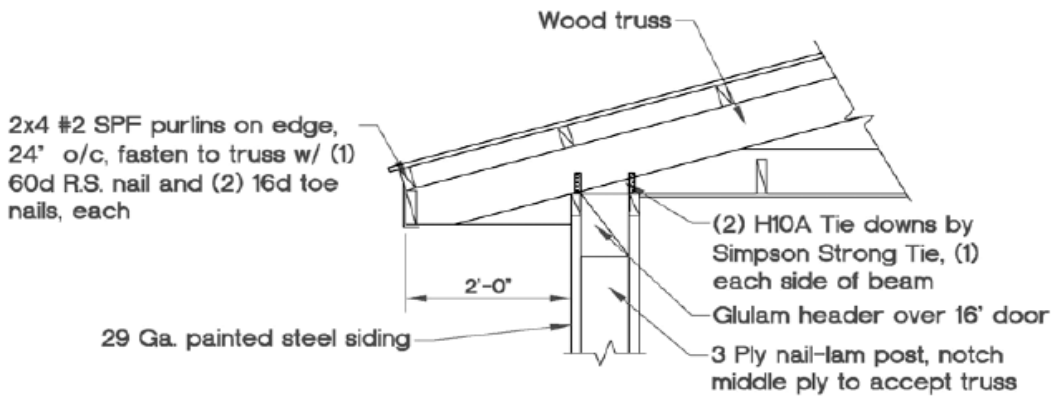
Scale 1/2" = 1'-0"



View 1

Truss Connection

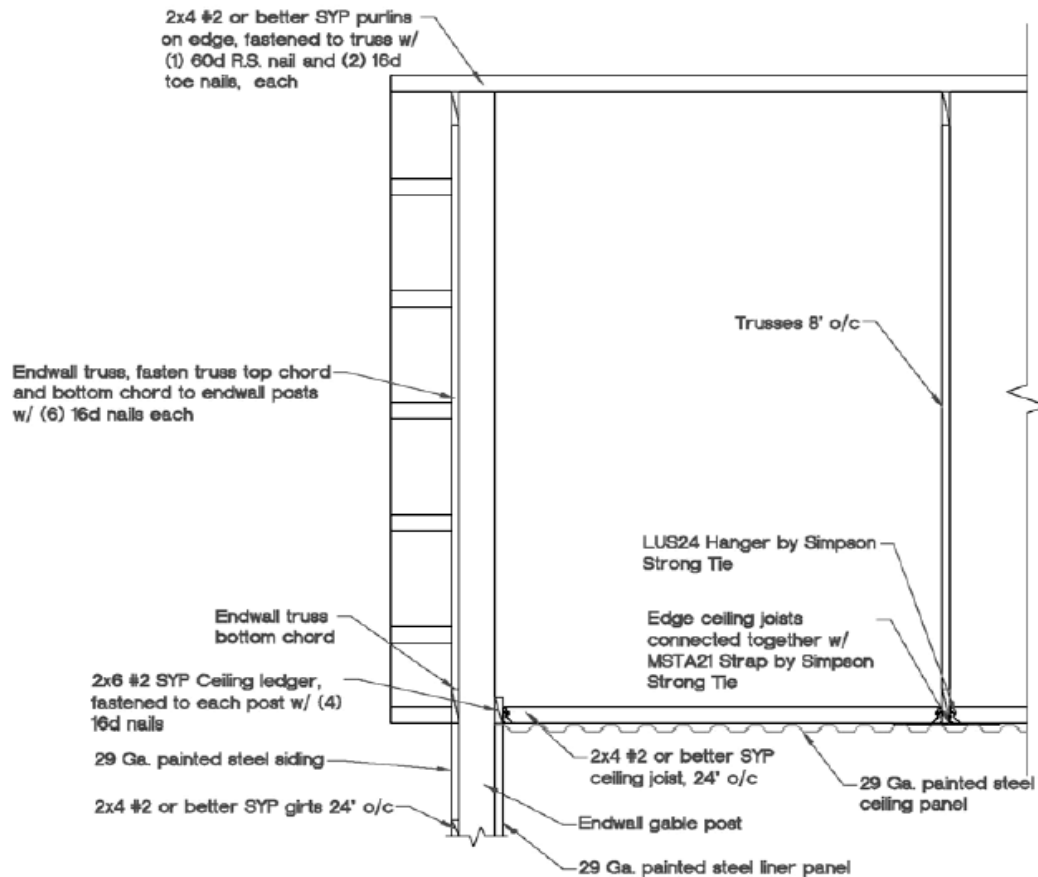
Scale 1/2" = 1'-0"



Truss Connection

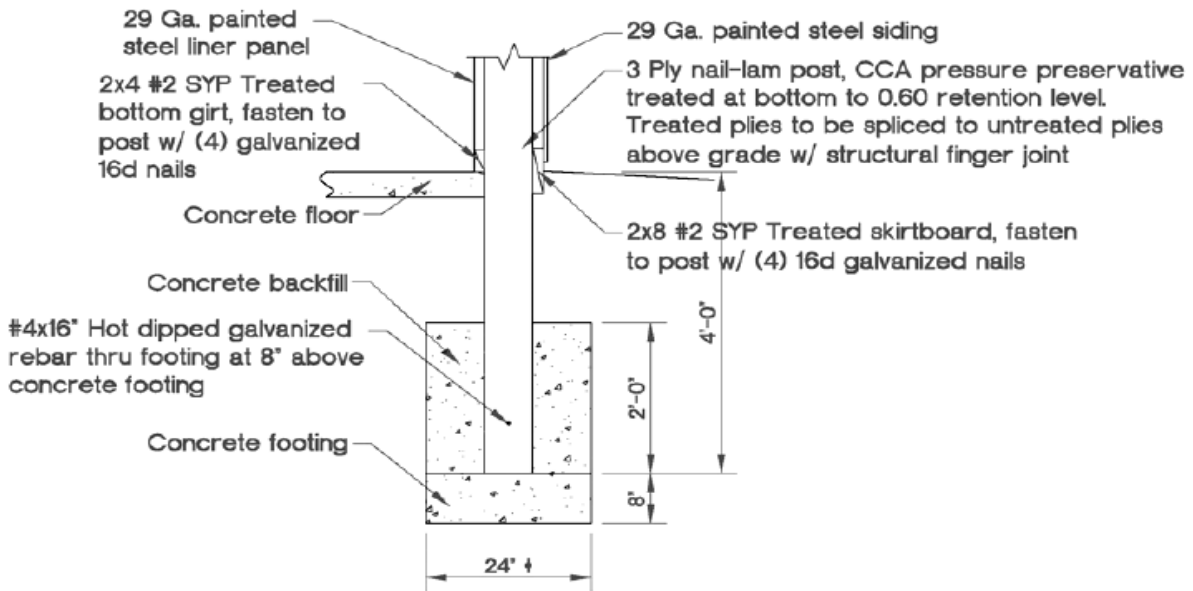
Scale $\frac{1}{2}'' = 1'-0''$

At glulam beam



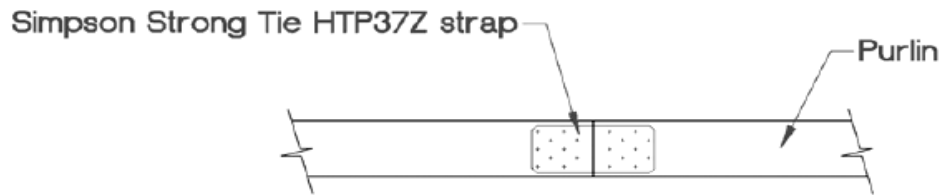
Endwall Detail

Scale $\frac{1}{2}'' = 1'-0''$



Foundation Detail

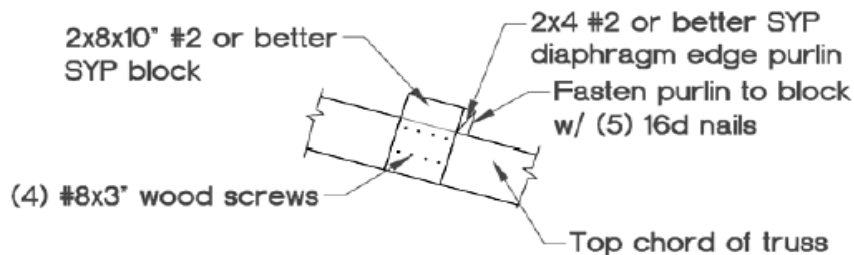
Scale $\frac{1}{2}$ " = 1'-0"



Connection Detail

Scale 1" = 1'-0"

Edge Purlin to Purlin Splice



Connection Detail

Scale $\frac{1}{2}$ " = 1'-0"

Edge Purlin To Endwall Truss Connection

Section 7: Purlin & Girt Design

7.1 Purlin Design

2x4 #2 S. Pine on edge @ 24" on center (Design details are provided in Appendix D).

Purlins are positioned on edge on top of top chord of truss and typically span over two spans for the total length of 16ft. Due to the sloping roof, the gravity loads are not aligned with the strong axis of the purlin. At the same time, the purlin can only move about its strong axis as the movement about the weak axis is restricted by the attached rigid roof panels. Thus, the gravity loads on the purlin should be broken down into strong axis and weak axis components, or 'y' and 'x' components on the sloping coordinate system. The results of the design are provided in the Appendix D.

Dead Load, DL =	2.5 psf	<i>(on surface area, see Roof Dead Load Section)</i>
Roof Live Load, LL _r =	20 psf	<i>(on horizontal projection, does not control)</i>
Snow Load, SL _{balanced} =	11.6 psf	<i>(on horizontal projection)</i>
Snow Load, SL _{unbalanced, windward} =	3.5 psf	<i>(on horizontal projection)</i>
Snow Load, SL _{unbalanced, leeward} =	26.4 psf,	<i>for a distance of 8.5 ft from ridge, then 11.6 psf</i>
Interior Roof Wind Load, q _{interior} =	-11.7 psf	<i>(component and cladding)</i>
Edge Roof Wind Load, q _{edge} =	-19.2 psf	<i>(component and cladding)</i>
Corner Roof Wind Load, q _{corner} =	-28.9 psf	<i>(component and cladding)</i>
Deflection Criterion = 1/150 and 1/120 for Live and Dead + Live Loads		
Roof angle, θ _r =	16.3 degrees	

STRONG AXIS LOADING ON PURLIN

Purlin	θ _r	Spacing	Dead Load	Roof Live Load	Balanced Snow Load	Unbalanced Snow Load	Interior Wind Load	Edge Wind Load	Corner Wind Load
	(deg)	(ft)	(lb/ft)	(lb/ft)	(lb/ft)	(lb/ft)	(lb/ft)	(lb/ft)	(lb/ft)
Typical Purlin	16.3	2	5	38.4	22.2	50.8	-23.3	-38.4	-57.7
Purlin in Unbalanced Snow Area	16.3	2	5.00	38.4	22.2	50.8	-23.3	-38.4	-57.7

7.2 Girt Design

2x4 #2 S. Pine flat against posts @ 24" on center continuous over two spans (Design details are provided in Appendix D).

Interior Wall Wind Load, q _{interior} =	-13.7 psf	<i>(component and cladding)</i>
Edge Wall Wind Load, q _{edge} =	-16.4 psf	<i>(component and cladding)</i>
Deflection Criterion = 1/90 <i>(IBC 2009, 1604.3, Footnote a)</i>		

Section 8: Other Design Considerations

This design example focused on resistance to lateral loads. Some other important connections not contained in this example, may include:

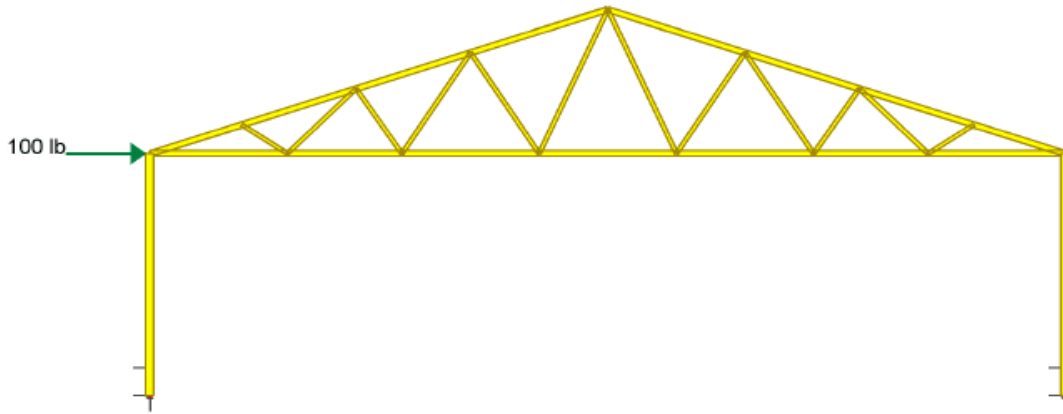
- Overhead Door Header(s) to Post Connection - Vertical and Horizontal shear
- Girt to Post Connection - Withdrawal due to wind suction
- Purlin to Truss Connection -Withdrawal (uplift) due to wind suction
- Roof Sheathing to Purlins Connection -Withdrawal (uplift) due to wind suction
- Wall Sheathing to Girts Connection - Withdrawal due to wind suction

It is also important to note that the truss design will be performed by the truss designer using the loading and geometry provided by the building designer. Guidelines for handling, bracing, and installing metal plate connected wood trusses are contained in the Building Component Safety Information (BCSI) booklet published jointly by TPI and WTCA. The truss bracing design for this building should take into account the bottom chord and compression web lateral restraint requirements shown on the truss design drawings, as well as the on center spacing of the trusses.

