DOUGLAS D. HARDIN, P.E.

STRUCTURAL ENGINEERING CALCULATIONS

FOR

WOODHOUSE – THE TIMBER FRAME COMPANY

THE BURNS ADDITION

CLARK, CO

TGE PROJECT NUMBER: 24-24439

COLORADO LICENSED PROFESSIONAL ENGINEER
DOUGLAS DEAN HARDIN
57366
09/05/2024

REVIEWED FOR CODE COMPLIANCE
10/15/2024

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1 PROJECT INFORMATION

Tamarack Grove Engineering Information:

- Company Address: 812 S. La Cassia Dr. Boise, ID 83705
- Date: 5 September 2024
- Firm Registration Number: 20151163829 for Colorado
- Engineer of Record: Douglas D. Hardin, P.E.
- Project Manager: Guilherme Rodrigues Vieira, E.I.
- Direct Phone: 208.219.5961
- Direct Email: guilherme.rodrigues@tamarackgrove.com

Project Client Information:

- Company: Woodhouse – The Timber Frame Company
- Contact: Tood Mahosky
- Address: 3295 Route 549, PO Box 219 Mansfield, PA 16933
- Phone: (570) 549-6232
- Logo:

Woodhouse
THE TIMBER FRAME COMPANY™

Project Information:

- Name: The Burns Addition
- Address: Clark, CO 80428
- Local Jurisdiction: Routt
- Enforced Code Used: 2021 IRC
- Jurisdictional Contact Information: www.co.routt.co.us/119/Building-Codes-and-Policies
- Scope of Work:

1. Provide **CLIENT** with structural engineering and professional engineering stamp for the **PROJECT** in accordance with the latest edition of IBC/IRC and any local jurisdiction amendments, including:
   - Structural calculations for full gravity/lateral load path analysis.
   - Structural calculations for timber member elements, joinery, and fastener connections associated with a complete gravity only load path analysis.
   - 'Redline' drawings provided by **CLIENT**.
2. Respond to Agency and Owner comments required for issuance of building permits, limited to two rounds of plan revisions.
3. Provide Construction Administration services to the **CLIENT**, building contractor, and/or Owner, including:
   - Shop drawing/deferred submittal review.
   - Construction assistance by responding to questions and requests for information (RFI's) that arise during the construction phase of the **PROJECT**.

2 WIND ANALYSIS

Tamarack Grove Engineering
812 S. La Cassia Drive
Boise, ID 83705
208.345.8941

JOB TITLE The Burns Addition

JOB NO. 24-24439 SHEET NO.
CALCULATED BY GRV DATE
CHECKED BY DATE

Wind Loads - MWFRS h≤60' (Low-rise Buildings) except for open buildings

		Kz = Kh =	0.700	Edge Strip (a) =	3.0 ft
Base pressure (qh) =	**20.1 psf**			End Zone (2a) =	6.0 ft
GCpi =	+/-0.18			Zone 2 length =	12.0 ft

Wind Pressure Coefficients

	CASE A				CASE B		
		θ = 22.6 deg					
Surface	GCpf	w/-GCpi	w/+GCpi		GCpf	w/-GCpi	w/+GCpi
1	0.54	0.72	0.36		-0.45	-0.27	-0.63
2	-0.45	-0.27	-0.63		-0.69	-0.51	-0.87
3	-0.47	-0.29	-0.65		-0.37	-0.19	-0.55
4	-0.41	-0.23	-0.59		-0.45	-0.27	-0.63
5					0.40	0.58	0.22
6					-0.29	-0.11	-0.47
1E	0.77	0.95	0.59		-0.48	-0.30	-0.66
2E	-0.72	-0.54	-0.90		-1.07	-0.89	-1.25
3E	-0.65	-0.47	-0.83		-0.53	-0.35	-0.71
4E	-0.60	-0.42	-0.78		-0.48	-0.30	-0.66
5E					0.61	0.79	0.43
6E					-0.43	-0.25	-0.61

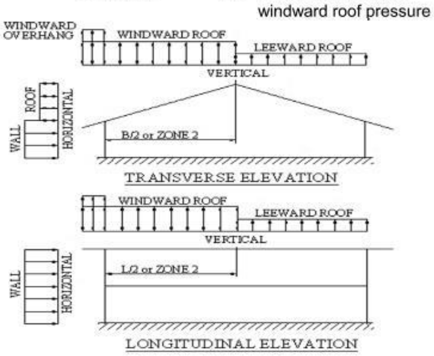
Ultimate Wind Surface Pressures (psf)

1	14.5	7.2		-5.4	-12.7
2	-5.5	-12.8		-10.3	-17.5
3	-5.8	-13.0		-3.8	-11.1
4	-4.7	-12.0		-5.4	-12.7
5				11.7	4.4
6				-2.2	-9.5
1E	19.2	11.9		-6.0	-13.3
2E	-10.9	-18.1		-17.9	-25.2
3E	-9.4	-16.7		-7.1	-14.3
4E	-8.4	-15.7		-6.0	-13.3
5E				15.9	8.7
6E				-5.0	-12.3

Parapet

Windward parapet = 0.0 psf (GCpn = +1.5)
Leeward parapet = 0.0 psf (GCpn = -1.0)

Windward roof overhangs = 14.1 psf (upward) add to windward roof pressure



Horizontal MWFRS Simple Diaphragm Pressures (psf)

Transverse direction (normal to L)

Interior Zone: Wall 19.2 psf
Roof 0.3 psf
End Zone: Wall 27.6 psf
Roof -1.4 psf **

Longitudinal direction (parallel to L)

Interior Zone: Wall 13.9 psf
End Zone: Wall 21.0 psf

** NOTE: Total horiz force shall not be less than that determined by neglecting roof forces (except for MWFRS moment frames).

The code requires the MWFRS be designed for a min ultimate force of 16 psf multiplied by the wall area plus an 8 psf force applied to the vertical projection of the roof.

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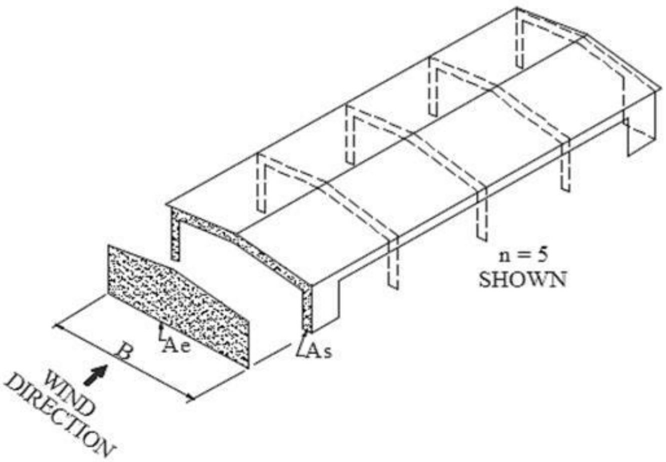
Wind Loads - h≤60' Longitudinal Direction MWFRS On Open or Partially Enclosed Buildings with Transverse Frames and Pitched Roofs

ASCE 7-16+ procedure

Base pressure (qh) = **20.1 psf**
GCpi = +/-0.18 Enclosed bldg, procdure doesn't apply
Roof Angle (θ) = 22.6 deg



B=	24.0 ft
# of frames (n) =	4
Solid are of end wall including fascia (As) =	26.0 sf
Roof ridge height =	23.5 ft
Roof eave height =	18.5 ft
Total end wall area if soild (Ae) =	504.0 sf

Longidinal Directional Force (F) = pAe
p= qh [(GCpf)windward -(GCpf)leeward] K_B K_S

Solidarity ratio (Φ) =	0.052
n =	4
KB =	1.56
KS =	0.679
Zones 5 & 6 area =	445 sf
5E & 6E area =	59 sf
(GCpf) windward - (GCpf) leeward] =	0.731
p =	15.6 psf

Total force to be resisted by MWFRS (F) = **7.9 kips** applied at the centroid of the end wall area Ae

Note: The longidudinal force acts in combination with roof loads calculated elsewhere for an open or partially enclosed building.

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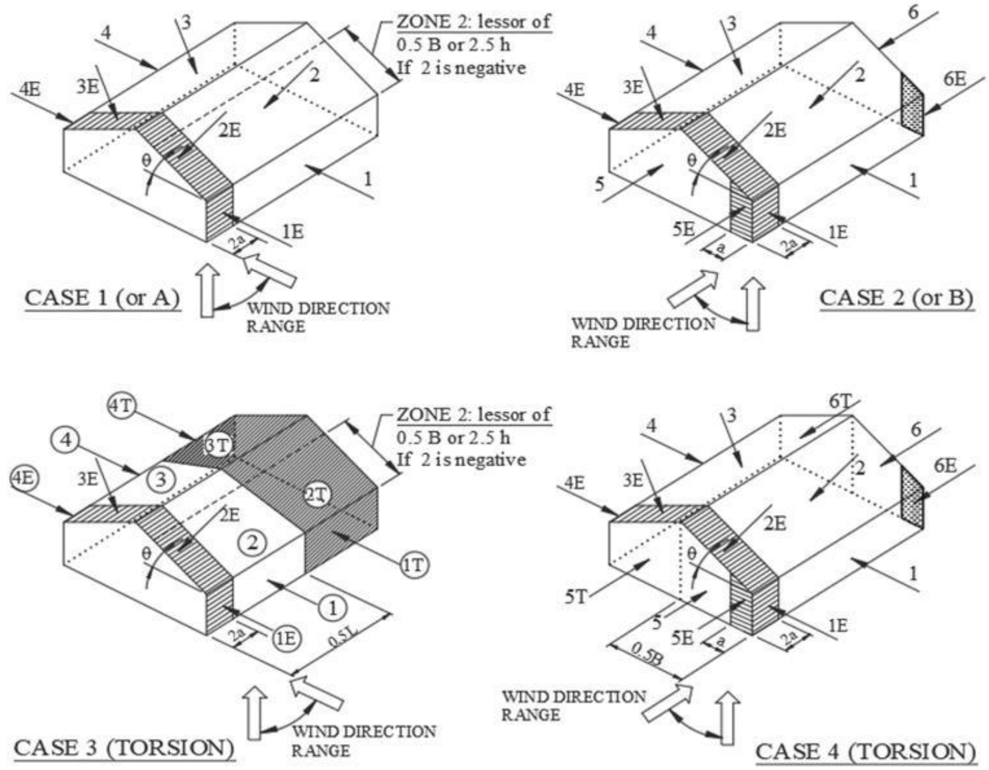
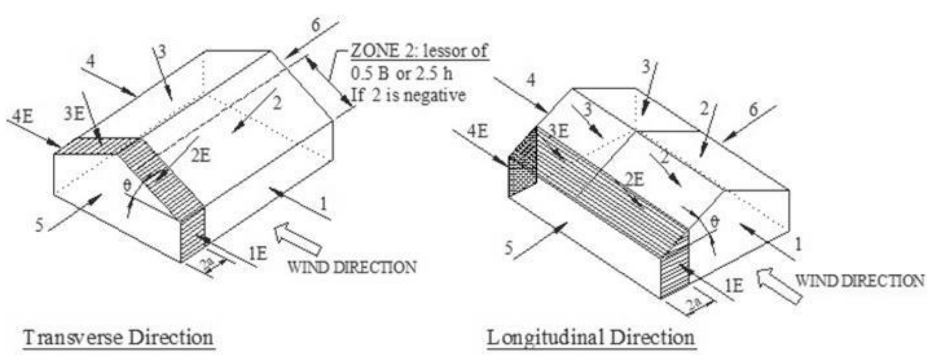


NOTE: Torsional loads are 25% of zones 1 - 6.
Exception: One story buildings h<30' and 1 to 2 storybuildings framed with light-frame construction or with flexible diaphragms need not be designed for the torsional load case.

ASCE 7-98 & ASCE 7-10 (& later) - MWFRS wind pressure zones



NOTE: Torsional loads are 25% of zones 1 - 4. See code for loading diagram.
Exception: One story buildings h<30' and 1 to 2 storybuildings framed with light-frame construction or with flexible diaphragms need not be designed for the torsional load case.

ASCE 7-02 and ASCE 7-05 - MWFRS wind pressure zones

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Ultimate Wind Pressures

Wind Loads - Components & Cladding : h ≤ 60'

		Kh =	0.633	~~100.0 ft~~	~~30.0 ft~~
Base pressure (qh) =	**18.2 psf**	h =	21.0 ft	~~100.0 ft~~	
Minimum parapet ht =	0.0 ft	a =	3.0 ft	~~100.0 ft~~	
Roof Angle (θ) =	22.6 deg	GCpi =	+/-0.18		
Type of roof =	Gable	qi = qh =	18.2 psf		

Roof

	Surface Pressure (psf)									User input	
Area	2 sf	4 sf	10 sf	20 sf	50 sf	100 sf	150 sf	300 sf	350 sf	70 sf	100 sf
Negative Zone 1 & 2e	-30.60	-30.60	-30.60	-30.60	-26.3	-23.0	-21.1	-17.8	-17.8	-24.7	-23.0
Negative Zone 2n, 2r &3e	-48.80	-48.80	-48.80	-42.70	-34.7	-28.7	-25.1	-25.1	-25.1	-31.8	-28.7
Negative Zone 3r	-68.80	-68.80	-56.90	-47.90	-36.1	-36.1	-36.1	-36.1	-36.1	-36.1	-36.1
Positive All Zones	16.00	16.00	16.00	16.00	16.0	16.0	16.0	16.0	16.0	16.0	16.0
Overhang Zone 1 & 2e	-36.40	-36.40	-36.40	-36.40	-35.2	-34.2	-33.7	-32.8	-32.8	-34.7	-34.2
Overhang Zone 2n & 2r	-54.60	-54.60	-54.60	-50.90	-46.0	-42.2	-40.1	-40.1	-40.1	-44.2	-42.2
Overhang Zone 3e	-65.50	-65.50	-65.50	-56.70	-45.0	-36.1	-31.0	-31.0	-31.0	-40.7	-36.1
Overhang Zone 3r	-85.60	-85.60	-69.70	-57.70	-41.9	-41.9	-41.9	-41.9	-41.9	-41.9	-41.9

Overhang pressures in the table above assume an internal pressure coefficient (Gcpi) of 0.0
Overhang soffit pressure equals adj wall pressure (which includes internal pressure of 3.3 psf)

Parapet

qp = 0.0 psf

	Surface Pressure (psf)							User input
Solid Parapet Pressure	4 sf	10 sf	20 sf	50 sf	150 sf	300 sf	500 sf	10 sf
CASE A: Zone 2e :	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Zone 2n, 2r & 3e :	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Zone 3r :	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
CASE B: Interior zone :	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Corner zone :	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Walls

	GCp +/- GCpi				Surface Pressure at h				User input	
Area	10 sf	100 sf	200 sf	500 sf	10 sf	100 sf	200 sf	500 sf	100 sf	100 sf
Negative Zone 4	-1.28	-1.10	-1.05	-0.98	-23.3	-20.1	-19.1	-17.8	-20.1	-20.1
Negative Zone 5	-1.58	-1.23	-1.12	-0.98	-28.8	-22.3	-20.4	-17.8	-22.3	-22.3
Positive Zone 4 & 5	1.18	1.00	0.95	0.88	21.5	18.3	17.3	16.0	18.3	18.3

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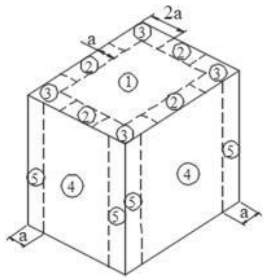
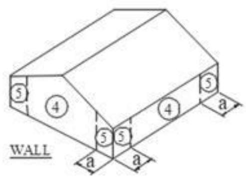
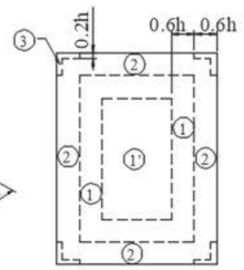
Location of C&C Wind Pressure Zones - ASCE 7-16



