The Structures (
Southwest, Inc.

Approved by City of Goodyear
Date: 07/01/21 5/14/2021
Sheet: 1 of 30
21-2119 - VIP PRODUCTS

VIP Products
Goodyear, AZ

PROJECT COVER SHEET

Registered Professional Engineer (Civil)
CERTIFICATE NO.
54537
WILLIAM T.
NANNI
5/14/2021
Date Signed
ARIZONA, U.S.A.

EXPIRES 06/30/2022

CITY CHANGES - Revised sheets 24, 25, and 26

1

WOOD ROOF LOADING (PSF):

Dead Load:

Roofing	2.5
Purlin	1.3
Stiffener (2x4 at 24")	1.1
Plywood (1/2" CDX)	1.5
Sprinkler	2.0
Insulation	0.5
Ceiling	1.0
Mechanical & Electrical	2.0
Miscellaneous	1.1
Additional Mechanical	2.0

Total Dead Load .... **15.0 PSF**
(beam wt. added in calc.)

Live Load (IBC) .... **20.0 PSF** (Reducible)

Load Duration Factor .... 1.25 (Construction Load)

Note: Minimum of 1/4" per lineal foot roof slope required for drainage.

3875 W. Adams Street   Phoenix, Arizona 85009   (602) 269-2458

The Structures Group

Roof:

Description	Span	depth	N	DL	LL	Joist Bay 1	Joist Bay 2	T.A.	Girder		Joist Bay 1 T.A.	Joist Bay 1 LL	Joist Bay 1 TL /ft.	Joist Bay 1 LL /ft.	Joist Bay 2 T.A.	Joist Bay 2 LL	Joist Bay 2 TL /ft.	Joist Bay 2 LL /ft.		Deck Span
Grid B	63	54	9	15	12	46.3	45	2877 FT	54G 9N 10.4K		370.6	16.59	253/	133	360	16.8	254/	134		8 FT
Grid B	54	54	7	15	12	45	45	2430 FT	54G 7N 10.3K		360	16.8	254/	134	360	16.8	254/	134		8 FT
Grid C and D	54	54	7	15	12	45	45	2430 FT	54G 7N 10.3K		360	16.8	254/	134	360	16.8	254/	134		8 FT
Grid C and D	63	54	9	15	12	45	45	2835 FT	54G 9N 10.3K		360	16.8	254/	134	360	16.8	254/	134		8 FT
Grid E	63	54	9	15	12	60	45	3308 FT	54G 9N 11.9K		480	14.4	235/	115	360	16.8	254/	134		8 FT
Grid E	54	54	7	15	12	60	45	2835 FT	54G 7N 11.9K		480	14.4	235/	115	360	16.8	254/	134		8 FT
Grid E	63	54	9	15	12	61.3	45	3347 FT	54G 9N 12.1K		490	14.2	234/	114	360	16.8	254/	134		8 FT
Joists																				
At Skew Wall				15		40	32				320	17.6	261/	141	256	18.88	271/	151		8 FT
At Skew Wall				15		24	16				192	20	280/	160	128	20	280/	160		8 FT

Wood Beam

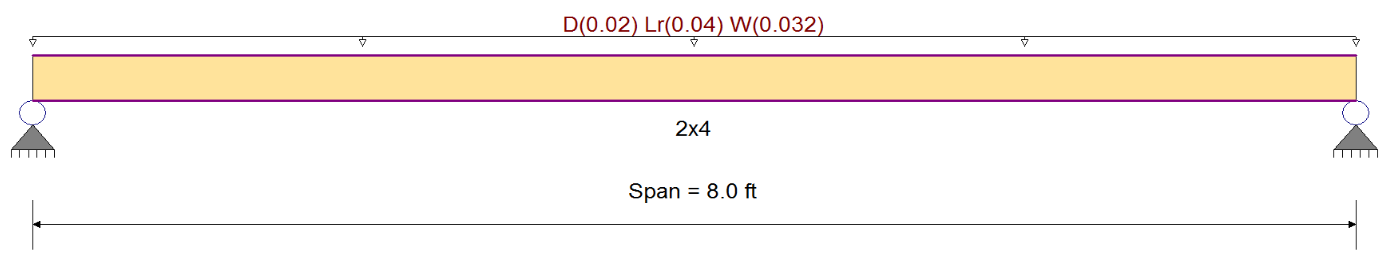
Description : Typical Subpurlin

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,500.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	1,500.0 psi	Ebend- xx	1,900.0 ksi
		Fc - Prll	1,700.0 psi	Eminbend - xx	690.0 ksi
Wood Species	: Douglas Fir - Larch	Fc - Perp	625.0 psi		
Wood Grade	: Select structural	Fv	180.0 psi		
		Ft	1,000.0 psi	Density	32.210 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling			Repetitive Member Stress Increase	



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.010, Lr = 0.020, W = 0.0160 ksf, Tributary Width = 2.0 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.593** : 1	Maximum Shear Stress Ratio	=	**0.290** : 1
Section used for this span		**2x4**	Section used for this span		**2x4**
fb : Actual	=	1,917.63 psi	fv : Actual	=	65.32 psi
FB : Allowable	=	3,234.38 psi	Fv : Allowable	=	225.00 psi
Load Combination		+D+Lr+H	Load Combination		+D+Lr+H
Location of maximum on span	=	4.000 ft	Location of maximum on span	=	7.737 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.364 in	Ratio =	263 >=180.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <180.0
Max Downward Total Deflection	0.597 in	Ratio =	160 >=100.
Max Upward Total Deflection	0.000 in	Ratio =	0 <100.0

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 = 8.0 ft	1	0.285	0.140	0.90	1.500	1.00	1.15	1.00	1.00	1.00	0.17	663.75	2328.75	0.08	22.61	162.00
+D+L+H					1.500	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.257	0.126	1.00	1.500	1.00	1.15	1.00	1.00	1.00	0.17	663.75	2587.50	0.08	22.61	180.00
+D+Lr+H					1.500	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.593	0.290	1.25	1.500	1.00	1.15	1.00	1.00	1.00	0.49	1,917.63	3234.38	0.23	65.32	225.00
+D+S+H					1.500	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.223	0.109	1.15	1.500	1.00	1.15	1.00	1.00	1.00	0.17	663.75	2975.63	0.08	22.61	207.00
+D+0.750Lr+0.750L+H					1.500	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.496	0.243	1.25	1.500	1.00	1.15	1.00	1.00	1.00	0.41	1,604.16	3234.38	0.19	54.64	225.00

The Structures Group Southwest, Inc
3875 E Adams Street
Phoenix, AZ 85009
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Wood Beam

Description : Typical Subpurlin

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+0.750L+0.750S+H					1.500	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.223	0.109	1.15	1.500	1.00	1.15	1.00	1.00	1.00	0.17	663.75	2975.63	0.08	22.61	207.00
+D+0.60W+H					1.500	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.306	0.150	1.60	1.500	1.00	1.15	1.00	1.00	1.00	0.32	1,265.61	4140.00	0.15	43.11	288.00
+D+0.70E+H					1.500	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.160	0.079	1.60	1.500	1.00	1.15	1.00	1.00	1.00	0.17	663.75	4140.00	0.08	22.61	288.00
+D+0.750Lr+0.750L+0.450W+H					1.500	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.497	0.243	1.60	1.500	1.00	1.15	1.00	1.00	1.00	0.52	2,055.55	4140.00	0.25	70.02	288.00
+D+0.750L+0.750S+0.450W+H					1.500	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.269	0.132	1.60	1.500	1.00	1.15	1.00	1.00	1.00	0.28	1,115.15	4140.00	0.13	37.99	288.00
+D+0.750L+0.750S+0.5250E+H					1.500	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.160	0.079	1.60	1.500	1.00	1.15	1.00	1.00	1.00	0.17	663.75	4140.00	0.08	22.61	288.00
+0.60D+0.60W+0.60H					1.500	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.242	0.118	1.60	1.500	1.00	1.15	1.00	1.00	1.00	0.26	1,000.11	4140.00	0.12	34.07	288.00
+0.60D+0.70E+0.60H					1.500	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.0 ft	1	0.096	0.047	1.60	1.500	1.00	1.15	1.00	1.00	1.00	0.10	398.25	4140.00	0.05	13.57	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750Lr+0.750L+0.450W+H	1	0.5970	4.029		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.262	0.262
Overall MINimum	0.128	0.128
+D+H	0.085	0.085
+D+L+H	0.085	0.085
+D+Lr+H	0.245	0.245
+D+S+H	0.085	0.085
+D+0.750Lr+0.750L+H	0.205	0.205
+D+0.750L+0.750S+H	0.085	0.085
+D+0.60W+H	0.161	0.161
+D+0.70E+H	0.085	0.085
+D+0.750Lr+0.750L+0.450W+H	0.262	0.262
+D+0.750L+0.750S+0.450W+H	0.142	0.142
+D+0.750L+0.750S+0.5250E+H	0.085	0.085
+0.60D+0.60W+0.60H	0.128	0.128
+0.60D+0.70E+0.60H	0.051	0.051
D Only	0.085	0.085
Lr Only	0.160	0.160
L Only		
S Only		
W Only	0.128	0.128
E Only		
H Only		

Wood Beam

Description : Sub Purlin w/ Conc load & 12.5lb light
300lbs = 48 psf = 100 plf across 2.5ft min.