APPENDIX A
FRAME STIFFNESS
POST DESIGN

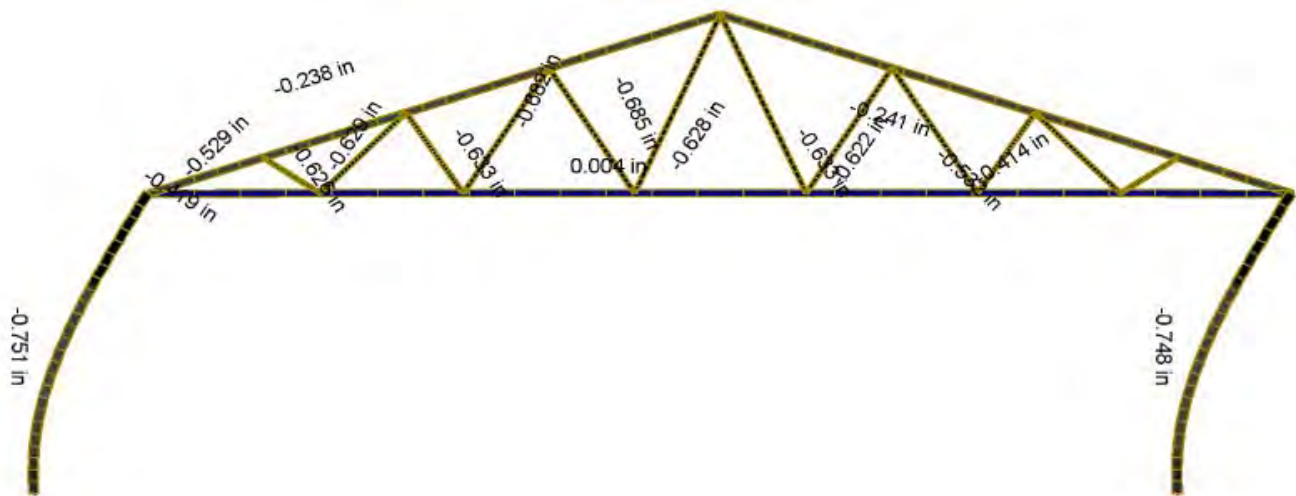
A.1 Frame Stiffness

Frame Stiffness
Timber Tech Engineering, Inc., Dmitry Reznik
Oct 04, 2010; 01:06 PM
Load Case: L
IES VisualAnalysis 7.00.0012



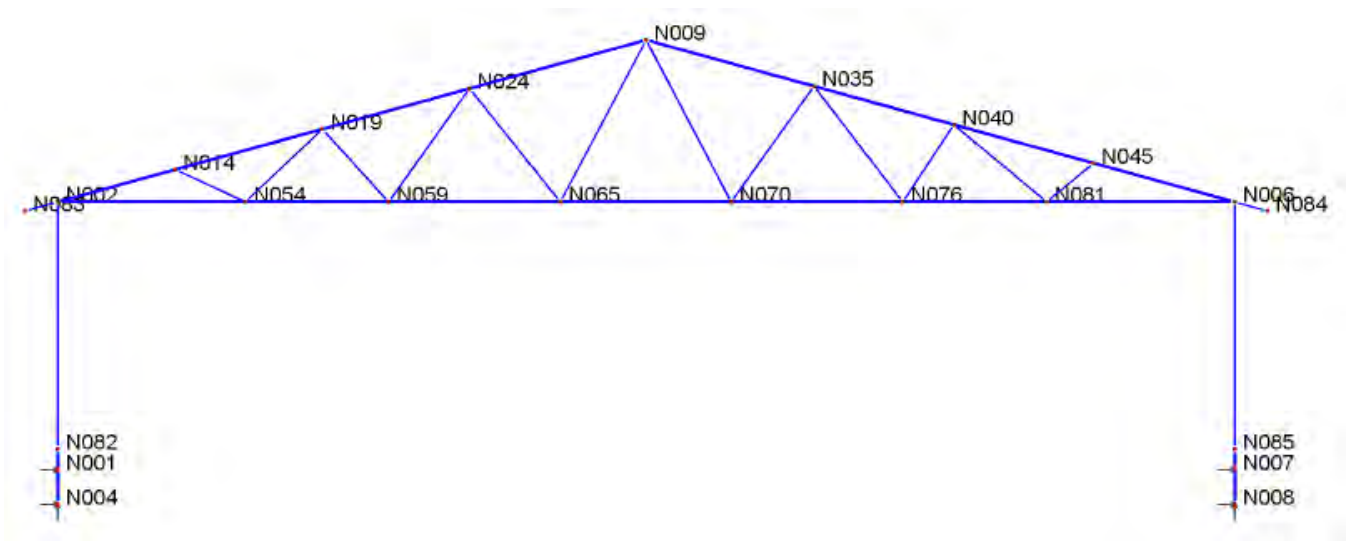
LOADING ON FRAME

Frame Stiffness
Timber Tech Engineering, Inc., Dmitry Reznik
Oct 04, 2010; 01:09 PM
Result Case: L
Member dy, deflection
IES VisualAnalysis 7.00.0012

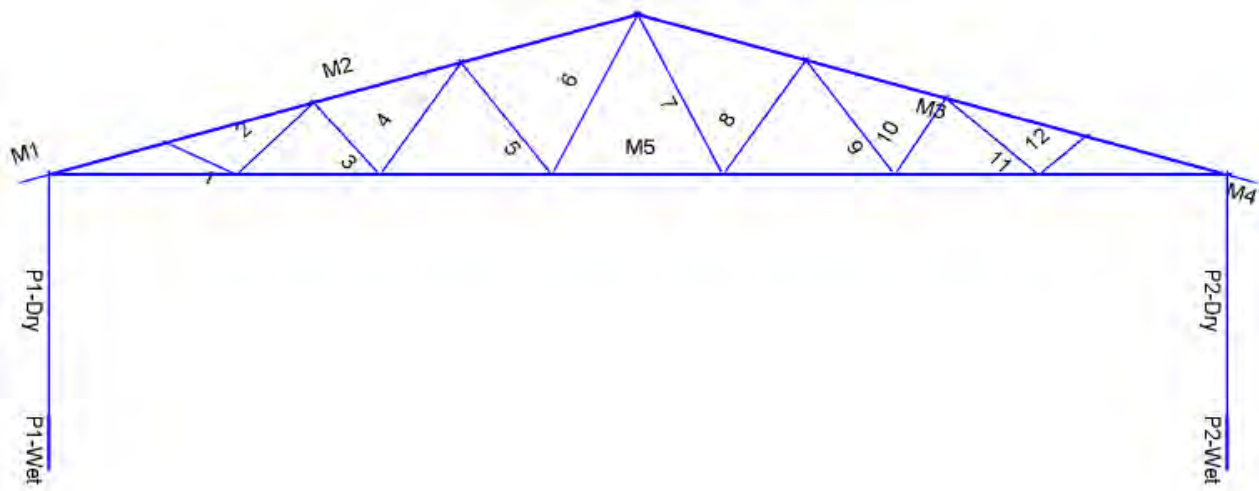


FRAME DEFLECTION FROM THE APPLIED 100 LB HORIZONTAL EAVE LOAD
(Eave Deflection = 0.751 in)

A.2 Post Design

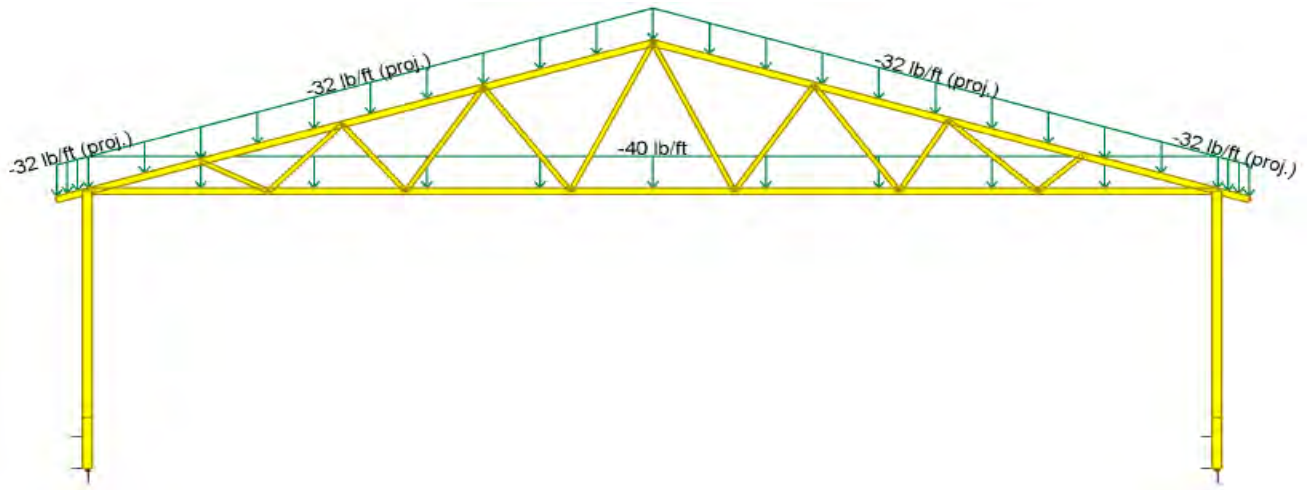


NODE NAMES
(IES Visual Analysis 7.0)



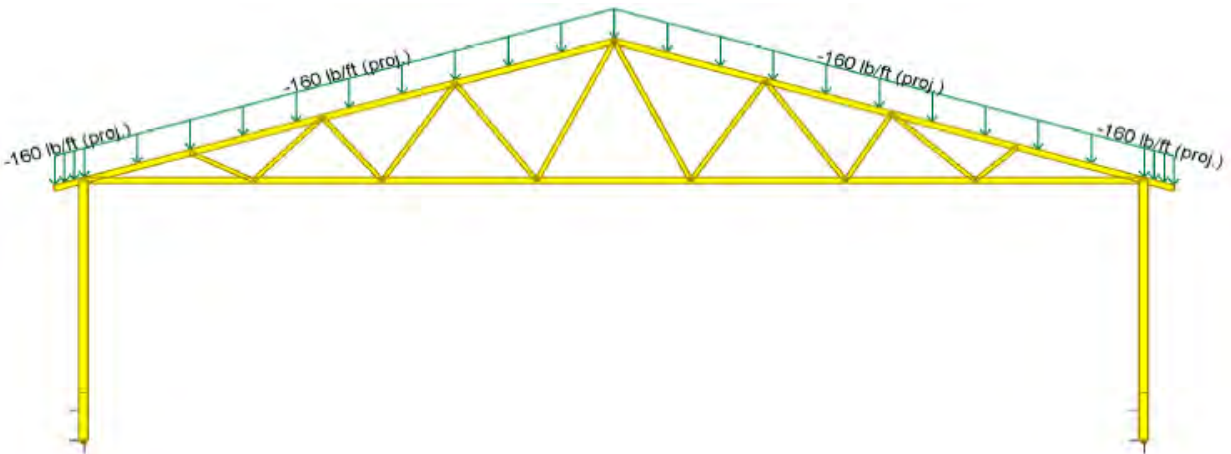
MEMBER NAMES
(IES Visual Analysis 7.0)

Sidewall Post Design.vap
Timber Tech Engineering, Inc., Dmitry Reznik
Oct 04, 2010; 12:34 PM
Load Case: Dead Load
IES VisualAnalysis 7.00.0012



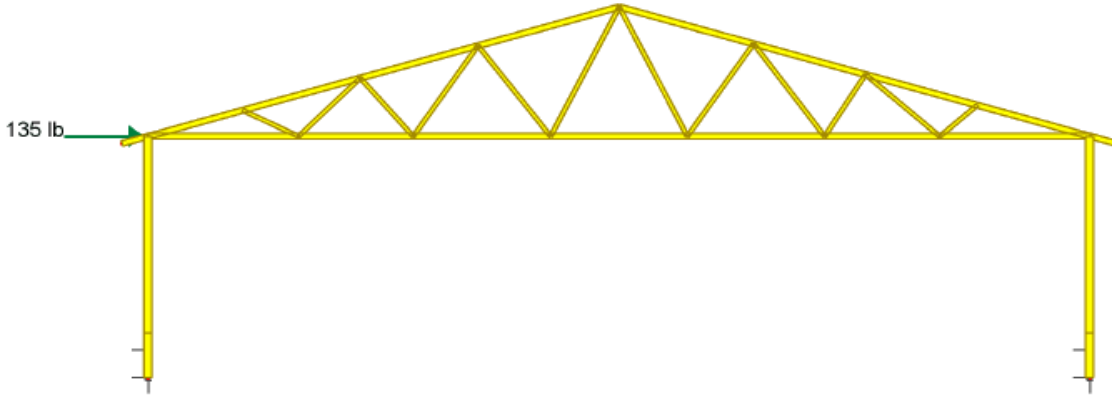
DEAD LOADS
(IES Visual Analysis 7.0)

Sidewall Post Design.vap
Timber Tech Engineering, Inc., Dmitry Reznik
Oct 04, 2010; 12:35 PM
Load Case: Roof Live Load
IES VisualAnalysis 7.00.0012



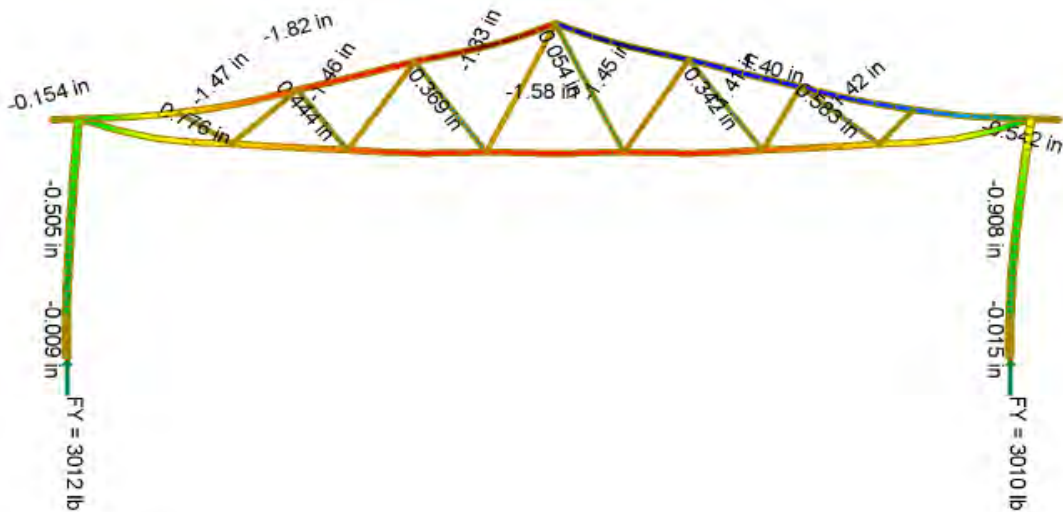
ROOF LIVE LOADS
(IES Visual Analysis 7.0)

Sidewall Post Design - Seismic.vap
 Timber Tech Engineering, Inc., Dmitry Reznik
 Oct 06, 2010; 11:16 AM
 Load Case: E+X
 IES VisualAnalysis 7.00.0012



SEISMIC LOAD
 (IES Visual Analysis 7.0)

Sidewall Post Design - Seismic.vap
 Timber Tech Engineering, Inc., Dmitry Reznik
 Oct 06, 2010; 11:19 AM
 Result Case: D+H+F+0.7E »+X
 Member dy, deflection
 IES VisualAnalysis 7.00.0012

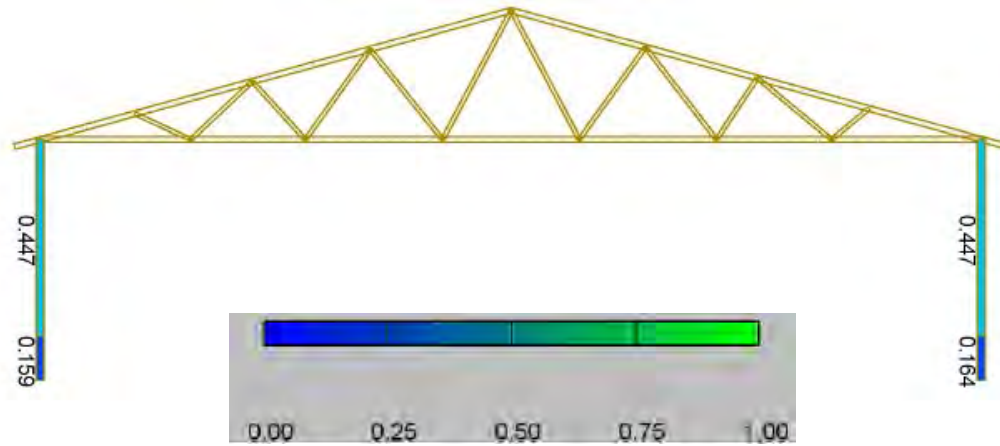


DEFLECTIONS DUE TO SEISMIC LOAD

Dead Load + 0.7Seismic Load

(IES Visual Analysis 7.0)