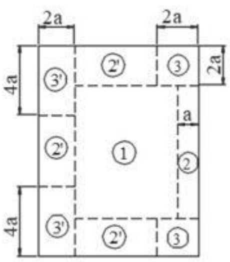
Roofs w/ θ ≤ 10°
and all walls
h > 60'



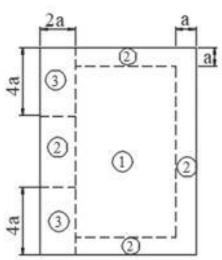
Walls h ≤ 60'
& alt design h<90'



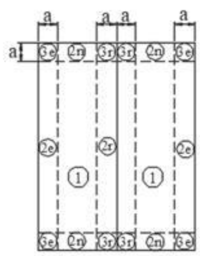
Multispan Gable & Sawtooth ≤ 10°
and Gable θ ≤ 7 degrees &
Monoslope ≤ 3 degrees
h ≤ 60' & alt design h<90'



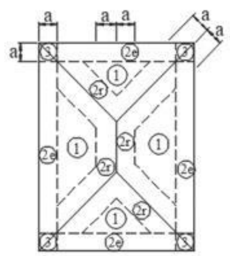
Monoslope roofs
3° < θ ≤ 10°
h ≤ 60' & alt design h<90'



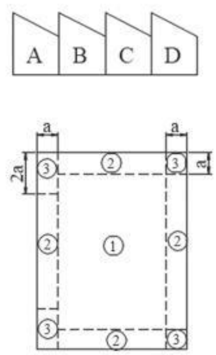
Monoslope roofs
10° < θ ≤ 30°
h ≤ 60' & alt design h<90'



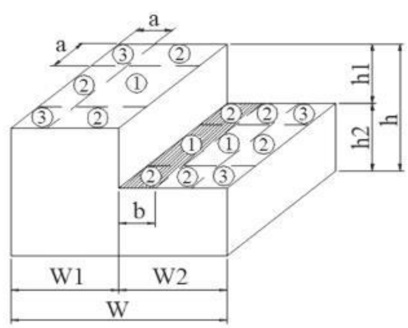
Multispan Gable > 10°
& Gable 7° < θ ≤ 45°

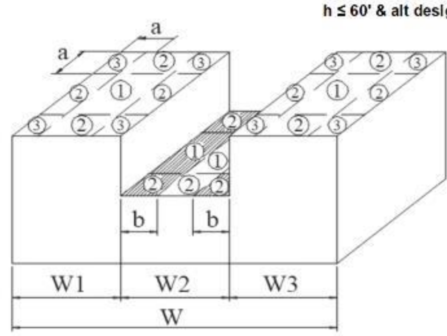


Hip 7° < θ ≤ 27°



Sawtooth 10° < θ ≤ 45°
h ≤ 60' & alt design h<90'





Note: The hatched area indicates where roof positive pressures are equal to the adjacent wall positive pressure.

Stepped roofs θ ≤ 3°
h ≤ 60' & alt design h<90'

Note: The stepped roof zones above are as shown in ASCE 7-16 . Prior editions didn't show zones, but the notes sent you to the low slope gable figure. The note in ASCE 7-16 still sends you to the low slope gable figure, but for some reason the zones shown are per editions prior to ASCE 7-16. Therefore, the above zones may be a code mistake and the correct zone locations may be per the low slope gable roof shown at the top of this page.

3 SEISMIC ANALYSIS

Tamarack Grove Engineering
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Seismic Loads: IBC 2021 — Strength Level Forces

Risk Category : II
Importance Factor (Ie) : 1.00
Site Class : D

Ss (0.2 sec) =	0.41 g	Fa =	1.473
S1 (1.0 sec) =	0.09 g	Fv =	2.400

Site specific ground motion analysis performed:

Sms =	0.602	S_{DS} =	0.402	Design Category =	C
Sm1 =	0.204	S_{D1} =	0.136	Design Category =	C

Seismic Design Category = **C**
Redundancy Coefficient ρ = 1.00
Number of Stories: 2

Structure Type: All other building systems
Horizontal Struct Irregularities: No plan Irregularity
Vertical Structural Irregularities: No vertical Irregularity

Flexible Diaphragms: Yes
Building System: **Building Frame Systems**
Seismic resisting system: **Light frame walls with shear panels-all other materials**
System Structural Height Limit: **Height not limited**
Actual Structural Height (hn) = 21.0 ft

DESIGN COEFFICIENTS AND FACTORS

Response Modification Coefficient (R) =	2.5	To = 0.2(Sd1/Sds) =	0.068
Over-Strength Factor (Ωo) =	2	Ts = Sd1/Sds =	0.339
Deflection Amplification Factor (Cd) =	2.5	Long Period Transition Period (TL) =	4 sec

S_{DS} = 0.402
S_{D1} = 0.136

Seismic Load Effect (E) = Eh +/-Ev = $\rho\, Q_E$ +/- $0.2S_{DS}$ D = Qe +/- 0.080D — Q_E = horizontal seismic force
Special Seismic Load Effect (Em) = Emh +/- Ev = $\Omega o\, Q_E$ +/- $0.2S_{DS}$ D = &G40&"(0.080D — D = dead load

ALLOWABLE STORY DRIFT

Structure Type: All other structures

Allowable story drift Δa = 0.020hsx where hsx is the story height below level ›

PERMITTED ANALYTICAL PROCEDURES

Index Force Analysis - Method Not Permitted (only applies to Seismic Category A

Model & Seismic Response Analysis - Permitted (see code for procedure)

Equivalent Lateral-Force (ELF) Analysis - Permitted

Building period coef. (C_T) =	0.020		Cu =	1.63
Approx fundamental period (Ta) =	$C_T h_n{}^x$ =	0.196 sec x= 0.75	Tmax = CuTa =	0.319 sec
User calculated fundamental period =			T =	0.196 sec

Seismic response coef. (Cs) =	SdsI/R =	0.161
need not exceed Cs =	Sd1 I /RT =	0.277
but not less than Cs =	0.044Sds*I =	0.018
USE Cs =		0.161

Design Base Shear V = 0.161W

4 SNOW ANALYSIS

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Snow Loads : ASCE 7- 16

Nominal Snow Forces

Roof slope	=	22.6 deg
Horiz. eave to ridge dist (W)	=	12.0 ft
Roof length parallel to ridge (L)	=	28.0 ft

Type of Roof		Hip and gable w/ rafters
Ground Snow Load	Pg =	152.0 psf
Risk Category	=	II
Importance Factor	I =	1.0
Roof R value	Rroof =	30
Thermal Factor	Ct =	1.000
Exposure Factor	Ce =	1.0
Pf = 0.7*Ce*Ct*I*Pg	=	103.7 psf
Unobstructed Slippery Surface		no
Sloped-roof Factor	Cs =	1.00
Balanced Snow Load	=	**103.7 psf**
Rain on Snow Surcharge Angle		0.24 deg
Code Maximum Rain Surcharge		5.0 psf
Rain on Snow Surcharge	=	0.0 psf
Ps plus rain surcharge	=	103.7 psf
Minimum Snow Load	Pm =	0.0 psf
Uniform Roof Design Snow Load	=	**103.7 psf**

Near ground level surface balanced snow load = **152.0 psf**

NOTE: Alternate spans of continuous beams shall be loaded with half the design roof snow load so as to produce the greatest possible effect - see code for loading diagrams and exceptions for gable roofs

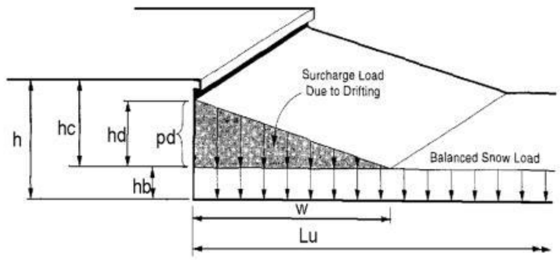
Unbalanced Snow Loads - for Hip & Gable roofs only

~~0.55~~

Required if slope is between 7 on 12 =	30.26 deg	
and 2.38 deg =	2.38 deg	**Unbalanced snow loads must be applied**
Windward snow load =	0.0 psf	
Leeward snow load =	152.0 psf	0.0 psf

Snow Drift 1 - Against roof projections, parapets, etc

Up or downwind fetch	lu =	220.0 ft
Projection height	h =	5.2 ft
Projection width/length	lp =	20.0 ft
Snow density	γ =	30.0 pcf
Balanced snow height	hb =	3.46 ft
	hd =	5.82 ft
	hc =	1.74 ft
hc/hb >0.2 = 0.5		**Therefore, design for drift**
Drift height (hc)	=	1.74 ft
Drift width	w =	13.95 ft
Surcharge load:	pd = γ*hd =	**52.3 psf**
Balanced Snow load:	=	103.7 psf
		156.0 psf



Note: If bottom of projection is at least 2 feet above hb then snow drift is not required.

Snow Drift 2- Against roof projections, parapets, etc

Up or downwind fetch	lu =	50.0 ft
Projection height	h =	4.0 ft
Projection width/length	lp =	20.0 ft
Snow density	γ =	30.0 pcf
Balanced snow height	hb =	3.46 ft
	hd =	3.11 ft
	hc =	0.54 ft
hc/hb <0.2 = 0.2		**Therefore, no drift**
Drift height (hc)	=	0.00 ft
Drift width	w =	4.35 ft
Surcharge load:	pd = γ*hd =	**0.0 psf**
Balanced Snow load:	=	103.7 psf
		103.7 psf

5 STRUCTURAL ENGINEERING CALCULATIONS

JURISDICTIONAL INFORMATION AND ABBREVIATIONS

BUILDING CODE USED: IBC 2021 w/ ASCE 7-16

GRAVITY LOAD DESIGN CRITERIA

$DL_{roof} := 18\ psf$	Roof dead load
$LL_{roof} := 20\ psf$	Roof live load (IBC Table 1607.1, occupancy 26)
$DL_{floor} := 12\ psf$	Floor dead load
$LL_{floor} := 40\ psf$	Floor live load (IBC Table 1607.1, occupancy 25)
$DL_{deck} := 10\ psf$	Deck dead load
$LL_{deck} := 1.5 \cdot LL_{floor} = 60\ psf$	Deck live load (IBC Table 1607.1, occupancy 5)
$DL_{wall} := 5\ psf$	Wall dead load (SIP)

SNOW LOAD CALCULATION

$p_g := 104\ psf$	Ground snow load
$\theta := 22.6\ °$	Roof slope angle, degrees
$p_{snow} := 104\ psf$	Design roof snow load (Excel)

***SEE ENERCALC FOR SNOW DRIFT, IF APPLICABLE**

LATERAL LOAD DESIGN CRITERIA - WIND

WIND PARAMETERS:

$p_{11} := 16.85\ psf$	Design wall wind pressure in 1-1 Direction (Wind Analysis)
$p_{22} := 13.8\ psf$	Design wall wind pressure in 2-2 Direction (Wind Analysis)
$p_{roof\_11} := 21.35\ psf$	Design roof wind pressure in 1-1 Direction (Wind Analysis)
$p_{roof\_22} := 21.35\ psf$	Design roof wind pressure in 2-2 Direction (Wind Analysis)

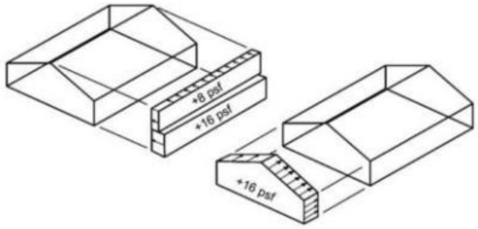


FIGURE C27.1-1. Application of Minimum Wind Load

$p_{11\_min} := 16\ psf$

$p_{roof\_min} := 8\ psf$ Minimum design wind loads (ASCE sec. 27.1.5)

$p_{22\_min} := 16\ psf$

LATERAL LOAD DESIGN CRITERIA - SEISMIC

SEISMIC PARAMETERS AND RESPONSE COEFFICENT CALCULATION:

D — Seismic site class

$S_S := 0.41$ — Spectral response acceleration parameter at short periods

$S_1 := 0.09$ — Spectral response acceleration parameter at a period of 1 s

$S_{DS} := 0.402$ — Design spectral response acceleration parameter at short periods

$F_v := 2.4$ — Long-period site coefficient (ASCE Table 11.4-2)

$S_{D1} := \frac{2}{3} \cdot F_v \cdot S_1 = 0.14$ — Design spectral response acceleration parameter at a period of 1 s (ASCE Eq. 11.4-4)

$SDC := \text{“C”}$ — Seismic design category (ASCE sec. 11.6)

$R_c := \text{“II”}$ — Risk category

$I_e := \text{if}\left(R_c = \text{“I”}, 1.0, \text{if}\left(R_c = \text{“II”}, 1.0, \text{if}\left(R_c = \text{“III”}, 1.25, 1.5\right)\right)\right) = 1$ — Seismic importance factor (ASCE Table 1.5-2)

$C_s := 0.161$ — Design seismic response coefficient (Seismic Analysis)

$W_{roof} := \text{if}\left(p_{snow} \le 30\ psf, DL_{roof}, DL_{roof} + 0.2 \cdot p_{snow}\right) = 38.8\ psf$ — Effective seismic weight, roof (ASCE sec. 12.7.2)

$A_{roof} := 913\ ft^2$ — Roof area

$A_{floor} := 793\ ft^2$ — Floor area

$L := 28\ ft$ — Building length

$B_1 := 36\ ft$ $\quad B_2 := 24\ ft$ — Building width

$h_{e\_2nd} := 11\ ft$ — Second story height

$h_{e\_1st} := 10\ ft$ — First story height

$$w_2 := W_{roof} \cdot A_{roof} + 2 \cdot B_1 \cdot DL_{wall} \cdot \frac{h_{e\_2nd}}{2} + 1.5 \cdot L \cdot DL_{wall} \cdot \frac{h_{e\_2nd}}{2} = 38559.4\ lbf$$

Effective seismic weight, roof (ASCE sec. 12.7.2)

$$w_1 := DL_{floor} \cdot A_{floor} + 2 \cdot B_2 \cdot DL_{wall} \cdot \frac{\left(h_{e\_1st} + h_{e\_2nd}\right)}{2} + 2.5 \cdot L \cdot DL_{wall} \cdot \frac{h_{e\_1st} + h_{e\_2nd}}{2} = 15711\ lbf$$

Effective seismic weight, level 1 (ASCE sec. 12.7.2)

$k := 1.0$ — Exponent related to structure period (ASCE sec. 12.8.3, T<0.5)

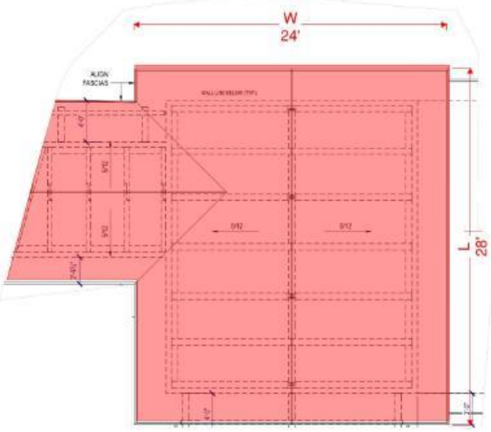
$V := C_s \cdot \left(w_2 + w_1\right) = 8737.53\ lbf$ — Seismic base shear (ASCE eq. 12.8-1)

$$F_1 := \left(\frac{w_1 \cdot h_{e\_1st}^{\ k}}{\left(w_1 \cdot h_{e\_1st}^{\ k} + w_2 \cdot \left(h_{e\_1st} + h_{e\_2nd}\right)^k\right)}\right) \cdot V = 1419.81\ lbf$$