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2018

Wood Species : Douglas Fir - Larch
Wood Grade : Select structural

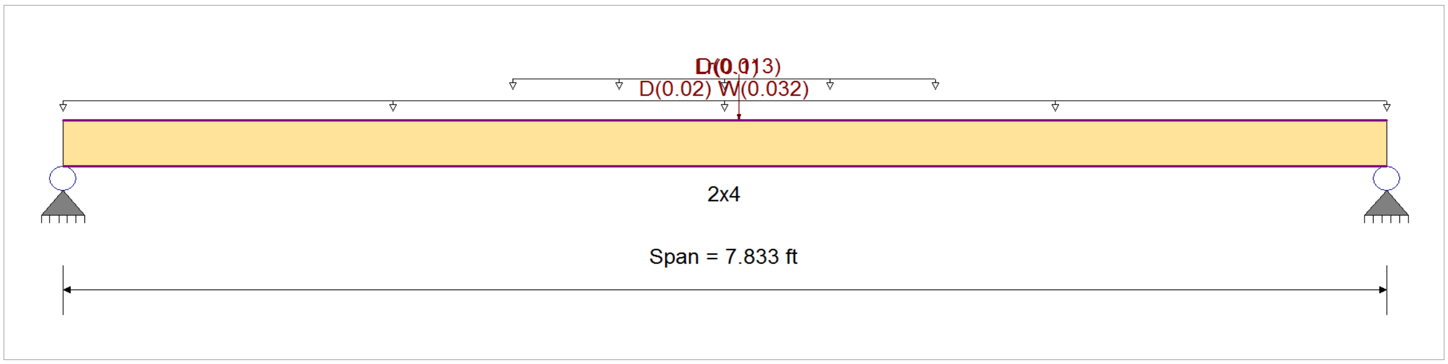
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	1,500.0	psi
Fb -	1,500.0	psi
Fc - Prll	1,700.0	psi
Fc - Perp	625.0	psi
Fv	180.0	psi
Ft	1,000.0	psi

E : Modulus of Elasticity

Ebend- xx	1,900.0	ksi
Eminbend - xx	690.0	ksi
Density	32.210	pcf

Repetitive Member Stress Increase



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.010, W = 0.0160 ksf, Tributary Width = 2.0 ft, (Roof)
Uniform Load : Lr = 0.10 k/ft, Extent = 2.667 -->> 5.167 ft, Tributary Width = 1.0 ft
Point Load : D = 0.0130 k @ 4.0 ft, (light)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.726** : 1	Maximum Shear Stress Ratio	=	**0.265** : 1
Section used for this span		**2x4**	Section used for this span		**2x4**
fb : Actual	=	2,346.90 psi	fv : Actual	=	59.58 psi
FB : Allowable	=	3,234.38 psi	Fv : Allowable	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	3.974 ft	Location of maximum on span	=	7.547 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.407 in	Ratio =	230 >=180.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <180.0
Max Downward Total Deflection	0.625 in	Ratio =	150 >=120.
Max Upward Total Deflection	0.000 in	Ratio =	0 <120.0

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 Only													0.00	0.00	0.00	0.00
Length = 7.833 ft	1	0.316	0.147	0.90	1.500	1.00	1.15	1.00	1.00	1.00	0.19	735.72	2328.75	0.08	23.86	162.00
+D+Lr					1.500	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.833 ft	1	0.726	0.265	1.25	1.500	1.00	1.15	1.00	1.00	1.00	0.60	2,346.90	3234.38	0.21	59.58	225.00
+D+0.750Lr					1.500	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.833 ft	1	0.601	0.225	1.25	1.500	1.00	1.15	1.00	1.00	1.00	0.50	1,944.00	3234.38	0.18	50.65	225.00
+D+0.60W					1.500	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.833 ft	1	0.317	0.152	1.60	1.500	1.00	1.15	1.00	1.00	1.00	0.33	1,312.48	4140.00	0.15	43.78	288.00

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Project Title: VIP Products
Engineer: BN
Project ID: 4018
Project Descr:

Printed: 21 MAR 2021, 12:38PM

Wood Beam

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Lic. # : KW-06002286 **Licensee : THE STRUCTURES GROUP**

Description : Sub Purlin w/ Conc load & 12.5lb light
300lbs = 48 psf = 100 plf across 2.5ft min.

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+0.750Lr+0.450W					1.500	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.833 ft	1	0.574	0.228	1.60	1.500	1.00	1.15	1.00	1.00	1.00	0.61	2,376.69	4140.00	0.23	65.59	288.00
+D+0.450W					1.500	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.833 ft	1	0.282	0.135	1.60	1.500	1.00	1.15	1.00	1.00	1.00	0.30	1,168.29	4140.00	0.14	38.80	288.00
+0.60D+0.60W					1.500	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.833 ft	1	0.246	0.119	1.60	1.500	1.00	1.15	1.00	1.00	1.00	0.26	1,018.19	4140.00	0.12	34.23	288.00
+0.60D					1.500	1.00	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.833 ft	1	0.107	0.050	1.60	1.500	1.00	1.15	1.00	1.00	1.00	0.11	441.43	4140.00	0.05	14.32	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750Lr+0.450W	1	0.6253	3.945		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.239	0.240
Overall MINimum	0.125	0.125
D Only	0.089	0.090
+D+Lr	0.214	0.215
+D+0.750Lr	0.183	0.183
+D+0.60W	0.164	0.165
+D+0.750Lr+0.450W	0.239	0.240
+D+0.450W	0.146	0.146
+0.60D+0.60W	0.129	0.129
+0.60D	0.054	0.054
Lr Only	0.125	0.125
W Only	0.125	0.125

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Wood Beam

Lic. # : KW-06002286 **Licensee : THE STRUCTURES GROUP**

Description : Roof wood beam

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2018

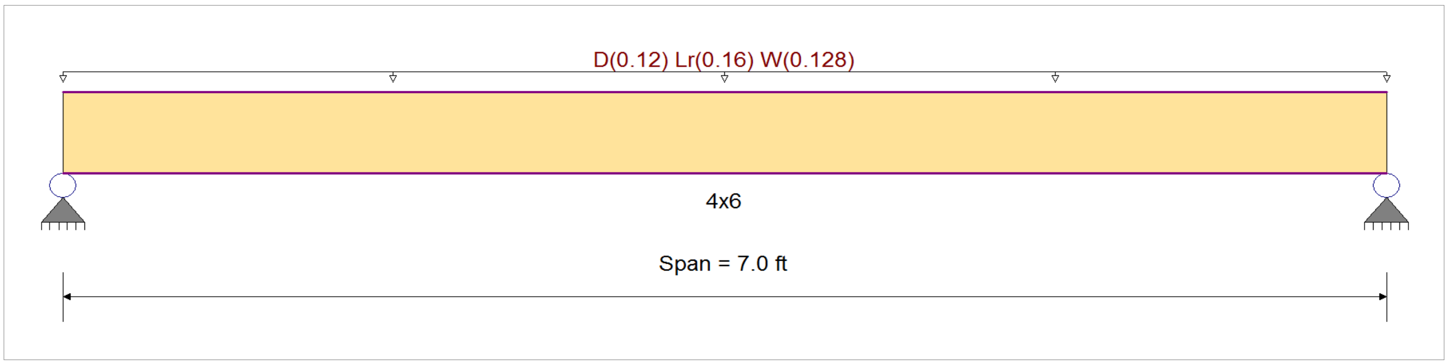
Wood Species : Douglas Fir-Larch
Wood Grade : No.2

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	900.0	psi
Fb -	900.0	psi
Fc - Prll	1,350.0	psi
Fc - Perp	625.0	psi
Fv	180.0	psi
Ft	575.0	psi

E : Modulus of Elasticity

Ebend- xx	1,600.0	ksi
Eminbend - xx	580.0	ksi
Density	31.210	pcf



Applied Loads

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

Uniform Load : D = 0.0150, Lr = 0.020, W = 0.0160 ksf, Tributary Width = 8.0 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.797**: 1	Maximum Shear Stress Ratio	=	**0.297** : 1
Section used for this span		**4x6**	Section used for this span		**4x6**
fb : Actual	=	1,166.28 psi	fv : Actual	=	66.89 psi
FB : Allowable	=	1,462.50 psi	Fv : Allowable	=	225.00 psi
Load Combination		+D+Lr+H	Load Combination		+D+Lr+H
Location of maximum on span	=	3.500 ft	Location of maximum on span	=	6.566 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.112 in	Ratio =	750 >=240
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240
Max Downward Total Deflection	0.208 in	Ratio =	403 >=120
Max Upward Total Deflection	0.000 in	Ratio =	0 <120