Sidewall Post Design - Seismic.vap
Timber Tech Engineering, Inc., Dmitry Reznik
Oct 06, 2010; 11:17 AM
Design View, Unity Checks
IES VisualAnalysis 7.00.0012



UNITY CHECK

*Applied Stresses from Controlling Load Combination Divided by Adjusted Allowable Stresses
(IES Visual Analysis 7.0)*

Sidewall Post Design - Seismic.vap

Company: Timber Tech Engineering, Inc. Engineer: Dimitry Reznik
VisualAnalysis 7.00 Report

Table of Contents

Project Header
Table of Contents
Load Cases
Load Combination Summary
Section Properties
Member Elements
Design Groups
Nodal Loads
Member Uniform Loads
Nodal Displacements
Member Unity Checks
Design Group Results
Nodal Reactions

Load Cases

Load Case	Design Checks	Seismic Type	Results	Analyze?	Envelope?
(1)Dead Load	-NA-	-NA-	Yes	Yes	No
(2)E+X	-NA-	-NA-	Yes	Yes	No
(11)Roof Live Load	-NA-	-NA-	Yes	Yes	No
(18)0.6D+0.7E+H »+X	Allowable (ASD)	-NA-	Yes	Yes	No
(19)D+F	Allowable (ASD)	-NA-	Yes	Yes	No
(20)D+H+F+0.75(0.7E+L+Lr)	Allowable (ASD)	-NA-	Yes	Yes	No
(21)D+H+F+0.75(0.7E+L+R)	Allowable (ASD)	-NA-	Yes	Yes	No
(22)D+H+F+0.75(L+Lr)	Allowable (ASD)	-NA-	Yes	Yes	No
(23)D+H+F+0.7E »+X	Allowable (ASD)	-NA-	Yes	Yes	No
(24)D+H+F+Lr	Allowable (ASD)	-NA-	Yes	Yes	No

Load Combination Summary

Equation Combination: 0.6D+0.7E+H »+X

Combination: 0.60D + H*0 + 0.70E+X

Contributing Cases & Source

Dead Load (D)

E+X (E+X)

Equation Combination: D+F

Combination: D + F*0

Contributing Cases & Source

Dead Load (D)

Equation Combination: D+H+F+0.75(0.7E+L+Lr) »+X

Combination: D + 0.75L*0 + 0.75Lpa*0 + 0.75Lr + H*0 + F*0 + 1.50Fa*0 + 0.53E+X

Contributing Cases & Source

Dead Load (D)

E+X (E+X)

Roof Live Load (Lr)

Equation Combination: D+H+F+0.75(0.7E+L+R) »+X

Combination: D + 0.75L*0 + 0.75Lpa*0 + 0.75R*0 + H*0 + F*0 + 1.50Fa*0 + 0.53E+X

Contributing Cases & Source

Dead Load (D)

E+X (E+X)

Equation Combination: D+H+F+0.75(L+Lr)

Combination: D + 0.75L*0 + 0.75Lpa*0 + 0.75Lr + H*0 + F*0 + 0.75T*0

Contributing Cases & Source

Dead Load (D)

Roof Live Load (Lr)

Equation Combination: D+H+F+0.7E »+X

Combination: D + H*0 + F*0 + 1.50Fa*0 + 0.70E+X

Contributing Cases & Source

Dead Load (D)

E+X (E+X)

Equation Combination: D+H+F+Lr

Combination: D + Lr + H*0 + F*0

Contributing Cases & Source

Dead Load (D)

Roof Live Load (Lr)

Section Properties

Section	Theta deg	Ax in^2	Iz ft^4	Sz(+y) ft^3	Sz(-y) ft^3
3plyX8	0.00	30.99	0.01	0.02	0.02
SS2x4	0.00	5.25	0.00	0.00	0.00
SS2x6	0.00	8.25	0.00	0.00	0.00

Member Elements

Member	Section	Material (1)	Node (2)	Length	Rz1	Rz2	One Way Framing
--------	---------	--------------	----------	--------	-----	-----	-----------------

ft

1	SS2x4	Southern N054	N014	4.79	Free	Free	Normal	Bracing
2	SS2x4	Southern N054	N019	6.66	Free	Free	Normal	Bracing
3	SS2x4	Southern N059	N019	6.22	Free	Free	Normal	Bracing
4	SS2x4	Southern N059	N024	8.86	Free	Free	Normal	Bracing
5	SS2x4	Southern N065	N024	9.21	Free	Free	Normal	Bracing
6	SS2x4	Southern N065	N009	11.74	Free	Free	Normal	Column
7	SS2x4	Southern N070	N009	11.72	Free	Free	Normal	Column
8	SS2x4	Southern N070	N035	9.06	Free	Free	Normal	Bracing
9	SS2x4	Southern N076	N035	9.26	Free	Free	Normal	Bracing
10	SS2x4	Southern N076	N040	5.90	Free	Free	Normal	Bracing
11	SS2x4	Southern N081	N040	7.57	Free	Free	Normal	Bracing
12	SS2x4	Southern N081	N045	3.82	Free	Free	Normal	Bracing
M1	SS2x6	Southern N083	N002	2.08	Fix	Fix	Normal	Beam
P1-Dry	3plyX8	Southern N082	N002	16.00	Fix	Free	Normal	Column
P1-Wet	3plyX8	Southern N004	N082	3.60	Fix	Fix	Normal	Column
M2	SS2x6	Southern N002	N009	37.50	Fix	Free	Normal	Beam
P2-Dry	3plyX8	Southern N085	N006	16.00	Fix	Free	Normal	Column
P2-Wet	3plyX8	Southern N008	N085	3.60	Fix	Fix	Normal	Column
M3	SS2x6	Southern N006	N009	37.50	Fix	Free	Normal	Beam
M4	SS2x6	Southern N084	N006	2.08	Fix	Fix	Normal	Beam
M5	SS2x6	Southern N002	N006	72.00	Free	Free	Normal	Beam

Design Groups

Group/Me	Elements	LL	Factor	Unity	Design	Shape	Overstrength
Post Abo	2	1.00	0.45	3plyX8			No
Post Bel	2	1.00	0.16	3plyX8			No

Nodal Loads

Load Case	Node	Direction	Force	Moment
			lb	lb-ft
E+X	N002	DX	135.0	0.00

Member Uniform Loads

Load Case	Member	Direction	Offset ft	End Offset ft	Force lb/ft	Moment ft-lb/ft
Dead Load	M1	DY proj.	0.00	2.08	-32.0	-NA-
Dead Load	M2	DY proj.	0.00	37.50	-32.0	-NA-
Dead Load	M3	DY proj.	0.00	37.50	-32.0	-NA-
Dead Load	M4	DY proj.	0.00	2.08	-32.0	-NA-
Dead Load	M5	DY	0.00	72.00	-40.0	-NA-
Roof Live Load	M1	DY proj.	0.00	2.08	-160.	-NA-
Roof Live Load	M2	DY proj.	0.00	37.50	-160.	-NA-
Roof Live Load	M3	DY proj.	0.00	37.50	-160.	-NA-
Roof Live Load	M4	DY proj.	0.00	2.08	-160.	-NA-

Nodal Displacements

Node	Result Case Name	DX in	DY in	RZ deg
N001	0.6D+0.7E+H »+X	0.0000	-0.0009	-0.02
N001	D+F	-0.0000	-0.0015	0.01
N001	D+H+F+0.75(0.7E+L+Lr) »+X	0.0000	-0.0039	-0.00
N001	D+H+F+0.75(0.7E+L+R) »+X	0.0000	-0.0015	-0.01
N001	D+H+F+0.75(L+Lr)	-0.0000	-0.0039	0.02
N001	D+H+F+0.7E »+X	0.0000	-0.0015	-0.02
N001	D+H+F+Lr	-0.0000	-0.0046	0.02
N001	Dead Load	-0.0000	-0.0015	0.01
N001	E+X	0.0000	0.0000	-0.03
N001	Roof Live Load	-0.0000	-0.0031	0.01
N002	0.6D+0.7E+H »+X	0.5868	-0.0079	-0.41
N002	D+F	-0.2048	-0.0131	-0.68
N002	D+H+F+0.75(0.7E+L+Lr) »+X	0.0164	-0.0335	-1.78
N002	D+H+F+0.75(0.7E+L+R) »+X	0.3326	-0.0131	-0.68
N002	D+H+F+0.75(L+Lr)	-0.5209	-0.0335	-1.79
N002	D+H+F+0.7E »+X	0.5049	-0.0131	-0.68
N002	D+H+F+Lr	-0.6263	-0.0403	-2.15
N002	Dead Load	-0.2048	-0.0131	-0.68
N002	E+X	1.0139	0.0000	0.00

N002	Roof Live Load	-0.4215	-0.0271	-1.47
N004	0.6D+0.7E+H »+X	-0.0000	-0.0000	0.01
N004	D+F	0.0000	-0.0000	-0.00
N004	D+H+F+0.75(0.7E+L+Lr) »+X	-0.0000	-0.0000	0.00
N004	D+H+F+0.75(0.7E+L+R) »+X	-0.0000	-0.0000	0.01
N004	D+H+F+0.75(L+Lr)	0.0000	-0.0000	-0.01
N004	D+H+F+0.7E »+X	-0.0000	-0.0000	0.01
N004	D+H+F+Lr	0.0000	-0.0000	-0.01
N004	Dead Load	0.0000	-0.0000	-0.00
N004	E+X	-0.0000	0.0000	0.02
N004	Roof Live Load	0.0000	-0.0000	-0.01
N006	0.6D+0.7E+H »+X	0.8274	-0.0079	0.42
N006	D+F	0.2007	-0.0131	0.70
N006	D+H+F+0.75(0.7E+L+Lr) »+X	1.0521	-0.0335	1.93
N006	D+H+F+0.75(0.7E+L+R) »+X	0.7360	-0.0131	0.70
N006	D+H+F+0.75(L+Lr)	0.5168	-0.0335	1.94
N006	D+H+F+0.7E »+X	0.9077	-0.0131	0.70
N006	D+H+F+Lr	0.6222	-0.0403	2.35
N006	Dead Load	0.2007	-0.0131	0.70
N006	E+X	1.0100	-0.0000	-0.00
N006	Roof Live Load	0.4215	-0.0271	1.65
N007	0.6D+0.7E+H »+X	0.0000	-0.0009	-0.03
N007	D+F	0.0000	-0.0015	-0.01
N007	D+H+F+0.75(0.7E+L+Lr) »+X	0.0000	-0.0039	-0.03
N007	D+H+F+0.75(0.7E+L+R) »+X	0.0000	-0.0015	-0.02
N007	D+H+F+0.75(L+Lr)	0.0000	-0.0039	-0.02
N007	D+H+F+0.7E »+X	0.0000	-0.0015	-0.03
N007	D+H+F+Lr	0.0000	-0.0046	-0.02
N007	Dead Load	0.0000	-0.0015	-0.01
N007	E+X	0.0000	-0.0000	-0.03
N007	Roof Live Load	0.0000	-0.0031	-0.01
N008	0.6D+0.7E+H »+X	-0.0000	-0.0000	0.01
N008	D+F	-0.0000	-0.0000	0.00
N008	D+H+F+0.75(0.7E+L+Lr) »+X	-0.0000	-0.0000	0.02
N008	D+H+F+0.75(0.7E+L+R) »+X	-0.0000	-0.0000	0.01
N008	D+H+F+0.75(L+Lr)	-0.0000	-0.0000	0.01
N008	D+H+F+0.7E »+X	-0.0000	-0.0000	0.01
N008	D+H+F+Lr	-0.0000	-0.0000	0.01
N008	Dead Load	-0.0000	-0.0000	0.00
N008	E+X	-0.0000	-0.0000	0.02
N008	Roof Live Load	-0.0000	-0.0000	0.01
N009	0.6D+0.7E+H »+X	0.7076	-0.9014	0.00
N009	D+F	-0.0012	-1.5101	0.00
N009	D+H+F+0.75(0.7E+L+Lr) »+X	0.5365	-3.8341	0.00
N009	D+H+F+0.75(0.7E+L+R) »+X	0.5351	-1.5065	0.00
N009	D+H+F+0.75(L+Lr)	0.0002	-3.8376	0.00

N009	D+H+F+0.7E »+X	0.7072	-1.5054	0.00
N009	D+H+F+Lr	0.0007	-4.6134	0.00
N009	Dead Load	-0.0012	-1.5101	0.00
N009	E+X	1.0119	0.0067	0.00
N009	Roof Live Load	0.0019	-3.1034	0.00
N014	0.6D+0.7E+H »+X	0.7080	-0.5366	-0.26
N014	D+F	-0.0016	-0.8985	-0.44
N014	D+H+F+0.75(0.7E+L+Lr) »+X	0.5336	-2.2946	-1.15
N014	D+H+F+0.75(0.7E+L+R) »+X	0.5352	-0.8966	-0.44
N014	D+H+F+0.75(L+Lr)	-0.0032	-2.2965	-1.15
N014	D+H+F+0.7E »+X	0.7074	-0.8960	-0.44
N014	D+H+F+Lr	-0.0037	-2.7625	-1.39
N014	Dead Load	-0.0016	-0.8985	-0.44
N014	E+X	1.0128	0.0036	0.00
N014	Roof Live Load	-0.0021	-1.8641	-0.95
N019	0.6D+0.7E+H »+X	0.7421	-0.7898	-0.05
N019	D+F	0.0558	-1.3227	-0.08
N019	D+H+F+0.75(0.7E+L+Lr) »+X	0.6792	-3.3607	-0.14
N019	D+H+F+0.75(0.7E+L+R) »+X	0.5924	-1.3199	-0.08
N019	D+H+F+0.75(L+Lr)	0.1427	-3.3636	-0.14
N019	D+H+F+0.7E »+X	0.7644	-1.3189	-0.08
N019	D+H+F+Lr	0.1717	-4.0439	-0.16
N019	Dead Load	0.0558	-1.3227	-0.08
N019	E+X	1.0123	0.0054	0.00
N019	Roof Live Load	0.1159	-2.7212	-0.08
N024	0.6D+0.7E+H »+X	0.7374	-0.8910	-0.08
N024	D+F	0.0483	-1.4924	-0.14
N024	D+H+F+0.75(0.7E+L+Lr) »+X	0.6613	-3.7919	-0.53
N024	D+H+F+0.75(0.7E+L+R) »+X	0.5847	-1.4891	-0.14
N024	D+H+F+0.75(L+Lr)	0.1250	-3.7953	-0.53
N024	D+H+F+0.7E »+X	0.7567	-1.4880	-0.14
N024	D+H+F+Lr	0.1505	-4.5630	-0.67
N024	Dead Load	0.0483	-1.4924	-0.14
N024	E+X	1.0120	0.0064	0.00
N024	Roof Live Load	0.1023	-3.0705	-0.53
N035	0.6D+0.7E+H »+X	0.6786	-0.8940	0.07
N035	D+F	-0.0494	-1.4975	0.11
N035	D+H+F+0.75(0.7E+L+Lr) »+X	0.4152	-3.8031	0.45
N035	D+H+F+0.75(0.7E+L+R) »+X	0.4869	-1.4941	0.11
N035	D+H+F+0.75(L+Lr)	-0.1210	-3.8065	0.45
N035	D+H+F+0.7E »+X	0.6589	-1.4931	0.11
N035	D+H+F+Lr	-0.1449	-4.5761	0.56
N035	Dead Load	-0.0494	-1.4975	0.11
N035	E+X	1.0118	0.0064	-0.00
N035	Roof Live Load	-0.0955	-3.0786	0.44
N040	0.6D+0.7E+H »+X	0.6714	-0.8075	0.05