Lateral seismic force, level 1 (ASCE sec. 12.8.3)

$F_2 := \left(\frac{w_2 \cdot (h_{e\_1st} + h_{e\_2nd})^k}{\left(w_1 \cdot h_{e\_1st}{}^k + w_2 \cdot (h_{e\_1st} + h_{e\_2nd})^k\right)} \right) \cdot V = 7317.72\ lbf$ Lateral seismic force, roof (ASCE sec. 12.8.3)

STRUCTURAL CALCULATIONS (LATERAL)



$A_{roof} = 913\ ft^2$ Roof area

$L = 28\ ft$

$W := 24\ ft$

$h_e := 9.2\ ft$ Eave height (top plate)

$h_r := 5.8\ ft$ Roof height (rigde to top plate)

DIAPHRAGM LOAD GENERATION - DIRECTION 1-1

$F_{p\_wind\_min\_1\_1} := L \cdot \left(p_{11\_min} \cdot \frac{h_e}{2} + p_{roof\_min} \cdot h_r \right) = 3360\ lbf$ Diaphragm load, wind (Minimum)

$F_{p\_wind\_1\_1} := L \cdot \left(p_{11} \cdot \frac{h_e}{2} + p_{roof\_11} \cdot (\cos(90° - \theta)) \cdot h_r \right) = 3502.72\ lbf$ Diaphragm load, wind (1-1 Direction)

$F_{p\_wind\_max\_1\_1} := \max(F_{p\_wind\_min\_1\_1}, F_{p\_wind\_1\_1}) = 3502.72\ lbf$ Design diaphragm load (Wind)

$F_{px\_1\_1} := F_2 = 7317.72\ lbf$ Diaphragm load, seismic

$Check := if(0.7 \cdot F_{px\_1\_1} \ge 0.6 \cdot F_{p\_wind\_max\_1\_1}, \text{"SEISMIC CONTROLS"}, \text{"WIND CONTROLS"}) = \text{"SEISMIC CONTROLS"}$

$F_{p\_11} := \max(0.6 \cdot F_{p\_wind\_max\_1\_1}, 0.7 \cdot F_{px\_1\_1}) = 5.12\ kip$ Controlling diaphragm load (Service level)

DIAPHRAGM LOAD GENERATION - DIRECTION 2-2

$F_{p\_wind\_min\_2\_2} := W \cdot \left(p_{22\_min} \cdot \frac{h_e}{2} + p_{22\_min} \cdot \frac{h_r}{2} \right) = 2880\ lbf$ Diaphragm load, wind (Minimum)

$F_{p\_wind\_2\_2} := W \cdot p_{22} \cdot \left(\frac{h_r}{2} + \frac{h_e}{2} \right) = 2484\ lbf$ Diaphragm load, wind (2-2 Direction)

$F_{p\_wind\_max\_2\_2} := \max(F_{p\_wind\_min\_2\_2}, F_{p\_wind\_2\_2}) = 2880\ lbf$ Design diaphragm load (Wind)

$F_{px\_2\_2} := F_2 = 7317.72\ lbf$ Diaphragm load, seismic

$Check := if(0.7 \cdot F_{px\_2\_2} \ge 0.6 \cdot F_{p\_wind\_max\_2\_2}, \text{"SEISMIC CONTROLS"}, \text{"WIND CONTROLS"}) = \text{"SEISMIC CONTROLS"}$

$F_{p\_22} := \max(0.6 \cdot F_{p\_wind\_max\_2\_2}, 0.7 \cdot F_{px\_2\_2}) = 5.12\ kip$ Controlling diaphragm load (Service level)

DIAPHRAGM CHECK

$L = 28\ ft$	Diaphragm length
$W = 24\ ft$	Diaphragm depth
$A_{Ratio} := \max\left(\frac{L}{W}, \frac{W}{L}\right) = 1.17$	Worst case aspect ratio
$Check := if\left(A_{Ratio} \le 3.0, \text{"Blk'g Not Req'd"}, if\left(3.0 < A_{Ratio} \le 4.0, \text{"Block Diaphragm"}, \text{"Shape Ratio Exceeded"}\right)\right) = \text{"Blk'g Not Req'd"}$	SDPWS Table 4.2.4
$V_{all\_11} := 330\ plf$	Unblocked Diaphragm Allowable Shear in 1-1 Direction (ESR-4696)
$V_{all\_22} := 330\ plf$	Unblocked Diaphragm Allowable Shear in 2-2 Direction (ESR-4696)
$V_{max\_11} := \frac{F_{p\_11}}{W} = 213.43\ plf$	Worst case diaphragm shear force in 1-1 direction (Service Level)
$V_{max\_22} := \frac{F_{p\_22}}{2 \cdot L} = 91.47\ plf$	Worst case diaphragm shear force in 2-2 direction

$Check := if\left(V_{all\_11} \ge V_{max\_11}, \text{"PASS"}, \text{"FAIL"}\right) = \text{"PASS"}$

$Check := if\left(V_{all\_22} \ge V_{max\_22}, \text{"PASS"}, \text{"FAIL"}\right) = \text{"PASS"}$

CHORD CHECK

$F_{chord} := \max\left(\frac{F_{p\_11} \cdot \frac{L}{2}}{W}, \frac{F_{p\_22} \cdot W}{8 \cdot L}\right) = 2988.07\ lbf$	Max chord force
$C_{D\_WL} := 1.60$	Load duration factor for seismic/wind (NDS Table 2.3.2)
$F_t := 450\ psi$	Reference tension parallel to grain design value (NDS Supp. Table 4A, SPF #1/#2)
$F_c := 1150\ psi$	Reference compression parallel to grain design value (NDS Supp. Table 4A, SPF #1/#2)
$C_{f\_t} := 1.3$	Size factor, tension (NDS Supp. Table 4A, 2X6)
$C_{f\_c} := 1.1$	Size factor, compression (NDS Supp. Table 4A, 2X6)
$F_t' := F_t \cdot C_{D\_WL} \cdot C_{f\_t} = 936\ psi$	Allowable tension capacity (NDS Table 4.3.1)
$F_c' := F_c \cdot C_{D\_WL} \cdot C_{f\_c} = 2024\ psi$	Allowable bearing capacity (NDS Table 4.3.1, NDS Sec. 3.10.1)
$A_{2x6} := 8.25\ in^2$	Area of section, 2X6
$f_{t\_c} := \frac{F_{chord}}{A_{2x6}} = 362.19\ psi$	Tensile/Compressive stress

$Check := if\left(F_t' \geq f_{t\_c}, \text{"PASS"}, \text{"FAIL"}\right) = \text{"PASS"}$

$Check := if\left(F_c' \geq f_{t\_c}, \text{"PASS"}, \text{"FAIL"}\right) = \text{"PASS"}$

SHEAR WALLS AT ROOF LEVEL

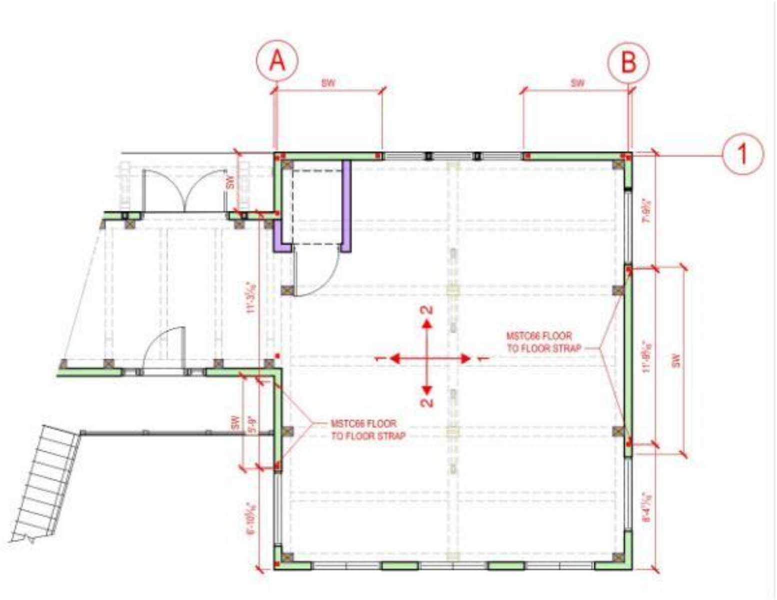
$A_{roof} = 913\ ft^2$ Roof area

$A_1 := 913\ ft^2$ $r_1 := \dfrac{A_1}{A_{roof}} = 1$

$A_A := 457\ ft^2$ $r_A := \dfrac{A_A}{A_{roof}} = 0.5$

Tributary Area Ratio of Each Grid Line

$A_B := 457\ ft^2$ $r_B := \dfrac{A_B}{A_{roof}} = 0.5$



$V_{max\_11} := F_{p\_11} = 5122.41\ lbf$ Story shear force in 1-1 direction (Service Level)

$V_{max\_22} := F_{p\_22} = 5122.41\ lbf$ Story shear force in 2-2 direction (Service Level)

SHEAR WALL - GRIDLINE 1 - SEGMENTED SHEAR WALL

$L_1 := 7\ ft$ $L_2 := 7\ ft$ Pier lengths

$h := 11.56\ ft$ Wall height

$A_{Ratio\_1} := \dfrac{h}{L_1} = 1.65$ $A_{Ratio\_2} := \dfrac{h}{L_2} = 1.65$ Aspect ratio

$Check := if\left(A_{Ratio\_1} \leq 2, \text{"Blk'g Not Req'd"}, if\left(2 < A_{Ratio\_1} \leq 3.5, \text{"Blocking Req'd"}, \text{"Shape Ratio Exceeded"}\right)\right) = \text{"Blk'g Not Req'd"}$ SDPWS Table 4.3.4

$Check := if\left(A_{Ratio\_2} \leq 2, \text{"Blk'g Not Req'd"}, if\left(2 < A_{Ratio\_2} \leq 3.5, \text{"Blocking Req'd"}, \text{"Shape Ratio Exceeded"}\right)\right) = \text{"Blk'g Not Req'd"}$ SDPWS Table 4.3.4

$V_{1\_R} := V_{max\_11} \cdot r_1 = 5122.41\ lbf$ Induced shear force in shear wall

$A_{ratio\_factor\_1} := \text{if}\left(A_{Ratio\_1} > 2, min\left(\frac{2 \cdot L_1}{h}, 1.25 - \frac{0.125 \cdot h}{L_1}\right), 1.0\right) = 1$ Aspect ratio factor, WSP (SDPWS sec. 4.3.3.4.1.1)

$A_{ratio\_factor\_2} := \text{if}\left(A_{Ratio\_2} > 2, min\left(\frac{2 \cdot L_2}{h}, 1.25 - \frac{0.125 \cdot h}{L_2}\right), 1.0\right) = 1$ Aspect ratio factor, WSP (SDPWS sec. 4.3.3.4.1.1)

$v_{1\_1} := \frac{V_{1\_R} \cdot (A_{ratio\_factor\_1} \cdot L_1)}{(A_{ratio\_factor\_1} \cdot L_1 + A_{ratio\_factor\_2} \cdot L_2) \cdot (A_{ratio\_factor\_1} \cdot L_1)} = 365.89\ plf$ Maximum induced unit shear wall force

$v_{1\_2} := \frac{V_{1\_R} \cdot (A_{ratio\_factor\_2} \cdot L_2)}{(A_{ratio\_factor\_1} \cdot L_1 + A_{ratio\_factor\_2} \cdot L_2) \cdot (A_{ratio\_factor\_2} \cdot L_2)} = 365.89\ plf$ Maximum induced unit shear wall force

$v_s := 380\ plf$ Nominal unit shear wall capacity (ESR-4696)

$Check := \text{if}\left(v_{1\_1} \ge v_s, \text{“Shear wall not adequate”}, \text{“OK”}\right) = \text{“OK”}$

$Check := \text{if}\left(v_{1\_2} \ge v_s, \text{“Shear wall not adequate”}, \text{“OK”}\right) = \text{“OK”}$

$T_{width\_roof} := 6\ ft$ Roof Tributary Width

$w_{DL} := DL_{roof} \cdot T_{width\_roof} + DL_{wall} \cdot h = 165.8\ plf$ Roof dead load

$T_{1\_1} := \frac{v_{1\_1} \cdot (A_{ratio\_factor\_1} \cdot L_1) \cdot h}{L_1} - \frac{(0.6 - 0.14\ S_{DS}) \cdot w_{DL} \cdot \frac{L_1^2}{2}}{L_1} = 3914.12\ lbf$ Required holdown capacity (Service level, SDPWS eq. 4.3-8)

$T_{1\_2} := \frac{v_{1\_2} \cdot (A_{ratio\_factor\_2} \cdot L_2) \cdot h}{L_2} - \frac{(0.6 - 0.14\ S_{DS}) \cdot w_{DL} \cdot \frac{L_2^2}{2}}{L_2} = 3914.12\ lbf$ Required holdown capacity (Service level, SDPWS eq. 4.3-8)

$T_{all} := 4340\ lbf$ Allowable holdown force (HDU5)

$Check := \text{if}\left(T_{1\_1} \ge T_{all}, \text{“Holdown not adequate”}, \text{“OK”}\right) = \text{“OK”}$

$Check := \text{if}\left(T_{1\_2} \ge T_{all}, \text{“Holdown not adequate”}, \text{“OK”}\right) = \text{“OK”}$

SHEAR WALL - GRIDLINE A - SEGMENTED SHEAR WALL

$L_1 := 5.75\ ft$ $L_2 := 4\ ft$ Pier lengths

$h := 9.5\ ft$ Wall height

$A_{Ratio\_1} := \frac{h}{L_1} = 1.65$ $A_{Ratio\_2} := \frac{h}{L_2} = 2.38$ Aspect ratio

$Check := \text{if}\left(A_{Ratio\_1} \le 2, \text{“Blk'g Not Req'd”}, \text{if}\left(2 < A_{Ratio\_1} \le 3.5, \text{“Blocking Req'd”}, \text{“Shape Ratio Exceeded”}\right)\right) = \text{“Blk'g Not Req'd”}$ SDPWS Table 4.3.4

$Check := \text{if}\left(A_{Ratio\_2} \le 2, \text{"Blk'g Not Req'd"}, \text{if}\left(2 < A_{Ratio\_2} \le 3.5, \text{"Blocking Req'd"}, \text{"Shape Ratio Exceeded"}\right)\right) = \text{"Blocking Req'd"}$ SDPWS Table 4.3.4

$V_{A\_R} := V_{max\_22} \cdot r_A = 2564.01\ \text{lbf}$ Induced shear force in shear wall

$A_{ratio\_factor\_1} := \text{if}\left(A_{Ratio\_1} > 2, min\left(\frac{2 \cdot L_1}{h}, 1.25 - \frac{0.125 \cdot h}{L_1}\right), 1.0\right) = 1$ Aspect ratio factor, WSP (SDPWS sec. 4.3.3.4.1.1)

$A_{ratio\_factor\_2} := \text{if}\left(A_{Ratio\_2} > 2, min\left(\frac{2 \cdot L_2}{h}, 1.25 - \frac{0.125 \cdot h}{L_2}\right), 1.0\right) = 0.84$ Aspect ratio factor, WSP (SDPWS sec. 4.3.3.4.1.1)

