Moreland Shop

116 Mill Road
Dover, Idaho

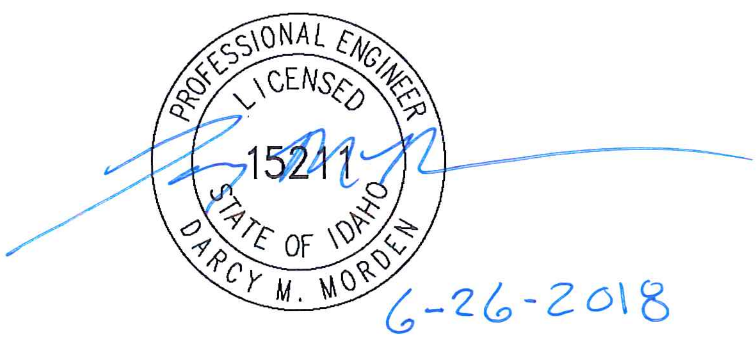
Structural Calculations

Prepared for:
Jim Moreland

Prepared by:
Darcy M Morden, P.E.

Project No.: 18-003

June 2018



6-26-2018

CITY OF DOVER
County of Bonner

RESIDENTIAL ROOF LOADS

SNOW LOAD: For all areas of **City of Dover** the **minimum** snow load shall be **55 psf.** It is recommended that the Calculation of Snow Load be determined by the Snow Study of the University of Idaho. GROUND SNOW LOAD 91.2 PSF
ROOF DESIGN SNOW LOAD => 80 PSF

DEAD LOAD: Is the vertical load due to the weight of all permanent structural and non-structural components of a building; such as walls, floors, roofs and fixed service equipment. The **typical** dead load for dwellings is **15 psf.**

WIND LOAD: Shall be based on basic wind speed of **90 mph (3-sec gust or 76mph fastest mile).** 115 MPH ULTIMATE WIND SPEED

EARTHQUAKE LOAD: Shall be based on **Seismic Zone C** of the International Residential Code.

Snow Loads : ASCE 7-10

Nominal Snow Forces

Roof slope	=	18.4 deg
Horiz. eave to ridge dist (W)	=	25.0 ft
Roof length parallel to ridge (L)	=	56.0 ft

Type of Roof			Hip and gable w/ trussed systems
Ground Snow Load	Pg	=	91.2 psf
Risk Category		=	II
Importance Factor	I	=	1.0
Thermal Factor	Ct	=	1.20
Exposure Factor	Ce	=	1.0
Pf = 0.7*Ce*Ct*I*Pg		=	76.6 psf
Unobstructed Slippery Surface			no
Sloped-roof Factor	Cs	=	1.00
Balanced Snow Load	**Ps**	=	**76.6 psf**
Rain on Snow Surcharge Angle			0.50 deg
Code Maximum Rain Surcharge			5.0 psf
Rain on Snow Surcharge		=	0.0 psf
Ps plus rain surcharge		=	76.6 psf
Minimum Snow Load	Pm	=	0.0 psf
Uniform Roof Design Snow Load		=	**76.6 psf** use 80.0

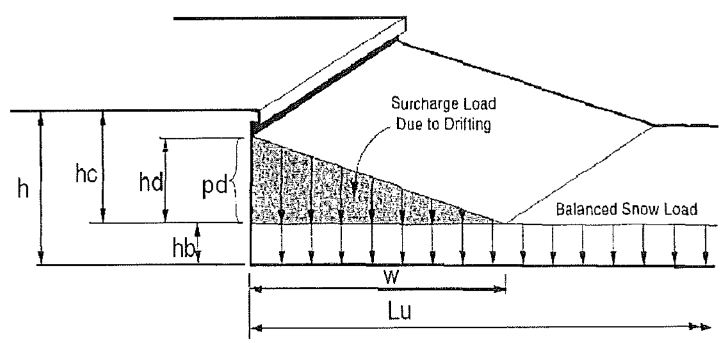
NOTE: Alternate spans of continuous beams and other areas shall be loaded with half the design roof snow load so as to produce the greatest possible effect - see code.

Unbalanced Snow Loads - for Hip & Gable roofs only

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 =	23.0 psf	= 0.3Ps
Leeward snow load from ridge to 11.49' =	113.7 psf	= hdγ / √S + Ps
Leeward snow load from 11.49' to the eave =	76.6 psf	= Ps

Windward Snow Drifts 1 - Against walls, parapets, etc more than 15' long

Upwind fetch	lu	=	28.0 ft
Projection height	h	=	12.0 ft
Snow density	g	=	25.9 pcf
Balanced snow height	hb	=	2.96 ft
	hd	=	1.98 ft
	hc	=	9.04 ft
hc/hb >0.2 = 3.1	**Therefore, design for drift**		
Drift height (hd)		=	1.98 ft
Drift width	w	=	7.92 ft
Surcharge load:	pd = γ*hd	=	**51.2 psf**
Balanced Snow load:		=	76.6 psf
			127.8 psf



Windward Snow Drifts 2 - Against walls, parapets, etc > 15'

Upwind fetch	lu	=	28.0 ft
Projection height	h	=	12.0 ft
Snow density	g	=	25.9 pcf
Balanced snow height	hb	=	2.96 ft
	hd	=	1.98 ft
	hc	=	9.04 ft
hc/hb >0.2 = 3.1	**Therefore, design for drift**		
Drift height (hd)		=	1.98 ft
Drift width	w	=	7.92 ft
Surcharge load:	pd = γ*hd	=	**51.2 psf**
Balanced Snow load:		=	76.6 psf
			127.8 psf

Wood Beam

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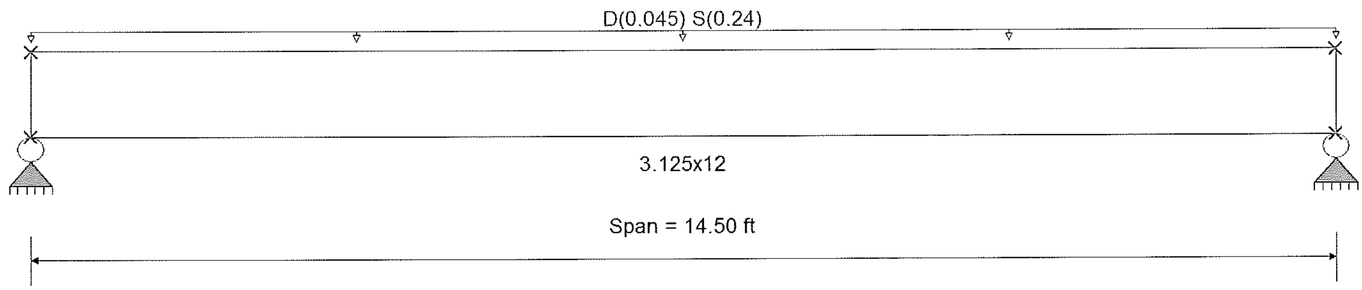
Description : Garage Door Header

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2400 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	2400 psi	Ebend- xx	1800 ksi
		Fc - Prll	1650 psi	Eminbend - xx	950 ksi
Wood Species	: DF/DF	Fc - Perp	650 psi	Ebend- yy	1600 ksi
Wood Grade	: 24F - V8	Fv	265 psi	Eminbend - yy	850 ksi
		Ft	1100 psi	Density	31.2 pcf
Beam Bracing	: Completely Unbraced				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.0150, S = 0.080 ksf, Tributary Width = 3.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.565 : 1	Maximum Shear Stress Ratio	=	0.242 : 1
Section used for this span		**3.125x12**	Section used for this span		**3.125x12**
fb : Actual	=	1,232.59psi	fv : Actual	=	73.84 psi
FB : Allowable	=	2,181.44psi	Fv : Allowable	=	304.75 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	7.250ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.296 in	Ratio =	587 >=240
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240
Max Downward Total Deflection	0.362 in	Ratio =	480 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 14.50 ft	1	0.118	0.056	0.90	1.000	1.00	1.00	1.00	1.00	0.88	1.40	223.39	1899.65	0.33	13.38	238.50
+D+L+H					1.000	1.00	1.00	1.00	1.00	0.88			0.00	0.00	0.00	0.00
Length = 14.50 ft	1	0.110	0.050	1.00	1.000	1.00	1.00	1.00	1.00	0.85	1.40	223.39	2031.90	0.33	13.38	265.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	0.85			0.00	0.00	0.00	0.00
Length = 14.50 ft	1	0.099	0.040	1.25	1.000	1.00	1.00	1.00	1.00	0.75	1.40	223.39	2252.96	0.33	13.38	331.25
+D+S+H					1.000	1.00	1.00	1.00	1.00	0.75			0.00	0.00	0.00	0.00
Length = 14.50 ft	1	0.565	0.242	1.15	1.000	1.00	1.00	1.00	1.00	0.79	7.70	1,232.59	2181.44	1.85	73.84	304.75
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	0.79			0.00	0.00	0.00	0.00