N040	D+F	-0.0612	-1.3523	0.08
N040	D+H+F+0.75(0.7E+L+Lr) »+X	0.3869	-3.4322	0.18
N040	D+H+F+0.75(0.7E+L+R) »+X	0.4749	-1.3493	0.08
N040	D+H+F+0.75(L+Lr)	-0.1492	-3.4352	0.18
N040	D+H+F+0.7E »+X	0.6469	-1.3484	0.08
N040	D+H+F+Lr	-0.1785	-4.1295	0.21
N040	Dead Load	-0.0612	-1.3523	0.08
N040	E+X	1.0116	0.0055	-0.00
N040	Roof Live Load	-0.1173	-2.7772	0.13
N045	0.6D+0.7E+H »+X	0.6956	-0.5947	0.21
N045	D+F	-0.0203	-0.9956	0.35
N045	D+H+F+0.75(0.7E+L+Lr) »+X	0.4920	-2.5309	0.84
N045	D+H+F+0.75(0.7E+L+R) »+X	0.5156	-0.9936	0.35
N045	D+H+F+0.75(L+Lr)	-0.0439	-2.5329	0.85
N045	D+H+F+0.7E »+X	0.6875	-0.9929	0.35
N045	D+H+F+Lr	-0.0518	-3.0453	1.01
N045	Dead Load	-0.0203	-0.9956	0.35
N045	E+X	1.0111	0.0038	-0.00
N045	Roof Live Load	-0.0315	-2.0498	0.67
N054	0.6D+0.7E+H »+X	0.6330	-0.6875	-0.08
N054	D+F	-0.1272	-1.1510	-0.14
N054	D+H+F+0.75(0.7E+L+Lr) »+X	0.2169	-2.9200	-0.59
N054	D+H+F+0.75(0.7E+L+R) »+X	0.4099	-1.1486	-0.14
N054	D+H+F+0.75(L+Lr)	-0.3202	-2.9223	-0.59
N054	D+H+F+0.7E »+X	0.5821	-1.1478	-0.14
N054	D+H+F+Lr	-0.3845	-3.5128	-0.74
N054	Dead Load	-0.1272	-1.1510	-0.14
N054	E+X	1.0133	0.0045	0.00
N054	Roof Live Load	-0.2573	-2.3619	-0.60
N059	0.6D+0.7E+H »+X	0.6643	-0.8516	-0.09
N059	D+F	-0.0745	-1.4261	-0.15
N059	D+H+F+0.75(0.7E+L+Lr) »+X	0.3507	-3.6184	-0.25
N059	D+H+F+0.75(0.7E+L+R) »+X	0.4623	-1.4230	-0.15
N059	D+H+F+0.75(L+Lr)	-0.1861	-3.6215	-0.25
N059	D+H+F+0.7E »+X	0.6345	-1.4220	-0.15
N059	D+H+F+Lr	-0.2233	-4.3532	-0.29
N059	Dead Load	-0.0745	-1.4261	-0.15
N059	E+X	1.0128	0.0059	0.00
N059	Roof Live Load	-0.1488	-2.9271	-0.14
N065	0.6D+0.7E+H »+X	0.6957	-0.9116	0.00
N065	D+F	-0.0214	-1.5270	0.01
N065	D+H+F+0.75(0.7E+L+Lr) »+X	0.4862	-3.8713	-0.03
N065	D+H+F+0.75(0.7E+L+R) »+X	0.5151	-1.5235	0.01
N065	D+H+F+0.75(L+Lr)	-0.0503	-3.8747	-0.03
N065	D+H+F+0.7E »+X	0.6872	-1.5224	0.01
N065	D+H+F+Lr	-0.0600	-4.6573	-0.04

N065	Dead Load	-0.0214	-1.5270	0.01
N065	E+X	1.0122	0.0065	0.00
N065	Roof Live Load	-0.0386	-3.1303	-0.05
N070	0.6D+0.7E+H »+X	0.7196	-0.9117	-0.00
N070	D+F	0.0190	-1.5271	-0.00
N070	D+H+F+0.75(0.7E+L+Lr) »+X	0.5884	-3.8711	0.03
N070	D+H+F+0.75(0.7E+L+R) »+X	0.5552	-1.5236	-0.01
N070	D+H+F+0.75(L+Lr)	0.0523	-3.8746	0.03
N070	D+H+F+0.7E »+X	0.7272	-1.5225	-0.01
N070	D+H+F+Lr	0.0633	-4.6571	0.04
N070	Dead Load	0.0190	-1.5271	-0.00
N070	E+X	1.0116	0.0065	-0.00
N070	Roof Live Load	0.0443	-3.1300	0.05
N076	0.6D+0.7E+H »+X	0.7506	-0.8530	0.09
N076	D+F	0.0715	-1.4285	0.14
N076	D+H+F+0.75(0.7E+L+Lr) »+X	0.7222	-3.6220	0.25
N076	D+H+F+0.75(0.7E+L+R) »+X	0.6074	-1.4254	0.14
N076	D+H+F+0.75(L+Lr)	0.1863	-3.6250	0.25
N076	D+H+F+0.7E »+X	0.7792	-1.4244	0.14
N076	D+H+F+Lr	0.2246	-4.3572	0.28
N076	Dead Load	0.0715	-1.4285	0.14
N076	E+X	1.0111	0.0058	-0.00
N076	Roof Live Load	0.1531	-2.9288	0.14
N081	0.6D+0.7E+H »+X	0.7815	-0.6914	0.08
N081	D+F	0.1235	-1.1575	0.14
N081	D+H+F+0.75(0.7E+L+Lr) »+X	0.8543	-2.9334	0.59
N081	D+H+F+0.75(0.7E+L+R) »+X	0.6592	-1.1552	0.14
N081	D+H+F+0.75(L+Lr)	0.3186	-2.9357	0.59
N081	D+H+F+0.7E »+X	0.8310	-1.1545	0.14
N081	D+H+F+Lr	0.3837	-3.5284	0.74
N081	Dead Load	0.1235	-1.1575	0.14
N081	E+X	1.0106	0.0044	-0.00
N081	Roof Live Load	0.2601	-2.3709	0.60
N082	0.6D+0.7E+H »+X	0.0099	-0.0015	-0.05
N082	D+F	-0.0035	-0.0025	0.02
N082	D+H+F+0.75(0.7E+L+Lr) »+X	0.0003	-0.0062	-0.00
N082	D+H+F+0.75(0.7E+L+R) »+X	0.0056	-0.0025	-0.03
N082	D+H+F+0.75(L+Lr)	-0.0088	-0.0062	0.05
N082	D+H+F+0.7E »+X	0.0085	-0.0025	-0.04
N082	D+H+F+Lr	-0.0106	-0.0074	0.05
N082	Dead Load	-0.0035	-0.0025	0.02
N082	E+X	0.0171	0.0000	-0.09
N082	Roof Live Load	-0.0071	-0.0050	0.04
N083	0.6D+0.7E+H »+X	0.5378	0.1603	-0.40
N083	D+F	-0.2869	0.2684	-0.67
N083	D+H+F+0.75(0.7E+L+Lr) »+X	-0.1973	0.6993	-1.74

N083	D+H+F+0.75(0.7E+L+R) »+X	0.2506	0.2678	-0.67
N083	D+H+F+0.75(L+Lr)	-0.7349	0.6999	-1.74
N083	D+H+F+0.7E »+X	0.4230	0.2677	-0.67
N083	D+H+F+Lr	-0.8842	0.8437	-2.10
N083	Dead Load	-0.2869	0.2684	-0.67
N083	E+X	1.0142	-0.0011	0.00
N083	Roof Live Load	-0.5973	0.5752	-1.43
N084	0.6D+0.7E+H »+X	0.8779	0.1651	0.41
N084	D+F	0.2851	0.2763	0.69
N084	D+H+F+0.75(0.7E+L+Lr) »+X	1.2841	0.7618	1.89
N084	D+H+F+0.75(0.7E+L+R) »+X	0.8203	0.2758	0.69
N084	D+H+F+0.75(L+Lr)	0.7490	0.7623	1.89
N084	D+H+F+0.7E »+X	0.9919	0.2756	0.69
N084	D+H+F+Lr	0.9036	0.9243	2.29
N084	Dead Load	0.2851	0.2763	0.69
N084	E+X	1.0097	-0.0010	-0.00
N084	Roof Live Load	0.6185	0.6481	1.60
N085	0.6D+0.7E+H »+X	0.0140	-0.0015	-0.07
N085	D+F	0.0034	-0.0025	-0.02
N085	D+H+F+0.75(0.7E+L+Lr) »+X	0.0178	-0.0062	-0.09
N085	D+H+F+0.75(0.7E+L+R) »+X	0.0124	-0.0025	-0.06
N085	D+H+F+0.75(L+Lr)	0.0087	-0.0062	-0.04
N085	D+H+F+0.7E »+X	0.0153	-0.0025	-0.08
N085	D+H+F+Lr	0.0105	-0.0074	-0.05
N085	Dead Load	0.0034	-0.0025	-0.02
N085	E+X	0.0171	-0.0000	-0.09
N085	Roof Live Load	0.0071	-0.0050	-0.04

Member Unity Checks

Unity Member	Status	Model Shape	Design Shape	Messages?
0.45 P1-Dry	Designed	3plyX8	3plyX8	None
0.45 P2-Dry	Designed	3plyX8	3plyX8	None
0.16 P1-Wet	Designed	3plyX8	3plyX8	None
0.16 P2-Wet	Designed	3plyX8	3plyX8	None

Design Group Results

NDS Wood Design Checks: Post Above Grade

Designed As: 3plyX8

Extreme Checks Only

Combined Stresses Check:

Member Name	Result Case	fa psf	fb1 psf	fb2 psf	Fa' psf	Fb1' psf	Fb2' psf	Unity Check
P1-Dry	18.01	8322	29144	0	96370	378786	399168	0.09
P1-Dry	19.01	13870	10171	0	89881	213417	224532	0.08
P1-Dry	20.01	35059	817	0	96370	378786	399168	0.14
P1-Dry	21.01	13870	16516	0	96370	378786	399168	0.07
P1-Dry	22.01	35059	25870	0	94171	296173	311850	0.27
P1-Dry	23.01	13870	25076	0	96370	378786	399168	0.10
P1-Dry	24.01	42122	31103	0	94171	296173	311850	0.38
P2-Dry	18.01	8319	41091	0	96370	378786	399168	0.13
P2-Dry	19.01	13865	9967	0	89881	213417	224532	0.08
P2-Dry	20.01	35054	52251	0	96370	378786	399168	0.34
P2-Dry	21.01	13865	36551	0	96370	378786	399168	0.13
P2-Dry	22.01	35054	25667	0	94171	296173	311850	0.27
P2-Dry	23.01	13865	45078	0	96370	378786	399168	0.16
P2-Dry	24.01	42117	30900	0	94171	296173	311850	0.38

Strong Flexure Check:

Member Name	Result Case #	Offset ft	fb psf	Fb psf	Fb' psf	Unity Check
P1-Dry	18.01	0	-29144	216000	378786	0.08
P1-Dry	19.01	0	10171	216000	213417	0.05
P1-Dry	21.01	0	-16516	216000	378786	0.04
P1-Dry	22.01	0	25870	216000	296173	0.09
P1-Dry	23.01	0	-25076	216000	378786	0.07
P1-Dry	24.01	0	31103	216000	296173	0.11
P2-Dry	18.01	0	-41091	216000	378786	0.11
P2-Dry	19.01	0	-9967	216000	213417	0.05
P2-Dry	20.01	0	-52251	216000	378786	0.14
P2-Dry	21.01	0	-36551	216000	378786	0.10
P2-Dry	22.01	0	-25667	216000	296173	0.09
P2-Dry	23.01	0	-45078	216000	378786	0.12
P2-Dry	24.01	0	-30900	216000	296173	0.10

Axial Check:

Member Name	Result Case #	Offset ft	Axial State	fa psf	Fa psf	Fa' psf	CP	Unity Check
P1-Dry	18.01	0.00	Comp.	-8322	237600	96370	0.25	0.09
P1-Dry	19.01	0.00	Comp.	-13870	237600	89881	0.42	0.15
P1-Dry	20.01	0.00	Comp.	-35059	237600	96370	0.25	0.36
P1-Dry	21.01	0.00	Comp.	-13870	237600	96370	0.25	0.14
P1-Dry	22.01	0.00	Comp.	-35059	237600	94171	0.32	0.37
P1-Dry	23.01	0.00	Comp.	-13870	237600	96370	0.25	0.14

P1-Dry	24.01	0.00	Comp.	-42122	237600	94171	0.32	0.45
P2-Dry	18.01	0.00	Comp.	-8319	237600	96370	0.25	0.09
P2-Dry	19.01	0.00	Comp.	-13865	237600	89881	0.42	0.15
P2-Dry	20.01	0.00	Comp.	-35054	237600	96370	0.25	0.36
P2-Dry	21.01	0.00	Comp.	-13865	237600	96370	0.25	0.14
P2-Dry	22.01	0.00	Comp.	-35054	237600	94171	0.32	0.37
P2-Dry	23.01	0.00	Comp.	-13865	237600	96370	0.25	0.14
P2-Dry	24.01	0.00	Comp.	-42117	237600	94171	0.32	0.45

Strong Shear Check:

Member	Result	Offset	fv	Fv	Fv'	CD	Unity
Name	Case #	ft	psf	psf	psf		Check
P1-Dry	18.01	0	273	25200	40320	1.60	0.01
P1-Dry	22.01	0	242	25200	31500	1.25	0.01
P1-Dry	23.01	0	235	25200	40320	1.60	0.01
P1-Dry	24.01	0	291	25200	31500	1.25	0.01
P2-Dry	18.01	0	385	25200	40320	1.60	0.01
P2-Dry	20.01	0	489	25200	40320	1.60	0.01
P2-Dry	21.01	0	342	25200	40320	1.60	0.01
P2-Dry	22.01	0	240	25200	31500	1.25	0.01
P2-Dry	23.01	0	422	25200	40320	1.60	0.01
P2-Dry	24.01	0	289	25200	31500	1.25	0.01

NDS Wood Design Checks: Post Below Grade

Designed As: 3plyX8

Extreme Checks Only

Combined Stresses Check:

Member	Result	fa	fb1	fb2	Fa'	Fb1'	Fb2'	Unity
Name	Case	psf	psf	psf	psf	psf	psf	Check
P1-Wet	18.01	8350	31621	0	327108	377552	399168	0.08
P1-Wet	19.01	13917	11035	0	198947	213057	224532	0.06
P1-Wet	20.01	35106	886	0	327108	377552	399168	0.01
P1-Wet	21.01	13917	17920	0	327108	377552	399168	0.05
P1-Wet	22.01	35106	28069	0	266412	295449	311850	0.11
P1-Wet	23.01	13917	27207	0	327108	377552	399168	0.07
P1-Wet	24.01	42169	33747	0	266412	295449	311850	0.14
P2-Wet	18.01	8347	44584	0	327108	377552	399168	0.12
P2-Wet	19.01	13912	10815	0	198947	213057	224532	0.06
P2-Wet	20.01	35101	56692	0	327108	377552	399168	0.16
P2-Wet	21.01	13912	39658	0	327108	377552	399168	0.11
P2-Wet	22.01	35101	27848	0	266412	295449	311850	0.11
P2-Wet	23.01	13912	48910	0	327108	377552	399168	0.13
P2-Wet	24.01	42164	33526	0	266412	295449	311850	0.14

Strong Flexure Check:

Member	Result	Offset	fb	Fb	Fb'	Unity
Name	Case #	ft	psf	psf	psf	Check
P1-Wet	18.01	2.24	-31621	216000	377552	0.08
P1-Wet	19.01	2.24	11035	216000	213057	0.05
P1-Wet	21.01	2.24	-17920	216000	377552	0.05
P1-Wet	22.01	2.24	28069	216000	295449	0.10
P1-Wet	23.01	2.24	-27207	216000	377552	0.07
P1-Wet	24.01	2.24	33747	216000	295449	0.11
P2-Wet	18.01	2.24	-44584	216000	377552	0.12
P2-Wet	19.01	2.24	-10815	216000	213057	0.05
P2-Wet	20.01	2.24	-56692	216000	377552	0.15
P2-Wet	21.01	2.24	-39658	216000	377552	0.11
P2-Wet	22.01	2.24	-27848	216000	295449	0.09
P2-Wet	23.01	2.24	-48910	216000	377552	0.13
P2-Wet	24.01	2.24	-33526	216000	295449	0.11

Axial Check:

Member	Result	Offset	Axial	fa	Fa	Fa'	CP	Unity
Name	Case #	ft	State	psf	psf	psf		Check
P1-Wet	18.01	0.00	Comp.	-8396	237600	327108	0.86	0.03
P1-Wet	19.01	0.00	Comp.	-13994	237600	198947	0.93	0.07
P1-Wet	20.01	0.00	Comp.	-35184	237600	327108	0.86	0.11
P1-Wet	21.01	0.00	Comp.	-13994	237600	327108	0.86	0.04
P1-Wet	22.01	0.00	Comp.	-35184	237600	266412	0.90	0.13
P1-Wet	23.01	0.00	Comp.	-13994	237600	327108	0.86	0.04
P1-Wet	24.01	0.00	Comp.	-42247	237600	266412	0.90	0.16
P2-Wet	18.01	0.00	Comp.	-8393	237600	327108	0.86	0.03
P2-Wet	19.01	0.00	Comp.	-13989	237600	198947	0.93	0.07
P2-Wet	20.01	0.00	Comp.	-35179	237600	327108	0.86	0.11
P2-Wet	21.01	0.00	Comp.	-13989	237600	327108	0.86	0.04
P2-Wet	22.01	0.00	Comp.	-35179	237600	266412	0.90	0.13
P2-Wet	23.01	0.00	Comp.	-13989	237600	327108	0.86	0.04
P2-Wet	24.01	0.00	Comp.	-42242	237600	266412	0.90	0.16

Strong Shear Check:

Member	Result	Offset	fv	Fv	Fv'	CD	Unity
Name	Case #	ft	psf	psf	psf		Check
P1-Wet	18.01	0	2115	25200	40320	1.60	0.05
P1-Wet	19.01	0	738	25200	22680	0.90	0.03
P1-Wet	21.01	0	1198	25200	40320	1.60	0.03
P1-Wet	22.01	0	1877	25200	31500	1.25	0.06
P1-Wet	23.01	0	1819	25200	40320	1.60	0.05
P1-Wet	24.01	0	2257	25200	31500	1.25	0.07
P2-Wet	18.01	0	2981	25200	40320	1.60	0.07
P2-Wet	19.01	0	723	25200	22680	0.90	0.03

P2-Wet	20.01	0	3791	25200	40320	1.60	0.09
P2-Wet	21.01	0	2652	25200	40320	1.60	0.07
P2-Wet	22.01	0	1862	25200	31500	1.25	0.06
P2-Wet	23.01	0	3271	25200	40320	1.60	0.08
P2-Wet	24.01	0	2242	25200	31500	1.25	0.07

Nodal Reactions

Node	Result Case Name	FX lb	FY lb	MZ lb-ft
N001	0.6D+0.7E+H »+X	-342.515	-NA-	-NA-
N001	D+F	119.534	-NA-	-NA-
N001	D+H+F+0.75(0.7E+L+Lr) »+X	-9.596	-NA-	-NA-
N001	D+H+F+0.75(0.7E+L+R) »+X	-194.102	-NA-	-NA-
N001	D+H+F+0.75(L+Lr)	304.039	-NA-	-NA-
N001	D+H+F+0.7E »+X	-294.702	-NA-	-NA-
N001	D+H+F+Lr	365.541	-NA-	-NA-
N001	Dead Load	119.534	-NA-	-NA-
N001	E+X	-591.765	-NA-	-NA-
N001	Roof Live Load	246.007	-NA-	-NA-
N004	0.6D+0.7E+H »+X	303.371	1806.916	-NA-
N004	D+F	-105.873	3011.526	-NA-
N004	D+H+F+0.75(0.7E+L+Lr) »+X	8.500	7571.522	-NA-
N004	D+H+F+0.75(0.7E+L+R) »+X	171.919	3011.526	-NA-
N004	D+H+F+0.75(L+Lr)	-269.292	7571.522	-NA-
N004	D+H+F+0.7E »+X	261.022	3011.526	-NA-
N004	D+H+F+Lr	-323.765	9091.521	-NA-
N004	Dead Load	-105.873	3011.526	-NA-
N004	E+X	524.135	-0.000	-NA-
N004	Roof Live Load	-217.892	6079.995	-NA-
N007	0.6D+0.7E+H »+X	-482.924	-NA-	-NA-
N007	D+F	-117.141	-NA-	-NA-
N007	D+H+F+0.75(0.7E+L+Lr) »+X	-614.074	-NA-	-NA-
N007	D+H+F+0.75(0.7E+L+R) »+X	-429.568	-NA-	-NA-
N007	D+H+F+0.75(L+Lr)	-301.647	-NA-	-NA-
N007	D+H+F+0.7E »+X	-529.781	-NA-	-NA-
N007	D+H+F+Lr	-363.148	-NA-	-NA-
N007	Dead Load	-117.141	-NA-	-NA-
N007	E+X	-589.485	-NA-	-NA-
N007	Roof Live Load	-246.007	-NA-	-NA-

N008	0.6D+0.7E+H »+X	427.733	1806.286	-NA-
N008	D+F	103.754	3010.477	-NA-
N008	D+H+F+0.75(0.7E+L+Lr) »+X	543.894	7570.473	-NA-
N008	D+H+F+0.75(0.7E+L+R) »+X	380.475	3010.477	-NA-
N008	D+H+F+0.75(L+Lr)	267.173	7570.473	-NA-
N008	D+H+F+0.7E »+X	469.235	3010.477	-NA-
N008	D+H+F+Lr	321.646	9090.472	-NA-
N008	Dead Load	103.754	3010.477	-NA-
N008	E+X	522.115	0.000	-NA-
N008	Roof Live Load	217.892	6079.995	-NA-

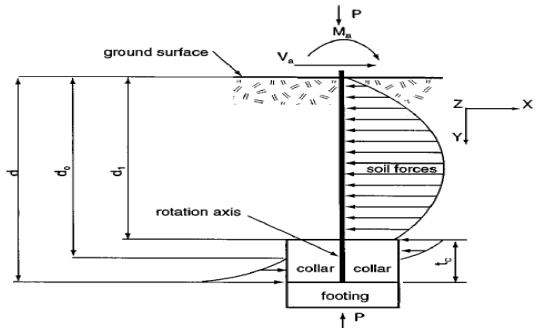
APPENDIX B
FOUNDATION DESIGN

NON CONSTRAINED FOOTING WITH BOTTOM CONCRETE COLLAR (EP 486.1)

REACTIONS	VERTICAL REACTIONS		HORIZONTAL REACTIONS		MOMENTS	
	Dead Load	(lbs) 3012	Dead Load	(lbs) 0	Dead Load	(ft-lbs) 0
	Live _R Load	(lbs) 6080	Live _R Load	(lbs) 0	Live _R Load	(ft-lbs) 0
	Snow Load	(lbs) 5985	Snow Load	(lbs) 0	Snow Load	(ft-lbs) 0
	Wind Uplift	(lbs) 2959	Wind Load	(lbs) 33.7	Wind Load	(ft-lbs) 539
	D	3012	D	0	D	0
	D+L _r	9092	D+L _r	0	D+L _r	0
	D+S	8997	D+S	0	D+S	0
W-0.6D (Uplift)	1152.258	D+W	33.7	D+W	539	
FINAL REACTIONS:						
Vertical, P (lbs):	9092	Horizontal, V _a (lbs):	33.7	Moment, M _a (ft-lb):	539	
Uplift, Q (lbs):	1152.26					

NON-CONSTRAINED FOOTING WITH BOTTOM CONCRETE COLLAR (ASAE EP486.1 2003)

REQUIRED POST EMBEDMENT DEPTH AND UPLIFT RESISTANCE CALCULATION	Initial Allowable vertical soil pressure, S _v	(psf)	2000	(EP486.1 Table 1, without the allowable increase factor)	
	Allowable lateral bearing soil pressure, S'	(psf/ft)	200	(EP486.1 Table 1)	
	Actual post embedment depth, d	(ft)	4		
	Post Width	(ft)	4.5		
	Collar width, w	(in)	24		
	Collar thickness, t _c	(in)	24		
	Footing thickness, t _f	(in)	8		
	Concrete Density, C	(pcf)	150		
	Soil Density, α	(pcf)	105		
	Gravity Acceleration, G	(lbf/lb)	1		
	Soil Friction Angle, q	(deg)	26		
	Post Cross Section Area, A _p	(in ²)	31		
	Enter d ₀ /d ratio calculated d values below are equal		0.665		
	INTERMEDIATE CALCULATIONS				
	Collar width, w	(ft)	2		
	Collar thickness, t _c	(ft)	2		
Post bearing width, b	(ft)	0.530			



ASAE EP486.1, 2003, FIG. 2

INTERMEDIATE CALCULATIONS

Collar width, w	(ft)	2
Collar thickness, t _c	(ft)	2
Post bearing width, b	(ft)	0.530

UPLIFT RESISTANCE

$$U = \alpha G [0.33\pi \{ [(d-t) + 0.5w/\tan\theta]^3 (\tan\theta)^2 - 0.125w^3/\tan\theta \} - A_p(d-t)] + 0.25C\pi w^2 tG$$

REQUIRED POST EMBEDMENT, d

(Shear Criterion) d = **2.25** ft

(Moment Criterion) d = **2.25** ft

$$d_1 = d - t_c$$

$$V_a = \frac{Sb}{2} \left(\frac{2d^3}{d_0} - 3d^2 \right) + \frac{S(w-b)}{2} \left[\frac{2(d^3 - d_1^3)}{d_0} - 3(d^2 - d_1^2) \right]$$

$$V_a d + M_a = -\frac{Sb}{4} \left(\frac{d^4}{d_0} - 2d^3 \right) - \frac{S(w-b)}{4} \left[\frac{d^4 - 3dd_1^3 + 3d_1^4}{d_0} - 2d^3 - 6dd_1^2 + 4d_1^3 \right]$$

Required post embedment, d **2.3** **ft** (based on shear and moment requirements)

SUMMARY	Required embedment < Actual Embedment		Vertical allowable soil pressures are increased per EP486 Footnotes, Tab.		
	U =	2394.7 lbs	> Q =	1152.3 lbs	Depth Increase: 73.3 % Width Increase: 20.0 %
	S _v =	3866.7 psf	> S _a =	2894.1 psf	S _s = P/π(w/2) ² S _v → ASAE EP486.1, 2003, Table 1
U = Uplift Resistance of Soil, S _v = Allowable Vertical Soil Pressure (including increases)					
V _a = Shear at Ground Surface, M _a = Moment at Ground Surface, Q = Uplift at Ground Surface, S _a = Actual Vertical Soil Pressure					

APPENDIX C

DESIGN OF CRITICAL CONNECTIONS

Truss to Post Connection - Uplift - Vertical Shear

REACTION	Tributary Width (ft)	b_t	38	<i>Half of building's length + overhang</i>
	Truss Spacing (ft)	s_t	8	
	Dead Load (psf)	D	4.5	<i>(50% of Design Dead Load)</i>
	Wind Load (psf)	L	9.74	
	Net Uplift (lbs)		2139	$\leftarrow (W-0.6D)(b_t)(s_t)$

NAILS - WOOD TO WOOD - SHEAR - NDS 2005, ASD

ALLOWABLE CAPACITY	Nail Size		16d		<i>Intermediate Calculations</i>	
	Number of nails	#	10		K_D	2.2
	Nail Diameter (in)	D	0.162		F_{yb} (psi)	90000
	Nail Length (in)	L	3.5		$2+R_e$	3.0
	Width of Side Member (in)	w	1.5		$1+2R_e$	3.0
	Width of Main Member (in)	w	1.5		k_1	1.05
	Nail Penetration into main member (in)	p	1.5		k_2	1.09
	Specific Gravity of Main Member	G	0.55		F_{em}	5526
	Specific Gravity of Side Member		0.55		F_{es}	5526
	Duration Factor	C_D	1.6	<i>(NDS 2005, Table 2.3.2)</i>	R_e	1.0
	Wet Service Factor	C_M	1		p	2.00
	Temperature Factor	C_t	1		$1+R_e$	2.0
	Toe Nail Factor	C_{tn}	1		I_s	610
	End Grain Factor	C_{eg}	1	<i>(NDS 2005, 11.5.2)</i>	III_m	286
	Depth Penetration Factor	C_d	0.9259259	<i>(Cd=p/10D)</i>	III_s	222
Lateral Design Value (lbs)	Z	154	<i>(controlling yield value)</i>	IV	154	
Total Allowable Lateral Capacity (lbs)	Z'	2275	$\leftarrow Z'=(\# \text{ of Nails}) \times Z \times C_D \times C_d \times C_{eg} \times C_{tn} \times C_M \times C_t$			

PASS $2138.66 \div 2275 = 0.9399167$ *Unity Check < 1*

Truss to Post Connection - Horizontal Shear at Frame 2, the Controlling Frame for Horizontal Shear