$v_{A\_1} := \frac{V_{A\_R} \cdot \left(A_{ratio\_factor\_1} \cdot L_1\right)}{\left(A_{ratio\_factor\_1} \cdot L_1 + A_{ratio\_factor\_2} \cdot L_2\right) \cdot \left(A_{ratio\_factor\_1} \cdot L_1\right)} = 281.19\ \text{plf}$ Maximum induced unit shear wall force

$v_{A\_2} := \frac{V_{A\_R} \cdot \left(A_{ratio\_factor\_2} \cdot L_2\right)}{\left(A_{ratio\_factor\_1} \cdot L_1 + A_{ratio\_factor\_2} \cdot L_2\right) \cdot \left(A_{ratio\_factor\_2} \cdot L_2\right)} = 281.19\ \text{plf}$ Maximum induced unit shear wall force

$v_s := 380\ \text{plf}$ Nominal unit shear wall capacity (ESR-4696)

$Check := \text{if}\left(v_{A\_1} \ge v_s, \text{"Shear wall not adequate"}, \text{"OK"}\right) = \text{"OK"}$

$Check := \text{if}\left(v_{A\_2} \ge v_s, \text{"Shear wall not adequate"}, \text{"OK"}\right) = \text{"OK"}$

$T_{width\_roof} := 2\ \text{ft}$ Roof Tributary Width

$w_{DL} := DL_{roof} \cdot T_{width\_roof} + DL_{wall} \cdot h = 83.5\ \text{plf}$ Roof dead load

$T_{A\_1} := \frac{v_{A\_1} \cdot \left(A_{ratio\_factor\_1} \cdot L_1\right) \cdot h}{L_1} - \frac{\left(0.6 - 0.14\ S_{DS}\right) \cdot w_{DL} \cdot \frac{L_1^{\ 2}}{2}}{L_1} = 2540.78\ \text{lbf}$ Required holdown capacity (Service level, SDPWS eq. 4.3-8)

$T_{A\_2} := \frac{v_{A\_2} \cdot \left(A_{ratio\_factor\_2} \cdot L_2\right) \cdot h}{L_2} - \frac{\left(0.6 - 0.14\ S_{DS}\right) \cdot w_{DL} \cdot \frac{L_2^{\ 2}}{2}}{L_2} = 2158.72\ \text{lbf}$ Required holdown capacity (Service level, SDPWS eq. 4.3-8)

$T_{all} := 4340\ \text{lbf}$ Allowable holdown force (HDU5)

$T_{all} := 4130\ \text{lbf}$ Allowable holdown force (MSTC66)

$Check := \text{if}\left(T_{A\_1} \ge T_{all}, \text{"Holdown not adequate"}, \text{"OK"}\right) = \text{"OK"}$

$Check := \text{if}\left(T_{A\_2} \ge T_{all}, \text{"Holdown not adequate"}, \text{"OK"}\right) = \text{"OK"}$

SHEAR WALL - GRIDLINE B - INDIVIDUAL FULL-HEIGHT SHEAR WALL

$L_{tot} := 11.75\ \text{ft}$ Wall length

$h := 9.5\ \text{ft}$ Wall height

$A_{Ratio} := \frac{h}{L_{tot}} = 0.81$ Worst case aspect ratio

$Check := \text{if}\left(A_{Ratio} \le 2, \text{"Blk'g Not Req'd"}, \text{if}\left(2 < A_{Ratio} \le 3.5, \text{"Blocking Req'd"}, \text{"Shape Ratio Exceeded"}\right)\right) = \text{"Blk'g Not Req'd"}$ SDPWS Table 4.3.4

$V_{B\_R} := V_{max\_22} \cdot r_B = 2564.01\ \text{lbf}$ Induced shear force in shear wall

$v_B := \frac{V_{B\_R}}{L_{tot}} = 218.21\ \text{plf}$ Maximum induced unit shear wall force

$v_s := 380\ \text{plf}$ Nominal unit shear wall capacity (ESR-4696)

$Check := \text{if}\left(v_B \ge v_s, \text{"Shear wall not adequate"}, \text{"OK"}\right) = \text{"OK"}$

$T_{width\_roof} := 3\ \text{ft}$ Roof Tributary Width

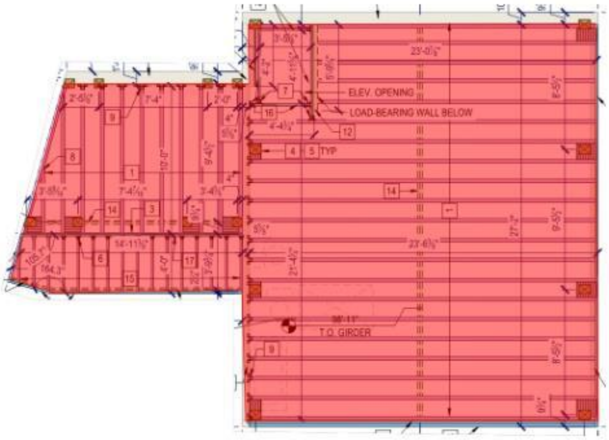
$w_{DL} := DL_{roof} \cdot T_{width\_roof} + DL_{wall} \cdot h = 101.5\ \text{plf}$ Roof dead load

$T_B := \frac{V_{B\_R} \cdot h}{L_{tot}} - \frac{\left(0.6 - 0.14\ S_{DS}\right) \cdot w_{DL} \cdot \frac{L_{tot}^{\ 2}}{2}}{L_{tot}} = 1748.8\ \text{lbf}$ Required holdown capacity (Service level, SDPWS eq. 4.3-8)

$T_{all} := 4130\ \text{lbf}$ Allowable holdown force (MSTC66)

$Check := \text{if}\left(T_B \ge T_{all}, \text{"Holdown not adequate"}, \text{"OK"}\right) = \text{"OK"}$

STRUCTURAL CALCULATIONS (LATERAL)



$A_{floor} = 793\ \text{ft}^2$ Floor area

$L = 28\ \text{ft}$

$W := 24\ \text{ft}$

$h_e := 18.5\ \text{ft}$ Eave height (top plate)

DIAPHRAGM LOAD GENERATION - DIRECTION 1-1

$F_{p\_wind\_min\_1\_1\_floor} := L \cdot \left(p_{11\_min} \cdot \frac{h_e}{2}\right) = 4144\ \text{lbf}$ Diaphragm load, wind (Minimum)

$F_{p\_wind\_1\_1\_floor} := L \cdot \left(p_{11} \cdot \frac{h_e}{2}\right) = 4364.15\ \text{lbf}$ Diaphragm load, wind (1-1 Direction)

$F_{p\_wind\_max\_1\_1\_floor} := \max\left(F_{p\_wind\_min\_1\_1\_floor}, F_{p\_wind\_1\_1\_floor}\right) = 4364.15\ lbf$ Design diaphragm load (Wind)

$F_{px\_1\_1} := F_1 = 1419.81\ lbf$ Diaphragm load, seismic

$Check := if\left(0.7 \cdot F_{px\_1\_1} \geq 0.6 \cdot F_{p\_wind\_max\_1\_1\_floor}, \text{"SEISMIC CONTROLS"}, \text{"WIND CONTROLS"}\right) = \text{"WIND CONTROLS"}$

$F_{p\_11\_floor} := \max\left(0.6 \cdot F_{p\_wind\_max\_1\_1\_floor}, 0.7 \cdot F_{px\_1\_1}\right) = 2.62\ kip$ Controlling diaphragm load (Service level)

DIAPHRAGM LOAD GENERATION - DIRECTION 2-2

$F_{p\_wind\_min\_2\_2\_floor} := W \cdot \left(p_{22\_min} \cdot \frac{h_e}{2}\right) = 3552\ lbf$ Diaphragm load, wind (Minimum)

$F_{p\_wind\_2\_2\_floor} := W \cdot p_{22} \cdot \left(\frac{h_e}{2}\right) = 3063.6\ lbf$ Diaphragm load, wind (2-2 Direction)

$F_{p\_wind\_max\_2\_2\_floor} := \max\left(F_{p\_wind\_min\_2\_2\_floor}, F_{p\_wind\_2\_2\_floor}\right) = 3552\ lbf$ Design diaphragm load (Wind)

$F_{px\_2\_2} := F_1 = 1419.81\ lbf$ Diaphragm load, seismic

$Check := if\left(0.7 \cdot F_{px\_2\_2} \geq 0.6 \cdot F_{p\_wind\_max\_2\_2\_floor}, \text{"SEISMIC CONTROLS"}, \text{"WIND CONTROLS"}\right) = \text{"WIND CONTROLS"}$

$F_{p\_22\_floor} := \max\left(0.6 \cdot F_{p\_wind\_max\_2\_2\_floor}, 0.7 \cdot F_{px\_2\_2}\right) = 2.13\ kip$ Controlling diaphragm load (Service level)

DIAPHRAGM CHECK

$L = 28\ ft$ Diaphragm length

$W = 24\ ft$ Diaphragm depth

$A_{Ratio} := \max\left(\frac{L}{W}, \frac{W}{L}\right) = 1.17$ Worst case aspect ratio

$Check := if\left(A_{Ratio} \leq 3.0, \text{"Blk'g Not Req'd"}, if\left(3.0 < A_{Ratio} \leq 4.0, \text{"Block Diaphragm"}, \text{"Shape Ratio Exceeded"}\right)\right) = \text{"Blk'g Not Req'd"}$ SDPWS Table 4.2.4

$\Omega_{ASD} := 2$ ASD reduction factor (SDPWS sec. 4.2.3)

$V_{all\_11} := 300\ plf$ Unblocked Diaphragm Allowable Shear in 1-1 Direction (SDPWS)

$V_{all\_22} := 300\ plf$ Unblocked Diaphragm Allowable Shear in 2-2 Direction (SDPWS)

$V_{max\_11} := \frac{F_{p\_11}}{2 \cdot W} = 106.72\ plf$ Worst case diaphragm shear force in 1-1 direction (Service Level)

$V_{max\_22} := \frac{F_{p\_22}}{2 \cdot L} = 91.47\ plf$ Worst case diaphragm shear force in 2-2 direction

$Check := if\left(V_{all\_11} \geq V_{max\_11}, \text{"PASS"}, \text{"FAIL"}\right) = \text{"PASS"}$

$Check := if\left(V_{all\_22} \geq V_{max\_22}, \text{"PASS"}, \text{"FAIL"}\right) = \text{"PASS"}$

CHORD CHECK

$F_{chord} := \max\left(\dfrac{F_{p\_11\_floor} \cdot \dfrac{L}{2}}{W}, \dfrac{F_{p\_22\_floor} \cdot W}{8 \cdot L}\right) = 1527.45\ lbf$ Max chord force

$C_{D\_WL} := 1.60$ Load duration factor for seismic/wind (NDS Table 2.3.2)

$F_t := 450\ psi$ Reference tension parallel to grain design value (NDS Supp. Table 4A, SPF #1/#2)

$F_c := 1150\ psi$ Reference compression parallel to grain design value (NDS Supp. Table 4A, SPF #1/#2)

$C_{f\_t} := 1.3$ Size factor, tension (NDS Supp. Table 4A, 2X6)

$C_{f\_c} := 1.1$ Size factor, compression (NDS Supp. Table 4A, 2X6)

$F_t' := F_t \cdot C_{D\_WL} \cdot C_{f\_t} = 936\ psi$ Allowable tension capacity (NDS Table 4.3.1)

$F_c' := F_c \cdot C_{D\_WL} \cdot C_{f\_c} = 2024\ psi$ Allowable bearing capacity (NDS Table 4.3.1, NDS Sec. 3.10.1)

$A_{2x6} := 8.25\ in^2$ Area of section, 2X6

$f_{t\_c} := \dfrac{F_{chord}}{A_{2x6}} = 185.15\ psi$ Tensile/Compressive stress

$Check := \text{if}\left(F_t' \ge f_{t\_c}, \text{“PASS”}, \text{“FAIL”}\right) = \text{“PASS”}$

$Check := \text{if}\left(F_c' \ge f_{t\_c}, \text{“PASS”}, \text{“FAIL”}\right) = \text{“PASS”}$

SHEAR WALLS AT FLOOR LEVEL

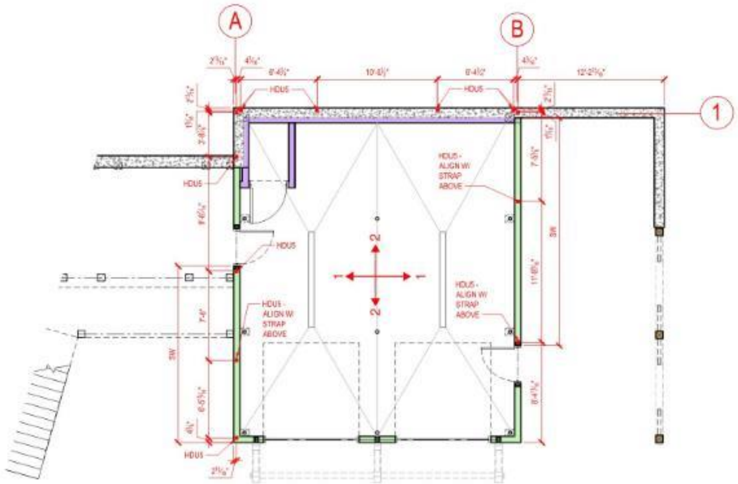
$A_{floor} = 793\ ft^2$ Floor area

$A_1 := 672\ ft^2$ $r_1 := \dfrac{A_1}{A_{floor}} = 0.85$

$A_A := 336\ ft^2$ $r_A := \dfrac{A_A}{A_{floor}} = 0.42$

$A_B := 336\ ft^2$ $r_B := \dfrac{A_B}{A_{floor}} = 0.42$

Tributary Area Ratio of Each Grid Line



$V_{max\_11} := F_{p\_11\_floor} = 2618.49\ \text{lbf}$ — Story shear force in 1-1 direction (Service Level)

$V_{max\_22} := F_{p\_22\_floor} = 2131.2\ \text{lbf}$ — Story shear force in 2-2 direction (Service Level)

SHEAR WALL - GRIDLINE 1 - ORDINARY REINFORCED CONCRETE SHEAR WALL (OSW)

$L_w := 22\ \text{ft}$ — Length of wall

$h := 10\ \text{in}$ — Wall thickness

$d := 0.8 \cdot L_w = 17.6\ \text{ft}$ — Distance from extreme compression fiber to centroid of longitudinal tension reinforcement (ACI sec. 11.5.4.2)

$\lambda := 1.0$ — Modification factor (ACI Table 19.2.4.2, Normalweight)

$f'_c := 2500\ \text{psi}$ — Specified compressive strength of concrete

$f_y := 60000\ \text{psi}$ — Specified yield strength of reinforcement

$\phi := 0.75$ — Strength reduction factor (ACI Table 21.2.1, Shear)

$V_c := 2 \cdot \dfrac{\text{lbf}^{\frac{1}{2}}}{\text{in}} \cdot \lambda \cdot \sqrt{f'_c} \cdot h \cdot d = 211200\ \text{lbf}$ — Nominal shear strength provided by concrete (ACI sec. 11.5.4.4)

$V_{1\_R\_floor} := V_{max\_11} \cdot r_1 + V_{1\_R} = 7341.35\ \text{lbf}$ — Induced shear force in shear wall

$$Check := \text{if}\left(\phi \cdot V_c \geq \frac{V_{1\_R}}{0.7}, \text{“Shear wall adequate”}, \text{“Shear wall not adequate, add reinforcement”}\right) = \text{“Shear wall adequate”}$$

$$Check := \text{if}\left(0.5 \cdot \phi \cdot V_c \geq \frac{V_{1\_R}}{0.7}, \text{“Minimum reinforcement required”}, \text{“Check reinforcement”}\right) = \text{“Minimum reinforcement required”}$$

CHECK REINFORCEMENT

$A_s := 0.21\ \text{in}^2$ — Steel reinforcement area, #4

$s := 10\ \text{in}$ — Minimum reinforcement spacing (ACI sec. 11.7.2.1, sec. 11.7.3.1)

$\rho_{tmin} := 0.002$ — Minimum transverse reinforcement ratio (ACI Table 11.6.1)

$$Check := \text{if}\left(\frac{A_s}{s \cdot h} \geq \rho_{tmin}, \text{“OK”}, \text{“Check reinforcement”}\right) = \text{“OK”}$$

$\rho_{lmin} := 0.002$ — Minimum longitudinal reinforcement ratio (ACI Table 11.6.1)

$$Check := \text{if}\left(\frac{A_s}{s \cdot h} \geq \rho_{lmin}, \text{“OK”}, \text{“Check reinforcement”}\right) = \text{“OK”}$$

CHECK SHEAR FRICTION

$A_{vf} := A_s \cdot \left(\frac{L_w}{s} - 1\right) = 5.33 \text{ in}^2$ Total area of reinforcement

$\mu := 0.6$ Coefficient of friction

$V_n := min\left(\mu \cdot f_y \cdot A_{vf}, min\left(0.2 \cdot f'_c \cdot L_w \cdot h, 800 \text{ psi} \cdot L_w \cdot h\right)\right) = 192024 \text{ lbf}$

$Check := \text{if}\left(\phi \cdot V_n \ge \frac{V_{1\_R}}{0.7}, \text{“Reinforcement adequate”}, \text{“Check reinforcement”}\right) = \text{“Reinforcement adequate”}$

SUMMARY: THEREFORE, USE 10 in CONCRETE SHEAR WALL W/ #5 @10" O.C. EACH WAY.