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 = 7.0 ft	1	0.475	0.177	0.90	1.300	1.00	1.00	1.00	1.00	1.00	0.74	499.83	1053.00	0.37	28.67	162.00
+D+L+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.427	0.159	1.00	1.300	1.00	1.00	1.00	1.00	1.00	0.74	499.83	1170.00	0.37	28.67	180.00
+D+Lr+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.797	0.297	1.25	1.300	1.00	1.00	1.00	1.00	1.00	1.72	1,166.28	1462.50	0.86	66.89	225.00
+D+S+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.371	0.138	1.15	1.300	1.00	1.00	1.00	1.00	1.00	0.74	499.83	1345.50	0.37	28.67	207.00
+D+0.750Lr+0.750L+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.684	0.255	1.25	1.300	1.00	1.00	1.00	1.00	1.00	1.47	999.67	1462.50	0.74	57.33	225.00
+D+0.750L+0.750S+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.371	0.138	1.15	1.300	1.00	1.00	1.00	1.00	1.00	0.74	499.83	1345.50	0.37	28.67	207.00

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Wood Beam

Description : Roof wood beam

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+0.60W+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.438	0.163	1.60	1.300	1.00	1.00	1.00	1.00	1.00	1.21	819.73	1872.00	0.60	47.01	288.00
+D+0.70E+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.267	0.100	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.74	499.83	1872.00	0.37	28.67	288.00
+D+0.750Lr+0.750L+0.450W+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.662	0.247	1.60	1.300	1.00	1.00	1.00	1.00	1.00	1.82	1,239.59	1872.00	0.91	71.09	288.00
+D+0.750L+0.750S+0.450W+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.395	0.147	1.60	1.300	1.00	1.00	1.00	1.00	1.00	1.09	739.76	1872.00	0.54	42.43	288.00
+D+0.750L+0.750S+0.5250E+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.267	0.100	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.74	499.83	1872.00	0.37	28.67	288.00
+0.60D+0.60W+0.60H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.331	0.123	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.91	619.80	1872.00	0.46	35.55	288.00
+0.60D+0.70E+0.60H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.0 ft	1	0.160	0.060	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.44	299.90	1872.00	0.22	17.20	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750Lr+0.750L+0.450W+H	1	0.2083	3.526		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.042	1.042
Overall MINimum	0.448	0.448
+D+H	0.420	0.420
+D+L+H	0.420	0.420
+D+Lr+H	0.980	0.980
+D+S+H	0.420	0.420
+D+0.750Lr+0.750L+H	0.840	0.840
+D+0.750L+0.750S+H	0.420	0.420
+D+0.60W+H	0.689	0.689
+D+0.70E+H	0.420	0.420
+D+0.750Lr+0.750L+0.450W+H	1.042	1.042
+D+0.750L+0.750S+0.450W+H	0.622	0.622
+D+0.750L+0.750S+0.5250E+H	0.420	0.420
+0.60D+0.60W+0.60H	0.521	0.521
+0.60D+0.70E+0.60H	0.252	0.252
D Only	0.420	0.420
Lr Only	0.560	0.560
L Only		
S Only		
W Only	0.448	0.448
E Only		
H Only		

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Project Title: VIP Products
Engineer: BN
Project ID: 4018
Project Descr:

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Wood Beam

Lic. # : KW-06002286 Licensee : THE STRUCTURES GROUP

Description : AC 800 lbs across 7'

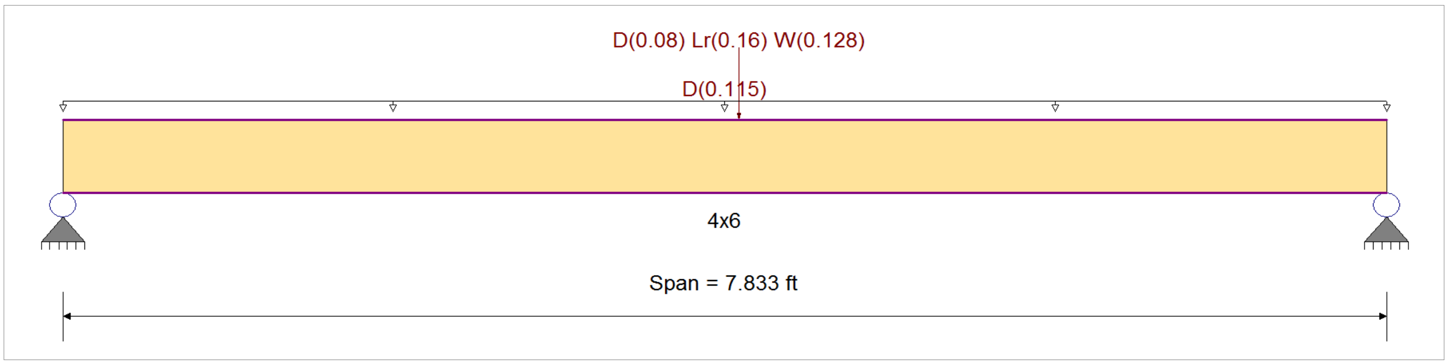
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2018	Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Wood Species	: Douglas Fir - Larch	Fc - Perp	625.0 psi		
Wood Grade	: No.2	Fv	180.0 psi		
		Ft	575.0 psi	Density	32.210 pcf

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.1150 k/ft, Extent = 0.0 -->> 7.833 ft, Tributary Width = 1.0 ft, (MU)
Point Load : D = 0.080, Lr = 0.160, W = 0.1280 k @ 4.0 ft, (header)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.670: 1**	Maximum Shear Stress Ratio	=	**0.211 : 1**
Section used for this span		**4x6**	Section used for this span		**4x6**
fb : Actual	=	705.93 psi	fv : Actual	=	34.18 psi
FB : Allowable	=	1,053.00 psi	Fv : Allowable	=	162.00 psi
Load Combination		D Only	Load Combination		D Only
Location of maximum on span	=	4.002 ft	Location of maximum on span	=	7.376 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.036 in	Ratio =	2623 >=180.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <180.0
Max Downward Total Deflection	0.184 in	Ratio =	511 >=100.
Max Upward Total Deflection	0.000 in	Ratio =	0 <100.0

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 Only													0.00	0.00	0.00	0.00
Length = 7.833 ft	1	0.670	0.211	0.90	1.300	1.00	1.00	1.00	1.00	1.00	1.04	705.93	1053.00	0.44	34.18	162.00
+D+Lr					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.833 ft	1	0.628	0.180	1.25	1.300	1.00	1.00	1.00	1.00	1.00	1.35	918.78	1462.50	0.52	40.55	225.00
+D+0.750Lr					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.833 ft	1	0.592	0.173	1.25	1.300	1.00	1.00	1.00	1.00	1.00	1.27	865.57	1462.50	0.50	38.96	225.00
+D+0.60W					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.833 ft	1	0.432	0.129	1.60	1.300	1.00	1.00	1.00	1.00	1.00	1.19	808.10	1872.00	0.48	37.24	288.00
+D+0.750Lr+0.450W					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

Description : AC 800 lbs across 7'

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
Length = 7.833 ft	1	0.503	0.143	1.60	1.300	1.00	1.00	1.00	1.00	1.00	1.39	942.20	1872.00	0.53	41.25	288.00
+D+0.450W					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.833 ft	1	0.418	0.127	1.60	1.300	1.00	1.00	1.00	1.00	1.00	1.15	782.56	1872.00	0.47	36.47	288.00
+0.60D+0.60W					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.833 ft	1	0.281	0.082	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.77	525.73	1872.00	0.30	23.56	288.00
+0.60D					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.833 ft	1	0.226	0.071	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.62	423.56	1872.00	0.26	20.51	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750Lr+0.450W	1	0.1839	3.945		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.576	0.582
Overall MINimum	0.063	0.065
D Only	0.490	0.491
+D+Lr	0.568	0.573
+D+0.750Lr	0.548	0.553
+D+0.60W	0.527	0.530
+D+0.750Lr+0.450W	0.576	0.582
+D+0.450W	0.518	0.521
+0.60D+0.60W	0.331	0.334
+0.60D	0.294	0.295
Lr Only	0.078	0.082
W Only	0.063	0.065

Wood Beam

Lic. # : KW-06002286 **Licensee : THE STRUCTURES GROUP**

Description : AC 2274 lbs across 8'

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2018

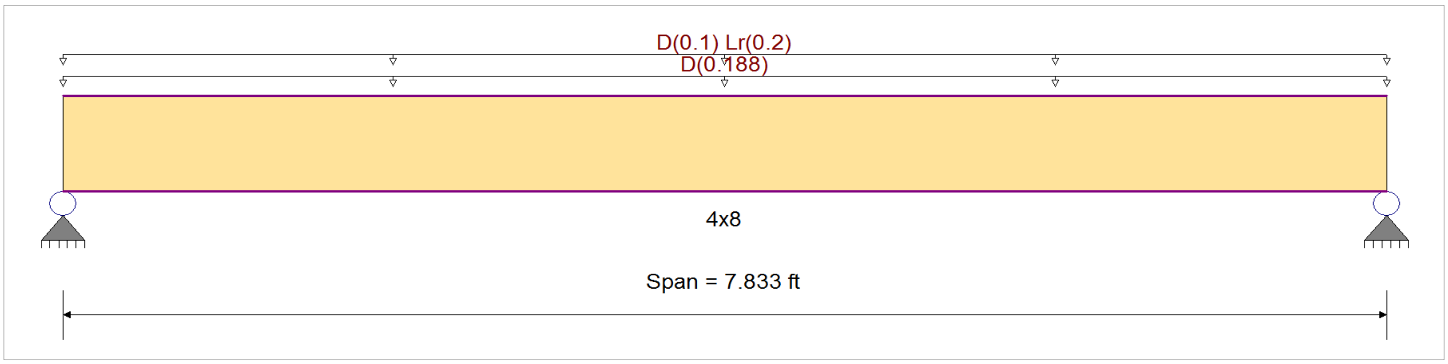
Wood Species : Douglas Fir-Larch (North)
Wood Grade : No. 1 & Btr