REACTION	Seismic Eave Load (lbs)	P_i	955	$V_g = (P_i + Q_s) / 2 \text{ posts (this is service load)}$
	Load Resisted by Diaphragm, (lbs)	Q_s	-820	
	Seismic Shear at Top of Post, (lbs)	V_g	67.3	
	D+0.7E Shear at Top of Post (lbs)		47	
NAILS - WOOD TO WOOD - SHEAR - NDS 2005, ASD				
ALLOWABLE CAPACITY	Nail Size		16d	<i>Intermediate Calculations</i>
	Number of nails	#	10	K_D 2.2
	Nail Diameter (in)	D	0.162	F_{yb} (psi) 90000
	Nail Length (in)	L	3.5	$2+R_e = 3.0$
	Width of Side Member (in)	w	1.5	$1+2R_e = 3.0$
	Width of Main Member (in)	w	1.5	$k_1 = 1.05$
	Nail Penetration into main member (in)	p	1.5	$k_2 = 1.09$
	Specific Gravity of Main Member	G	0.55	$F_{em} = 5526$
	Specific Gravity of Side Member		0.55	$F_{es} = 5526$
	Duration Factor	C_D	1.6	(NDS 2005, Table 2.3.2) $R_e = 1.0$
	Wet Service Factor	C_M	1	$p = 2.00$
	Temperature Factor	C_t	1	$1+R_e = 2.0$
	Toe Nail Factor	C_{tn}	1	I_s 610
	End Grain Factor	C_{eg}	1	(NDS 2005, 11.5.2) III_m 286
	Depth Penetration Factor	C_d	0.9259259	($Cd=p/10D$) III_s 222
Lateral Design Value (lbs)	Z	154	(controlling yield value) IV 154	
Total Allowable Lateral Capacity (lbs)	Z'	2275	← $Z'=(\# \text{ of Nails}) \times Z \times C_D \times C_d \times C_{eg} \times C_{tn} \times C_M \times C_t$	
PASS		$47.0967 \div 2275$	= 0.0206985	<i>Unity Check < 1</i>

Truss to Glulam Header Connection - Uplift

REACTION	Tributary Width (ft)	b_t	38	<i>Half of building's length + overhang</i>
	Truss Spacing (ft)	s_t	8	
	Dead Load (psf)	D	4.5	<i>(50% of Design Dead Load)</i>
	Wind Load (psf)	L	9.74	
	Net Uplift (lbs)		2139	$\leftarrow (W-0.6D)(b_t)(s_t)$
ALLOWABLE CAPACITY	H10A Simpson Hurricane Tie			
	Number of straps/brackets/hangers		2	
	Specific Gravity	G	0.55	
	Duration Factor	C_D	1	<i>C_D is included by bracket manufacturer</i>
	Wet Service Factor	C_M	1	
	Design Value per strap/bracket/hanger		1140	
Total Allowable Capacity (lbs)		2280		
PASS		$2138.66 \div 2280$	$= 0.9380078$	<i>Unity Check < 1</i>

Ceiling Ledger to Endwall Posts Connection - Shear Transfer from Ceiling Diaphragm to Endwall

REACTION	Number of Endwall Posts		10	
	Number of Nails per Post		4	
	Maximum Shear (lbs)	V_{max}	2116	<i>(see Ceiling Diaphragm Shear Strength Check Section)</i>
ALLOWABLE CAPACITY	NAILS - WOOD TO WOOD - SHEAR - NDS 2005, ASD			
	Nail Size		16d	<i>Intermediate Calculations</i>
	Number of nails	#	40	$K_D = 2.2$
	Nail Diameter (in)	D	0.162	$F_{yb} \text{ (psi)} = 90000$
	Nail Length (in)	L	3.5	$2+R_e = 3.0$
	Width of Side Member (in)	w	1.5	$1+2R_e = 3.0$
	Width of Main Member (in)	w	7	$k_1 = 1.05$
	Nail Penetration into main member (in)	p	2	$k_2 = 1.09$
	Specific Gravity of Main Member	G	0.55	$F_{em} = 5526$
	Specific Gravity of Side Member		0.55	$F_{es} = 5526$
	Duration Factor	C_D	1.6	<i>(NDS 2005, Table 2.3.2)</i> $R_e = 1.0$
	Wet Service Factor	C_M	1	$p = 2.00$
	Temperature Factor	C_t	1	$1+R_e = 2.0$
	Toe Nail Factor	C_{tn}	1	$I_s = 610$
	End Grain Factor	C_{eg}	1	<i>(NDS 2005, 11.5.2)</i> $III_m = 286$
Depth Penetration Factor	C_d	1	<i>(Cd=p/10D)</i> $III_s = 222$	
Lateral Design Value (lbs)	Z	154	<i>(controlling yield value)</i> $IV = 154$	
Total Allowable Lateral Capacity (lbs)	Z'	9830	$\leftarrow Z'=(\# \text{ of Nails}) \times Z \times C_D \times C_d \times C_{eg} \times C_{tn} \times C_M \times C_t$	
PASS		$2116 \div 9830$	$= 0.215305$	<i>Unity Check < 1</i>

Bottom Girt to Endwall Posts Connection - Shear Transfer from Wall Sheathing to Endwall Posts

REACTION	Number of Endwall Posts	10	
	Number of Nails per Post	4	
	Maximum Shear (lbs) V_{max}	2205	(see Endwall Shear Strength Check Section)
NAILS - WOOD TO WOOD - SHEAR - NDS 2005, ASD			
ALLOWABLE CAPACITY	Nail Size	16d	<i>Intermediate Calculations</i>
	Number of nails	# 40	K_D 2.2
	Nail Diameter (in)	D 0.162	F_{yb} (psi) 90000
	Nail Length (in)	L 3.5	$2+R_e = 3.0$
	Width of Side Member (in)	w 1.5	$1+2R_e = 3.0$
	Width of Main Member (in)	w 7	$k_1 = 1.05$
	Nail Penetration into main member (in)	p 2	$k_2 = 1.09$
	Specific Gravity of Main Member	G 0.55	$F_{em} = 5526$
	Specific Gravity of Side Member	0.55	$F_{es} = 5526$
	Duration Factor	C_D 1.6	(NDS 2005, Table 2.3.2) $R_e = 1.0$
	Wet Service Factor	C_M 0.7	$p = 2.00$
	Temperature Factor	C_t 1	$1+R_e = 2.0$
	Toe Nail Factor	C_{tn} 1	I_s 610
	End Grain Factor	C_{eg} 1	(NDS 2005, 11.5.2) III_m 286
	Depth Penetration Factor	C_d 1	($C_d=p/10D$) III_s 222
Lateral Design Value (lbs)	Z 154	(controlling yield value) IV 154	
Total Allowable Lateral Capacity (lbs)	Z' 6881	$\leftarrow Z'=(\# \text{ of Nails}) \times Z \times C_D \times C_d \times C_{eg} \times C_m \times C_M \times C_t$	
PASS	$2205 \div 6881 = 0.320406$	<i>Unity Check < 1</i>	

Purlin to Purlin Connection at Splice, Diaphragm Tension Chord

ALLOWABLE CAPACITY	REACTION			
	Tension Load in Diaphragm Chord (lbs)	T_{max}	1	← Tension from Roof Diaphragm Chord Forces Section
ALLOWABLE CAPACITY	HTP37Z Simpson Strap			
	Number of straps/brackets/hangers		1	<i>C_D is included by the manufacturer</i>
	Specific Gravity	G	0.55	
	Duration Factor	C _D	1	
	Wet Service Factor	C _M	1	
	Design Value per strap/bracket/hanger		1850	
Total Allowable Capacity (lbs)		1850		
PASS	$0.73752 \div 1850$	$=$	0.0003987	<i>Unity Check < 1</i>

Ceiling Joist to Ceiling Joist Connection at Splice, Diaphragm Tension Chord

ALLOWABLE CAPACITY	REACTION			
	Tension Load in Diaphragm Chord (lbs)	T_{max}	1	← Tension from Roof Diaphragm Chord Forces Section
ALLOWABLE CAPACITY	MSTA21 Simpson Strap			
	Number of straps/brackets/hangers		1	<i>C_D is included by the manufacturer</i>
	Specific Gravity	G	0.55	
	Duration Factor	C _D	1	
	Wet Service Factor	C _M	1	
	Design Value per strap/bracket/hanger		1505	
Total Allowable Capacity (lbs)		1505		
PASS	$0.76825 \div 1505$	$=$	0.0005105	<i>Unity Check < 1</i>

Purlin to Endwall Truss Connection, Diaphragm Tension Chord

REACTION	Tension Load in Diaphragm Chord (lbs)	T_{max}	1	← Tension from Roof Diaphragm Chord Forces Section
	NAILS - WOOD TO WOOD - SHEAR - NDS 2005, ASD			
ALLOWABLE CAPACITY	Nail Size		16d	<i>Intermediate Calculations</i>
	Number of nails	#	5	K_D 2.2
	Nail Diameter (in)	D	0.162	F_{yb} (psi) 90000
	Nail Length (in)	L	3.5	$2+R_e = 3.0$
	Width of Side Member (in)	w	1.5	$1+2R_e = 3.0$
	Width of Main Member (in)	w	7	$k_1 = 1.05$
	Nail Penetration into main member (in)	p	2	$k_2 = 1.09$
	Specific Gravity of Main Member	G	0.55	$F_{em} = 5526$
	Specific Gravity of Side Member		0.55	$F_{es} = 5526$
	Duration Factor	C_D	1.6	(NDS 2005, Table 2.3.2) $R_e = 1.0$
	Wet Service Factor	C_M	1	$p = 2.00$
	Temperature Factor	C_t	1	$1+R_e = 2.0$
	Toe Nail Factor	C_{tn}	1	I_s 610
	End Grain Factor	C_{eg}	1	(NDS 2005, 11.5.2) III_m 286
	Depth Penetration Factor	C_d	1	($Cd=p/10D$) III_s 222
Lateral Design Value (lbs)	Z	154	(controlling yield value) IV 154	
Total Allowable Lateral Capacity (lbs)	Z'	1229	← $Z'=(\# \text{ of Nails}) \times Z \times C_D \times C_d \times C_{eg} \times C_m \times C_M \times C_t$	
NAILS - WOOD TO WOOD - SHEAR - NDS 2005, ASD				
ALLOWABLE CAPACITY	Nail Size		60d	RS nail <i>Intermediate Calculations</i>
	Number of nails	#	1	K_D 2.27
	Nail Diameter (in)	D	0.177	F_{yb} (psi) 90000
	Nail Length (in)	L	6	$2+R_e = 3.0$
	Width of Side Member (in)	w	3.5	$1+2R_e = 3.0$
	Width of Main Member (in)	w	5.5	$k_1 = 1.04$
	Nail Penetration into main member (in)	p	2.5	$k_2 = 1.02$
	Specific Gravity of Main Member	G	0.55	$F_{em} = 5526$
	Specific Gravity of Side Member		0.55	$F_{es} = 5526$
	Duration Factor	C_D	1.6	(NDS 2005, Table 2.3.2) $R_e = 1.0$
	Wet Service Factor	C_M	1	$p = 2.50$
	Temperature Factor	C_t	1	$1+R_e = 2.0$
	Toe Nail Factor	C_{tn}	1	I_s 1508
	End Grain Factor	C_{eg}	1	(NDS 2005, 11.5.2) III_m 374
	Depth Penetration Factor	C_d	1	($Cd=p/10D$) III_s 513
Lateral Design Value (lbs)	Z	178	(controlling yield value) IV 178	
Total Allowable Lateral Capacity (lbs)	Z'	284	← $Z'=(\# \text{ of Nails}) \times Z \times C_D \times C_d \times C_{eg} \times C_m \times C_M \times C_t$	
		1229 + 284	= 1513	<i>Unity Check < 1</i>
PASS		1 ÷ 1513	= 0.0004875	<i>Unity Check < 1</i>

Purlin to Endwall Truss Connection, Diaphragm Tension Chord - BLOCKING TO TRUSS

REACTION	Tension Load in Diaphragm Chord (lbs)	T_{max}	1	← Tension from Roof Diaphragm Chord Forces Section	
	WOOD SCREWS - WITHDRAWAL - NDS 2005, ASD				
ALLOWABLE CAPACITY	Screw Size #		8	(# of effective screwsn not total) Note: if a certain adjustment factor is not shown, its value is assumed to be 1.0	
	Number of screws	#	4		
	Screw Diameter (in)	D	0.164		
	Screw Length (in)	L	3		
	Width of Side Member (in)	w	1.5		
	Width of Main Member (in)	w	1.5		
	Screw Penetration into main member (in)	p	1.5		
	Specific Gravity	G	0.55		
	Duration Factor	C_D	1.6		← NDS 2005, Table 2.3.2
	Toe Nail Factor	C_{tn}	1		
	Temperature Factor	C_t	1		
	Wet Service Factor	C_M	1		
	Reference Withdrawal Design Value	W	141		← NDS 2005, Equation 11.2-2
Total Allowable Withdrawal Capacity (lbs)	W'	1357	← $W'=(\# \text{ of Screws}) \times Z \times C_D \times C_t \times C_m \times C_M$		
$0.73752 \div 1357 = 0.0005434$					

Ceiling Joist to Ceiling Joist Connection at Splice, Diaphragm Tension Chord

REACTION	Tension Load in Diaphragm Chord (lbs)	T_{max}	1	← Tension from Roof Diaphragm Chord Forces Section
	NAILS - WOOD TO WOOD - SHEAR - NDS 2005, ASD			
ALLOWABLE CAPACITY	Nail Size		16d	<u>Intermediate Calculations</u>
	Number of nails	#	6	K_D 2.2
	Nail Diameter (in)	D	0.162	$F_{yb} \text{ (psi)}$ 90000
	Nail Length (in)	L	3.5	$2+R_e =$ 3.0
	Width of Side Member (in)	w	1.5	$1+2R_e =$ 3.0
	Width of Main Member (in)	w	7	$k_1 =$ 1.05
	Nail Penetration into main member (in)	p	2	$k_2 =$ 1.09
	Specific Gravity of Main Member	G	0.55	$F_{em} =$ 5526
	Specific Gravity of Side Member	G	0.55	$F_{es} =$ 5526
	Duration Factor	C_D	1.6	(NDS 2005, Table 2.3.2) $R_e =$ 1.0
	Wet Service Factor	C_M	1	$p =$ 2.00
	Temperature Factor	C_t	1	$1+R_e =$ 2.0
	Toe Nail Factor	C_{tn}	1	I_s 610
	End Grain Factor	C_{eg}	1	(NDS 2005, 11.5.2) III_m 286
	Depth Penetration Factor	C_d	1	($Cd=p/10D$) III_s 222
	Lateral Design Value (lbs)	Z	154	(controlling yield value) IV 154
	Total Allowable Lateral Capacity (lbs)	Z'	1474	← $Z'=(\# \text{ of Nails}) \times Z \times C_D \times C_d \times C_{eg} \times C_m \times C_M \times C_t$
PASS $0.76825 \div 1474 = 0.000521$ Unity Check < 1				

APPENDIX D

PURLIN DESIGN
GIRT DESIGN



COMPANY	PROJECT
Oct. 6, 2010 10:50	Purlin - D+Lr

Design Check Calculation Sheet
Size: 8.3

LOADS:

Load	Type	Distribution	Pat-tern	Location [ft]		Magnitude		Unit
				Start	End	Start	End	
Load1	Dead	Full Area	No			2.50	(24.0)'	psf
Load2	Roof constr.	Full UDL	No			33.4		plf

*Tributary Width (in)

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :

Category	0'	8'	16'
Unfactored:			
Dead	18	54	18
Other	115	354	115
Factored:			
Total	134	448	134
Bearing:			
Load Comb	#2	#2	#2
Length	0.50'	0.50'	0.50'
Cb	1.00	1.75	1.00