$T_{width\_floor} := 2 \text{ ft}$ Floor Tributary Width

$w_{DL} := DL_{floor} \cdot T_{width\_floor} = 24 \text{ plf}$ Floor dead load

$H_{wall} := 8.83 \text{ ft}$ Wall height

$T_s := \frac{\frac{V_{1\_R}}{0.7} \cdot H_{wall}}{d} - \frac{(0.9 - 0.2\, S_{DS}) \cdot (w_{DL} + DL_{wall} \cdot H_{wall}) \cdot \frac{L_w^{\,2}}{2}}{d} = 2903.32 \text{ lbf}$ Required steel reinforcement capacity (Strength level)

$V := F_{p\_wind\_1\_1\_floor} \cdot r_1 = 3698.25 \text{ lbf}$ Induced shear force in perforated shear wall

$T_w := \frac{V \cdot H_{wall}}{d} - \frac{0.9 \cdot (w_{DL} + DL_{wall} \cdot H_{wall}) \cdot \frac{(L_w)^2}{2}}{d} + F_{p\_wind\_1\_1} = 4514.79 \text{ lbf}$ Required steel reinforcement capacity (Strength level)

$T_1 := \max(T_s, T_w) = 4514.79 \text{ lbf}$ Required steel reinforcement capacity (Strength level)

$$A_{s\_req} := \frac{0.85 \cdot f'_c \cdot d \cdot h \cdot \left(1 - \left(\sqrt{1 - \frac{\max\left(\frac{V_{1\_R}}{0.7} \cdot H_{wall}, V \cdot H_{wall}\right)}{0.383 \cdot f'_c \cdot h \cdot d^2}}\right)\right)}{f_y} = 0.07 \text{ in}^2$$

Required steel reinforcement at end of shear wall

$A_g := 2 \cdot 0.79 \text{ in}^2$ Area of rebar, (2)#8

$F_y := 60 \text{ ksi}$ Tensile strength

$\phi_t := 0.9$ LRFD factor, AISC 360-16 Sec. D2

$P_n := F_y \cdot A_g = 94800 \text{ lbf}$ Nominal tensile yielding of rebar

$Check := \text{if}(\phi_t \cdot P_n \ge T_1, \text{“PASS”}, \text{“FAIL”}) = \text{“PASS”}$

$Check := \text{if}(A_g \ge A_{s\_req}, \text{“PASS”}, \text{“FAIL”}) = \text{“PASS”}$

SHEAR WALL - GRIDLINE A - INDIVIDUAL FULL-HEIGHT SHEAR WALL

$L_{tot} := 14.5\ \text{ft}$ — Wall length

$h := 8.83\ \text{ft}$ — Wall height

$A_{Ratio} := \dfrac{h}{L_{tot}} = 0.61$ — Worst case aspect ratio

$Check := \text{if}\left(A_{Ratio} \le 2, \text{“Blk'g Not Req'd”}, \text{if}\left(2 < A_{Ratio} \le 3.5, \text{“Blocking Req'd”}, \text{“Shape Ratio Exceeded”}\right)\right) = \text{“Blk'g Not Req'd”}$ — SDPWS Table 4.3.4

$V_{A\_R\_floor} := V_{max\_22} \cdot r_A + V_{A\_R} = 3467.01\ \text{lbf}$ — Induced shear force in shear wall

$v_{A\_floor} := \dfrac{V_{A\_R\_floor}}{L_{tot}} = 239.1\ \text{plf}$ — Maximum induced unit shear wall force

$v_s := 380\ \text{plf}$ — Nominal unit shear wall capacity (ESR-4696)

$Check := \text{if}\left(v_B \ge v_s, \text{“Shear wall not adequate”}, \text{“OK”}\right) = \text{“OK”}$

$T_{width\_floor} := 12\ \text{ft}$ — Floor Tributary Width

$w_{DL} := DL_{floor} \cdot T_{width\_floor} + DL_{wall} \cdot h = 188.15\ \text{plf}$ — Floor dead load

$V_{min} := F_{p\_wind\_min\_2\_2\_floor} \cdot r_A = 1505.01\ \text{lbf}$ — Minimum Induced shear force in shear wall (ASCE Fig. C27.1-1)

$$T_{min} := \frac{0.6 \cdot V_{min} \cdot h}{L_{tot}} - \frac{0.6 \cdot w_{DL} \cdot \dfrac{\left(L_{tot}\right)^2}{2}}{L_{tot}} = -268.55\ \text{lbf}$$ Required holdown capacity, service level

$V := F_{p\_wind\_2\_2\_floor} \cdot r_A = 1298.07\ \text{lbf}$ — Induced shear force in perforated shear wall

$$T_{A\_floor} := \frac{0.6 \cdot V \cdot h}{L_{tot}} - \frac{0.6 \cdot w_{DL} \cdot \dfrac{\left(L_{tot}\right)^2}{2}}{L_{tot}} + T_{A\_2} = 1814.55\ \text{lbf}$$ Required holdown capacity, service level

$T_{all} := 4350\ \text{lbf}$ — Allowable holdown force (HDU5)

$Check := \text{if}\left(T_{A\_floor} \ge T_{all}, \text{“Holdown not adequate”}, \text{“OK”}\right) = \text{“OK”}$

SHEAR WALL - GRIDLINE B - INDIVIDUAL FULL-HEIGHT SHEAR WALL

$L_{tot} := 18.5\ \text{ft}$ — Wall length

$h := 8.83\ \text{ft}$ — Wall height

$A_{Ratio} := \dfrac{h}{L_{tot}} = 0.48$ — Worst case aspect ratio

$Check := \text{if}\left(A_{Ratio} \le 2, \text{“Blk'g Not Req'd”}, \text{if}\left(2 < A_{Ratio} \le 3.5, \text{“Blocking Req'd”}, \text{“Shape Ratio Exceeded”}\right)\right) = \text{“Blk'g Not Req'd”}$ — SDPWS Table 4.3.4

$V_{B\_R\_floor} := V_{max\_22} \cdot r_B + V_{B\_R} = 3467.01\ lbf$ Induced shear force in shear wall

$v_{B\_floor} := \frac{V_{B\_R\_floor}}{L_{tot}} = 187.41\ plf$ Maximum induced unit shear wall force

$v_s := 380\ plf$ Nominal unit shear wall capacity (ESR-4696)

$Check := if\left(v_{B\_floor} \ge v_s, \text{“Shear wall not adequate”}, \text{“OK”}\right) = \text{“OK”}$

$T_{width\_floor} := 12\ ft$ Floor Tributary Width

$w_{DL} := DL_{floor} \cdot T_{width\_floor} + DL_{wall} \cdot h = 188.15\ plf$ Floor dead load

$V_{min} := F_{p\_wind\_min\_2\_2\_floor} \cdot r_B = 1505.01\ lbf$ Minimum Induced shear force in shear wall (ASCE Fig. C27.1-1)

$T_{min} := \frac{0.6 \cdot V_{min} \cdot h}{L_{tot}} - \frac{0.6 \cdot w_{DL} \cdot \frac{\left(L_{tot}\right)^2}{2}}{L_{tot}} = -613.23\ lbf$ Required holdown capacity, service level

$V := F_{p\_wind\_2\_2\_floor} \cdot r_B = 1298.07\ lbf$ Induced shear force in perforated shear wall

$T_{B\_floor} := \frac{0.6 \cdot V \cdot h}{L_{tot}} - \frac{0.6 \cdot w_{DL} \cdot \frac{\left(L_{tot}\right)^2}{2}}{L_{tot}} + T_B = 1076.31\ lbf$ Required holdown capacity, service level

$T_{all} := 4350\ lbf$ Allowable holdown force (HDU5)

$Check := if\left(T_{B\_floor} \ge T_{all}, \text{“Holdown not adequate”}, \text{“OK”}\right) = \text{“OK”}$

CHECK SILL ATTACHMENT

$C_{D\_WL} := 1.60$ Load duration factor for seismic/wind (NDS Table 2.3.2)

$S := 6\ in$ Spacing of 16d nails

$N := 2$ Number of 16d nails

$Z := 120\ lbf$ Reference lateral design value (NDS Table 12N, 16d nail, SPF, ts=1-1/2 in)

$D_{nail} := 0.162\ in$ Diameter of 16d nail (NDS App. L, Table L4)

$L_{nail} := 3.5\ in$ Length of 16d nail (NDS App. L, Table L4)

$p := L_{nail} - 1.5\ in - 0.75\ in = 1.25\ in$ Penetration of 16d nails

$p' := if\left(p \ge 10 \cdot D_{nail}, 1.0, if\left(6 \cdot D_{nail} \le p < 10 \cdot D_{nail}, \frac{p}{10 \cdot D_{nail}}, \text{“Select Longer Nail”}\right)\right) = 0.77$

$Z' := \frac{C_{D\_WL} \cdot Z \cdot N \cdot p'}{S} = 592.59\ plf$ Allowable nail shear capacity per linear foot

$V_{max} := \max\left(v_{1\_1}, v_{1\_2}, v_{A\_floor}, v_{B\_floor}\right) = 365.89 \text{ plf}$

$Check := \text{if}\left(Z' \geq V_{max}, \text{"PASS"}, \text{"FAIL"}\right) = \text{"PASS"}$

CONNECTION TO DOUBLE PLATE CHECK

$N := 1$	Number of A35 clips
$S := 12 \text{ in}$	Spacing of A35 clips
$F_{all} := \dfrac{N \cdot 560 \text{ lbf}}{S} = 560 \text{ plf}$	Allowable A35 clip Load (Simpson Catalog)

$Check := \text{if}\left(F_{all} \geq V_{max}, \text{"PASS"}, \text{"FAIL"}\right) = \text{"PASS"}$

CHECK SILL ATTACHMENT

$Z := 590 \text{ lbf}$	Reference Lateral Design Value (NDS Table 12E, 1/2", SPF, ts=1-1/2 in)
$Z' := C_{D\_WL} \cdot Z = 944 \text{ lbf}$	Allowable Anchor Bolt Capacity

MAXIMUM SHEAR	REQUIRED AND ACTUAL BOLT SPACING	
$v_{1\_1} = 365.89 \text{ plf}$	$S_{1\_1} := \dfrac{Z'}{v_{1\_1}} = 30.96 \text{ in}$	30" O.C.
$v_{1\_2} = 365.89 \text{ plf}$	$S_{1\_2} := \dfrac{Z'}{v_{1\_2}} = 30.96 \text{ in}$	30" O.C.
$v_{A\_floor} = 239.1 \text{ plf}$	$S_A := \dfrac{Z'}{v_{A\_floor}} = 47.38 \text{ in}$	30" O.C.
$v_{B\_floor} = 187.41 \text{ plf}$	$S_B := \dfrac{Z'}{v_{B\_floor}} = 60.45 \text{ in}$	30" O.C.

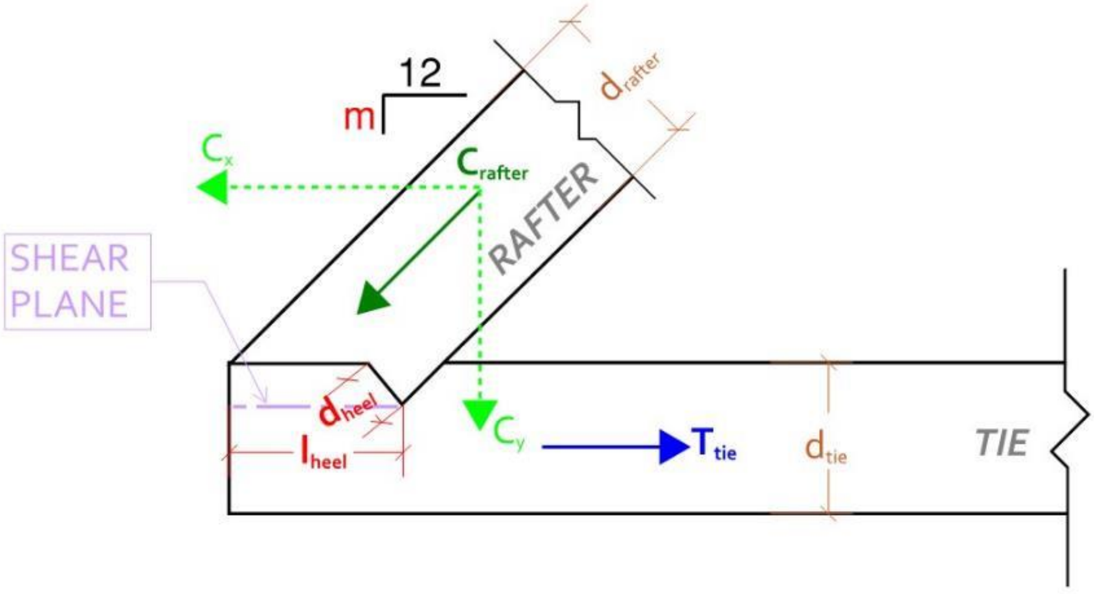
WOOD PROPERTIES - NDS SUPPLEMENT (5" x 5" AND LARGER - TABLE 4D)

Species & Grade: DF #1

$E := 1600\ ksi$	Modulus of Elasticity		
$SG := 0.5$	Specific Gravity		
$Green\_Timber := 0$	In-Service Moisture Content of Timbers (1 = Yes, 0 = No)		
$F_{b\_BS} := 1350\ psi$	Bending Capacity of Beam and Stringer	$F_{b\_PT} := 1200\ psi$	Bending Capacity of Post and Timber
$F_{t\_BS} := 675\ psi$	Tension Capacity of Beam and Stringer	$F_{t\_PT} := 825\ psi$	Tension Capacity of Post and Timber
$F_{v\_BS} := 170\ psi$	Shear Capacity of Beam and Stringer	$F_{v\_PT} := 170\ psi$	Shear Capacity of Post and Timber
$F_{cpll\_BS} := 925\ psi$	Compression Capacity Parallel to Grain of Beam and Stringer	$F_{cpll\_PT} := 1000\ psi$	Compression Capacity Parallel to Grain of Post and Timber
$F_{cperp\_BS} := 625\ psi$	Compression Capacity Perpendicular to Grain of Beam and Stringer	$F_{cperp\_PT} := 625\ psi$	Compression Capacity Perpendicular to Grain of Post and Timber