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	1,150.0	psi
Fb -	1,150.0	psi
Fc - Prll	1,800.0	psi
Fc - Perp	625.0	psi
Fv	180.0	psi
Ft	750.0	psi

E : Modulus of Elasticity

Ebend- xx	1,800.0	ksi
Eminbend - xx	660.0	ksi
Density	30.590	pcf



Applied Loads

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

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.1880 k/ft, Extent = 0.0 -->> 7.833 ft, Tributary Width = 1.0 ft, (MU)
Uniform Load : D = 0.10, Lr = 0.20 , Tributary Width = 1.0 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.792** : 1	Maximum Shear Stress Ratio	=	**0.430** : 1
Section used for this span		**4x8**	Section used for this span		**4x8**
fb : Actual	=	1,480.97 psi	fv : Actual	=	96.72 psi
FB : Allowable	=	1,868.75 psi	Fv : Allowable	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	3.917 ft	Location of maximum on span	=	7.233 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.085 in	Ratio =	1103 >=180.
Max Upward Transient Deflection	0.000 in	Ratio =	0 <180.0
Max Downward Total Deflection	0.210 in	Ratio =	447 >=100.
Max Upward Total Deflection	0.000 in	Ratio =	0 <100.0

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 Only													0.00	0.00	0.00	0.00
Length = 7.833 ft	1	0.655	0.355	0.90	1.300	1.00	1.00	1.00	1.00	1.00	2.25	880.64	1345.50	0.97	57.51	162.00
+D+Lr					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.833 ft	1	0.792	0.430	1.25	1.300	1.00	1.00	1.00	1.00	1.00	3.78	1,480.97	1868.75	1.64	96.72	225.00
+D+0.750Lr					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.833 ft	1	0.712	0.386	1.25	1.300	1.00	1.00	1.00	1.00	1.00	3.40	1,330.89	1868.75	1.47	86.92	225.00
+0.60D					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 7.833 ft	1	0.221	0.120	1.60	1.300	1.00	1.00	1.00	1.00	1.00	1.35	528.39	2392.00	0.58	34.51	288.00

The Structures Group Southwest, Inc
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Phoenix, AZ 85009
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602-269-2458

Wood Beam

Description : AC 2274 lbs across 8'

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.2101	3.945		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.932	1.932
Overall MINimum	0.783	0.783
D Only	1.149	1.149
+D+Lr	1.932	1.932
+D+0.750Lr	1.737	1.737
+0.60D	0.689	0.689
Lr Only	0.783	0.783

ATC Hazards by Location

Search Information

Coordinates: 33.45555356697871, -112.35344237354126

Elevation: 985 ft

Timestamp: 2021-03-05T00:45:32.397Z

Hazard Type: Wind

ASCE 7-16

MRI 10-Year	71 mph
MRI 25-Year	77 mph
MRI 50-Year	82 mph
MRI 100-Year	87 mph
Risk Category I	95 mph
Risk Category II	101 mph
Risk Category III	108 mph
Risk Category IV	112 mph

ASCE 7-10

MRI 10-Year	76 mph
MRI 25-Year	84 mph
MRI 50-Year	90 mph
MRI 100-Year	96 mph
Risk Category I	105 mph
Risk Category II	115 mph
Risk Category III-IV	120 mph

ASCE 7-05

ASCE 7-05 Wind Speed	90 mph

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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Wind Loads:

ASCE 7-16 Chapter 28, Part 1:

MWFRS

Given: Roof Slope < 5 deg.

		Long Face		Trans Face	
T.O.P=	Front	41	ft	41	ft
	Back	41	ft	41	ft
T.O.D=	Front	33 7/12	ft	35 5/12	ft
	Back	34 10/12	ft	35 5/12	ft

Long Face 344 ft

Trans Face 195 ft

Roof Area= 67080 sf

Step 1	Risk Category	II				Table 1.5-1
Step 2	Basic Wind Speed	102 mph				Fig 26.5-1B
Step 3						
	Wind Directionality Factor Kd		=	0.85	(Buildings)	26.6
	Exposure	C				26.7
	Topo Factor Kzt	1		Flat		26.8
	Ground Elevavation Factor Ke	0.96				Table 26.9-1
	Enclosure Classification	Partially Open				26.12
	Internal Pressure Coeff Gcpi=		=	0.18		Table 26.13-1
Step 4	Kh= 2.01(z/zg)^2/a=		=	1.006	Long Face	Table 28.2-1
			=	1.017	Trans Face	
Step 5	qh= $.00256K_zK_{zt}K_dK_eV^2$		=	21.86 psf	Long Face	(28.3-1)
			=	22.11 psf	Trans Face	

Step 6 Note for low slope roofs GCpf factors are the same for load case A (Surface 1, 4, 1E, 4E) and load case B (5, 6, 5E, 6E).

Table 28.3-1

Surface	1	4	1E	4E
GCpf	0.4	-0.29	0.61	-0.43

a= 33 7/12 x 0.4 Or 195 x 0.1 = 13.444 ft

Step 7 p= qh[(GCpf)-(Gcpi)] (28.3-1)

=	**15.09** psf	Long Face	Int zone
=	**22.74** psf	Long Face	End zone
=	**15.26** psf	Trans Face	Int zone
=	**22.99** psf	Trans Face	End zone

Parapets

28.3.2

Pp= (qp)(GCpn) (28.3-2)

Kz= 2.01(z/zg)^2/a=	Front	=	1.049	Long Face	Table 28.2-1
	Back	=	1.049	Long Face	
	Front	=	1.049	Trans Face	
	Back	=	1.049	Trans Face	
qp= $.00256K_zK_{zt}K_dK_eV^2$	Front	=	22.8 psf	Long Face	(28.3-1)
	Back	=	22.8 psf	Long Face	
	Front	=	22.8 psf	Trans Face	
	Back	=	22.8 psf	Trans Face	
GCpn=		=	1.5	Windward	28.3.2
		=	-1	Leeward	
Pp= (qp)(GCpn)	Front	=	**34.20** psf	Long Face	(28.3-2)
	Back	=	**-22.8** psf	Long Face	
	Front	=	**34.2** psf	Trans Face	
	Back	=	**-22.8** psf	Trans Face	

Diaphragm Loads: Sum moments about ground/slab

W=(	8.74586 x		33.61 /2		) +		Front	Below TOD
(	6.34075 x		34.81 /2		) +		Back	Below TOD
[	34.20 x	(	41 -		33.61) x		Front	Parapet
((	7.39 /		2) +	33.61)] /	33.61]+			
[	-22.80 x	(	41 -		34.81) x		Back	Parapet
((	6.19 /		2) +	34.81)] /	34.81]			
				=		692 plf	Long Face	Int zone
Similarly								
W=				=		822 plf	Long Face	End zone
W=				=		612 plf	Trans Fac	Int zone
W=				=		749 plf	Trans Fac	End zone

Allowable Stress Design loads are 60% of Strength Design loads 2.4.1

W=	691.52	x	0.60	=	414.9 plf	Long Face	Int zone
W=	821.96	x	0.60	=	493.2 plf	Long Face	End zone
W=	612.10	x	0.60	=	367.3 plf	Trans Fac	Int zone
W=	749.23	x	0.60	=	449.5 plf	Trans Fac	End zone

ASCE 7-16 Chapter 30.3, Part 1:
Components and Cladding

Note: Steps 1-5 are unchanged from the MWFRS:

Step 6 GCp based on Trib Area of component. Note Wall-tie forces are due to leeward wind onl Fig 30.3-1
Therefore, only negative GCp values are needed from Figure 30.3-1, except for parapet loads
Per Figure 30.3-1 note 5, GCp values reduced 10% for walls when Ø ≤ 10°.
See tables below for trib area and GCp Interpolations

Step 7 p= qh[(GCp)-(Gcpi)] (30.3-1)
See tables below based on trib area and incorporating parapte loads.

Parapets: 30.8

p= qp[(GCp)-(Gcpi)] (30.9-1)

qp= $.00256K_zK_{zt}K_dV^2$ Same as MWFRS (30.3-1)

Front	=	22.8	psf	Long Face
Back	=	22.8	psf	Long Face
Front	=	22.8	psf	Trans Face
Back	=	22.8	psf	Trans Face

Gcpi= Solid parapets = 0

Note: Tables below show Fig 30.9-1, Load Case B only as wall-ties need only consider leeward wind.