Description : Garage Door Header

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
Length = 14.50 ft	1	0.099	0.040	1.25	1.000	1.00	1.00	1.00	1.00	0.75	1.40	223.39	2252.96	0.33	13.38	331.25
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	0.75			0.00	0.00	0.00	0.00
Length = 14.50 ft	1	0.449	0.193	1.15	1.000	1.00	1.00	1.00	1.00	0.79	6.13	980.29	2181.44	1.47	58.72	304.75
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	0.79			0.00	0.00	0.00	0.00
Length = 14.50 ft	1	0.093	0.032	1.60	1.000	1.00	1.00	1.00	1.00	0.62	1.40	223.39	2394.40	0.33	13.38	424.00
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	0.62			0.00	0.00	0.00	0.00
Length = 14.50 ft	1	0.093	0.032	1.60	1.000	1.00	1.00	1.00	1.00	0.62	1.40	223.39	2394.40	0.33	13.38	424.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	0.62			0.00	0.00	0.00	0.00
Length = 14.50 ft	1	0.093	0.032	1.60	1.000	1.00	1.00	1.00	1.00	0.62	1.40	223.39	2394.40	0.33	13.38	424.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	0.62			0.00	0.00	0.00	0.00
Length = 14.50 ft	1	0.409	0.138	1.60	1.000	1.00	1.00	1.00	1.00	0.62	6.13	980.29	2394.40	1.47	58.72	424.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	0.62			0.00	0.00	0.00	0.00
Length = 14.50 ft	1	0.409	0.138	1.60	1.000	1.00	1.00	1.00	1.00	0.62	6.13	980.29	2394.40	1.47	58.72	424.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	0.62			0.00	0.00	0.00	0.00
Length = 14.50 ft	1	0.056	0.019	1.60	1.000	1.00	1.00	1.00	1.00	0.62	0.84	134.03	2394.40	0.20	8.03	424.00
+0.60D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	0.62			0.00	0.00	0.00	0.00
Length = 14.50 ft	1	0.056	0.019	1.60	1.000	1.00	1.00	1.00	1.00	0.62	0.84	134.03	2394.40	0.20	8.03	424.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.3620	7.303		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.125	2.125
Overall MINimum	1.740	1.740
+D+H	0.385	0.385
+D+L+H	0.385	0.385
+D+Lr+H	0.385	0.385
+D+S+H	2.125	2.125
+D+0.750Lr+0.750L+H	0.385	0.385
+D+0.750L+0.750S+H	1.690	1.690
+D+0.60W+H	0.385	0.385
+D+0.70E+H	0.385	0.385
+D+0.750Lr+0.750L+0.450W+H	0.385	0.385
+D+0.750L+0.750S+0.450W+H	1.690	1.690
+D+0.750L+0.750S+0.5250E+H	1.690	1.690
+0.60D+0.60W+0.60H	0.231	0.231
+0.60D+0.70E+0.60H	0.231	0.231
D Only	0.385	0.385
Lr Only		
L Only		
S Only	1.740	1.740
W Only		
E Only		
H Only		

Wood Beam

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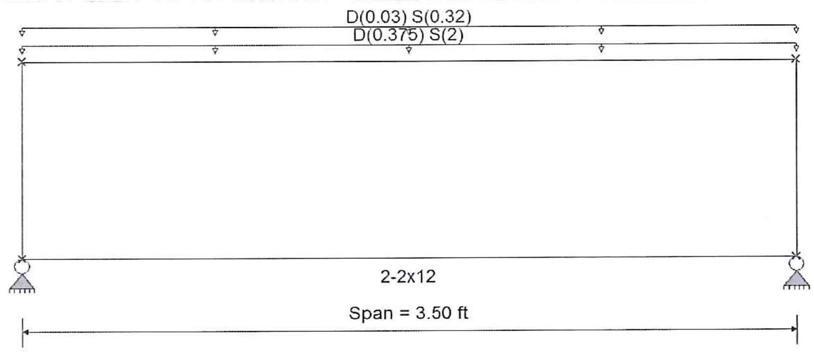
Description : Man Door Header OPTION 1

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	850.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	850.0 psi	Ebend- xx	1,300.0 ksi
		Fc - Prll	1,300.0 psi	Eminbend - xx	470.0 ksi
Wood Species	: Hem Fir	Fc - Perp	405.0 psi		
Wood Grade	: No.2	Fv	150.0 psi		
		Ft	525.0 psi	Density	26.830 pcf
Beam Bracing	: Completely Unbraced				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads

Uniform Load : D = 0.0150, S = 0.080 ksf, Tributary Width = 25.0 ft

Uniform Load : D = 0.0150, S = 0.160 ksf, Tributary Width = 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.821: 1	Maximum Shear Stress Ratio	=	0.575 : 1
Section used for this span		**2-2x12**	Section used for this span		**2-2x12**
fb : Actual	=	793.09psi	fv : Actual	=	99.24 psi
FB : Allowable	=	966.51psi	Fv : Allowable	=	172.50 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	1.750ft	Location of maximum on span	=	2.568 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.017 in	Ratio =	2466 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.020 in	Ratio =	2095 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.157	0.111	0.90	1.000	1.00	1.00	1.00	1.00	0.99	0.63	119.43	758.54	0.34	14.94	135.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.142	0.100	1.00	1.000	1.00	1.00	1.00	1.00	0.99	0.63	119.43	841.90	0.34	14.94	150.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.114	0.080	1.25	1.000	1.00	1.00	1.00	1.00	0.99	0.63	119.43	1049.28	0.34	14.94	187.50

Wood Beam

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Description : Man Door Header

Load Combination Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+S+H					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.821	0.575	1.15	1.000	1.00	1.00	1.00	1.00	0.99	4.18	793.09	966.51	2.23	99.24	172.50
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.114	0.080	1.25	1.000	1.00	1.00	1.00	1.00	0.99	0.63	119.43	1049.28	0.34	14.94	187.50
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.646	0.453	1.15	1.000	1.00	1.00	1.00	1.00	0.99	3.29	624.67	966.51	1.76	78.17	172.50
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.089	0.062	1.60	1.000	1.00	1.00	1.00	1.00	0.98	0.63	119.43	1336.95	0.34	14.94	240.00
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	0.98			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.089	0.062	1.60	1.000	1.00	1.00	1.00	1.00	0.98	0.63	119.43	1336.95	0.34	14.94	240.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	0.98			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.089	0.062	1.60	1.000	1.00	1.00	1.00	1.00	0.98	0.63	119.43	1336.95	0.34	14.94	240.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	0.98			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.467	0.326	1.60	1.000	1.00	1.00	1.00	1.00	0.98	3.29	624.67	1336.95	1.76	78.17	240.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	0.98			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.467	0.326	1.60	1.000	1.00	1.00	1.00	1.00	0.98	3.29	624.67	1336.95	1.76	78.17	240.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	0.98			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.054	0.037	1.60	1.000	1.00	1.00	1.00	1.00	0.98	0.38	71.66	1336.95	0.20	8.97	240.00
+0.60D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	0.98			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.054	0.037	1.60	1.000	1.00	1.00	1.00	1.00	0.98	0.38	71.66	1336.95	0.20	8.97	240.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.0200	1.763		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.780	4.780
Overall MINimum	4.060	4.060
+D+H	0.720	0.720
+D+L+H	0.720	0.720
+D+Lr+H	0.720	0.720
+D+S+H	4.780	4.780
+D+0.750Lr+0.750L+H	0.720	0.720
+D+0.750L+0.750S+H	3.765	3.765
+D+0.60W+H	0.720	0.720
+D+0.70E+H	0.720	0.720
+D+0.750Lr+0.750L+0.450W+H	0.720	0.720
+D+0.750L+0.750S+0.450W+H	3.765	3.765
+D+0.750L+0.750S+0.5250E+H	3.765	3.765
+0.60D+0.60W+0.60H	0.432	0.432
+0.60D+0.70E+0.60H	0.432	0.432
D Only	0.720	0.720
Lr Only		
L Only		
S Only	4.060	4.060
W Only		
E Only		
H Only		

Wood Beam

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Lic. # : KW-06000628 Licensee : WOMER & ASSOCIATES