JOINERY ANALYSIS

RAFTER TO TIE BEAM



TIMBER PARAMETERS

Rafter:

$b_{rafter} := 7.5\ in$	Width of Rafter
$d_{rafter} := 11.5\ in$	Depth of Rafter
$size_r := d_{rafter} - b_{rafter} \le 2\ in = 0$	If 1= Post & Timber, 0= Beam &Stringer
$m := 5$	Rise of Rafter Slope

Tie:

$b_{tie} := 7.5\ in$	Width of tie
$d_{tie} := 13.5\ in$	Depth of tie
$size_{tie} := d_{tie} - b_{tie} \le 2\ in = 0$	If 1= Post & Timber, 0= Beam &Stringer

$d_{heel} := 3\ in$ Depth of Heel

$l_{heel} := 15\ in$ Length to Heel

LOADS

$C_{rafter} := 20.3\ kip$ Compression fom Rafter (RISA)

$\vartheta := \operatorname{atan}\left(\frac{m}{12}\right) = 22.62\ °$ Angle of Rafter

$C_y := C_{rafter} \cdot \sin(\vartheta) = 7807.7\ lbf$ Veritical Compression Force Component

$C_x := C_{rafter} \cdot \cos(\vartheta) = 18738.5\ lbf$ Horizontal Compression Force Component

$T_{tie} := 17.7\ kip$ Tension in Tie (RISA)

ANALYSIS

Adjustments:

$C_D := 1.15$ Load Adjustment Factor for Governing Load Combination (NDS Table 2.3.2)

$C_t := 1.0$ Temperature Factor (NDS Table 2.3.3)

$C_{F\_c} := 1.0$ Size Factor for Compressive Allowable (NDS Supplement - Table 4D Adjustment Factors)

$C_{M\_cperp} := \text{if}\left(Green\_Timber \le 0, 1, 0.67\right) = 1$ Wet Service Factor to Apply to Compression Perpendicular to Grain (NDS Supplement - Table 4D Adjustments)

$C_{M\_cpll} := \text{if}\left(Green\_Timber \le 0, 1, 0.91\right) = 1$ Wet Service Factor to Apply to Compression Parallel to Grain (NDS Supplement - Table 4D Adjustments)

Properties:

$F_{cperp\_r} := \text{if}\left(size_r \le 0, F_{cperp\_BS}, F_{cperp\_PT}\right) = 625\ psi$ Allowable Compression Perpendicular to Grain for Rafter Timber

$F_{cpll\_r} := \text{if}\left(size_r \le 0, F_{cpll\_BS}, F_{cpll\_PT}\right) = 925\ psi$ Allowable Compression Parallel to Grain for Rafter Timber

$F_{v\_t} := \text{if}\left(size_{tie} \le 0, F_{v\_BS}, F_{v\_PT}\right) = 170\ psi$ Allowable Shear for Tie Timber

Adjusted Properties:

$F'_{cperp\_r} := F_{cperp\_r} \cdot C_{M\_cperp} \cdot C_t = 625\ psi$ Adjusted Allowable Compression Perpendicular to Grain for Rafter Timber

$F'_{cpll\_r} := F_{cpll\_r} \cdot C_D \cdot C_{M\_cperp} \cdot C_t \cdot C_{F\_c} = \left(1.06 \cdot 10^3\right)\ psi$ Adjusted Allowable Compression Perpendicular to Grain for Rafter Timber

$F_{\vartheta\_r} := \frac{F'_{cpll\_r} \cdot F'_{cperp\_r}}{F'_{cpll\_r} \cdot \sin(\vartheta)^2 + F'_{cperp\_r} \cdot \cos(\vartheta)^2} = 963.68\ psi$ Allowable Compression at Angle to Grain for Rafter Timber

$F'_{v\_t} := F_{v\_t} \cdot C_D \cdot C_t = 195.5\ psi$ Adjusted Allowable Shear for TieTimber

Vertical Component Resolution/Check:

$R_{eg} := 1.5$ End Grain Shear Reduction Factor

$A_{comp\_V} := \left(l_{heel} \cdot b_{rafter}\right) = 112.5\ in^2$ Area of Plane resisting Vertical Compressive Component

$C_{y\_allow} := \frac{A_{comp\_V} \cdot F_{\vartheta\_r}}{R_{eg}} = 72275.7\ lbf$ Allowable Vertical Compression

Check: $C_{y\_allow} \ge C_y = 1$ **1 = OK, 0 = N.G.**

Horizontal Component Resolution/Check:

$A_{comp\_H} := d_{heel} \cdot b_{rafter} = 22.5\ in^2$ — Area of Plane resisting Horizontal Compressive Component

$A_{v\_H} := b_{tie} \cdot l_{heel} = 112.5\ in^2$ — Area of Shear Plane resisting Horizontal Tension Component

$C_{x\_allow} := A_{comp\_H} \cdot F_{\theta\_r} = 21682.7\ lbf$ — Allowable Horizontal Compression

$T_{all} := A_{v\_H} \cdot F'_{v\_t} = 21994\ lbf$ — Allowable Horizontal Tension due to Shear Block

Check: $C_{x\_allow} \geq C_x = 1$ — 1 = OK, 0 = N.G.

$T_{all} \geq T_{tie} = 1$ — 1 = OK, 0 = N.G.

SUMMARY: THE RAFTER AND TIE GEOMETRY, GRADE, AND SPECIES ARE ADEQUATE TO RESIST ALL DESIGN LOADS.

Kingpost to rafter connection

Post Tension

$W := 7\ in$ — Width of kingpost btw tapered housing

$D := 8\ in$ — Thickness of kingpost

$T_{act} := 7000\ lbf$ — Actual kingpost tension

$A_{tp} := W \cdot D = 56\ in^2$ — Tension plan area

$T_{allow} := F_{t\_PT} \cdot A_{tp} = \left(4.62 \cdot 10^4\right)\ lbf$ — Allowable tension

$T_{act} \leq T_{allow} = 1$

Kingpost failure due to rafter compression

$F_{cperp\_BS} = 625\ psi$

$F_{c\_rafter} := 20300\ lbf$ — Rafter compression toward KP

$L := 11\ in$ — Length of kingpost housing (subtract any mortise w/in kingpost)

$W := 5\ in$ — Width of timber (subtract any mortise w/in kingpost)

$A_B := L \cdot W = 55\ in^2$ — Bearing surface area

$Bearing\_strength := A_B \cdot F_{cperp\_BS} = \left(3.44 \cdot 10^4\right)\ psi \cdot in^2$ — Bearing capacity

$F_{c\_rafter} \leq Bearing\_strength = 1$

Wood Peg Capacity Calculation

Per TFEC 2009-01 (incorporated into TFEC 1-2010 Standard for Design of Timber Frame Structures)

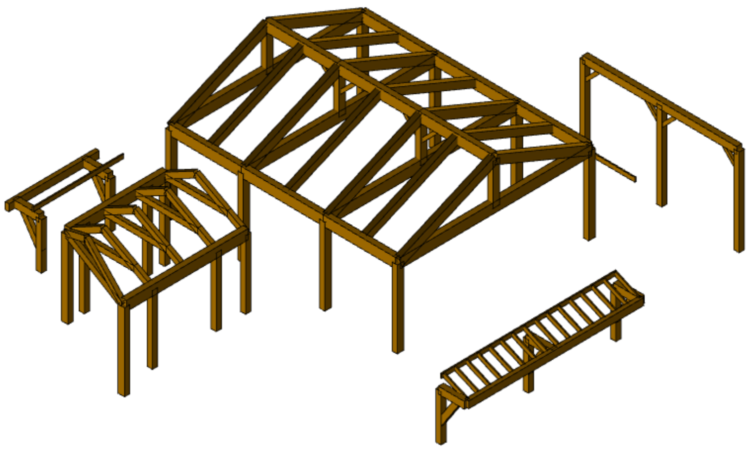
Input			
Peg diameter	D	1	in.
Specific Gravity - Peg	Gp	0.73	
Specific Gravity - Timber	Gt	0.5	
Tenon / mortise width	Lm	2	in.
Mortise side wall thickness	Ls	3	in.
Max. angle of load to grain	Theta	90	deg.
Calculations			
	Fe par	3148.5	psi
	Fe perp	2575.8	psi
	Fe theta	2575.8	psi
Tenon dowel bearing	Fem	3148.5	psi
Mortise dowel bearing	Fes	2575.8	psi
Dowel shear strength	Fyv	2105.2	psi
Bending yield strength - peg	Fyb	17413.3	psi
Constants	Re	1.222	
	k3	1.226	
	Ktheta	1.250	
	Rd(Im,Is)	5	
	Rd(IIIs)	4	
	Rd(V)	3.5	
Yield Mode Values	Z (Im)	1259	lbs.
	Z (Is)	3091	lbs.
	Z (IIIs)	1797	lbs.
	Z (V)	945	lbs.
Nominal Design Capacity	Z	945	lbs.
Load Duration Factor	Cd	1.15	
Allowable Design Capacity	Z'	**2173**	lbs.

6 SOFTWARE PRINTOUTS (ENERCALC)

7 SOFTWARE PRINTOUTS (RISA)

7.1 MEMBER SPECIES, SIZE, AND GRADE SUMMARY

- All Members were analyzed using DFL #1, U.N.O.



3-D Model

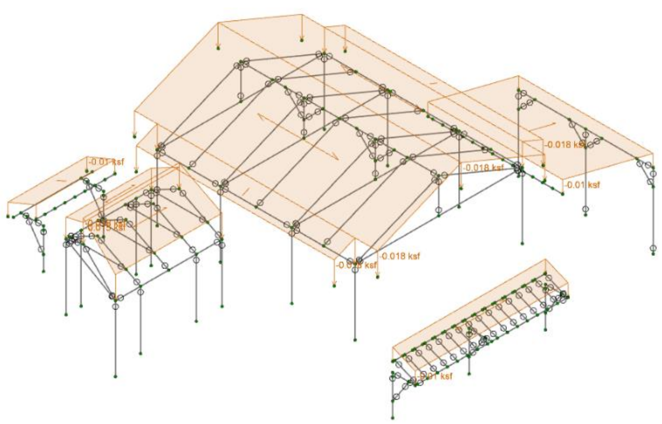
Member	*Size (FS)*	*Worst Case Axial (kips)*	*Worst Case Shear (kips)*
8x14 Plate	7.25X13.25FS	0	3.7
6x8 Tie	5.25X7.25FS	-3.1	0
6x10 Tie	5.25X9.25FS	-1.4	0.6
8x14 Tie	7.25X13.25FS	-17.7	0.3
6x8 Rafter	5.25X7.25FS	3.7	0.8
8x10 Rafter	7.25X9.25FS	1.5	2.9
8x12 Top Chord	7.25X11.25FS	20.3	3.1
8x10 Post	7.25X9.25FS	18	0
8x8 Post	7.25X7.25FS	8.2	0
8x14 Ridge	7.25X13.25FS	-3.6	2.1
8x10 KP	7.25X9.25FS	-7	0.1
8x10 plate	7.25X9.25FS	2.4	1.5
6x8 Bracket diagonal	5.25X7.25FS	2.7	0
2x Ledger	2X6	0	0.4
4x6 Brace	3.25X4.25FS	5	0

7.2 SERVICE LOADS

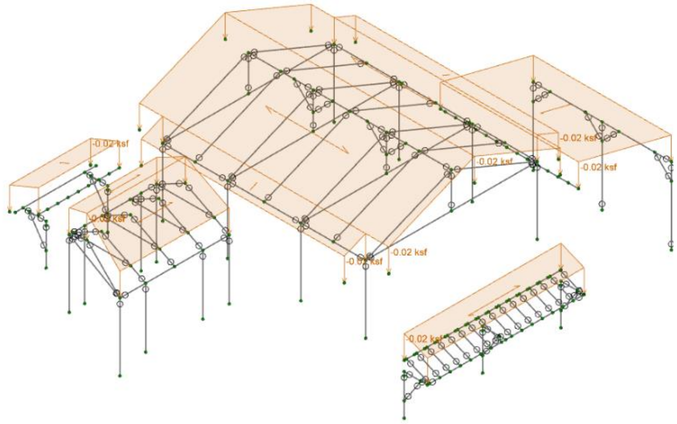
Location	DL (psf)	RLL (psf)	SL (psf)	WL (psf)
Roof	18	20	104	See Wind Analysis

7.3 LOAD COMBINATIONS

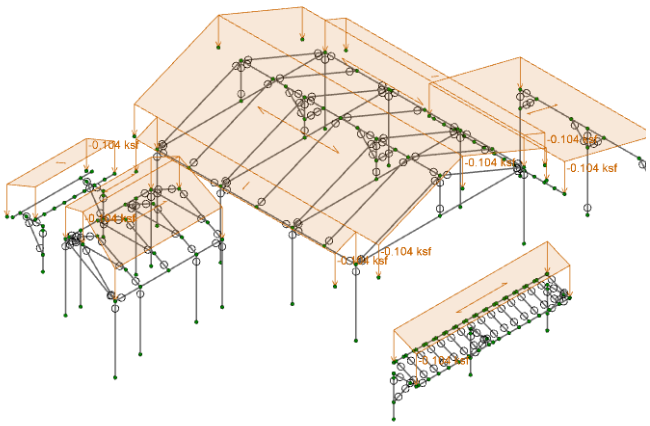
	Description	Solve	P-Delta	SRSS	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	ASCE ASD 1	✓			DL	1										
2	ASCE ASD 2	✓			DL	1	LL	1	LLS	1						
3	ASCE ASD...	✓			DL	1	RLL	1								
4	ASCE ASD...	✓			DL	1	SL	1	SLN	1						
5	ASCE ASD...	✓			DL	1	LL	0.75	LLS	0.75	RLL	0.75				
6	ASCE ASD...	✓			DL	1	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75		
7	ASCE ASD...	✓			DL	1	WL	0.6								
8	ASCE ASD...	✓			DL	1	WL	0.45	LL	0.75	LLS	0.75	RLL	0.75		
9	ASCE ASD...	✓			DL	1	WL	0.45	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75
10	ASCE ASD...	✓			DL	1	WL	0.45	LL	0.75	LLS	0.75				
11	ASCE ASD 7	✓			DL	0.6	WL	0.6								