*Min. bearing length for joists is 1/2" for exterior supports and 1/2" for intermediate supports

Lumber-soft, S. Pine, No.2, 2x4"

Roof joist spaced at 24" c/c. Self-weight of 1.36 plf included in loads;
Lateral support: top= full, bottom= at supports; Repetitive factor: applied where permitted (refer to online help);

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2005 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	Fv = 60	Fv' = 215	Fv/Fv' = 0.28
Bending(+)	Fb = 789	Fb' = 2158	Fb/Fb' = 0.37
Bending(-)	Fb = 1408	Fb' = 2129	Fb/Fb' = 0.66
Live Defl'n	0.47 = L/881	0.64 = L/150	0.73
Total Defl'n	0.21 = L/465	0.80 = L/120	0.26

ADDITIONAL DATA:

FACTORS:	F/E	CD	CM	Ct	CL	CF	Cfu	Cv	Cft	CL	Cu	LC+
Fv'	175	1.25	1.00	1.00	-	-	-	-	1.00	1.00	1.00	1.00
Fb'+	978	1.25	1.00	1.00	1.000	1.558	2.00	2.15	1.88	2.08	-	-
Fb'-	978	1.25	1.00	1.00	0.995	1.558	2.00	2.15	1.88	2.08	-	-
Fcp'	869	-	1.00	1.00	-	-	-	-	1.00	2.00	-	-
E'	1.6 million	1.00	1.00	-	-	-	-	-	1.00	2.00	-	-
Emin'	0.58 million	1.00	1.00	-	-	-	-	-	1.79	2.00	-	-

Shear : LC #2 = D+Lr, V = 334, V design = 211 lbs
 Bending(+): LC #2 = D+Lr, M = 201 lbs-ft
 Bending(-): LC #2 = D+Lr, M = 359 lbs-ft
 Deflection: LC #2 = D+Lr (live)
 LC #2 = D+Lr (total)
 EI = 3e06 lb-in²
 Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection.
 D=dead L=Live S=snow W=wind I=impact L=roof constr. C=concentrated
 All LC's are listed in the Analysis output
 Load combinations: ASCE 7-05

DESIGN NOTES:

- Please verify that the default deflection limits are appropriate for your application.
- Continuous or Cantilevered Beams: NDS Clause 4.2.5.5 requires that normal grading provisions be extended to the middle 2/3 of 2 span beams and to the full length of cantilevers and other spans.
- Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4.1.



COMPANY

PROJECT

Oct. 6, 2010 10:50

Purlin - D+Lr Unbalanced

Design Check Calculation Sheet
Size: 8.3

LOADS:

Load	Type	Distribution	Pat-tern	Location (ft) Start End	Magnitude Start End	Unit
Load1	Dead	Full Area	Nc		2.50 (24.0)*	pcf
Load2	Roof constr.	Partial UDL	Nc	0.00 9.00	38.4 38.4	plf

*Tributary Width (in)

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :

Category	0'	8'	16'
Unfactored:			
Dead	66	64	19
Other	124	122	
Factored:			
Uplift			14
Total	190	186	19
Bearing:			
Load Comb	#2	#2	#1
Length	0.50*	0.50*	0.50*
Cb	1.00	1.75	1.00

*Min. bearing length for joists is 1/2" for exterior supports and 1/2" for intermediate supports

Lumber-soft, S. Pine, No.2, 2x4"

Roof joist spaced at 24" c/c; Self-weight of 1.36 pcf included in loads;

Lateral support: top= full, bottom= at supports; Repetitive factor, applied where permitted (refer to online help);

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2005 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	Fv = 58	Fv' = 219	Fv/Fv' = 0.26
Bending(+)	Fb = 1031	Fb' = 2196	Fb/Fb' = 0.48
Bending(-)	Fb = 801	Fb' = 1898	Fb/Fb' = 0.48
Live Defl'n	0.29 = L/392	0.64 = L/150	0.48
Total Defl'n	0.33 = L/299	0.80 = L/120	0.41

ADDITIONAL DATA:

FACTORS:	F/E	CD	CM	Co	CL	CF	Cfu	Cg	CFRt	Ci	Cm	LC+
Fv'	178	1.25	1.00	1.00	-	-	-	-	1.00	1.00	1.00	1.00
Fb'+	978	1.25	1.00	1.00	1.000	1.839	1.00	1.45	1.00	1.00	-	1.00
Fb'-	978	1.25	1.00	1.00	0.874	1.839	1.00	1.39	1.00	1.00	-	1.00
Fcg'	568	-	1.00	1.00	-	-	-	-	1.00	1.00	-	1.00
E'	1.8 million	1.00	1.00	1.00	-	-	-	-	1.00	1.00	-	1.00
Emin'	0.58 million	1.00	1.00	1.00	-	-	-	-	1.00	1.00	-	1.00

Shear : LC #2 = D+Lr, V = 206, V design = 192 lbs
 Bending(+): LC #2 = D+Lr, M = 268 lbs-ft
 Bending(-): LC #2 = D+Lr, M = 204 lbs-ft
 Deflection: LC #2 = D+Lr (Live)
 LC #2 = D+Lr (total)
 EI = 9e08 lb-in²
 Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection.
 D=dead L=Live S=snow W=wind I=impact Lr=roof constr. Lc=concentrated
 All LC's are listed in the Analysis output
 Load combinations: ASCE 7-05

DESIGN NOTES:

- Please verify that the default deflection limits are appropriate for your application.
- Continuous or Cantilevered Beams: NDS Clause 4.2.5.5 requires that normal grading provisions be extended to the middle 2/3 of 2 span beams and to the full length of cantilevers and other spans.
- Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4.1.



COMPANY	PROJECT
Oct. 6, 2010 10:54	Purlin - D+S balanced

Design Check Calculation Sheet
Size 8.3

LOADS:

Load	Type	Distribution	Far-tern	Location (ft)	Magnitude	Unit
				Start End	Start End	
Load1	Dead	Full Area	No		2.50 (24.0)'	psf
Load2	Snow	Full UDL	No		22.2	psf

*Tributary Width (in)

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :

	0'	8'	16'
Unfactored:			
Dead	18	64	18
Other	87	222	87
Factored:			
Total	86	286	86
Bearing:			
Load Comb	#2	#2	#2
Length	0.80'	0.80'	0.80'
Cb	1.00	1.75	1.00

*Min. bearing length for joists is 1/2" for exterior supports and 1/2" for intermediate supports

Lumber-soft, S. Pine, No.2, 2x4"

Roof joist spaced at 24" o/c, Self-weight of 1.36 plf included in loads;
Lateral support: top= full, bottom= at supports; Repetitive factor applied where permitted (refer to online help).

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2005 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	$F_v = 38$	$F_v' = 201$	$F_v/F_v' = 0.19$
Bending(+)	$F_b = 304$	$F_b' = 1983$	$F_b/F_b' = 0.25$
Bending(-)	$F_b = 388$	$F_b' = 1986$	$F_b/F_b' = 0.46$
Live Defl'n	0.10 = $L/996$	0.64 = $L/150$	0.15
Total Defl'n	0.14 = $L/703$	0.80 = $L/120$	0.17

ADDITIONAL DATA:

FACTORS:	F/E	CD	CM	Cp	CL	CF	Cfu	Ci	Cftc	Ct	Cn	DC+
F_v'	175	1.15	1.00	1.00	-	-	-	-	1.00	1.00	1.00	2
F_b'	975	1.15	1.00	1.00	1.000	1.538	1.00	1.15	1.00	1.00	-	2
F_b'	975	1.15	1.00	1.00	0.996	1.538	1.00	1.15	1.00	1.00	-	2
F_{cp}'	568	-	1.00	1.00	-	-	-	-	1.00	1.00	-	2
E'	1.6 million	1.00	1.00	-	-	-	-	-	1.00	1.00	-	2
E_{min}'	0.80 million	1.00	1.00	-	-	-	-	-	1.00	1.00	-	2

Shear : LC #2 = D+S, V = 193, V design = 184 lbs

Bending(+): LC #2 = D+S, M = 128 lbs-ft

Bending(-): LC #2 = D+S, M = 323 lbs-ft

Deflection: LC #2 = D+S (live)

LC #2 = D+S (total)

ET = 9606 lb-in²

Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection.

D=dead L=Live S=snow W=wind I=impact L=roof const. C=concentrated

All LC's are listed in the Analysis output.

Load combinations: ASCE 7-05

DESIGN NOTES:

- Please verify that the default deflection limits are appropriate for your application.
- Continuous or Cantilevered Beams: NDS Clause 4.2.5.5 requires that normal grading provisions be extended to the middle 2/3 of 2 span beams and to the full length of cantilevers and other spans.
- Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4.1.



COMPANY

PROJECT

Oct. 6, 2010 10:53

Purlin- D+S Unbalanced

Design Check Calculation Sheet
Size 8.3

LOADS:

Load	Type	Distribution	Pat-tern	Location [Ft]	Magnitude	Unit
				Start End	Start End	
Load1	Dead	Full Area	No		2.50 (24.0)'	psf
Load2	Snow	Full UDL	No		50.5	plf

*Tributary Width (in)

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :

Unfactored:						
Dead	18			84		18
Other	150			808		150
Factored:						
Total	178			892		178
Bearing:						
Load Comb	#2			#2		#2
Length	0.50'			0.50'		0.50'
Cb	1.00			1.00		1.00

*Min. bearing length for joists is 1/2" for exterior supports and 1/2" for intermediate supports

Lumber-soft, S. Pine, No.2, 2x4"

Roof joist spaced at 24" o/c; Self-weight of 1.36 plf included in loads;
Lateral support: top= full, bottom= at supports; Repetitive factor: applied where permitted (refer to online help)

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2005 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	Fv = 77	Fv' = 201	Fv/Fv' = 0.38
Bending(+)	Fb = 2008	Fb' = 1883	Fb/Fb' = 0.51
Bending(-)	Fb = 1752	Fb' = 1886	Fb/Fb' = 0.92
Live Defl'n	0.22 = L/435	0.64 = L/150	0.24
Total Defl'n	0.26 = L/370	0.80 = L/120	0.32

ADDITIONAL DATA:

FACTORS:	F/E	CD	CM	Ct	CL	CE	Cfu	Cr	Cftb	Ci	Ch	LC#
Fv'	175	1.15	1.00	1.00	-	-	-	-	1.00	1.00	1.00	1
Fb'+	375	1.15	1.00	1.00	1.000	1.583	1.00	1.15	1.00	1.00	-	1
Fb'-	375	1.15	1.00	1.00	0.886	1.583	1.00	1.15	1.00	1.00	-	1
Fcp'	565	-	1.00	1.00	-	-	-	-	1.00	1.00	-	1
E'	1.6 million	1.00	1.00	-	-	-	-	-	1.00	1.00	-	1
Emin'	0.92 million	1.00	1.00	-	-	-	-	-	1.00	1.00	-	1

Shear : LC #2 = D+S, V = 266, V design = 269 lbs

Bending(+): LC #2 = D+S, M = 267 lbs-ft

Bending(-): LC #2 = D+S, M = 457 lbs-ft

Deflection: LC #2 = D+S (live)

LC #2 = D+S (total)

ED = 9e06 lb-in²

Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection.

D=dead L=live S=snow W=wind I=impact L=roof live Lc=concentrated

All LC's are listed in the Analysis output

Load combinations: ASCE 7-05

DESIGN NOTES:

- Please verify that the default deflection limits are appropriate for your application.
- Continuous or Cantilevered Beams: NDS Clause 4.2.5.5 requires that normal grading provisions be extended to the middle 2/3 of 2 span beams and to the full length of cantilevers and other spans.
- Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4.1.



COMPANY

PROJECT

Oct. 6, 2010 10:51

Purim - Wind Interior Zone

Design Check Calculation Sheet
Size: 8.3

LOADS:

Load	Type	Distribution	Sap-tern	Location [ft]		Magnitude		Unit
				Start	End	Start	End	
Load1	Dead	Full Area	No			1.00	(24.0)'	psf
Load2	Wind	Full Area	No			-11.70	(24.0)'	psf

*Tributary Width (in):

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :

Unfactored:								
Dead	10			84				10
Other								
Factored:								
Uplift	83			203				83
Total	10			34				10
Bearing:								
Load Comb	#1			#1				#1
Length	0.50'			0.50'				0.50'
Cb	1.00			1.75				1.00

*Min. bearing length for joists is 1/2" for exterior supports and 1/2" for intermediate supports

Lumber-soft, S. Pine, No.2, 2x4"

Roof joist spaced at 24" c/c; Self-weight of 1.36 plf included in loads;
Lateral support: top= full, bottom= at supports; Repetitive factor applied where permitted (refer to online help)

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2005 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	$F_v = 30$	$F_v' = 250$	$F_v/F_v' = 0.11$
Bending(+)	$F_b = 653$	$F_b' = 2753$	$F_b/F_b' = 0.24$
Bending(-)	$F_b = 367$	$F_b' = 2441$	$F_b/F_b' = 0.15$
Live Defl'n	$-0.10 = L/822$	$0.64 = L/150$	0.16
Total Defl'n	$-0.08 = L/998$	$0.80 = L/120$	0.10

ADDITIONAL DATA:

FACTORS:	S/E	CD	CM	Ct	CL	CF	Cfu	Cc	Cftc	Cb	Cn	LC#
F_v'	175	1.60	1.00	1.00	-	-	-	-	1.00	1.00	1.00	2
$F_b'+$	575	1.60	1.00	1.00	1.000	1.536	1.00	1.15	1.00	1.00	-	2
$F_b'-$	575	1.60	1.00	1.00	0.895	1.536	1.00	0.15	1.00	1.00	-	2
F_{cp}'	568	-	1.00	1.00	-	-	-	-	1.00	1.00	-	-
E'	1.6 million	1.00	1.00	1.00	-	-	-	-	1.00	1.00	-	2
E_{min}'	0.88 million	1.00	1.00	1.00	-	-	-	-	1.00	1.00	-	2

Shear : LC #2 = .6D+W, V = 104, V design = 104 lbs
 Bending(+): LC #2 = .6I+W, M = 167 lbs-ft
 Bending(-): LC #2 = .6I+W, M = 94 lbs-ft
 Deflection: LC #2 = .6D+W (live)
 LC #2 = .6D+W (total)
 EI = 9e06 lb-in²
 Total Deflection = 1.80(Dead Load Deflection) + Live Load Deflection.
 D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated
 All LC's are listed in the Analysis output
 Load combinations: ASCE 7-05

DESIGN NOTES:

- Please verify that the default deflection limits are appropriate for your application.
- Continuous or Cantilevered Beams: NDS Clause 4.2.5.5 requires that normal grading provisions be extended to the middle 2/3 of 2 span beams and to the full length of cantilevers and other spans.
- Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4.1.