First row = load from ground to roof deck
Second row = load from parapet (includes pos load on back of parapet)

Purlin Anchors:

Trib Area= 8 x 41.00 ^2 /2 / 33.61 = 200.06 sf

GCp Interpolation				Zone	GCp	p(psf)	w (plf)	Front	Long Face
-0.792	-0.72	200.00	500.00	4	-0.792	-21.25	**-357**		
0.702	0.63	200.00	500.00	4	0.70199	-34.06	**-279**		
								-636.52 plf	Strength
				x	0.6	=		-381.91 plf	ASD

Trib Area= 8 x 41.00 ^2 /2 / 33.61 = 200.06 sf

GCp Interpolation				Zone	GCp	p(psf)	w (plf)	Back	Long Face
-0.792	-0.72	200.00	500.00	4	-0.792	-21.25	**-357**		
0.702	0.63	200.00	500.00	4	0.70199	-34.06	**-279**		
								-636.52 plf	Strength
				x	0.6	=		-381.91 plf	ASD

Trib Area= 8 x 41.00 ^2 /2 / 33.61 = 200.06 sf

GCp Interpolation				Zone	GCp	p(psf)	w (plf)	Front	Long Face
-0.855	-0.72	200.00	500.00	5	-0.855	-22.63	**-380**		
0.702	0.63	200.00	500.00	5	0.70199	-35.50	**-291**		
								-671.45 plf	Strength
				x	0.6	=		-402.87 plf	ASD

Trib Area= 8 x 41.00 ^2 /2 / 33.61 = 200.06 sf

GCp Interpolation				Zone	GCp	p(psf)	w (plf)	Back	Long Face
-0.855	-0.72	200.00	500.00	5	-0.855	-22.63	**-380**		
0.702	0.63	200.00	500.00	5	0.70199	-35.50	**-291**		
								-671.45 plf	Strength
				x	0.6	=		-402.87 plf	ASD

Sub-Purlin Anchors:

	Trib Area=	4	x	41.00 ^2 /2 /		35.44 =		94.86 sf	
	GCp Interpolation			Zone	GCp	p(psf)	w (plf)	Front	Trans Face
-0.882	-0.828	50.00	100.00	4	-0.8335	-22.41	**-397**		
0.792	0.738	50.00	100.00	4	0.74355	-35.96	**-216**		
								-612.69 plf	Strength
					x	0.6	=	-367.62 plf	ASD

	Trib Area=	4	x	41.00 ^2 /2 /		35.44 =		94.86 sf	
	GCp Interpolation			Zone	GCp	p(psf)	w (plf)	Back	Trans Face
-0.882	-0.828	50.00	100.00	4	-0.8335	-22.41	**-397**		
0.792	0.738	50.00	100.00	4	0.74355	-35.96	**-216**		
								-612.69 plf	Strength
					x	0.6	=	-367.62 plf	ASD

	Trib Area=	4	x	41.00 ^2 /2 /		35.44 =		94.86 sf	
	GCp Interpolation			Zone	GCp	p(psf)	w (plf)	Front	Trans Face
-1.035	-0.945	50.00	100.00	5	-0.9542	-25.08	**-444**		
0.792	0.738	50.00	100.00	5	0.74355	-38.71	**-232**		
								-676.48 plf	Strength
					x	0.6	=	-405.89 plf	ASD

	Trib Area=	4	x	41.00 ^2 /2 /		35.44 =		94.86 sf	
	GCp Interpolation			Zone	GCp	p(psf)	w (plf)	Back	Trans Face
-1.035	-0.945	50.00	100.00	5	-0.9542	-25.08	**-444**		
0.792	0.738	50.00	100.00	5	0.74355	-38.71	**-232**		
								-676.48 plf	Strength
					x	0.6	=	-405.89 plf	ASD

Sub-diaphragm:

	1 Trib Area=	60	x	41.00 ^2 /2 /		35.44 =		1422.97 sf	
	GCp Interpolation			Zone	GCp	p(psf)	w (plf)	Front	Trans Face
-0.72	-0.72	500.00	500.00	4	-0.72	-19.90	**-353**		
0.63	0.63	500.00	500.00	4	0.63	-30.78	**-185**		
								-537.16 plf	
-0.72	-0.72	500.00	500.00	5	-0.72	-19.90	**-353**		
0.63	0.63	500.00	500.00	5	0.63	-30.78	**-185**		
								-537.16 plf	Strength
					x	0.6	=	-322.3 plf	ASD

	2 Trib Area=	60	x	41.00 ^2 /2 /		35.44 =		1422.97 sf	
	GCp Interpolation			Zone	GCp	p(psf)	w (plf)	Front	Trans Face
-0.72	-0.72	500.00	500.00	4	-0.72	-19.90	**-353**		
0.63	0.63	500.00	500.00	4	0.63	-30.78	**-185**		
								-537.16 plf	
-0.72	-0.72	500.00	500.00	5	-0.72	-19.90	**-353**		
0.63	0.63	500.00	500.00	5	0.63	-30.78	**-185**		
								-537.16 plf	Strength
					x	0.6	=	-322.3 plf	ASD

3 Trib Area=		45	x	41.00 ^2 /2 /		35.44 =		1067.23 sf		
	GCp Interpolation			Zone	GCp	p(psf)	w (plf)		Front	Trans Face
-0.72	-0.72	500.00	500.00	4	-0.72	-19.90	**-353**			
0.63	0.63	500.00	500.00	4	0.63	-30.78	**-185**			
								-537.16 plf		
-0.72	-0.72	500.00	500.00	5	-0.72	-19.90	**-353**			
0.63	0.63	500.00	500.00	5	0.63	-30.78	**-185**			
								-537.16 plf		Strength
					x	0.6	=	-322.3 plf		ASD

4 Trib Area=		45	x	41.00 ^2 /2 /		35.44 =		1067.23 sf		
	GCp Interpolation			Zone	GCp	p(psf)	w (plf)		Back	Trans Face
-0.72	-0.72	500.00	500.00	4	-0.72	-19.90	**-353**			
0.63	0.63	500.00	500.00	4	0.63	-30.78	**-185**			
								-537.16 plf		
-0.72	-0.72	500.00	500.00	5	-0.72	-19.90	**-353**			
0.63	0.63	500.00	500.00	5	0.63	-30.78	**-185**			
								-537.16 plf		Strength
					x	0.6	=	-322.3 plf		ASD

Girder Axial:

0 Trib Area=		52.5	x	41.00 ^2 /2 /		33.61 =		1312.89 sf		
	GCp Interpolation			Zone	GCp	p(psf)	w (plf)		Front	Long Face
-0.72	-0.72	500.00	500.00	4	-0.72	-19.68	**-331**			
0.63	0.63	500.00	500.00	4	0.63	-30.78	**-252**			
								-583.15 plf		
-0.72	-0.72	500.00	500.00	5	-0.72	-19.68	**-331**			
0.63	0.63	500.00	500.00	5	0.63	-30.78	**-252**			
								-583.15 plf		Strength
					x	0.6	=	-349.89 plf		ASD

Uplift:

Zone 2 Dist=	21	ft
Zone 1 Dist=	21	ft

Purlins:

Trib Area= 8.00 x 47.00 = 376.00 sf

GCp Interpolation				Zone	GCp	Pu(psf)	0.6Pu	0.6DL	Pnet(psf)
-0.75	-0.55	200.00	500	1'	-0.6327	-17.97	-10.8	7.08	-3.70
-1.19	-1	200.00	500	1	-1.0785	-27.83	-16.7	7.08	-9.62
-1.6	-1.4	200.00	500	2	-1.4827	-36.76	-22.1	7.08	-14.98
-1.8	-1.4	200.00	500	3	-1.5653	-38.59	-23.2	7.08	-16.07

Girders:

Trib Area= 52.50 x 56.00 = 2940.00 sf

Use MWFRS pressures

p= qh[(GCpf)-(Gcpi)] (28.3-1)

Surface	qh	GCpf	Gcpi	Pu(psf)	0.6Pu	0.6DL	Pnet(psf)
2	22.11	-0.69	0.18	-19.24	-11.5	7.08	-4.46
2E	22.11	-1.07	0.18	-27.64	-16.6	7.08	-9.50

Uplift on Stiffeners

Nails used - 10d common 0.148 dia w 1 1/2" penetration

One nail - 36#/in for withdrawl
36#/in * 1.5in = 54 #

Worst Case Uplift on Plywood Sheet

16.07 psf * 32ft^2 /2 = 281 # at stiffener

Nine nails required acreoss 4ft length of plywood
281 # / 9 nails = 31 # each nail
Less than 86# - Good