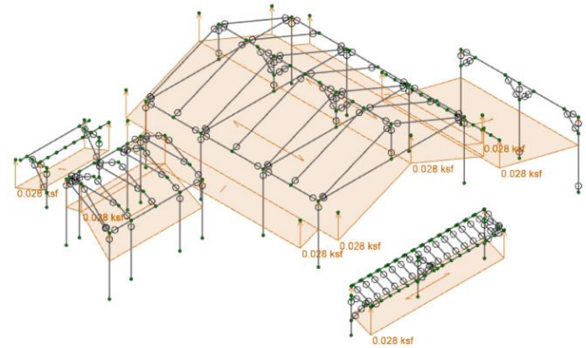
Applied Dead Load



Applied Roof Live Load



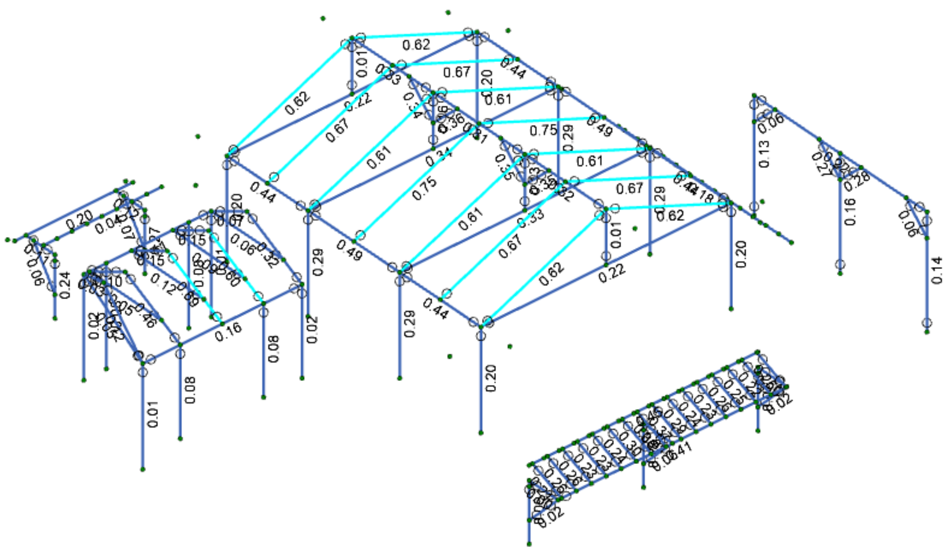
Applied Snow Load



Applied Wind Load

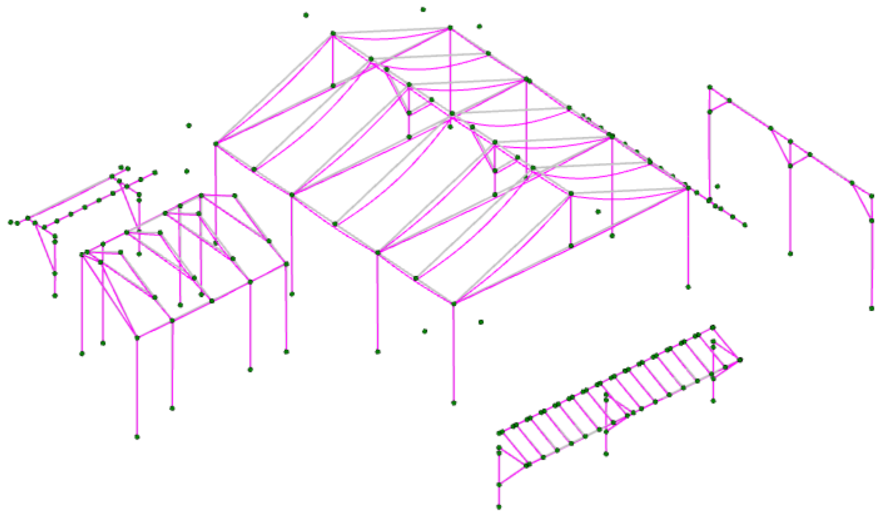
7.4 MOMENT BENDING CAPACITY CHECK

- Values must be ≤ 1.0 to pass

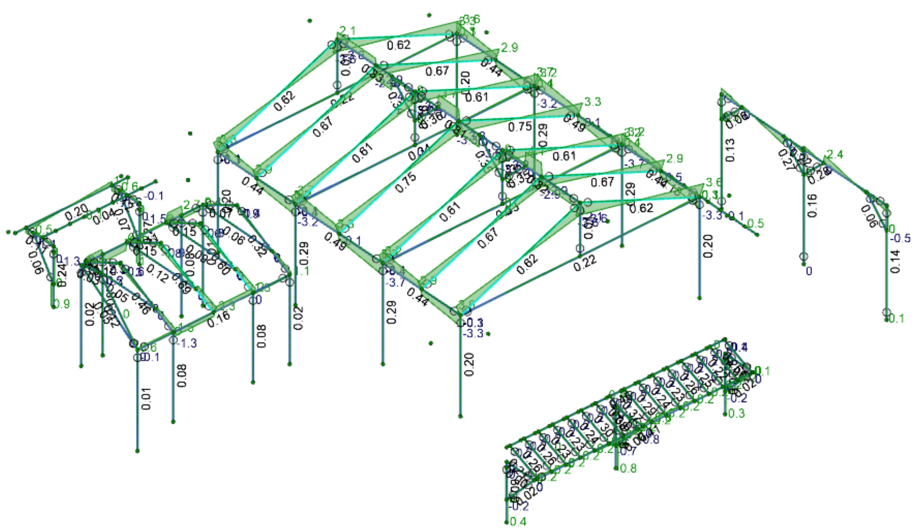


7.5 MEMBER DEFLECTION

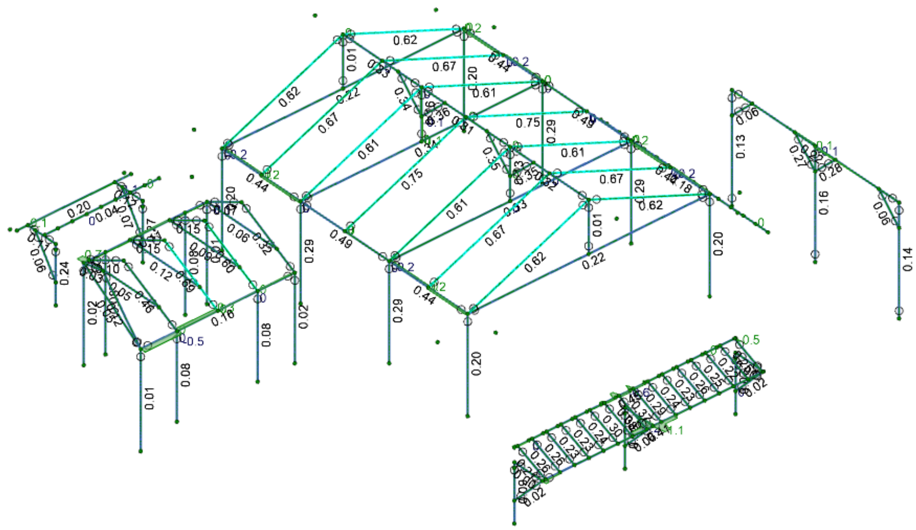
- Deflection magnified by 32 for clarity
- Maximum deflection due to Load Combination #4



7.6 MAXIMUM MEMBER SHEAR FORCE – "Y" DIRECTION

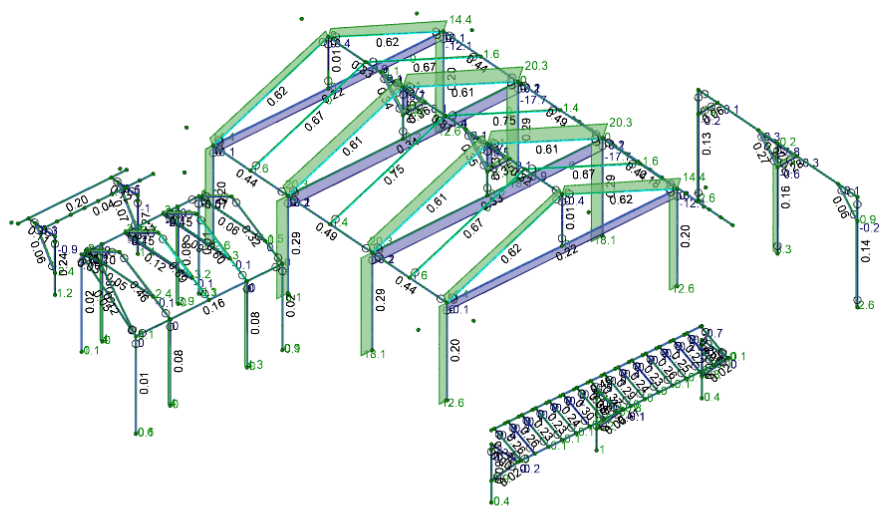


7.7 MAXIMUM MEMBER SHEAR FORCE – "Z" DIRECTION

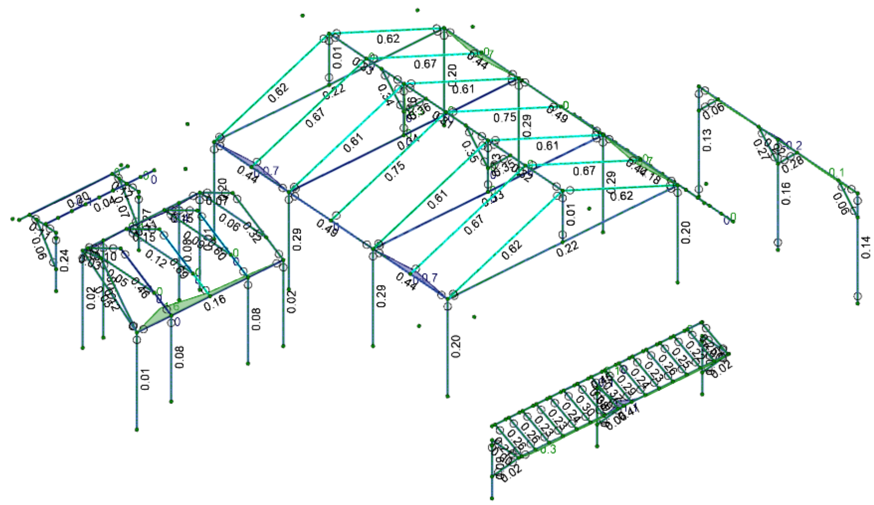


7.8 MAXIMUM MEMBER AXIAL FORCE

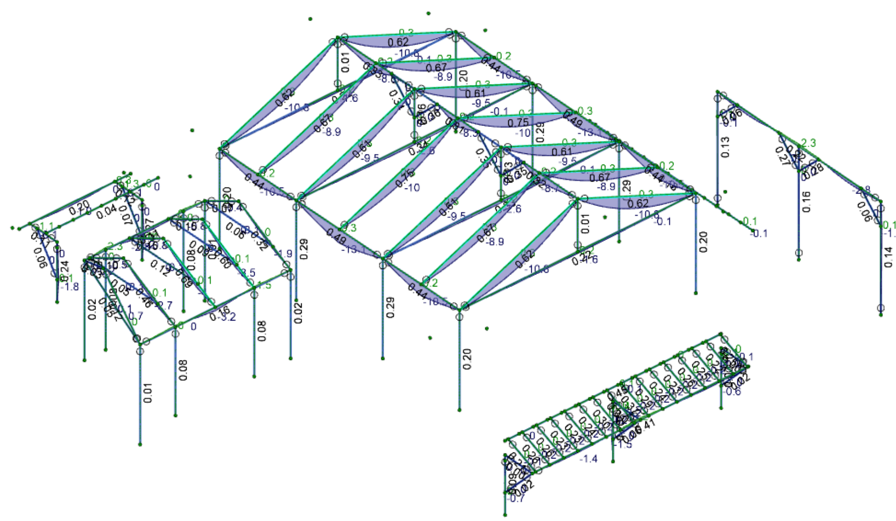
- (Compression = GREEN, Tension = BLUE)



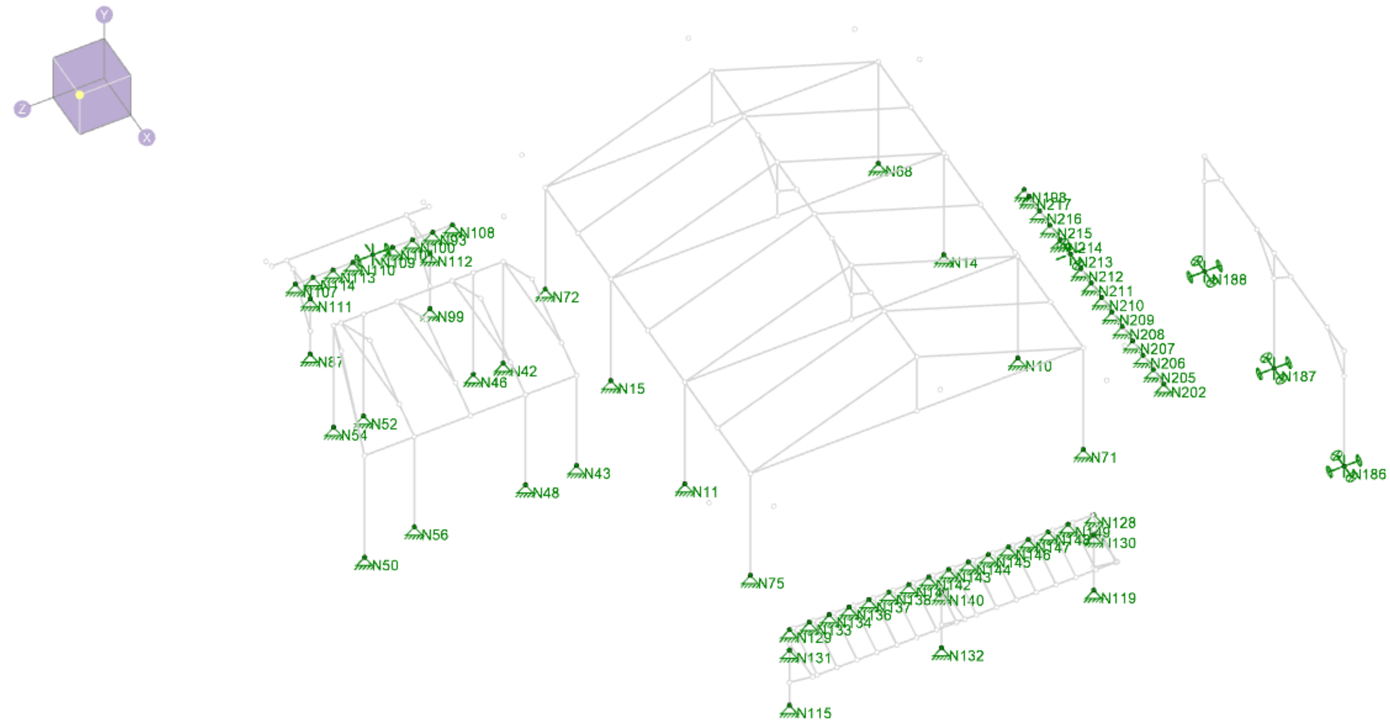
7.9 MAXIMUM MEMBER "Myy" MOMENT FORCES



7.10 MAXIMUM MEMBER "Mzz" MOMENT FORCES



7.11 POST BASE/POCKET MEMBER FORCE SUMMARY



RISA A NEMETSCHEK COMPANY

Company : TGE
Designer : GRV
Job Number : 24-24439
Model Name : The Bruns Addition

9/5/2024
10:20:29 AM
Checked By : ___________

Load Combinations

	Description	Solve	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	ASCE ASD 1		DL	1										
2	ASCE ASD 2		DL	1	LL	1	LLS	1						
3	ASCE ASD 3 (a)		DL	1	RLL	1								
4	ASCE ASD 3 (b)		DL	1	SL	1	SLN	1						
5	ASCE ASD 4 (a)		DL	1	LL	0.75	LLS	0.75	RLL	0.75				
6	ASCE ASD 4 (b)		DL	1	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75		
7	ASCE ASD 5 (a)		DL	1	WL	0.6								
8	ASCE ASD 6 (a)		DL	1	WL	0.45	LL	0.75	LLS	0.75	RLL	0.75		
9	ASCE ASD 6 (b)		DL	1	WL	0.45	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75
10	ASCE ASD 6 (c)		DL	1	WL	0.45	LL	0.75	LLS	0.75				
11	ASCE ASD 7		DL	0.6	WL	0.6								
12	DL	Yes	DL	1										
13	RLL	Yes	RLL	1										
14	SL	Yes	SL	1										
15	WL	Yes	WL	1										