COMPANY	PROJECT
Oct. 6, 2010 10:48	Purlin - Wind Edge Zone

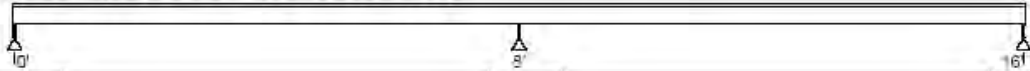
Design Check Calculation Sheet
Size 8.3

LOADS:

Load	Type	Distribution	Ser-tern	Location (ft)		Magnitude		Units
				Start	End	Start	End	
Load1	Dead	Full Area	No			1.00	(24.0)'	psf
Load2	Wind	Full Area	No			-19.20	(24.0)'	psf

*Tributary Width (in)

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :



	0'	8'	16'
Unfactored:			
Dead	10	34	10
Other			
Factored:			
Uplift	108	388	108
Total	10	34	10
Bearing:			
Load Comb	#1	#1	#1
Length	0.50"	0.50"	0.50"
Cb	1.00	1.00	1.00

*Min. bearing length for joists is 1/2" for exterior supports and 1/2" for intermediate supports

Lumber-soft, S. Pine, No.2, 2x4"

Roof joist spaced at 24" c/c; Self-weight of 1.36 pf included in loads;
Lateral support: top= full, bottom= at supports; Repetitive factor: applied where permitted (refer to online help).

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2005 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	$F_v = 51$	$F_v' = 280$	$F_v/F_v' = 0.18$
Bending(+)	$F_b = 1123$	$F_b' = 2789$	$F_b/F_b' = 0.41$
Bending(-)	$F_b = 832$	$F_b' = 2441$	$F_b/F_b' = 0.34$
Live Defl'n	$-0.17 = L/568$	$0.24 = L/180$	0.26
Total Defl'n	$-0.18 = L/668$	$0.30 = L/120$	0.18

ADDITIONAL DATA:

FACTORS:	S/E	CD	CM	Ce	CL	CF	Cfu	Cp	Cpfd	Ci	Cm	EC#
F_v'	175	1.00	1.00	1.00	-	-	-	-	1.00	1.00	1.00	2
F_b'	878	1.00	1.00	1.00	1.000	1.558	1.00	1.18	1.00	1.00	-	2
F_b'	878	1.00	1.00	1.00	0.838	1.558	1.00	1.18	1.00	1.00	-	2
F_{cp}'	888	-	1.00	1.00	-	-	-	-	1.00	1.00	-	2
E'	1.6 million	1.00	1.00	1.00	-	-	-	-	1.00	1.00	-	2
E_{min}'	0.58 million	1.00	1.00	1.00	-	-	-	-	1.00	1.00	-	2

Shear : LC #2 = .6D+W, V = 179, V design = 179 lbs
 Bending(+): LC #2 = .6D+W, M = 297 lbs-ft
 Bending(-): LC #2 = .6D+W, M = 161 lbs-ft
 Deflection: LC #2 = .6D+W (live)
 LC #2 = .6D+W (total)
 ED = 9e06 lb-in²
 Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection.
 D=dead L=live S=snow W=wind I=impact Ir=roof live Lc=concentrated
 All LC's are listed in the Analysis output
 Load combinations: ASCE 7-08

DESIGN NOTES:

- Please verify that the default deflection limits are appropriate for your application.
- Continuous or Cantilevered Beams: NDS Clause 4.2.5.5 requires that normal grading provisions be extended to the middle 2/3 of 2 span beams and to the full length of cantilevers and other spans.
- Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4.1.



COMPANY	PROJECT
Oct 6, 2010 10:51	Purlin - Wind Corner Zone

Design Check Calculation Sheet
Size 8/3

LOADS:

Load	Type	Distribution	Pat-tern	Location [ft]	Magnitude	Unit
				Start End	Start End	
Load1	Dead	Full Area	No		1.00 (24.0)'	psf
Load3	Wind	Partial Area	No	0.00 3.00	-28.90 (24.0)'	psf
Load4	Wind	Partial Area	No	8.00 16.00	-18.20 (24.0)'	psf

*Tributary Width (in)

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :

Category	0'	8'	16'
Unfactored:			
Dead	11	24	10
Other			
Factored:			
Uplift	152	374	105
Total	10	34	10
Bearing:			
Load Comb	#1	#1	#1
Length	0.60"	0.60"	0.60"
Cb	1.00	1.75	1.00

*Min. bearing length for joists is 1/2" for exterior supports and 1/2" for intermediate supports

Lumber-soft, S. Pine, No.2, 2x4"

Roof joist spaced at 24" o/c; Self-weight of 1.36 plf included in loads;
Lateral support: top= full, bottom= at supports; Repetitive factor: applied where permitted (refer to online help)

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2005 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	Fv = 58	Fv' = 290	Fv/Fv' = 0.20
Bending(+)	Fb = 1208	Fb' = 2759	Fb/Fb' = 0.44
Bending(-)	Fb = 822	Fb' = 2435	Fb/Fb' = 0.34
Live Defl'n	-0.22 = L/444	0.64 = L/150	0.34
Total Defl'n	-0.25 = L/494	0.60 = L/120	0.24

ADDITIONAL DATA:

FACTORS:	FvE	CD	CM	Ct	CL	CF	Cfu	Cr	Cftt	Ci	Cm	LC
Fv'	175	1.60	1.00	1.00	-	-	-	-	1.00	1.00	1.00	1.00
Fb'+	975	1.60	1.00	1.00	1.000	1.583	1.00	1.15	1.00	1.00	-	1.00
Fb'-	975	1.60	1.00	1.00	0.833	1.583	1.00	1.15	1.00	1.00	-	1.00
Fcp'	865	-	1.00	1.00	-	-	-	-	1.00	1.00	-	1.00
E'	1.6 million	1.00	1.00	-	-	-	-	-	1.00	1.00	-	1.00
Emin'	0.99 million	1.00	1.00	-	-	-	-	-	1.00	1.00	-	1.00

Shear : LC #1 = .6D+W, V = 193, V design = 193 lbs
 Bending(+): LC #1 = .6D+W, M = 807 lbs-ft
 Bending(-): LC #1 = .6D+W, M = 210 lbs-ft
 Deflection: LC #1 = .6D+W (live)
 LC #1 = .6D+W (total)
 EI = 9e06 lb-in²
 Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection
 D=dead L=live S=snow W=wind I=impact L=roof live Lc=concentrated
 All LC's are listed in the Analysis output
 Load combinations: ASCE 7-08

DESIGN NOTES:

- Please verify that the default deflection limits are appropriate for your application.
- Continuous or Cantilevered Beams: NDS Clause 4.2.5.5 requires that normal grading provisions be extended to the middle 2/3 of 2 span beams and to the full length of cantilevers and other spans.
- Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4.1.



COMPANY

PROJECT

Oct. 6, 2010 10:49

Girt - Interior/Wind Zone

Design Check Calculation Sheet
Size 8.3

LOADS:

Load	Type	Distribution	Pat-tern	Location (ft)	Magnitude	Unit
				Start End	Start End	
Load1	Wind	Full Area	No		18.73 (1.00)'	psf

*Tributary Width (ft)

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :

Unfactored:						
Dead						
Other	32					32
Factored:						
Total	32				274	32
Bearing:						
Load Comb	#2				#2	#2
Length	0.50'				0.50'	0.50'
Cc	1.00				1.75	1.00

*Min. bearing length for beams is 1/2" for exterior supports and 1/2" for intermediate supports

Lumber n-ply, S. Pine, No.2, 2x4", 1-ply

Lateral support: top= at supports, bottom= at supports; Oblique angle: 90.0 [deg];

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2005 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear	$F_v = 0$	$F_v' = 280$	$F_v/F_v' = 0.00$
	$F_v = 33$	$F_v' = 280$	$F_v/F_v' = 0.14$
Biaxial	$F_{vx} + F_{vy}$	$F_{vx}' + F_{vy}'$	$F_{vx}/F_{vx}' + F_{vy}/F_{vy}' = 0.14$
Bending(+) x-x	$F_b = 0$	$F_b' = 2155$	$F_b/F_b' = 0.00$
	$F_b = 1127$	$F_b' = 2639$	$F_b/F_b' = 0.43$
Biaxial	3.9-3: $F_{b1}/F_{b1}' + F_{b2}/F_{b2}' / (1 - F_{b1}/F_{b1}' - F_{b2}/F_{b2}')$		$F_{b1}/F_{b1}' + F_{b2}/F_{b2}' = 0.43$
Bending(-) x-x	$F_b = 0$	$F_b' = 2353$	$F_b/F_b' = 0.00$
	$F_b = 2004$	$F_b' = 2639$	$F_b/F_b' = 0.76$
Biaxial	3.9-3: $F_{b1}/F_{b1}' + F_{b2}/F_{b2}' / (1 - F_{b1}/F_{b1}' - F_{b2}/F_{b2}')$		$F_{b1}/F_{b1}' + F_{b2}/F_{b2}' = 0.76$
Live Defl'n	-0.64 = L/149	1.07 = L/90	0.60
Total Defl'n	-0.64 = L/149	1.07 = L/90	0.60

ADDITIONAL DATA:

FACTORS:	F/E	CD	CM	Cc	CL	CF	Cfu	Cr	Cfrc	Ct	Cv	LC#
F_v'	178	1.00	1.00	1.00	-	-	-	-	1.00	1.00	1.00	
F_{vy}'	178	1.00	1.00	1.00	-	-	-	-	1.00	1.00	-	
F_{b+}	578	1.00	1.00	1.00	0.915	1.538	2.00	2.00	1.00	1.00	-	
F_{b1}'	578	1.00	1.00	1.00	1.000	1.538	1.10	1.00	1.00	1.00	-	
F_{b-}	578	1.00	1.00	1.00	0.983	1.538	1.00	1.00	1.00	1.00	-	
F_{b1}'	578	1.00	1.00	1.00	1.000	1.538	1.10	1.00	1.00	1.00	-	
F_{cp}	568	-	1.00	1.00	-	-	-	-	1.00	1.00	-	
E'	1.6 million	1.00	1.00	-	-	-	-	-	1.00	1.00	-	
E_{min}'	0.58 million	1.00	1.00	-	-	-	-	-	1.00	1.00	-	

Shear : LC #2 = .6D+W, V = 137, V design = 134 lbs
 Bending(+): LC #2 = .6D+W, M = 123 lbs-ft
 Bending(-): LC #2 = .6D+W, M = 219 lbs-ft
 Deflection: LC #2 = .6D+W (live)
 LC #2 = .6D+W (total)
 $EI = 9e06 \text{ lb-in}^2$ $EI_y = 2e06 \text{ lb-in}^2$
 Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection.
 D=dead L=live S=snow W=wind I=impact L=roof live Lc=concentrated
 All LC's are listed in the Analysis output
 Load combinations: ASCE 7-05

DESIGN NOTES:

- Please verify that the default deflection limits are appropriate for your application.
- Continuous or Cantilevered Beams: NDS Clause 4.2.5.5 requires that normal grading provisions be extended to the middle 2/3 of 2 span beams and to the full length of cantilevers and other spans.
- Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4.1.



COMPANY	PROJECT
Oct. 6, 2010 10:49	Girt - Edge Wind Zone

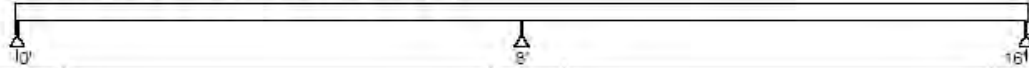
Design Check Calculation Sheet
Size 8.3

LOADS:

Load	Type	Distribution	Pat-tern	Location [ft]	Magnitude	Unit
				Start End	Start End	
Load1	Wind	Partial Area	No	7.00 16.00	19.70 (2.00)*	psf
Load2	Wind	Partial Area	No	0.00 7.00	16.45 (2.00)*	psf

*Tributary Width (ft)

MAXIMUM REACTIONS (lbs) and BEARING LENGTHS (in) :



Category	Value	Value	Value
Unfactored: Dead			
Other	101		85
Factored: Total	101	256	80
Bearing: Load Comb	#2	#2	#2
Length	0.50*	0.50*	0.50*
Cb	1.00	1.75	1.35

*Min. bearing length for beams is 1/2" for exterior supports and 1/2" for intermediate supports

Lumber n-ply, S. Pine, No.2, 2x4", 1-ply

Lateral support: top= at supports, bottom= at supports, Oblique angle: 90.0 [deg].

Analysis vs. Allowable Stress (psi) and Deflection (in) using NDS 2005 :

Criterion	Analysis Value	Design Value	Analysis/Design
Shear x-x	Fv = 7	Fv' = 280	Fv/Fv' = 0.02
y-y	Fv = 44	Fv' = 280	Fv/Fv' = 0.16
Biaxial	Fvx / Fvx' + Fvy / Fvy'	Fv' = 2198	Fv/Fv' = 0.16
Bending(+) x-x	Fb = 0	Fb' = 2638	Fb/Fb' = 0.00
y-y	Fb = 1418	Fb' = 2638	Fb/Fb' = 0.54
Biaxial	3.9-8: Fb1/Fb1' + Fb2/Fb2' / (1 - Fb1/Fb1' * 2)	Fb' = 2638	Fb/Fb' = 0.54
Bending(-) x-x	Fb = 0	Fb' = 2638	Fb/Fb' = 0.00
y-y	Fb = 2191	Fb' = 2638	Fb/Fb' = 0.83
Biaxial	3.9-8: Fb1/Fb1' + Fb2/Fb2' / (1 - Fb1/Fb1' * 2)	Fb' = 2638	Fb/Fb' = 0.83
Live Defl'n	-0.86 = L/112	1.07 = L/90	0.80
Total Defl'n	-0.86 = L/112	1.07 = L/90	0.80

ADDITIONAL DATA:

FACTORS: F/E	CD	CM	Cc	CL	CP	CEu	CE	C&rt	Ci	Cw	LC4
Fv'	1.75	1.00	1.00	-	-	-	-	1.00	1.00	1.70	-
Fvy'	1.75	1.00	1.00	-	-	-	-	1.00	1.00	-	-
Fb'	0.75	1.00	1.00	0.914	1.833	1.00	1.00	1.00	1.00	-	-
Fb1'	0.75	1.00	1.00	1.000	1.833	1.10	1.00	1.00	1.00	-	-
Fb2'	0.75	1.00	1.00	0.984	1.833	1.00	1.00	1.00	1.00	-	-
Fb3'	0.75	1.00	1.00	1.000	1.833	1.10	1.00	1.00	1.00	-	-
Fcp'	0.65	-	1.00	1.00	-	-	-	1.00	1.00	-	-
E'	1.6 million	1.00	1.00	-	-	-	-	1.00	1.00	-	-
Emin'	0.98 million	1.00	1.00	-	-	-	-	1.00	1.00	-	-

Shear : LC #2 = .6D+W, V = 186, V design = 183 lbs
 Bending(+): LC #2 = .6D+W, M = 158 lbe-ft
 Bending(-): LC #2 = .6D+W, M = 240 lbe-ft
 Deflection: LC #2 = .6D+W (live)
 LC #2 = .6D+W (total)
 EI = 9e06 lb-in² Ely = 2e06 lb-in²
 Total Deflection = 1.50(Dead Load Deflection) + Live Load Deflection.
 D=dead L=live S=snow W=wind I=impact L=roof live Lc=concentrated
 All LC's are listed in the Analysis output
 Load combinations: ASCE 7-08

DESIGN NOTES:

- Please verify that the default deflection limits are appropriate for your application.
- Continuous or Cantilevered Beams: NDS Clause 4.2.5.5 requires that normal grading provisions be extended to the middle 2/3 of 2 span beams and to the full length of cantilevers and other spans.
- Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4.1.