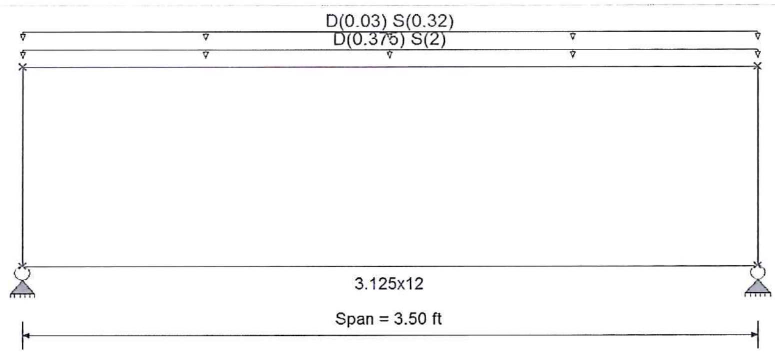
Description : Man Door Header OPTION 2

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2400 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	1850 psi	Ebend- xx	1800 ksi
		Fc - Prll	1650 psi	Eminbend - xx	950 ksi
Wood Species	: DF/DF	Fc - Perp	650 psi	Ebend- yy	1600 ksi
Wood Grade	: 24F - V4	Fv	265 psi	Eminbend - yy	850 ksi
		Ft	1100 psi	Density	31.2 pcf
Beam Bracing	: Completely Unbraced				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads

Uniform Load : D = 0.0150, S = 0.080 ksf, Tributary Width = 25.0 ft

Uniform Load : D = 0.0150, S = 0.160 ksf, Tributary Width = 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.247: 1	Maximum Shear Stress Ratio	=	0.270 : 1
Section used for this span		**3.125x12**	Section used for this span		**3.125x12**
fb : Actual	=	669.62 psi	fv : Actual	=	82.39 psi
FB : Allowable	=	2,706.81 psi	Fv : Allowable	=	304.75 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	1.750 ft	Location of maximum on span	=	2.504 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.010 in	Ratio =	4317 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.011 in	Ratio =	3665 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.048	0.052	0.90	1.000	1.00	1.00	1.00	1.00	0.99	0.63	101.22	2129.62	0.31	12.45	238.50
+D+L+H					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.043	0.047	1.00	1.000	1.00	1.00	1.00	1.00	0.98	0.63	101.22	2361.45	0.31	12.45	265.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	0.98			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.034	0.038	1.25	1.000	1.00	1.00	1.00	1.00	0.98	0.63	101.22	2935.30	0.31	12.45	331.25

Wood Beam

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Lic. # : KW-06000628 — Licensee : WOMER & ASSOCIATES

Description : Man Door Header

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
+D+S+H					1.000	1.00	1.00	1.00	1.00	0.98			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.247	0.270	1.15	1.000	1.00	1.00	1.00	1.00	0.98	4.19	669.62	2706.81	2.06	82.39	304.75
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	0.98			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.034	0.038	1.25	1.000	1.00	1.00	1.00	1.00	0.98	0.63	101.22	2935.30	0.31	12.45	331.25
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	0.98			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.195	0.213	1.15	1.000	1.00	1.00	1.00	1.00	0.98	3.30	527.52	2706.81	1.62	64.91	304.75
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	0.98			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.027	0.029	1.60	1.000	1.00	1.00	1.00	1.00	0.97	0.63	101.22	3722.04	0.31	12.45	424.00
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	0.97			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.027	0.029	1.60	1.000	1.00	1.00	1.00	1.00	0.97	0.63	101.22	3722.04	0.31	12.45	424.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	0.97			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.027	0.029	1.60	1.000	1.00	1.00	1.00	1.00	0.97	0.63	101.22	3722.04	0.31	12.45	424.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	0.97			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.142	0.153	1.60	1.000	1.00	1.00	1.00	1.00	0.97	3.30	527.52	3722.04	1.62	64.91	424.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	0.97			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.142	0.153	1.60	1.000	1.00	1.00	1.00	1.00	0.97	3.30	527.52	3722.04	1.62	64.91	424.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	0.97			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.016	0.018	1.60	1.000	1.00	1.00	1.00	1.00	0.97	0.38	60.73	3722.04	0.19	7.47	424.00
+0.60D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	0.97			0.00	0.00	0.00	0.00
Length = 3.50 ft	1	0.016	0.018	1.60	1.000	1.00	1.00	1.00	1.00	0.97	0.38	60.73	3722.04	0.19	7.47	424.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.0115	1.763		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 — Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.783	4.783
Overall MINimum	4.060	4.060
+D+H	0.723	0.723
+D+L+H	0.723	0.723
+D+Lr+H	0.723	0.723
+D+S+H	4.783	4.783
+D+0.750Lr+0.750L+H	0.723	0.723
+D+0.750L+0.750S+H	3.768	3.768
+D+0.60W+H	0.723	0.723
+D+0.70E+H	0.723	0.723
+D+0.750Lr+0.750L+0.450W+H	0.723	0.723
+D+0.750L+0.750S+0.450W+H	3.768	3.768
+D+0.750L+0.750S+0.5250E+H	3.768	3.768
+0.60D+0.60W+0.60H	0.434	0.434
+0.60D+0.70E+0.60H	0.434	0.434
D Only	0.723	0.723
Lr Only		
L Only		
S Only	4.060	4.060
W Only		
E Only		
H Only		

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Wall Footing

Lic. # : KW-06000628 Licensee : WOMER & ASSOCIATES

Description : Typical Foundation

Code References

Calculations per ACI 318-14, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : IBC 2018

General Information

Material Properties

f'c : Concrete 28 day strength	=	2.50	ksi
fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	3,122.0	ksi
Concrete Density	=	145.0	pcf
φ Values Flexure	=	0.90	
Shear	=	0.750	

Analysis Settings

Min Steel % Bending Reinf.	=		
Min Allow % Temp Reinf.	=	0.00180	
Min. Overturning Safety Factor	=	1.0 : 1	
Min. Sliding Safety Factor	=	1.0 : 1	
AutoCalc Footing Weight as DL	:	Yes	

Soil Design Values

Allowable Soil Bearing	=	1.550	ksf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	250.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

Reference Depth below Surface	=	2.0	ft
Allow. Pressure Increase per foot of depth	=	0.0	ksf
when base footing is below	=	0.0	ft

Increases based on footing Width

Allow. Pressure Increase per foot of width	=	0.0	ksf
when footing is wider than	=	0.0	ft

Adjusted Allowable Bearing Pressure = 1.550 ksf

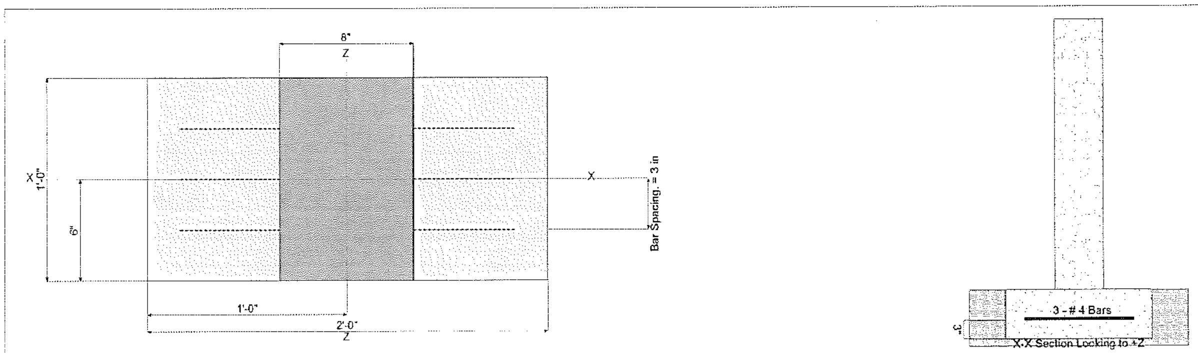
Dimensions

Footing Width	=	2.0	ft
Wall Thickness	=	8.0	in
Wall center offset from center of footing	=	0	in
Footing Thickness	=	8.0	in
Rebar Centerline to Edge of Concrete... at Bottom of footing	=	3.0	in

Reinforcing

Bars along X-X Axis

# of Bars in 12" Width	=	3
Reinforcing Bar Size	=	# 4



Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	0.5650	0.0	0.0	2.320	0.0	0.0	0.0	k
OB : Overburden	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0	ksf
V-x	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0	k
M-zz	=	0.0	0.0	0.0	0.0	0.0	0.0	0.0	k-ft

Vx applied = 0.0 in above top of footing

Description : Typical Foundation

DESIGN SUMMARY

Design OK

	Factor of Safety	Item	Applied	Capacity	Governing Load Combination
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift

	Utilization Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.9930	Soil Bearing	1.539 ksf	1.550 ksf	+D+S+H
PASS	0.04429	Z Flexure (+X)	0.5136 k-ft	11.594 k-ft	+1.20D+0.50L+1.60S+1
PASS	0.006541	Z Flexure (-X)	0.07583 k-ft	11.594 k-ft	+0.90D+E+0.90H
PASS	0.1233	1-way Shear (+X)	9.244 psi	75.0 psi	+1.20D+0.50L+1.60S+1
PASS	0.1233	1-way Shear (-X)	9.244 psi	75.0 psi	+1.20D+0.50L+1.60S+1

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc	Actual Soil Bearing Stress -X	Actual Soil Bearing Stress +X	Actual / Allowable Ratio
, +D+H	1.550 ksf	0.0 in	0.3792 ksf	0.3792 ksf	0.245
, +D+L+H	1.550 ksf	0.0 in	0.3792 ksf	0.3792 ksf	0.245
, +D+Lr+H	1.550 ksf	0.0 in	0.3792 ksf	0.3792 ksf	0.245
, +D+S+H	1.550 ksf	0.0 in	1.539 ksf	1.539 ksf	0.993
, +D+0.750Lr+0.750L+H	1.550 ksf	0.0 in	0.3792 ksf	0.3792 ksf	0.245
, +D+0.750L+0.750S+H	1.550 ksf	0.0 in	1.249 ksf	1.249 ksf	0.806
, +D+0.60W+H	1.550 ksf	0.0 in	0.3792 ksf	0.3792 ksf	0.245
, +D+0.70E+H	1.550 ksf	0.0 in	0.3792 ksf	0.3792 ksf	0.245
, +D+0.750Lr+0.750L+0.450W+H	1.550 ksf	0.0 in	0.3792 ksf	0.3792 ksf	0.245
, +D+0.750L+0.750S+0.450W+H	1.550 ksf	0.0 in	1.249 ksf	1.249 ksf	0.806
, +D+0.750L+0.750S+0.5250E+H	1.550 ksf	0.0 in	1.249 ksf	1.249 ksf	0.806
, +0.60D+0.60W+0.60H	1.550 ksf	0.0 in	0.2275 ksf	0.2275 ksf	0.147
, +0.60D+0.70E+0.60H	1.550 ksf	0.0 in	0.2275 ksf	0.2275 ksf	0.147

Overturning Stability

Units : k-ft

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status

Footing Has NO Overturning

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Sliding SafetyRatio	Status

Footing Has NO Sliding

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Which Side ?	Tension @ Bot. or Top ?	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
, +1.40D+1.60H	0.118	-X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +1.40D+1.60H	0.118	+X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +1.20D+0.50Lr+1.60L+1.60H	0.1011	-X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +1.20D+0.50Lr+1.60L+1.60H	0.1011	+X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +1.20D+1.60L+0.50S+1.60H	0.23	-X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +1.20D+1.60L+0.50S+1.60H	0.23	+X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +1.20D+1.60Lr+0.50L+1.60H	0.1011	-X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +1.20D+1.60Lr+0.50L+1.60H	0.1011	+X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +1.20D+1.60Lr+0.50W+1.60H	0.1011	-X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +1.20D+1.60Lr+0.50W+1.60H	0.1011	+X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +1.20D+0.50L+1.60S+1.60H	0.5136	-X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +1.20D+0.50L+1.60S+1.60H	0.5136	+X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +1.20D+1.60S+0.50W+1.60H	0.5136	-X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +1.20D+1.60S+0.50W+1.60H	0.5136	+X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +1.20D+0.50Lr+0.50L+W+1.60H	0.1011	-X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +1.20D+0.50Lr+0.50L+W+1.60H	0.1011	+X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +1.20D+0.50L+0.50S+W+1.60H	0.23	-X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +1.20D+0.50L+0.50S+W+1.60H	0.23	+X	Bottom	0.1728	Min Temp %	0.6	11.594	OK

Description : Typical Foundation

, +1.20D+0.50L+0.70S+E+1.60H	0.2816	-X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +1.20D+0.50L+0.70S+E+1.60H	0.2816	+X	Bottom	0.1728	Min Temp %	0.6	11.594	OK

Wall Footing

Description : Typical Foundation

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Which Side ?	Tension @ Bot. or Top ?	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
, +0.90D+W+0.90H	0.07583	-X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +0.90D+W+0.90H	0.07583	+X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +0.90D+E+0.90H	0.07583	-X	Bottom	0.1728	Min Temp %	0.6	11.594	OK
, +0.90D+E+0.90H	0.07583	+X	Bottom	0.1728	Min Temp %	0.6	11.594	OK

One Way Shear

Units : k

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D+1.60H	2.123 psi	2.123 psi	2.123 psi	75 psi	0.02831	OK
+1.20D+0.50Lr+1.60L+1.60H	1.82 psi	1.82 psi	1.82 psi	75 psi	0.02427	OK
+1.20D+1.60L+0.50S+1.60H	4.14 psi	4.14 psi	4.14 psi	75 psi	0.0552	OK
+1.20D+1.60Lr+0.50L+1.60H	1.82 psi	1.82 psi	1.82 psi	75 psi	0.02427	OK
+1.20D+1.60Lr+0.50W+1.60H	1.82 psi	1.82 psi	1.82 psi	75 psi	0.02427	OK
+1.20D+0.50L+1.60S+1.60H	9.244 psi	9.244 psi	9.244 psi	75 psi	0.1233	OK
+1.20D+1.60S+0.50W+1.60H	9.244 psi	9.244 psi	9.244 psi	75 psi	0.1233	OK
+1.20D+0.50Lr+0.50L+W+1.60H	1.82 psi	1.82 psi	1.82 psi	75 psi	0.02427	OK
+1.20D+0.50L+0.50S+W+1.60H	4.14 psi	4.14 psi	4.14 psi	75 psi	0.0552	OK
+1.20D+0.50L+0.70S+E+1.60H	5.068 psi	5.068 psi	5.068 psi	75 psi	0.06757	OK
+0.90D+W+0.90H	1.365 psi	1.365 psi	1.365 psi	75 psi	0.0182	OK
+0.90D+E+0.90H	1.365 psi	1.365 psi	1.365 psi	75 psi	0.0182	OK

JOB TITLE	116 Mill Rd Shop
JOB NO.	18-003
SHEET NO.	
CALCULATED BY	DMM
DATE	
CHECKED BY	
DATE	

Wind Loads : ASCE 7- 10

Ultimate Wind Speed	115 mph
Nominal Wind Speed	89.1 mph
Risk Category	II
Exposure Category	C
Enclosure Classif.	Enclosed Building
Internal pressure	+/-0.18
Directionality (Kd)	0.85
Kh case 1	0.905
Kh case 2	0.905
Type of roof	Gable

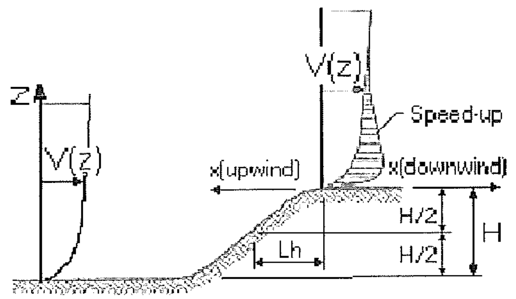
Topographic Factor (Kzt)

Topography	2D Escarpment
Hill Height (H)	270.0 ft
Half Hill Length (Lh)	900.0 ft
Actual H/Lh =	0.30
Use H/Lh =	0.30
Modified Lh =	900.0 ft
From top of crest: x =	10.0 ft
Bldg up/down wind?	upwind

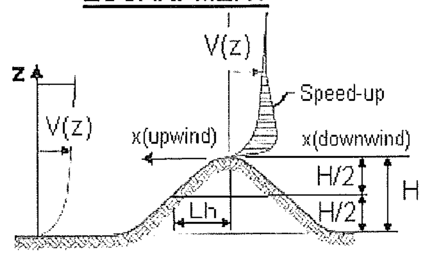
H/Lh= 0.30	K_1 = 0.255
x/Lh = 0.01	K_2 = 0.993
z/Lh = 0.02	K_3 = 0.945

At Mean Roof Ht:

Kzt = $(1+K_1K_2K_3)$^2 = 1.54



ESCARPMENT



2D RIDGE or 3D AXISYMMETRICAL HILL

Gust Effect Factor

h =	20.3 ft
B =	50.0 ft
/z (0.6h) =	15.0 ft

Flexible structure if natural frequency < 1 Hz (T > 1 second).
However, if building h/B < 4 then probably rigid structure (rule of thumb).
h/B = 0.41 Rigid structure

G = 0.85 Using rigid structure default

Rigid Structure

ē =	0.20
ℓ =	500 ft
z_{min} =	15 ft
c =	0.20
g_Q, g_v =	3.4
L_z =	427.1 ft
Q =	0.91
I_z =	0.23
G =	0.88 use G = 0.85

Flexible or Dynamically Sensitive Structure

Natural Frequency (η_1) =	0.0 Hz				
Damping ratio (β) =	0				
/b =	0.65				
/α =	0.15				
Vz =	97.1				
N_1 =	0.00				
R_n =	0.000				
R_h =	28.282	η =	0.000	h =	20.3 ft
R_B =	28.282	η =	0.000		
R_L =	28.282	η =	0.000		
g_R =	0.000				
R =	0.000				
G =	0.000				

Enclosure Classification

Test for Enclosed Building: A building that does not qualify as open or partially enclosed.