ATC Hazards by Location

Search Information

Coordinates: 33.45555356697871, -112.35344237354126

Elevation: 985 ft

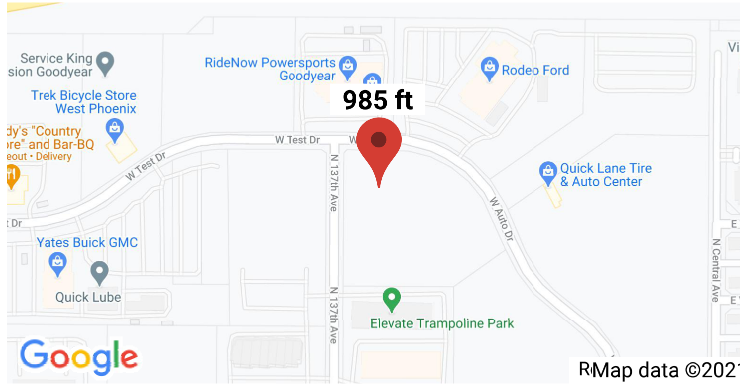
Timestamp: 2021-03-05T00:47:24.038Z

Hazard Type: Seismic

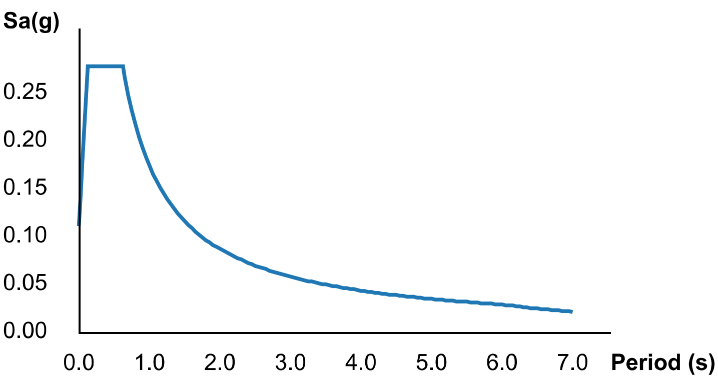
Reference Document: ASCE7-16

Risk Category: II

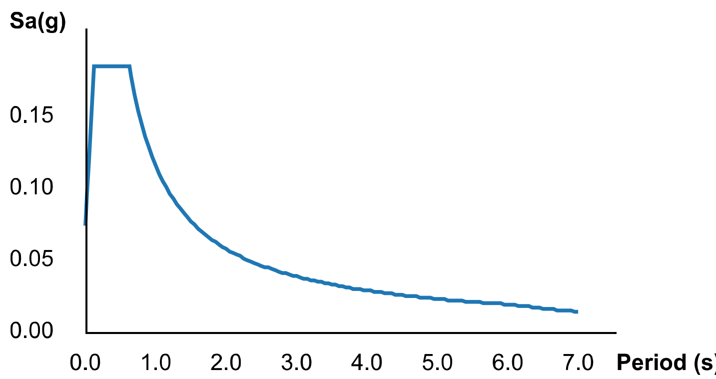
Site Class: D



MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	0.174	MCE_R ground motion (period=0.2s)
S_1	0.072	MCE_R ground motion (period=1.0s)
S_{MS}	0.278	Site-modified spectral acceleration value
S_{M1}	0.174	Site-modified spectral acceleration value
S_{DS}	0.185	Numeric seismic design value at 0.2s SA
S_{D1}	0.116	Numeric seismic design value at 1.0s SA

▾Additional Information

Name	Value	Description
SDC	B	Seismic design category
F_a	1.6	Site amplification factor at 0.2s
F_v	2.4	Site amplification factor at 1.0s
CR_S	0.931	Coefficient of risk (0.2s)

Seismic Loads:

Overall Diaphragm loads

T.O.P.=	41	ft	Ss=	0.174		
T.O.D.=	33 7/12	ft	S1=	0.072		
Long =	344	ft	Sms=	0.278		(11.4-1)
Trans =	195	ft	Sm1=	0.174		(11.4-2)
hn=	33 7/12	ft	R=	3		
Wwall=	112.5	psf	I=	1		
Wroof=	15	psf	SDC=	B		

	Sds=	0.67	x	0.278	=	0.185333			(11.4-3)
	Sd1=	0.67	x	0.174	=	0.116			(11.4-4)
	Cs=	Sds	/	R/I	=	0.061778			(12.8-2)
	Ta=	0.02	x	hn^3/4	=	0.28			(12.8-7)
	Max Cs=	Sd1	/	(R/I)T	=	0.14			(12.8-3)
	Min Cs=	0.044	x	Sds I	=	0.01			(12.8-5)
	W=	112.5	x	41	^2 /2 /	33.61	x 2 +		
		15	x	399	=	11611.67	plf	Trans Face	
	W=	112.5	x	41	^2 /2 /	33 7/12	x 2 +		
		15	x	240	=	9226.674	plf	Long Face	
	V=	Cs	x	W	=	717.34	plf	Trans Face	(12.8-1)
	V=	Cs	x	W	=	570.00	plf	Long Face	
Strength	V=	0.7	x	E	=	502.14	plf	**Trans Face**	
	V=	0.7	x	E	=	399.00	plf	**Long Face**	

Wall-Tie loads

	Wwall=	41	^2	/2 /	33.6			
			x	112.5	=	2813.337	plf	
	Fp=	0.4	x	Sds(Ka)I(W)	=	417.12407	plf	(12.11-1)
	Min Fp=	0.2	x	KaIWp	=	1125.3347	plf	(12.11-1)
	Ka=	1	+	Lf/100	=	2	Max	(12.11-2)
Strength	Fp=	0.7	x	1125.33	=	787.7343	plf	

Wall Ties:

Seismic: V= **788** plf (See previous sheet)

Wind: V= **Varies** (See previous sheet)

CMU/Conc: Concrete

Nails: 10d x 2 1/8"

V=	119	x	1.48	/	1.48	x	1.6	=	**190.40** #/nail	(1 1/2" embed)
V=	119	x	1.48	/	1.48	x	1.6	=	**190.40** #/nail	(1 1/2" embed)

Purlin Tie:

Front	Long Face	P=	8 x 788	=	6302	pounds	Seismic
Interior			Use 3/16" x 2.26 Fillet				
Front	Long Face	P=	8 x 788	=	6302	pounds	Seismic
Corner			Use 3/16" x 2.26 Fillet				
a=	13.444 ft						
Back	Long Face	P=	8 x 788	=	6302	pounds	Seismic
Interior			Use 3/16" x 2.26 Fillet				
Back	Long Face	P=	8 x 788	=	6302	pounds	Seismic
Corner			Use 3/16" x 2.26 Fillet				
a=	13.444 ft						

Stiffener Tie:

Front	Trans Face	P=	4 x 788	=	3151	pounds	Seismic
Interior		N=	3151 / 3220 x 16	=	15.66	Nails	
			Use PA23 w/ 16- 10 d x 2 1/8 Nails at 4 ft o.c.				
Front	Trans Face	P=	4 x 788	=	3151	pounds	Seismic
Corner		N=	3151 / 3220 x 16	=	15.66	Nails	
a=	13.444 ft		Use PA23 w/ 16- 10 d x 2 1/8 Nails at 4 ft o.c.				
Back	Trans Face	P=	4 x 788	=	3151	pounds	Seismic
Interior		N=	3151 / 3220 x 16	=	15.66	Nails	
			Use PA23 w/ 16- 10 d x 2 1/8 Nails at 4 ft o.c.				
Back	Trans Face	P=	4 x 788	=	3151	pounds	Seismic
Corner		N=	3151 / 3220 x 16	=	15.66	Nails	
a=	13.444 ft		Use PA23 w/ 16- 10 d x 2 1/8 Nails at 4 ft o.c.				

Cross-Tie at Purlin

T= 788 x 8 = 6302

Use 3/16" x 2.3 Fillet Transfer thru Girder

Sub-Diaphragms

1

Front	Trans Face

L=	40									
W=	60									
V=	788	x	60	/	2	/	40 =		590.80 plf	Seismic
T=	788	/	40	x (	40	-	8)			
		x	4				=		2520.75 lbs	

Use ST6224 Across 1st Purlin

Note: See below for additional strap information

2

Front	Trans Face

L=	40									
W=	60									
V=	788	x	60	/	2	/	40 =		590.80 plf	Seismic
T=	788	/	40	x (	32	-	8)			
		x	4				=		1890.56 lbs	

Use MSTA30 Across 2nd - 4th Purlins

Note: 1/2 spacing across 4th purlin

3

Front	Trans Face

L=	32									
W=	45									
V=	788	x	45	/	2	/	32 =		553.88 plf	Seismic
T=	788	/	32	x (	32	-	8)			
		x	4				=		2363.20 lbs	

Use ST6224 Across 1st Purlin

Note: See below for additional strap information

4

Back	Trans Face

L=	32									
W=	45									
V=	788	x	45	/	2	/	32 =		553.88 plf	Seismic
T=	788	/	32	x (	24	-	8)			
		x	4				=		1575.47 lbs	

Use LSTA30 Across 2nd and 3rd Purlins

Note: 1/2 spacing across 3rd purlin

Girder Axial Load

Front	Long Face

Bay W=	52.5	FT						
T=	788	x	52.5	/	1000	=	41.36 K	Seismic

Design Girder for 41.4 K Axial Load

Overall Diaphragm East End

Loads perp to
Long Face

1

Seismic: V= **399** PLF Long Face

Diaphragm Shear:

v= 399 x 396 / 2 / 195 = **405.14** plf

					Nailing Pattern			
	from exterior wall				B.N.	E.N.	Field	
Distance to v=	650 =		ft.	3x	2 1/2	4	12	D
	575 =	0	ft.	2x	2 1/2	4	12	C
	385 =	10	ft.	2x	4	6	12	B
	290 =	56	ft.	2x	6	6	12	A

C-D

Use 15/32 sheathing, see above for grade and nail spacing
All nails shall be 10d x 2 1/8" (1 5/8" penetration)

Wind: End V= **493** plf (See previous sheet) a= 13.444 ft
Interior V= **415** plf (See previous sheet)