Node Reactions

	LC	Node Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
1	12	N68	0	2.63	0	0	0	0
2	12	N71	0	2.627	0	0	0	0
3	12	N72	0	2.63	0	0	0	0
4	12	N75	0	2.627	0	0	0	0
5	12	N10	0	3.714	0	0	0	0
6	12	N11	0	3.714	0	0	0	0
7	12	N14	0	3.718	0	0	0	0
8	12	N15	0	3.718	0	0	0	0
9	12	N42	0	0.173	0	0	0	0
10	12	N43	0	0.277	0	0	0	0
11	12	N46	0	0.86	0	0	0	0
12	12	N48	0	0.944	0	0	0	0
13	12	N52	0	0.883	0	0	0	0
14	12	N54	0	0.37	0	0	0	0
15	12	N56	0	0.907	0	0	0	0
16	12	N50	0	0.304	0	0	0	0
17	12	N87	-0.136	0.213	0	0	0	0
18	12	N99	-0.151	0.232	0	0	0	0
19	12	N111	0.136	0.116	0	0	0	0
20	12	N112	0.151	0.121	0	0	0	0
21	12	N108	0	0.009	0	0	0	0
22	12	N107	0	0.008	0	0	0	0
23	12	N93	0	0.026	0	0	0	0
24	12	N100	0	0.025	0	0	0	0
25	12	N101	0	0.026	0	0	0	0
26	12	N109	0	0.026	0	0	0	0
27	12	N110	0	0.026	0	0	0	0
28	12	N113	0	0.025	0	0	0	0
29	12	N114	0	0.024	0	0	0	0
30	12	N115	0.074	0.119	0	0	0	0
31	12	N119	0.058	0.109	0	0	0	0
32	12	N128	-0.105	0.061	0	0	0	0
33	12	N129	-0.039	0.028	0	0	0	0
34	12	N130	0.065	0.116	0	0	0	0
35	12	N131	0.006	0.124	0	0	0	0
36	12	N132	0.132	0.202	0	0	0	0

Node Reactions (Continued)

	LC	Node Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
37	12	N140	0.245	0.182	0	0	0	0
38	12	N133	-0.048	0.048	0	0	0	0
39	12	N134	-0.002	0.03	0	0	0	0
40	12	N136	0.007	0.026	0	0	0	0
41	12	N137	0.006	0.025	0	0	0	0
42	12	N138	-0.006	0.03	0	0	0	0
43	12	N141	-0.047	0.047	0	0	0	0
44	12	N142	-0.119	0.076	0	0	0	0
45	12	N143	-0.131	0.081	0	0	0	0
46	12	N144	-0.067	0.056	0	0	0	0
47	12	N145	-0.013	0.033	0	0	0	0
48	12	N146	0.004	0.026	0	0	0	0
49	12	N147	0.003	0.028	0	0	0	0
50	12	N148	-0.007	0.029	0	0	0	0
51	12	N149	-0.015	0.04	0	0	0	0
52	12	N186	-0.015	0.401	0	0	0	0
53	12	N187	0	1.028	0	0	0	0
54	12	N188	0.015	0.397	0	0	0	0
55	12	N202	0	0.034	0	0	0	0
56	12	N198	0	0.01	0	0	0	0
57	12	N205	0	0.095	0	0	0	0
58	12	N206	0	0.083	0	0	0	0
59	12	N207	0	0.086	0	0	0	0
60	12	N208	0	0.085	0	0	0	0
61	12	N209	0	0.085	0	0	0	0
62	12	N210	0	0.085	0	0	0	0
63	12	N211	0	0.085	0	0	0	0
64	12	N212	0	0.085	0	0	0	0
65	12	N213	0	0.085	0	0	0	0
66	12	N214	0	0.085	0	0	0	0
67	12	N215	0	0.085	0	0	0	0
68	12	N216	0	0.088	0	0	0	0
69	12	N217	0	0.071	0	0	0	0
70	12	COG (ft):	X: 7.725	Y: 8.619	Z: 26.193			
71	13	N68	0	1.917	0	0	0	0
72	13	N71	0	1.915	0	0	0	0
73	13	N72	0	1.917	0	0	0	0
74	13	N75	0	1.915	0	0	0	0
75	13	N10	0	2.769	0	0	0	0
76	13	N11	0	2.769	0	0	0	0
77	13	N14	0	2.772	0	0	0	0
78	13	N15	0	2.772	0	0	0	0
79	13	N42	0	0.022	0	0	0	0
80	13	N43	0	0.112	0	0	0	0
81	13	N46	0	0.583	0	0	0	0
82	13	N48	0	0.646	0	0	0	0
83	13	N52	0	0.595	0	0	0	0
84	13	N54	0	0.124	0	0	0	0
85	13	N56	0	0.599	0	0	0	0
86	13	N50	0	0.06	0	0	0	0
87	13	N87	-0.15	0.19	0	0	0	0
88	13	N99	-0.17	0.216	0	0	0	0
89	13	N111	0.15	0.047	0	0	0	0
90	13	N112	0.17	0.054	0	0	0	0
91	13	N108	0	0.015	0	0	0	0

Node Reactions (Continued)

	LC	Node Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
92	13	N107	0	0.013	0	0	0	0
93	13	N93	0	0.046	0	0	0	0
94	13	N100	0	0.044	0	0	0	0
95	13	N101	0	0.046	0	0	0	0
96	13	N109	0	0.046	0	0	0	0
97	13	N110	0	0.046	0	0	0	0
98	13	N113	0	0.044	0	0	0	0
99	13	N114	0	0.043	0	0	0	0
100	13	N115	0.056	0.058	0	0	0	0
101	13	N119	0.044	0.05	0	0	0	0
102	13	N128	-0.078	0.059	0	0	0	0
103	13	N129	-0.03	0.031	0	0	0	0
104	13	N130	0.051	0.035	0	0	0	0
105	13	N131	0.007	0.041	0	0	0	0
106	13	N132	0.126	0.152	0	0	0	0
107	13	N140	0.235	0.107	0	0	0	0
108	13	N133	-0.037	0.058	0	0	0	0
109	13	N134	-0.003	0.047	0	0	0	0
110	13	N136	0.004	0.043	0	0	0	0
111	13	N137	0.004	0.042	0	0	0	0
112	13	N138	-0.006	0.046	0	0	0	0
113	13	N141	-0.045	0.062	0	0	0	0
114	13	N142	-0.112	0.089	0	0	0	0
115	13	N143	-0.124	0.094	0	0	0	0
116	13	N144	-0.064	0.07	0	0	0	0
117	13	N145	-0.013	0.049	0	0	0	0
118	13	N146	0.002	0.042	0	0	0	0
119	13	N147	0.001	0.046	0	0	0	0
120	13	N148	-0.007	0.045	0	0	0	0
121	13	N149	-0.012	0.056	0	0	0	0
122	13	N186	-0.024	0.421	0	0	0	0
123	13	N187	0.001	1.391	0	0	0	0
124	13	N188	0.023	0.414	0	0	0	0
125	13	N202	0	0.066	0	0	0	0
126	13	N198	0	0.019	0	0	0	0
127	13	N205	0	0.185	0	0	0	0
128	13	N206	0	0.161	0	0	0	0
129	13	N207	0	0.166	0	0	0	0
130	13	N208	0	0.165	0	0	0	0
131	13	N209	0	0.165	0	0	0	0
132	13	N210	0	0.165	0	0	0	0
133	13	N211	0	0.165	0	0	0	0
134	13	N212	0	0.165	0	0	0	0
135	13	N213	0	0.165	0	0	0	0
136	13	N214	0	0.165	0	0	0	0
137	13	N215	0	0.164	0	0	0	0
138	13	N216	0	0.171	0	0	0	0
139	13	N217	0	0.137	0	0	0	0
140	13	COG (ft):	X: 8.187	Y: 8.525	Z: 23.93			
141	14	N68	0	9.971	0	0	0	0
142	14	N71	0	9.96	0	0	0	0
143	14	N72	0	9.971	0	0	0	0
144	14	N75	0	9.96	0	0	0	0
145	14	N10	0	14.4	0	0	0	0
146	14	N11	0	14.4	0	0	0	0

Node Reactions (Continued)

	LC	Node Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
147	14	N14	0	14.417	0	0	0	0
148	14	N15	0	14.417	0	0	0	0
149	14	N42	0	0.117	0	0	0	0
150	14	N43	0	0.583	0	0	0	0
151	14	N46	0	3.03	0	0	0	0
152	14	N48	0	3.361	0	0	0	0
153	14	N52	0	3.094	0	0	0	0
154	14	N54	0	0.646	0	0	0	0
155	14	N56	0	3.114	0	0	0	0
156	14	N50	0	0.313	0	0	0	0
157	14	N87	-0.781	0.99	0	0	0	0
158	14	N99	-0.885	1.122	0	0	0	0
159	14	N111	0.781	0.246	0	0	0	0
160	14	N112	0.885	0.279	0	0	0	0
161	14	N108	0	0.079	0	0	0	0
162	14	N107	0	0.068	0	0	0	0
163	14	N93	0	0.239	0	0	0	0
164	14	N100	0	0.228	0	0	0	0
165	14	N101	0	0.24	0	0	0	0
166	14	N109	0	0.24	0	0	0	0
167	14	N110	0	0.239	0	0	0	0
168	14	N113	0	0.231	0	0	0	0
169	14	N114	0	0.222	0	0	0	0
170	14	N115	0.291	0.303	0	0	0	0
171	14	N119	0.232	0.264	0	0	0	0
172	14	N128	-0.412	0.302	0	0	0	0
173	14	N129	-0.154	0.159	0	0	0	0
174	14	N130	0.268	0.186	0	0	0	0
175	14	N131	0.038	0.213	0	0	0	0
176	14	N132	0.654	0.792	0	0	0	0
177	14	N140	1.222	0.557	0	0	0	0
178	14	N133	-0.191	0.304	0	0	0	0
179	14	N134	-0.015	0.244	0	0	0	0
180	14	N136	0.02	0.222	0	0	0	0
181	14	N137	0.02	0.218	0	0	0	0
182	14	N138	-0.031	0.239	0	0	0	0
183	14	N141	-0.233	0.322	0	0	0	0
184	14	N142	-0.583	0.464	0	0	0	0
185	14	N143	-0.643	0.489	0	0	0	0
186	14	N144	-0.331	0.363	0	0	0	0
187	14	N145	-0.069	0.255	0	0	0	0
188	14	N146	0.011	0.22	0	0	0	0
189	14	N147	0.007	0.241	0	0	0	0
190	14	N148	-0.037	0.231	0	0	0	0
191	14	N149	-0.063	0.286	0	0	0	0
192	14	N186	-0.124	2.187	0	0	0	0
193	14	N187	0.003	7.231	0	0	0	0
194	14	N188	0.121	2.155	0	0	0	0
195	14	N202	0	0.344	0	0	0	0
196	14	N198	0	0.098	0	0	0	0
197	14	N205	0	0.961	0	0	0	0
198	14	N206	0	0.839	0	0	0	0
199	14	N207	0	0.863	0	0	0	0
200	14	N208	0	0.858	0	0	0	0
201	14	N209	0	0.859	0	0	0	0

Node Reactions (Continued)

	LC	Node Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
202	14	N210	0	0.859	0	0	0	0
203	14	N211	0	0.859	0	0	0	0
204	14	N212	0	0.859	0	0	0	0
205	14	N213	0	0.859	0	0	0	0
206	14	N214	0	0.86	0	0	0	0
207	14	N215	0	0.853	0	0	0	0
208	14	N216	0	0.89	0	0	0	0
209	14	N217	0	0.713	0	0	0	0
210	14	COG (ft):	X: 8.187	Y: 8.525	Z: 23.93			
211	15	N68	0	-2.684	0	0	0	0
212	15	N71	0	-2.682	0	0	0	0
213	15	N72	0	-2.684	0	0	0	0
214	15	N75	0	-2.682	0	0	0	0
215	15	N10	0	-3.877	0	0	0	0
216	15	N11	0	-3.877	0	0	0	0
217	15	N14	0	-3.881	0	0	0	0
218	15	N15	0	-3.881	0	0	0	0
219	15	N42	0	-0.031	0	0	0	0
220	15	N43	0	-0.157	0	0	0	0
221	15	N46	0	-0.816	0	0	0	0
222	15	N48	0	-0.905	0	0	0	0
223	15	N52	0	-0.833	0	0	0	0
224	15	N54	0	-0.174	0	0	0	0
225	15	N56	0	-0.838	0	0	0	0
226	15	N50	0	-0.084	0	0	0	0
227	15	N87	0.181	-0.246	0	0	0	0
228	15	N99	0.205	-0.279	0	0	0	0
229	15	N111	-0.053	-0.061	0	0	0	0
230	15	N112	-0.06	-0.069	0	0	0	0
231	15	N108	0.008	-0.02	0	0	0	0
232	15	N107	0.007	-0.017	0	0	0	0
233	15	N93	0.025	-0.059	0	0	0	0
234	15	N100	0.023	-0.057	0	0	0	0
235	15	N101	0.025	-0.06	0	0	0	0
236	15	N109	0.025	-0.06	0	0	0	0
237	15	N110	0.025	-0.059	0	0	0	0
238	15	N113	0.024	-0.057	0	0	0	0
239	15	N114	0.023	-0.055	0	0	0	0
240	15	N115	-0.08	-0.085	0	0	0	0
241	15	N119	-0.062	-0.074	0	0	0	0
242	15	N128	0.098	-0.079	0	0	0	0
243	15	N129	0.033	-0.041	0	0	0	0
244	15	N130	-0.097	-0.052	0	0	0	0
245	15	N131	-0.03	-0.06	0	0	0	0
246	15	N132	-0.182	-0.225	0	0	0	0
247	15	N140	-0.39	-0.158	0	0	0	0
248	15	N133	0.016	-0.072	0	0	0	0
249	15	N134	-0.044	-0.051	0	0	0	0
250	15	N136	-0.055	-0.044	0	0	0	0
251	15	N137	-0.054	-0.043	0	0	0	0
252	15	N138	-0.037	-0.05	0	0	0	0
253	15	N141	0.025	-0.076	0	0	0	0
254	15	N142	0.132	-0.12	0	0	0	0
255	15	N143	0.151	-0.127	0	0	0	0
256	15	N144	0.055	-0.089	0	0	0	0

Node Reactions (Continued)

	LC	Node Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
257	15	N145	-0.026	-0.055	0	0	0	0
258	15	N146	-0.051	-0.044	0	0	0	0
259	15	N147	-0.052	-0.049	0	0	0	0
260	15	N148	-0.04	-0.046	0	0	0	0
261	15	N149	-0.021	-0.067	0	0	0	0
262	15	N186	0.032	-0.571	0	0	0	0
263	15	N187	-0.001	-1.889	0	0	0	0
264	15	N188	-0.032	-0.563	0	0	0	0
265	15	N202	0	-0.09	0.022	0	0	0
266	15	N198	0	-0.026	0.006	0	0	0
267	15	N205	0	-0.251	0.063	0	0	0
268	15	N206	0	-0.219	0.054	0	0	0
269	15	N207	0	-0.225	0.056	0	0	0
270	15	N208	0	-0.224	0.056	0	0	0
271	15	N209	0	-0.224	0.056	0	0	0
272	15	N210	0	-0.224	0.056	0	0	0
273	15	N211	0	-0.224	0.056	0	0	0
274	15	N212	0	-0.224	0.056	0	0	0
275	15	N213	0	-0.224	0.056	0	0	0
276	15	N214	0	-0.225	0.056	0	0	0
277	15	N215	0	-0.223	0.055	0	0	0
278	15	N216	0	-0.232	0.058	0	0	0
279	15	N217	0	-0.186	0.047	0	0	0
280	15	COG (ft):	X: 8.136	Y: 8.59	Z: 23.972			