Test for Open Building: All walls are at least 80% open.
$A_o \geq 0.8A_g$

Test for Partially Enclosed Building:

	Input			Test	
Ao	100000.0	sf	Ao ≥ 1.1Aoi	YES	
Ag	0.0	sf	Ao > 4' or 0.01Ag	YES	
Aoi	0.0	sf	Aoi / Agi ≤ 0.20	NO	Building is NOT Partially Enclosed
Agi	0.0	sf			

ERROR: Ag must be greater than Ao

Conditions to qualify as Partially Enclosed Building. Must satisfy all of the following:

$A_o \geq 1.1A_{oi}$

$A_o >$ smaller of 4' or 0.01 Ag

$A_{oi} / A_{gi} \leq 0.20$

Where:

Ao = the total area of openings in a wall that receives positive external pressure.
Ag = the gross area of that wall in which Ao is identified.
Aoi = the sum of the areas of openings in the building envelope (walls and roof) not including Ao.
Agi = the sum of the gross surface areas of the building envelope (walls and roof) not including Ag.

Reduction Factor for large volume partially enclosed buildings (Ri) :

If the partially enclosed building contains a single room that is unpartitioned , the internal pressure coefficient may be multiplied by the reduction factor Ri.

Total area of all wall & roof openings (Aog):		0 sf
Unpartitioned internal volume (Vi) :		0 cf
	Ri =	1.00

Altitude adjustment to constant 0.00256 (caution - see code) :

Altitude = 0 feet Average Air Density = 0.0765 lbm/ft^3
Constant = 0.00256

JOB TITLE	116 Mill Rd Shop		
JOB NO.	18-003	SHEET NO.	
CALCULATED BY	DMM	DATE	
CHECKED BY		DATE	

Wind Loads - MWFRS all h (Enclosed/partially enclosed only)

Kh (case 2) =	0.91	h =	20.3 ft	GCpi =	+/-0.18
Base pressure (q_h) =	**40.0 psf**	ridge ht =	24.5 ft	G =	0.85
Roof Angle (θ) =	18.4 deg	L =	56.0 ft	qi = qh	
Roof tributary area - (h/2)*L:	569 sf	B =	50.0 ft		
(h/2)*B:	508 sf				

Ultimate Wind Surface Pressures (psf)

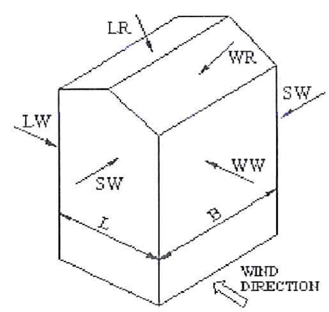
Surface	Wind Normal to Ridge				Wind Parallel to Ridge				
	B/L = 0.89		h/L = 0.41			L/B = 1.12		h/L = 0.36	
	Cp	q_hGC_p	w/+q_iGC_{pi}	w/-q_hGC_{pi}	Dist.*	Cp	q_hGC_p	w/ +q_iGC_{pi}	w/ -q_hGC_{pi}
Windward Wall (WW)	0.80	27.2	see table below			0.80	27.2	see table below	
Leeward Wall (LW)	-0.50	-17.0	-24.2	-9.8		-0.48	-16.2	-23.4	-9.0
Side Wall (SW)	-0.70	-23.8	-31.0	-16.6		-0.70	-23.8	-31.0	-16.6
Leeward Roof (LR)	-0.57	-19.3	-26.5	-12.1		Included in windward roof			
Neg Windward Roof pressure	-0.44	-15.1	-22.3	-7.9	0 to h/2*	-0.90	-30.6	-37.8	-23.4
Pos/min Windward Roof press.	0.02	0.5	-6.7	7.7	h/2 to h*	-0.90	-30.6	-37.8	-23.4
					h to 2h*	-0.50	-17.0	-24.2	-9.8
					> 2h*	-0.30	-10.2	-17.4	-3.0
					Min press.	-0.18	-6.1	-13.3	1.1

*Horizontal distance from windward edge

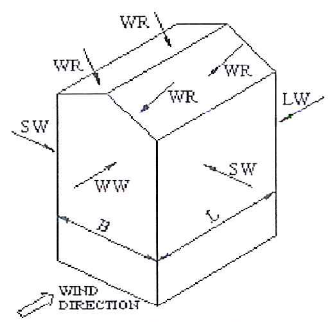
Windward Wall Pressures at "z" (psf)

	z	Kz	Kzt	q_zGC_p	Windward Wall w/+q_iGC_{pi}	Windward Wall w/-q_hGC_{pi}	Combined WW + LW Normal to Ridge	Combined WW + LW Parallel to Ridge
	0 to 15'	0.85	1.56	25.9	18.7	33.1	42.9	42.1
	20.0 ft	0.90	1.54	27.2	20.0	34.4	44.2	43.4
h=	20.3 ft	0.91	1.54	27.2	20.0	34.4	44.2	43.4
ridge =	24.5 ft	0.94	1.53	28.2	21.0	35.4	45.2	44.4

ULTIMATE LOADS



WIND NORMAL TO RIDGE



WIND PARALLEL TO RIDGE

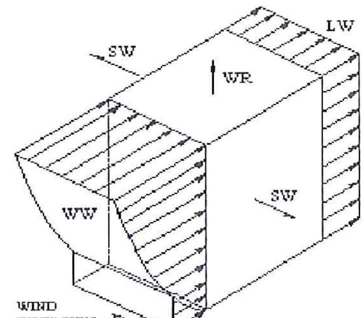
NOTE:
See figure in ASCE7 for the application of full and partial loading of the above wind pressures. There are 4 different loading cases.

Parapet

z	Kz	Kzt	qp (psf)
0.0 ft	0.85	1.57	0.0

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

Windward roof overhangs (add to windward roof pressure) : 27.2 psf (upward)



TYPICAL WIND LOADING

JOB TITLE 116 Mill Rd Shop

JOB NO.	18-003	SHEET NO.	
CALCULATED BY	DMM	DATE	
CHECKED BY		DATE	

ASD LOADS

Wind Loads - MWFRS all h (Enclosed/partially enclosed only)

Kh (case 2) =	0.91	h =	20.3 ft	GCpi =	+/-0.18
Base pressure (q_h) =	**24.0 psf**	ridge ht =	24.5 ft	G =	0.85
Roof Angle (θ) =	18.4 deg	L =	56.0 ft	qi = qh	
Roof tributary area - (h/2)*L:	569 sf	B =	50.0 ft		
(h/2)*B:	508 sf				

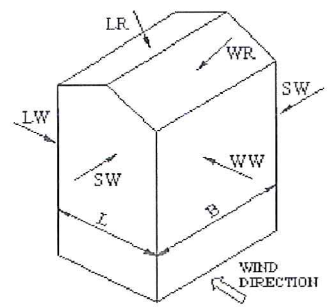
Nominal Wind Surface Pressures (psf)

Surface	Wind Normal to Ridge				Wind Parallel to Ridge				
	B/L = 0.89		h/L = 0.41		L/B = 1.12			h/L = 0.36	
	Cp	q_hGC_p	w/+q_iGC_{pi}	w/-q_hGC_{pi}	Dist.*	Cp	q_hGC_p	w/ +q_iGC_{pi}	w/ -q_hGC_{pi}
Windward Wall (WW)	0.80	16.3	see table below			0.80	16.3	see table below	
Leeward Wall (LW)	-0.50	-10.2	-14.5	-5.9		-0.48	-9.7	-14.0	-5.4
Side Wall (SW)	-0.70	-14.3	-18.6	-10.0		-0.70	-14.3	-18.6	-10.0
Leeward Roof (LR)	-0.57	-11.6	-15.9	-7.3		Included in windward roof			
Neg Windward Roof pressure	-0.44	-9.1	-13.4	-4.8	0 to h/2*	-0.90	-18.4	-22.7	-14.0
Pos/min Windward Roof press.	0.02	0.3	-4.0	4.6	h/2 to h*	-0.90	-18.4	-22.7	-14.0
					h to 2h*	-0.50	-10.2	-14.5	-5.9
					> 2h*	-0.30	-6.1	-10.4	-1.8
					Min press.	-0.18	-3.7	-8.0	0.6