Diaphragm Shear:

v= 415 x 396 / 2 / 240 = **342.30** plf

End v=[78 x 26.89 x (396 - 13.4)] / 396 /240 **=8** plf

v= 342 + 8 = **350.77** plf

					Nailing Pattern			
	from exterior wall				B.N.	E.N.	Field	
Distance to v=	910 =		ft.	3x	2 1/2	4	12	D
	805 =		ft.	2x	2 1/2	4	12	C
	540 =		ft.	2x	4	6	12	B
	405 =		ft.	2x	6	6	12	A

C-D

Use 15/32 sheathing, see above for grade and nail spacing
All nails shall be 10d x 2 1/8" (1 5/8" penetration)

Note: Allowable shear values from SDPWS ruduced for ASD loads by factor of 2 per Sec 4.2.3

Overall Diaphragm West End of Building

Loads perp to
Long Face

1

Seismic: V= **399** PLF Long Face

Diaphragm Shear:

v= 399 x 396 / 2 / 240 = **329.18** plf

				Nailing Pattern			
	from exterior wall			B.N.	E.N.	Field	
Distance to v=	650 =	ft.	3x	2 1/2	4	12	D
	575 =	ft.	2x	2 1/2	4	12	C
	385 = 0	ft.	2x	4	6	12	B
	290 = 24	ft.	2x	6	6	12	A

C-D

Use 15/32 sheathing, see above for grade and nail spacing
All nails shall be 10d x 2 1/8" (1 5/8" penetration)

Wind: End V= **493** plf (See previous sheet) a= 13.444 ft
Interior V= **415** plf (See previous sheet)

Diaphragm Shear:

v= 415 x 396 / 2 / 240 = **342.30** plf

End v=[78 x 26.89 x (396 - 13.4)] / 396 /240 **=8** plf

v= 342 + 8 = **350.77** plf

				Nailing Pattern			
	from exterior wall			B.N.	E.N.	Field	
Distance to v=	910 =	ft.	3x	2 1/2	4	12	D
	805 =	ft.	2x	2 1/2	4	12	C
	540 =	ft.	2x	4	6	12	B
	405 =	ft.	2x	6	6	12	A

C-D

Use 15/32 sheathing, see above for grade and nail spacing
All nails shall be 10d x 2 1/8" (1 5/8" penetration)

Note: Allowable shear values from SDPWS ruduced for ASD loads by factor of 2 per Sec 4.2.3

Overall Diaphragm

Loads perp to
Trans Face

1

Seismic: V= **502** PLF Trans Face

Diaphragm Shear:

v= 502 x 240 / 2 / 344 = **175.17** plf

				Nailing Pattern			
	from exterior wall			B.N.	E.N.	Field	
Distance to v=	650 =	ft.	3x	2 1/2	4	12	D
	575 =	ft.	2x	2 1/2	4	12	C
	385 =	ft.	2x	4	6	12	B
	290 =	ft.	2x	6	6	12	A

C-D

Use 15/32 sheathing, see above for grade and nail spacing
All nails shall be 10d x 2 1/8" (1 5/8" penetration)

Wind: End V= **450** plf (See previous sheet) a= 13.444 ft
Interior V= **367** plf (See previous sheet)

Diaphragm Shear:

v= 367 x 240 / 2 / 344 = **128.11** plf

End v=[82 x 26.89 x (240 - 13.4)] / 240 /344 **=6** plf

v= 128 + 6 = **134.18** plf

				Nailing Pattern			
	from exterior wall			B.N.	E.N.	Field	
Distance to v=	910 =	ft.	3x	2 1/2	4	12	D
	805 =	ft.	2x	2 1/2	4	12	C
	540 =	ft.	2x	4	6	12	B
	405 =	ft.	2x	6	6	12	A

C-D

Use 15/32 sheathing, see above for grade and nail spacing
All nails shall be 10d x 2 1/8" (1 5/8" penetration)

Overall Diaphragm

Loads perp to
Long Face

Seismic: V= **399** PLF Long Face

Diaphragm Shear:

v= 399 x 396 / 2 / 240 = **329.18** plf

Wind: End V= **493** plf (See previous sheet) a= 13.444 ft

Interior V= **415** plf (See previous sheet)

Diaphragm Shear:

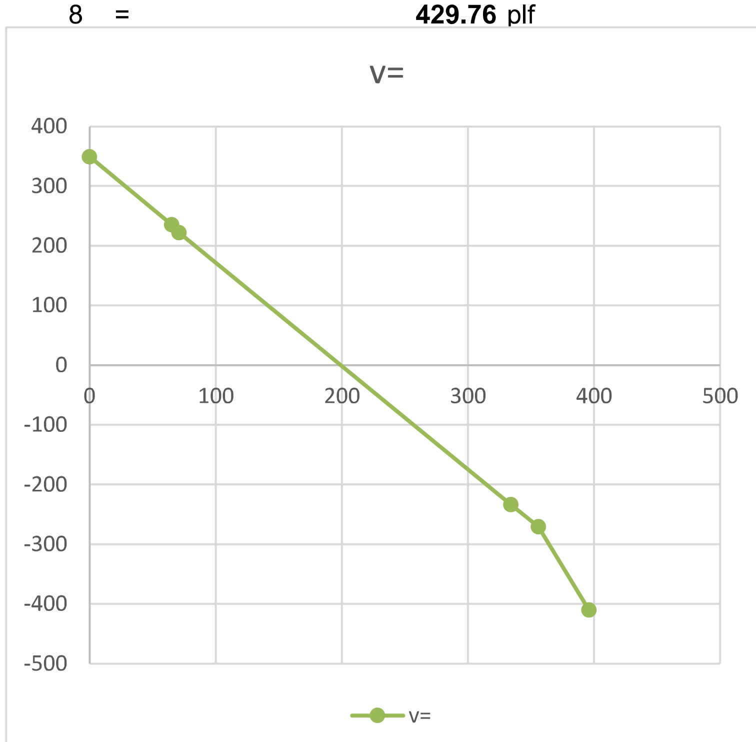
v= 415 x 396 / 2 / 195 = **421.29** plf

End v=[78 x 26.89 x (396 - 13.4)] / 396 /240 **=8** plf

v= 421 + 8 = **429.76** plf

Base Shears at locations x

X	depth	V	v=
0	240	83804.06	349.2
65	237	55782.69	235.4
71	240	53293.22	222.1
334	239	-55828.3	-234
356	240	-64956.3	-271
396	199	-81552.8	-410
0	171	83804.06	490.1
0	188	83804.06	445.8
0	188	83804.06	445.8
0	142	83804.06	590.2
0	142	83804.06	590.2
0	142	83804.06	590.2
0	142	83804.06	590.2
0	142	83804.06	590.2
0	142	83804.06	590.2
0	142	83804.06	590.2
0	142	83804.06	590.2



Overall Diaphragm

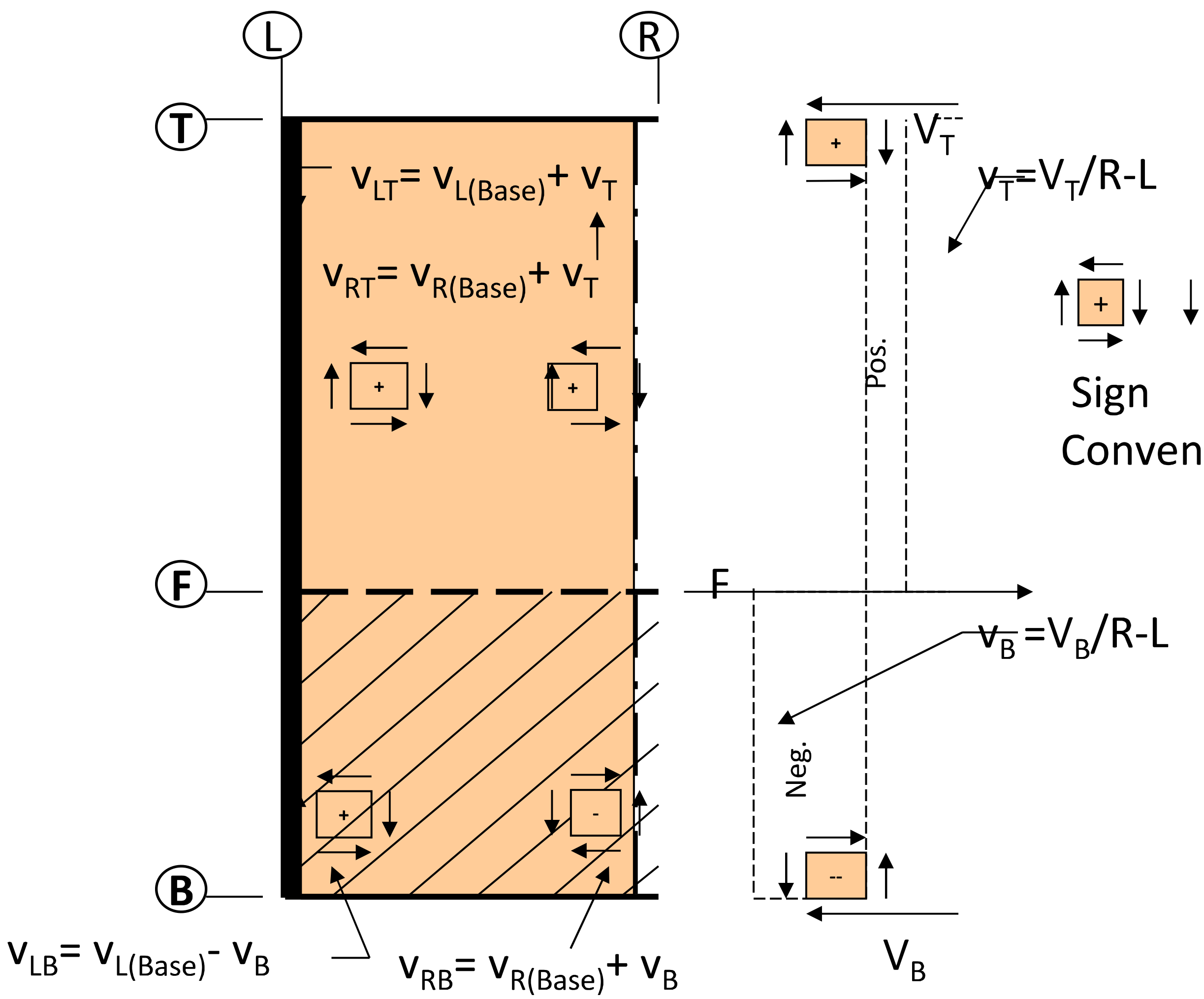
Transfer Diaphragm and transfer shears
T=Top, F=Chord/Strut, B=Bottom, L=left, R=Right