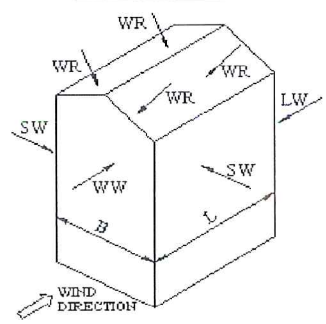
*Horizontal distance from windward edge

Windward Wall Pressures at "z" (psf)

	z	Kz	Kzt	Windward Wall q_zGC_p	Windward Wall w/+q_iGC_{pi}	Windward Wall w/-q_hGC_{pi}	Combined WW + LW Normal to Ridge	Combined WW + LW Parallel to Ridge
	0 to 15'	0.85	1.56	15.5	11.2	19.8	25.7	25.2
	20.0 ft	0.90	1.54	16.3	12.0	20.6	26.5	26.0
h=	20.3 ft	0.91	1.54	16.3	12.0	20.6	26.5	26.0
ridge =	24.5 ft	0.94	1.53	16.9	12.6	21.3	27.1	26.6



WIND NORMAL TO RIDGE



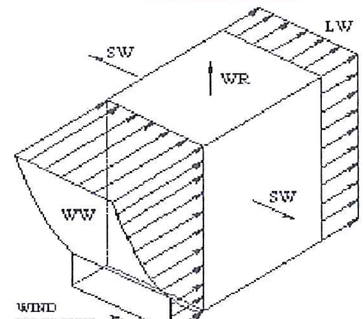
WIND PARALLEL TO RIDGE

NOTE:
See figure in ASCE7 for the application of full and partial loading of the above wind pressures. There are 4 different loading cases.

Parapet

z	Kz	Kzt	qp (psf)
0.0 ft	0.85	1.57	0.0

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



TYPICAL WIND LOADING

Windward roof overhangs (add to windward roof pressure) : 16.3 psf (upward)

Job#: 18-003

STUD WALL CALCULATIONS

Stud Width (dy)	1.50	in
Stud Depth (dx)	7.25	in
Stud Length (L)	16.00	ft
Stud Spacing	16	in
Stud Species and Grade	2X8 DF No. 2	
Top/Sill Plt. Species	DF	

Vertical Loads

Wall LL (wLL)	2320	plf
Wall DL (wDL)	405	plf
Wall DL (wTL)	2725	plf
Trib. Length	1.33	ft
Pc	3633.33	lbs

Design Values

Fb	900	psi
Fc	1350	psi
Fc⊥	625	psi
E	1,600,000	psi
Emin	580,000	psi
CF_b	1.20	
CF_c	1.05	
A	10.88	in^2
Sx	13.14	in^3
Ix	47.63	in^4
Ct_c	1.00	
CM_c	1.00	
Ci_c	1.00	

Lateral Loads (Wind MWFRS)

Wind Load (windward wall)	45.00	psf
MWFRS Wind Load ASD	27.00	psf
Wind Atrib	21.33	ft^2
W	576.00	lbs
w	36.00	plf

Lateral Loads (Wind C&C)

Wind Load (Zone 4)	53.30	psf
CC Wind Load ASD	31.98	psf
W	682.24	lbs
w	42.64	plf

Load Case 1: Gravity Loads Only

ly (unbraced length)	1.0	ft
CD	1.15	(Snow Load)
(le/d)y	8.00	
(le/d)x	26.48	(governs)
E'min	580,000	psi
FcE	679.79	psi
Fc*	1630.13	psi
c	0.80	sawn lumber
FcE/Fc*	0.417	
1 + FcE/Fc*/2c	0.886	
Cp	0.373	
Fc'	607.58	psi
fc	334.10	psi
CSI (axial)	**0.55**	**OK**

Load Case 2: Lateral Loads Only (Wind C&C)

Mmax	1364.48	ft-lbs
	16373.76	in-lbs
fbx	1246.04	psi
CSI (bending C&C)	**0.64**	**OK**

Load Case 3: Gravity Loads and Lateral Loads

CD	1.60	(Wind/Seismic)
Mmax	1152.00	ft-lbs
	13824.00	in-lbs
CL	0.99	
Cr	1.15	@ 16 O/C
Fbx'	1958.90	psi
fbx	1052.00	psi
CSI (bending MWFRFS)	**0.54**	**OK**

Bearing on Stud Wall Plates

lb	1.50	in
Cb	1.00	(conservative)
Fc⊥'	625.00	psi
fc⊥	334.10	psi
CSI (bearing)	**0.53**	**OK**

Combined Stress

(re-evaluate compression values with CD = 1.6)

FcEx	679.79	psi
FcE	679.79	psi
Fc*	2268.00	psi
c	0.80	sawn lumber
FcE/Fc*	0.300	
1 + FcE/Fc*/2c	0.812	
Cp	0.278	
Fc'	631.12	psi

Deflection

E'	1,600,000	psi
ΔWIND (.42C&C)*	0.58	in
L/d**	332	OK

$$\left(\frac{f_c}{F_c'}\right)^2 + \left(\frac{1}{1-\frac{f_c}{F_{cEx}}}\right)\left(\frac{f_{bx}}{F_{bx}'}\right) = 0.59 \text{ OK}$$

*IBC 2015 Sec. 1604.3

**IRC 2015 Sec. 301.7

Load Case: LC5

*LCMAX takes 100% of all loads for axial and bending.

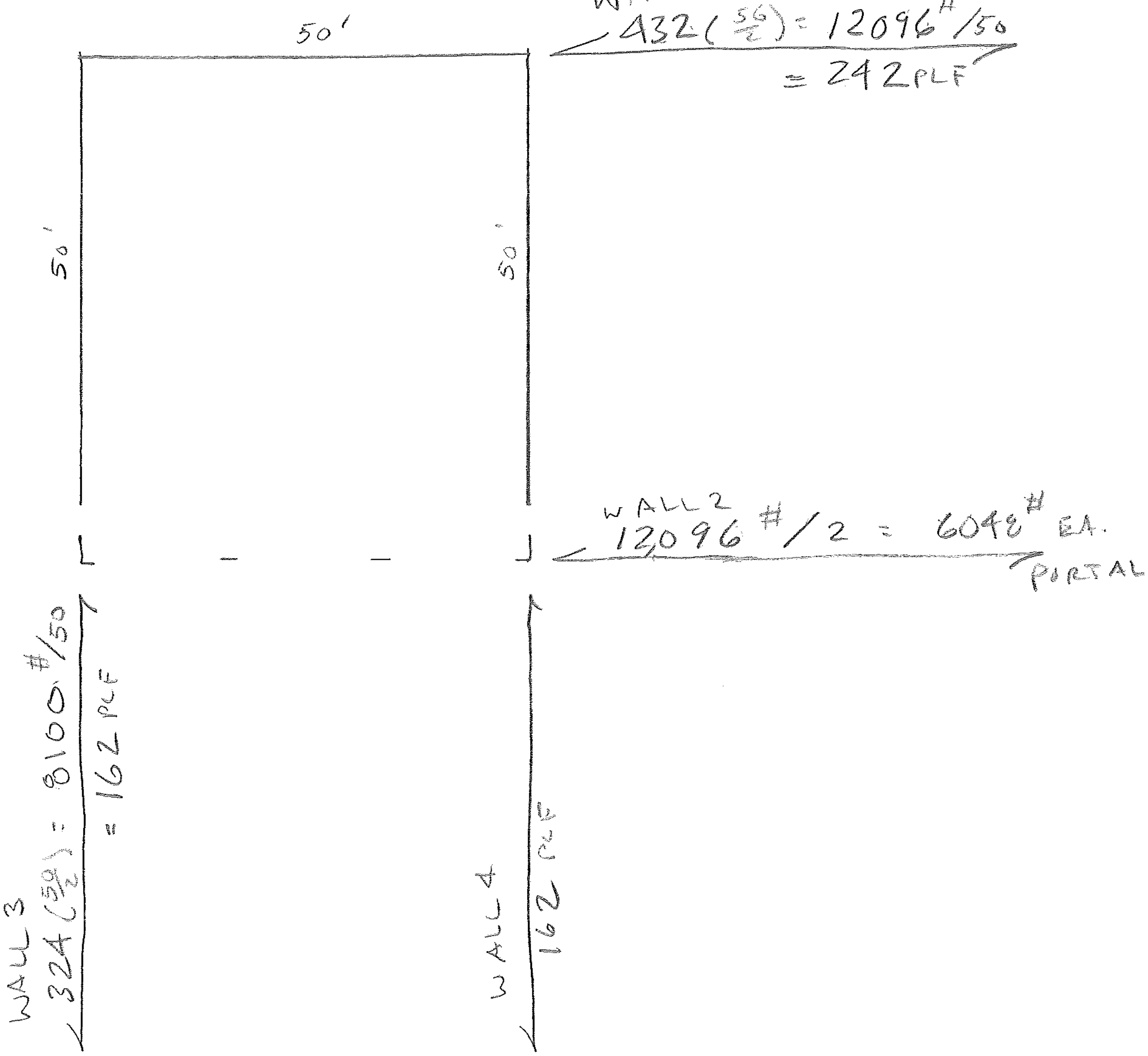
Location:	Arthur Shop Exterior Studs
Specification:	**Use 2X8 DF No. 2 Grade @16" o/c**

Rev. 1.3.0 - 04/08/2015

WIND CONTROLS

LONG = 27 psf (8+8) = 432 plf

TRANS = 27 psf [(8)(50) + ½ (8)(50)] = 16,200#/50 = 324 plf



PROJECT	Date	Design By
116 MILL ROAD - SHOP	6-25-18	DM
DOVER, IDAHO	Project No. 18-003	Sheet No. 18

	Project	Engineer: DMM	Project #
	116 Mill Road Shop	Date: 6/25/2018	18-003
	Subject	Checker:	Page
	Shearwall Design	Date:	

Floor Level: **First**
Wall Line: **1**

Unit Shear Calculations

Seismic Design Category E or F? **no** *(Prohibits Use of Gypsum Shear Walls)*

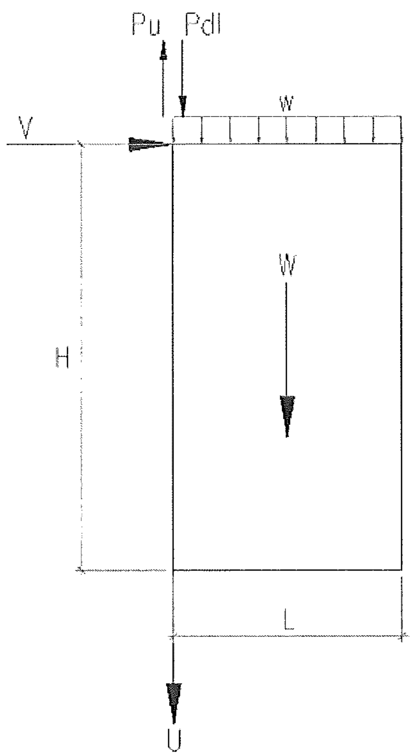
Lateral Load to Wall Line = **12,096** lbs
Total Length of Shearwalls = **50.0** ft

Unit Shear Load (v) = **242** plf ←

Use Shearwall Type	
Plywood	**Gypboard**
P1-6	**G2-6**

Reference attached shearwall schedule for more information.

Overturning Calculations



Seismic Controlled Design? **no** (Affects aspect ratio)

Terminology: V = Panel Shear (lbs)
W = Panel Self Weight (lbs)
w = Trib. Roof/Floor Dead Load (plf)
P_{dl} = DL Reaction from Header/Beam (lbs)
P_u = Uplift from Shearwall Above (lbs)
OTM = Overturning Moment (ft-lbs)
RM = DL Resisting Moment (ft-lbs)

Equations: $V = vL$
$OTM = VH$
$RM = 2/3[(W+wL)(L/2)+P_{dl}L]$
$U = (OTM-RM)/L + P_u$

Reference attached Simpson cutsheets for holdown descriptions, capacities, and installation requirements. Holdown capacities are based on SPF/HF values with 1.33 load duration factor.

Load Check, ΣV = 12,096 (Compare w/ Load Above)
Max. Aspect Ratio: 3.5
Check Aspect Ratio: OK
(Ref. IBC Table 2305.3.4, footnote (a), when aspect ratios are exceeded)

H (ft)	L (ft)	V	W	w	Pdl	Pu	Uplift (U)	Req'd Holdown	
								Anchor	**Strap**
16.0	**50.0**	12,096	8,000	**36**	**1,000**	**0**	-63	**NA**	**NA**
0.0	**0.0**	0	0	**0**	**0**	**0**	0	**NA**	**NA**
0.0	**0.0**	0	0	**0**	**0**	**0**	0	**NA**	**NA**
0.0	**0.0**	0	0	**0**	**0**	**0**	0	**NA**	**NA**
0.0	**0.0**	0	0	**0**	**0**	**0**	0	**NA**	**NA**
0.0	**0.0**	0	0	**0**	**0**	**0**	0	**NA**	**NA**
0.0	**0.0**	0	0	**0**	**0**	**0**	0	**NA**	**NA**
0.0	**0.0**	0	0	**0**	**0**	**0**	0	**NA**	**NA**

	Project 116 Mill Road Shop	Engineer: DMM Date: 6/25/2018	Project # 18-003
	Subject Shearwall Design	Checker: Date:	Page

Floor Level: **First**
Wall Line: 3 & 4

Unit Shear Calculations

Seismic Design Category E or F? **no** *(Prohibits Use of Gypsum Shear Walls)*

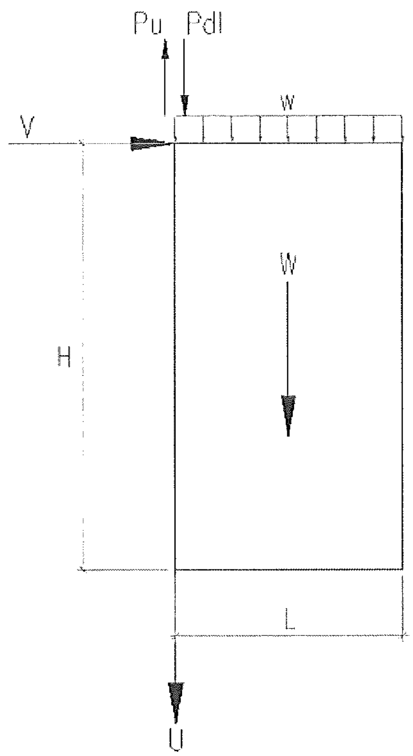
Lateral Load to Wall Line = **8,100** lbs
Total Length of Shearwalls = **50.0** ft

Unit Shear Load (v) = **162** plf ←

Use Shearwall Type	
Plywood	**Gypboard**
P1-6	**G2-6**

Reference attached shearwall schedule for more information.

Overturning Calculations



Seismic Controlled Design? **no** (Affects aspect ratio)

Terminology : V = Panel Shear (lbs)
W = Panel Self Weight (lbs)
w = Trib. Roof/Floor Dead Load (plf)
P_{dl} = DL Reaction from Header/Beam (lbs)
P_u = Uplift from Shearwall Above (lbs)
OTM = Overturning Moment (ft-lbs)
RM = DL Resisting Moment (ft-lbs)

Equations: $V = vL$
$OTM = VH$
$RM = 2/3[(W+wL)(L/2)+P_{dl}L]$
$U = (OTM-RM)/L + P_u$

Reference attached Simpson cutsheets for holdown descriptions, capacities, and installation requirements. Holdown capacities are based on SPF/HF values with 1.33 load duration factor.