Grids					Distance Between		Distance from X=0		Base v		
T	F	B	L	R	T-F	F-B	L	R	L	R	F
A	A.3	B	1.6	2.3	38	7	41	89	277	191	17

Vt & Vb: Sum moments about opposite side
Vt= F*(dist F to B)/(dist T to B) = 17 * 7 / 45 = 2.64444 K
Vb= 17 * 38 / 45 = 14.3556 K

vt & vb: Divide V by dist R-L
vt= 2.6444 / 48 = 55.0926 plf
vb= 14.356 / 48 = 299.074 plf

Find new total shear at each section: NOTE SIGN CONVENTION
vlt= 277 + 55.09259259 = 332.093
vrt= 191 + 55.09259259 = 246.093
vlb= 277 - 299.0740741 = -22.074
vrb= 191 - 299.0740741 = -108.07



Overall Diaphragm

Transfer Diaphragm and transfer shears
T=Top, F=Chord/Strut, B=Bottom, L=left, R=Right

Grids					Distance Between		Distance from X=0		Base v		
T	F	B	L	R	T-F	F-B	L	R	L	R	F
F	F.1	E	6.6	7.3	53	7	311	359	-194	-276	20.3

Vt & Vb: Sum moments about opposite side

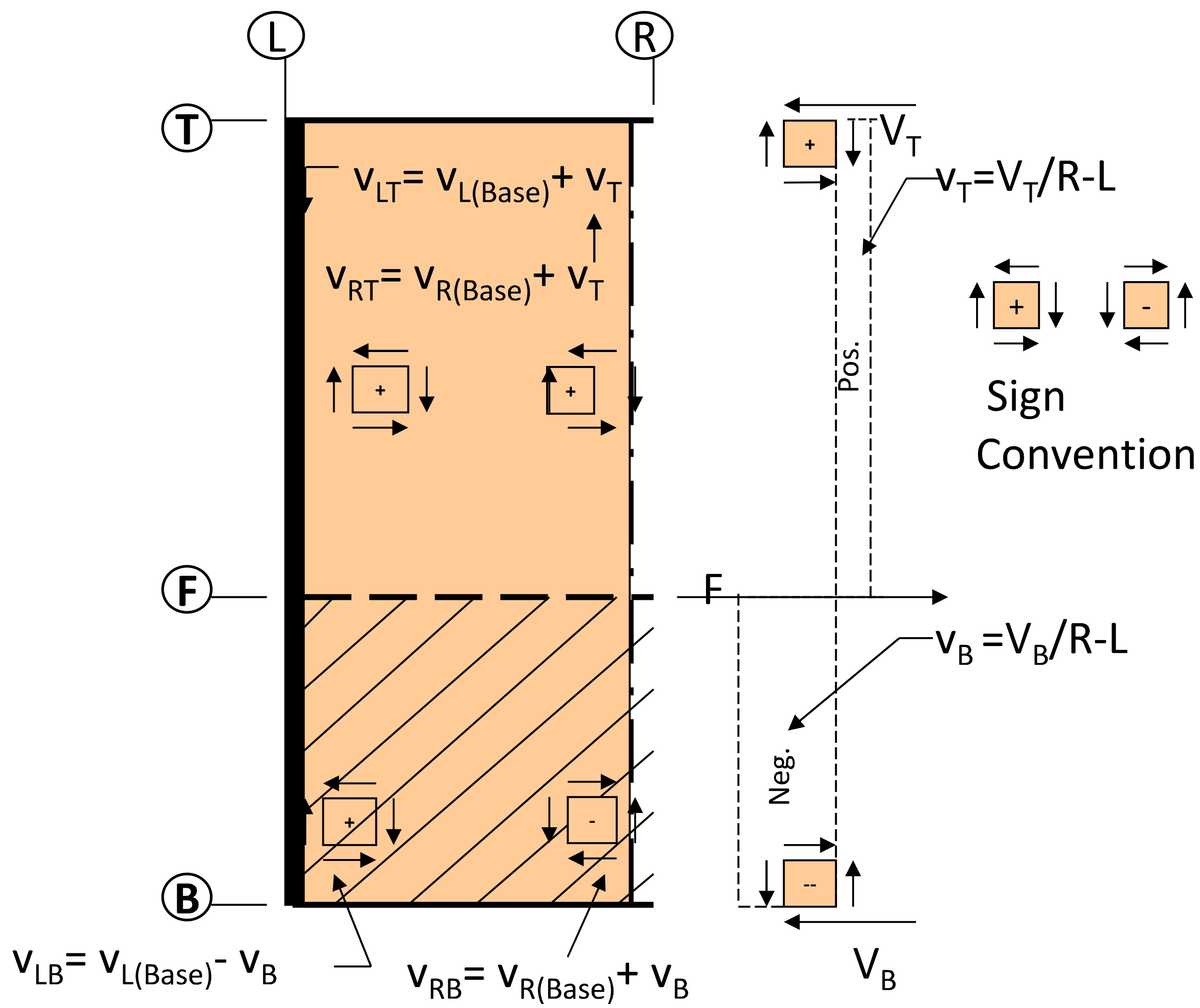
Vt= F*(dist F to B)/(dist T to B) =	20.3 *	7 /	60 =	2.36833 K
Vb=	20.3 *	53 /	60 =	17.9317 K

vt & vb: Divide V by dist R-L

vt=	2.368 /	48 =	49.3403 plf
vb=	17.93 /	48 =	373.576 plf

Find new total shear at each section: NOTE SIGN CONVENTION

vlt=	-194 +	49.3 =	-144.66
vrt=	-276 +	49.3 =	-226.66
vlb=	-194 -	374 =	-567.58
vrb=	-276 -	374 =	-649.58



Chord-Tie

				Front	Trans Face		
Seismic:		V=	**399**	PLF	Long.		
			502	PLF	Trans.		
Wind:	End	V=	**493**	plf (See previous sheet)		a=	13.444 ft
	Interior	V=	**415**	plf (See previous sheet)			

Gridline 2 & A

Chord-Splice

L= 396 Ft
w= 415 plf
x= 56 Ft
M= 415 x 56 /2 (396 - 56) = 3950 K-FT
T= 3950 / 233 = 16.9526 K

Design Girders for 17.0 K axial load

As= 16.95 / 36 / 0.60 / 1 = 0.78484 in^2

Use 1/2 x 1.57 Tie Plate

Lw= 16.95257483 / 2 / 4 / 1 / 0.93 = 2.28348 in

Use Fillet 1/4 x 2.28 each side

Note: Use HSS2 1/2 x2 1/2 x 3/16

Gridline F and 7

Chord-Splice

L= 396 Ft
w= 415 plf
x= 326 Ft
M= 415 x 326 /2 (396 - 326) = 4734 K-FT
T= 4734 / 233 = 20.3182 K

Design Girders for 20.3 K axial load

As= 20.32 / 36 / 0.60 / 1 = 0.94066 in^2

Use 1/2 x 1.88 Tie Plate

Lw= 20.31815954 / 2 / 4 / 1 / 0.93 = 2.73682 in

Use Fillet 1/4 x 2.74 each side

Note: Use HSS2 1/2 x2 1/2 x 3/16

Gridline B and 7

L= 396 Ft
w= 415 plf
x= 350 Ft
M= 415 x 350 /2 (396 - 350) = 3340 K-FT
T= 3340 / 188 = 17.7661 K

Design Girders for 17.8 K axial load

Note: less than typical Girder Axial (41.4K)

Gridline 1.1 and 7.3

L= 240 Ft
w= 466 plf
x= 120 Ft
M= 466 x 120 /2 (240 - 120) = 3355 K-FT
T= 3355 / 384 = 8.7375 K

Design Joist for 8.7 K axial load

As= 8.738 / 36 / 0.60 / 1 = 0.40451 in^2

Use 1/4 x 1.62 Tie Plate

Lw= 8.7375 / 2 / 3 / 1 / 0.93 = 1.56923 in

Use Fillet 3/16 x 1.57 each side