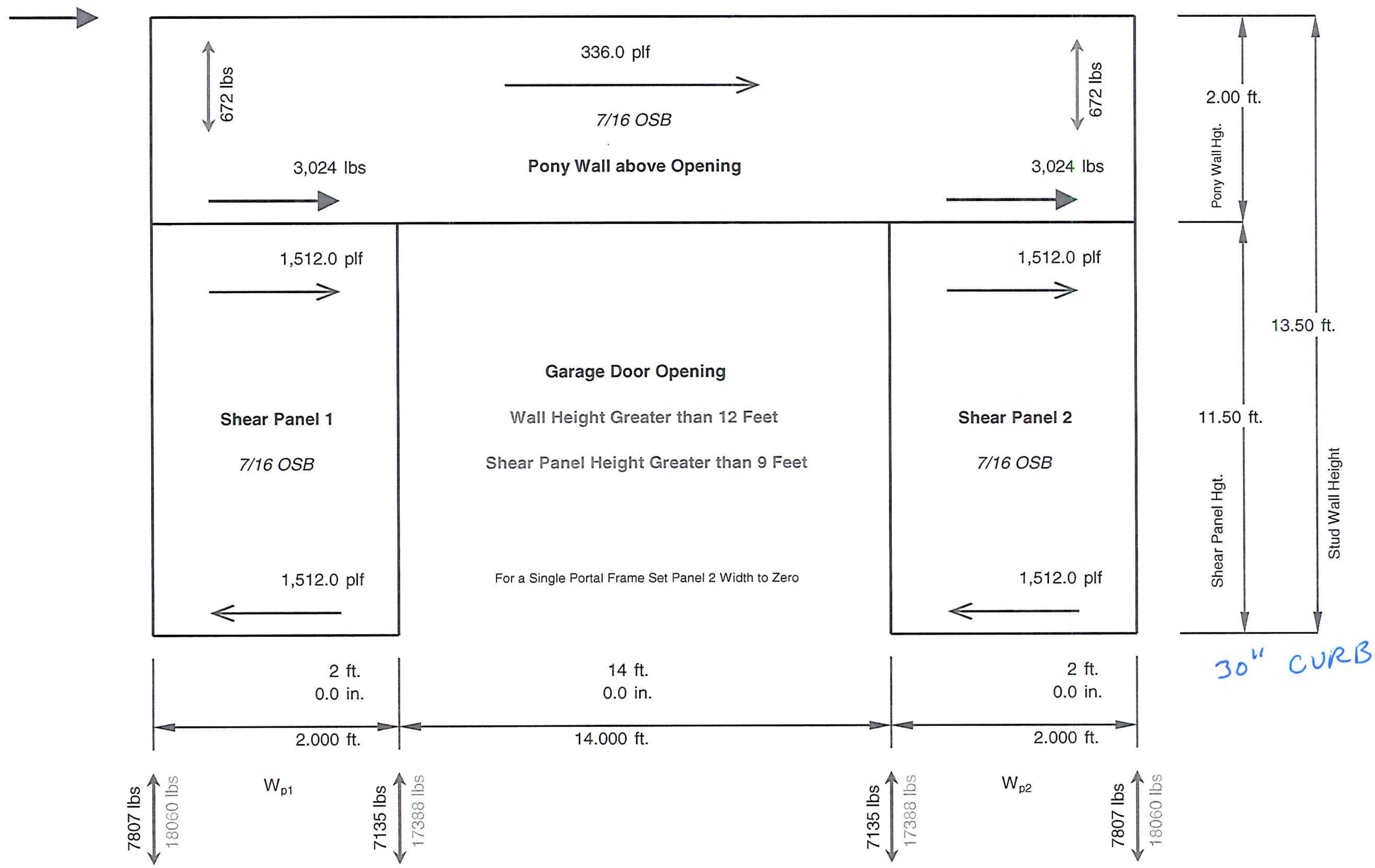
Load Check, ΣV = *8,100* *(Compare w/ Load Above)*
Max. Aspect Ratio: *3.5*
Check Aspect Ratio: *OK*
(Ref. IBC Table 2305.3.4, footnote (a), when aspect ratios are exceeded)

H (ft)	L (ft)	V	W	w	Pdl	Pu	Uplift (U)	Req'd Holdown	
								Anchor	**Strap**
16.0	50.0	8,100	8,000	320	1,000	0	-6,075	**NA**	**NA**
0.0	0.0	0	0	0	0	0	0	**NA**	**NA**
0.0	0.0	0	0	0	0	0	0	**NA**	**NA**
0.0	0.0	0	0	0	0	0	0	**NA**	**NA**
0.0	0.0	0	0	0	0	0	0	**NA**	**NA**
0.0	0.0	0	0	0	0	0	0	**NA**	**NA**
0.0	0.0	0	0	0	0	0	0	**NA**	**NA**
0.0	0.0	0	0	0	0	0	0	**NA**	**NA**

PORTAL FRAME CALCULATOR PFH1 Vs = 3,000 lbs (ASD) Vw = 6,048 lbs (ASD) **Job#: 18-003**

Wind Load Governs

Total Wall Length:	18.00 ft.
Portal Frame Deflection (ASD):	1.20 in.
Max. Tension of Header Strap:	5,312 lbs



*Notes: 1.) Shear Distribution based on equivalent deflection of each shear panel.
2.) Holdown forces shown in grey indicate loads if holdowns assume 100% of shear panel overturning moments.
3.) Holdown forces conservative since counteracting dead loads (LC7: 0.6D + 0.6W) are not considered in analysis.

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Job#: 18-003

<u>Design Criteria:</u>

Max. Shear @ Panels =	3,024 lbs	Sheathing Panel Thickness =	7/16 in
Max. Panel Width =	2.000 ft.	Panel Shear through Thickness: F_vt_v =	165 lbs/in.
Max. Unit Shear @ Panels =	1,512.0 plf	Sheathing both sides :	YES
		Nail Spacing: S =	3 in.
		Fastener Type :	8d
		Fastener Lateral Design Value: Z =	73 lbs (TABLE 11Q, 2012 NDS)

<u>Moment Distribution:</u>

Max. Moments @ Panels =	417,312 in-lbs	
Applied Moment @ Top of Shear Panel =	280,097 in-lbs	67%
Applied Moment @ Bottom of Shear Panel =	137,215 in-lbs	33%

Header Depth =	12.00 in.	
Header Strap :	MSTC66	(Simpson)
Header Strap Allowable Tension =	5,860 lbs	⟶ OK
Header Strap both sides :	YES	
Holdowns :	HDU8	(Simpson)
Holdown Allowable Tension =	7,870 lbs	⟶ OK
Shear Panel Posts :	(3) 2x6	
Shear Panel Posts Thickness =	4.50 in.	
Load Duration Factor: C_D =	1.6	(Seismic/Wind)
Sill Plate:	(1)-2x	
Number of Anchor Bolts per Panel =	2	
Anchor Bolt DIA =	0.875 in	
AB Lateral Design Value: Z_{ab} =	1,280 lbs	(TABLE 11E, 2012 NDS)
Allowable Edgewise Bending Stress: F_{be} =	600 psi	(TABLE 4, APA W345)
Foundation Type =	8" Stemwall	

<u>Shear Capacity of Sheathing (Shear-through-Thickness):</u>

Adjusted Panel Shear through Thickness: $F_vt_v' = F_vt_v(C_D)(C_M)(C_t)(C_G)$ = 6,336 plf

6,336 plf > 1,512 plf ⟶ OK

<u>Shear Capacity of Panel-to-Framing Nails:</u>

Number of Nails (based on two rows at 3" o/c) = 18.0 nails

Adjusted Lateral Capacity of Nails: $Z' = Z(C_D)(N)$ = 4,205 lbs

4,205 lbs > 3,024 lbs ⟶ OK

<u>Shear Capacity of Anchor Bolts:</u>

Adjusted Lateral Capacity of Bolts: $Z_{ab}' = Z_{ab}(C_D)(N)$ = 4,096 lbs

4,096 lbs > 3,024 lbs ⟶ OK

<u>Moment Capacity of Sheathing (Edgewise Bending @ Header/Top of Panel):</u>

Section Modulus of Sheathing Panel in Bending: $S = bh^2/6$ = 42.00 $in.^3$

Adjusted Moment Capacity of Sheathing: $M_{wsp}' = F_{be}(C_D)(S)$ = 80,640 in-lbs

<u>Moment Capacity of Header Strap (Tension @ Header/Top of Panel):</u>

Adjusted Moment Capacity of Header Strap: $M_{strap}' = T_{strap}(W_p - 1.5)$ = 263,700 in-lbs

<u>Moment Capacity of Nails into Header:</u> (nail spacing @ 3 x 3 grid pattern)

Distance from center of rotation to furthest fastener (longest moment arm): r_{max} = 11.74 in. (assume nail edge distance of 0.75")

Polar Moment of Inertia of Nail Group: $J = I_x + I_y = bh^3/12 + hb^3/12$ = 20,473 $in.^4$

Adjusted Moment Capacity of Header Nail Group: $M_{header}' = ZC_DJ/S^2r_{max}$ = 45,265 in-lbs

Combined Moment Capacity at Top of Portal Frame Shear Panel = **308,965 in-lbs** > 280,097 in-lbs ⟶ OK

<u>Moment Capacity of Holdown:</u>

Distance from centerline of holdown to panel edge: D_{center} = 6 in.

Adjusted Moment Capacity of Holdown: $M_{holdown}' = T_{holdown}(W_p - D_{center})$ = 141,660 in-lbs > 128,424 in-lbs ⟶ OK

<u>Moment Capacity of Nails into Sill Plate:</u> (only nails into bottom sill plate considered, conservative)

Distance from center of rotation to furthest fastener (longest moment arm): r_{max} = 11.25 in. (assume nail edge distance of 0.75")

Polar Moment of Inertia of Nail Group: $J = I_x + I_y = bh^3/12 + hb^3/12$ = 4,203 $in.^4$

Adjusted Moment Capacity of Sill Plate Nail Group: $M_{sill}' = ZC_DJ/S^2r_{max}$ = 9,696 in-lbs

Combined Moment Capacity at Bottom of Portal Frame Shear Panel = **151,356 in-lbs** > 137,215 in-lbs ⟶ OK

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